



IABEE 2026



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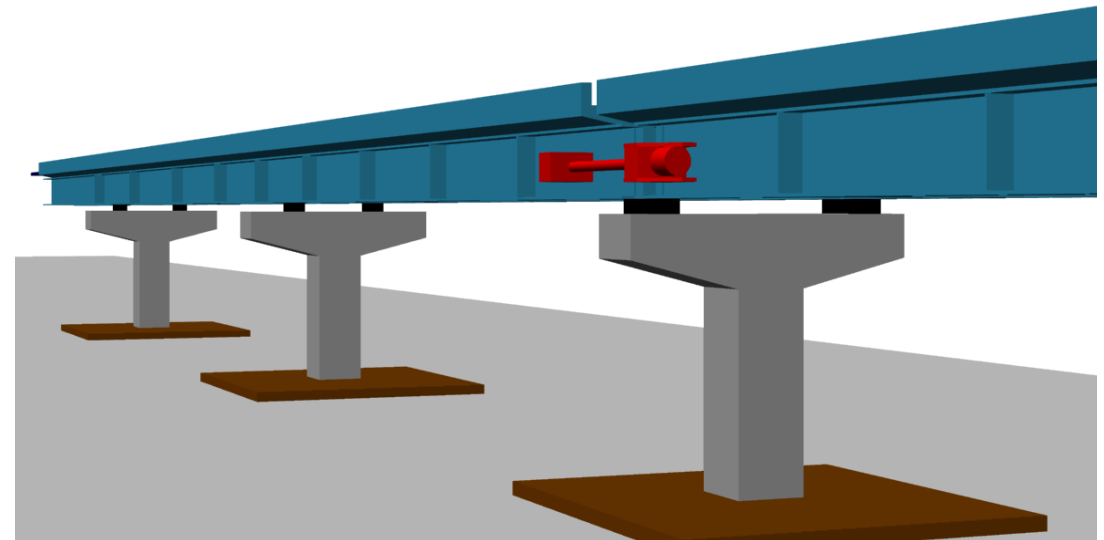


Nagoya Institute of Technology

Shaking Table Tests of Two-span Continuous Elavated-Girder Bridge Models Supported by Steel Piers

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Structural robustness against extreme earthquakes :

The structural robustness against extreme earthquakes is defined as the ability of structure to withstand local failure with suffering disproportionate extensive collapse in extreme earthquakes.



Local buckling in steel piers

1995 Kobe
Earthquake



Piltz type elevated-girder bridge(RC structure)

Disproportionate failure caused

I. Background

(1) Necessity to improve the robustness of bridges against imminent extreme earthquakes

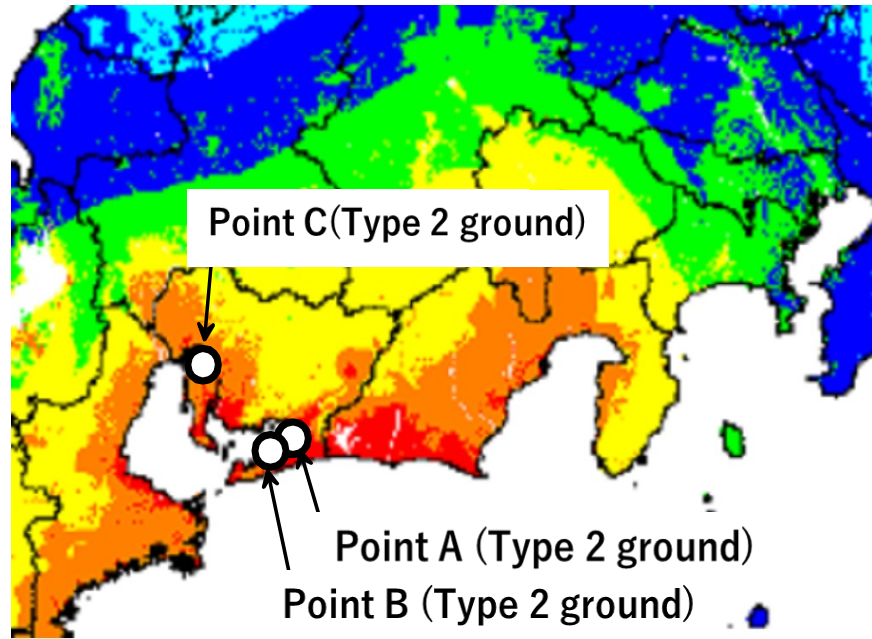
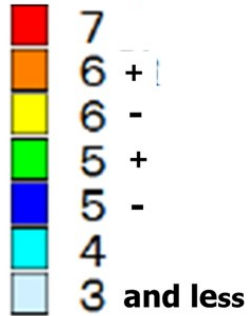
In Japan, for the past 30 years, major earthquakes have frequently occurred.

Recent major earthquakes	Magnitude	Casualties
1995 Kobe (Inland)	M=7.2	6,437
2004 Chuetsu (Inland)	M=6.8	68
2007 Chuetsu -oki (Inland)	M=6.8	15
2008 Iwate-miyagi (Inland)	M=7.2	25
2011 Tohoku (Offshore)	M=9.0	20,960
2016 Kumamoto (Inland)	M=7.3	216
2024 Noto (Inland)	M=7.6	240

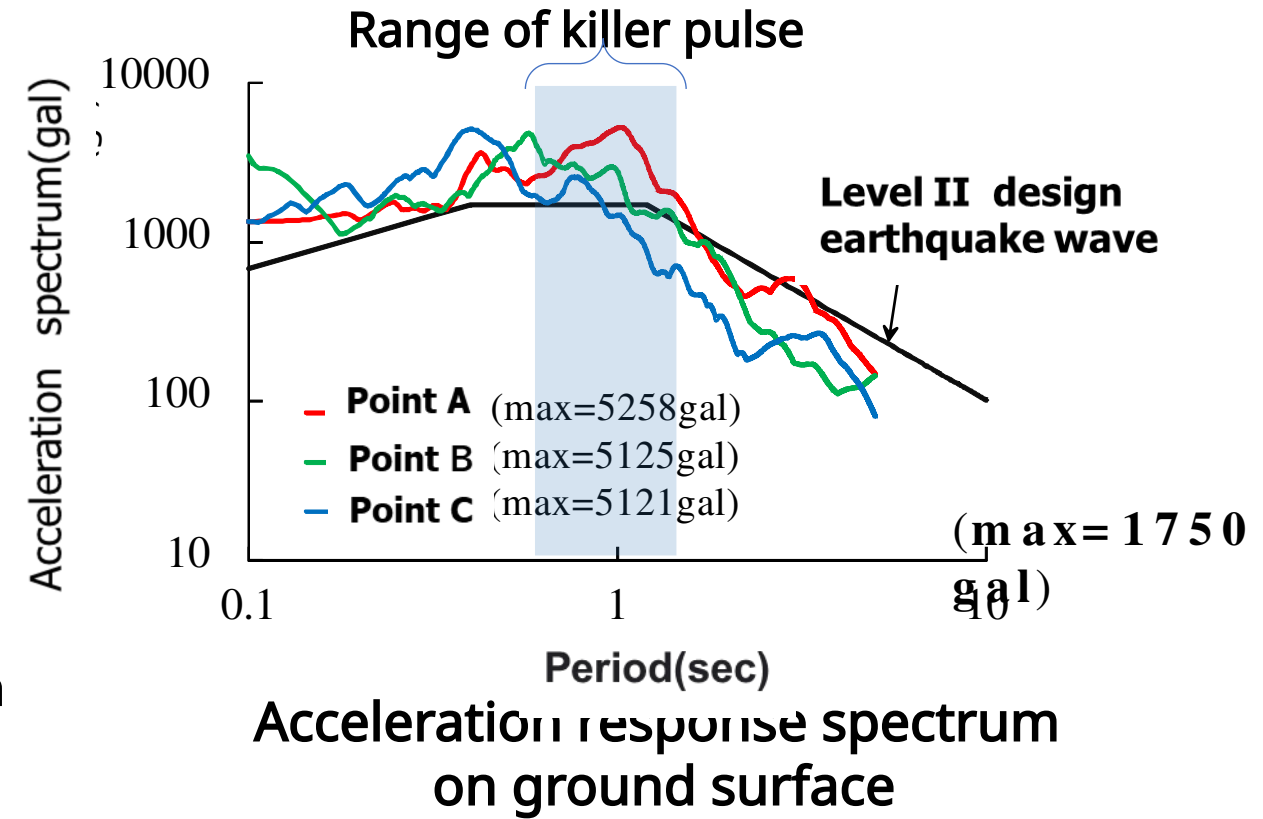
For the next 30 years, extreme earthquakes in Japan are expected occur with nearly 100 percent possibility of occurrence.

(2) Predicted extreme killer pulses of the Nankai trough mega earthquake

Seismic intensity scale



Nankai trough earthquake is expected to occur from 2030 to 2040.

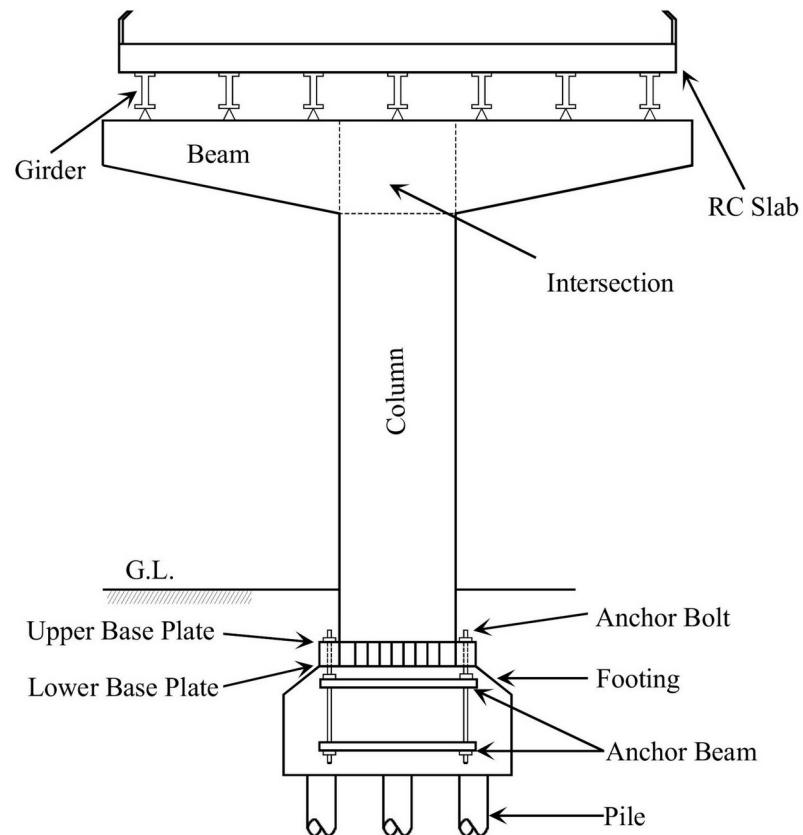


The acceleration response spectrums are located around the vibration period of 1 second and these peak values amounts to 3 times as large as the current design earthquake motion. These characteristics of the acceleration response spectrums indicate that the extreme killer pulses are included in the ground motions of the Nankai trough earthquake.

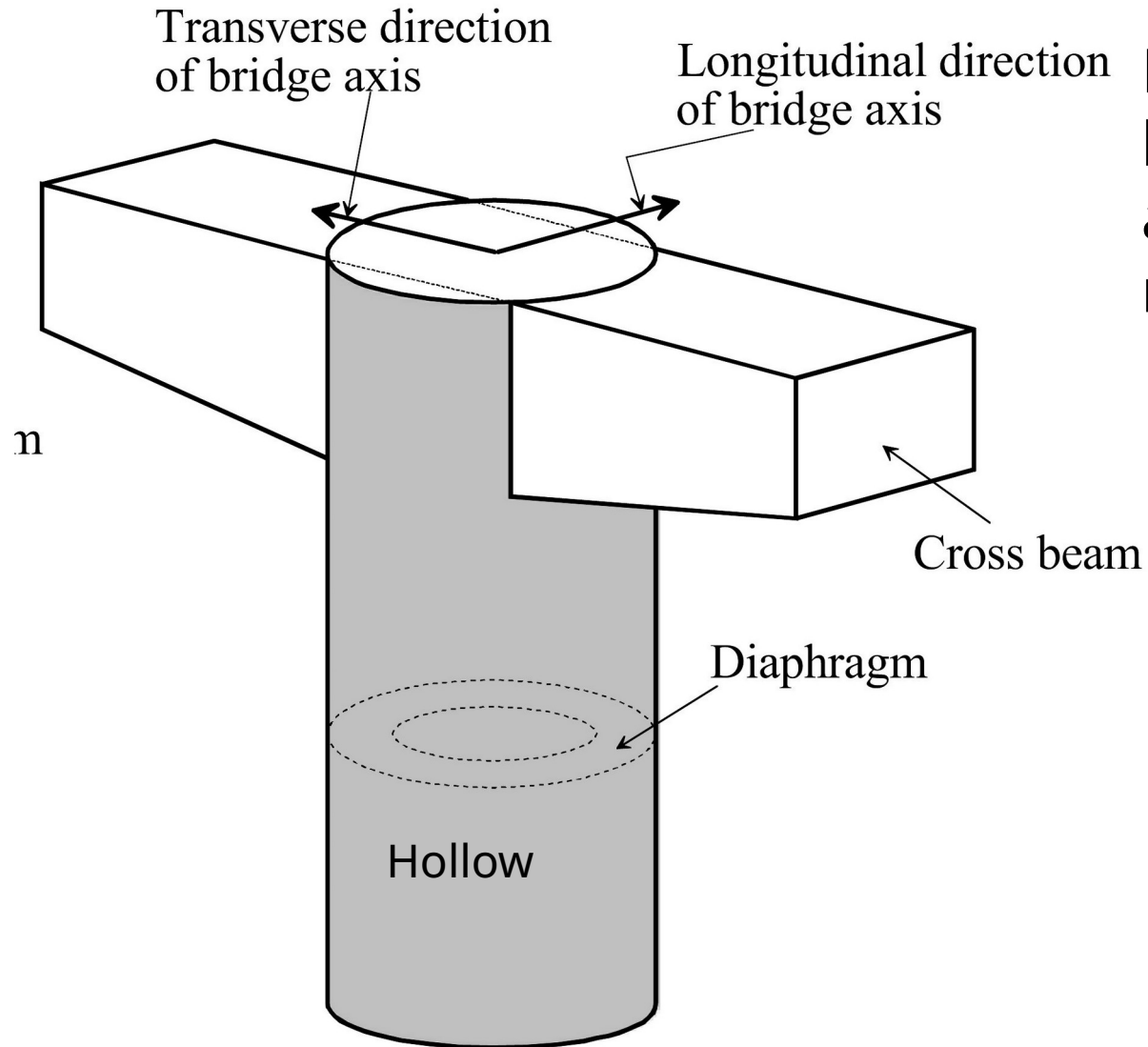
urgent necessity to consider some strategies to prevent the disproportionate collapse of piers

II. Objective

Improve the robustness of thin-walled steel piers in elevated-girder bridges, based on the concept of the collapse control design .



III. Examine robustness of existing thin-walled hollow steel piers



Radius to thickness parameter are limited by this equation for the pier to have appropriate deformation capacity in plastic range

$$R_t = \frac{R}{t} \frac{\sigma_y}{E} \sqrt{3(1 - \nu^2)} \leq 0.08$$

The hollow cross section of the pier is stiffened by diaphragm to prevent cross-sectional deformation .

(1) Experiment and analysis to assess the collapse behavior of single thin-walled hollow column under horizontal seismic accelerations



2008~2010



Column base(analysis)

Developed
collapse
analysis

$$R_t = 0.10$$

$$\bar{\lambda} = 0.52$$



柱基部

Column base(experiment)

As a preliminary research, a shake table test was conducted on a single thin-walled hollow column model with a weight at the top.

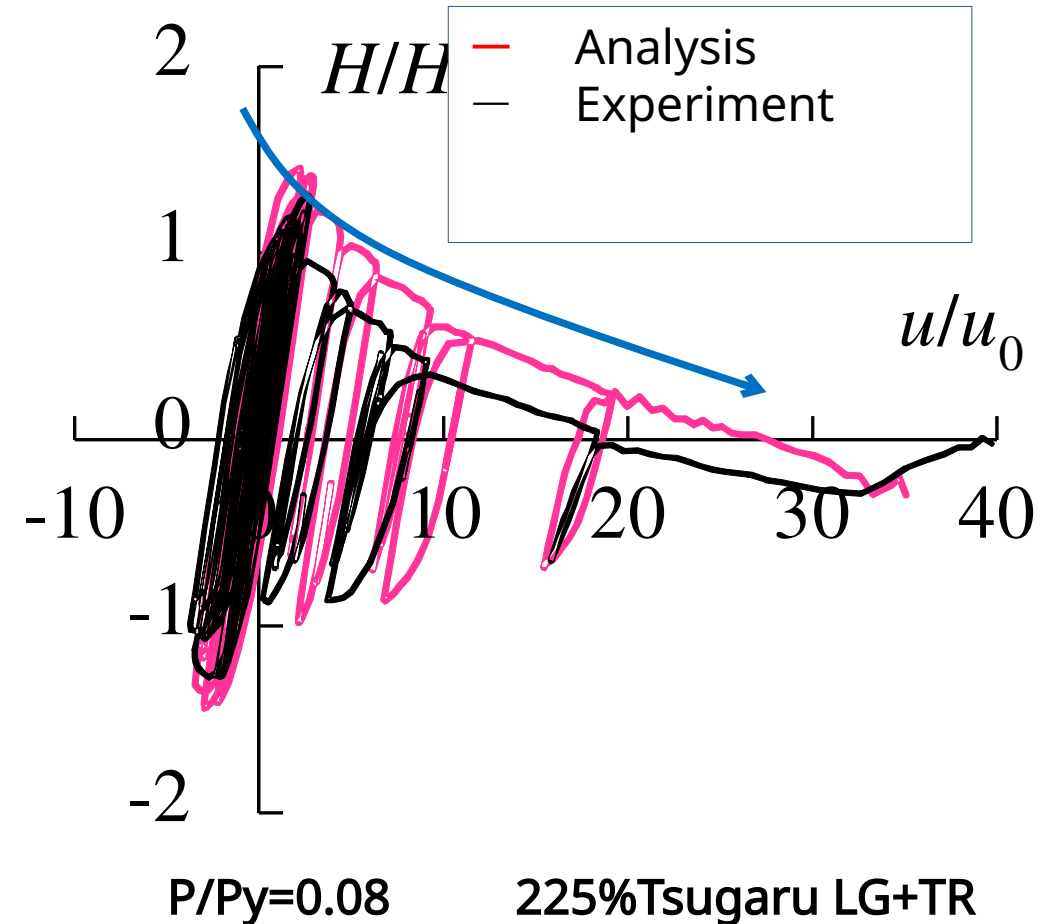
The pier collapsed due to the local buckling deformation near the column base of the thin-walled column.

Regarding the local buckling behavior, the experiment and analysis are in good agreement.

(1) Experiment and analysis to assess the collapse behavior of single thin-walled hollow column under horizontal seismic accelerations

The horizontal restoring force-horizontal displacement relations at the top of the column, obtained by the experiment and FE analysis. Both relations are also in good agreement.

The restoring force decreases significantly in the post-peak range and the column finally collapsed.



(2)-a Collapse analysis model for elevated-girder bridge systems with thin-walled hollow steel piers.

Structural component models validated by experiments

Piers

Bearings

Superstructure

Superstructure

Rubber bearing

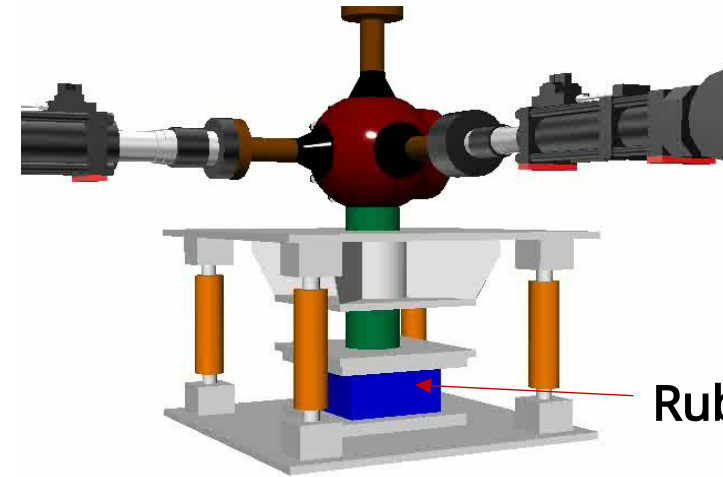
Steel or CFT bridge pier

Assemble

Entire elevated-girder bridge model

Calibration

Collapse experiment on entire bridge model

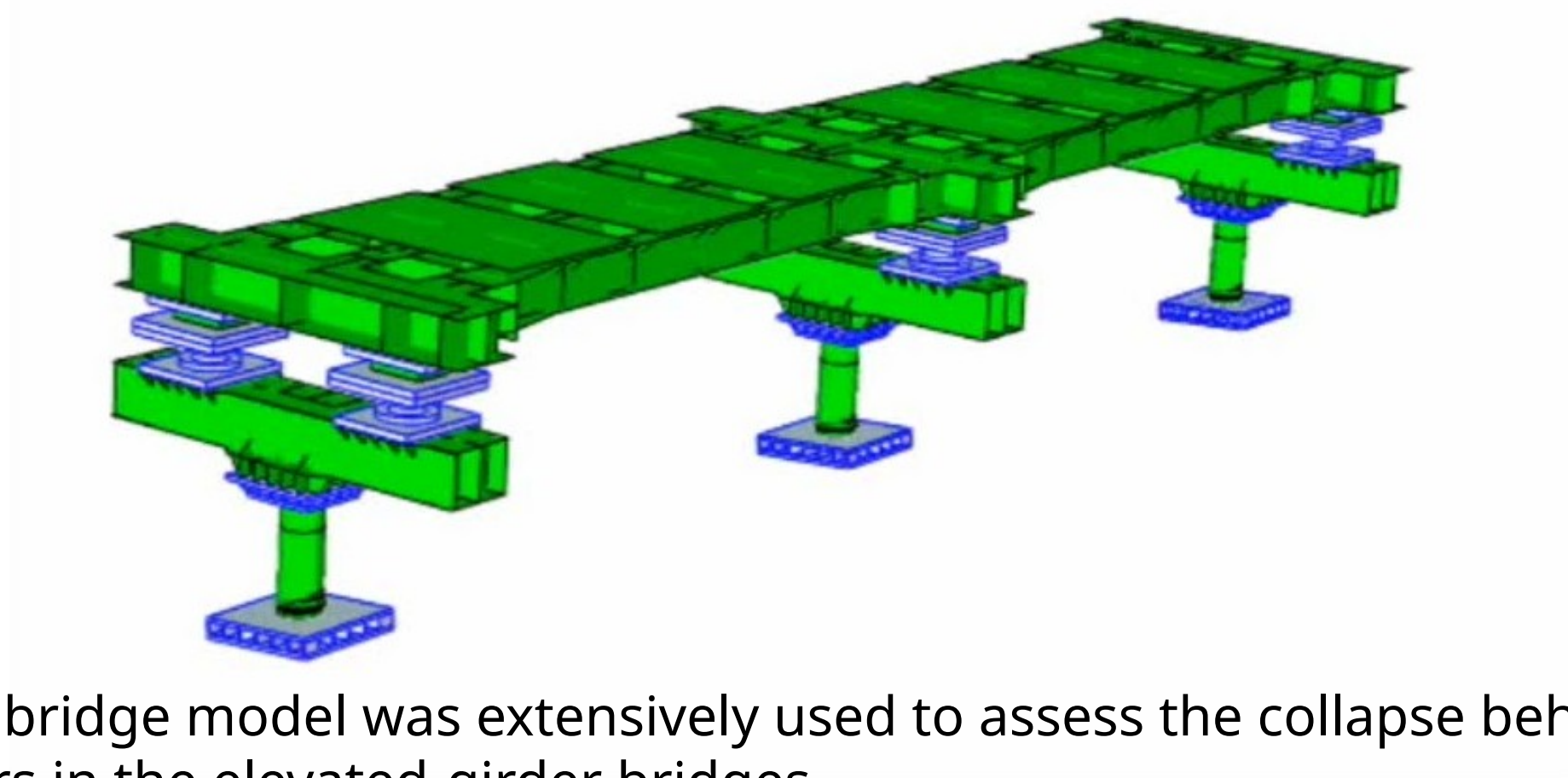


Rubber bearing

Bi-directional cyclic loading test on bearing

(2)-a Collapse analysis model for elevated-girder bridge systems with thin-walled hollow steel piers.

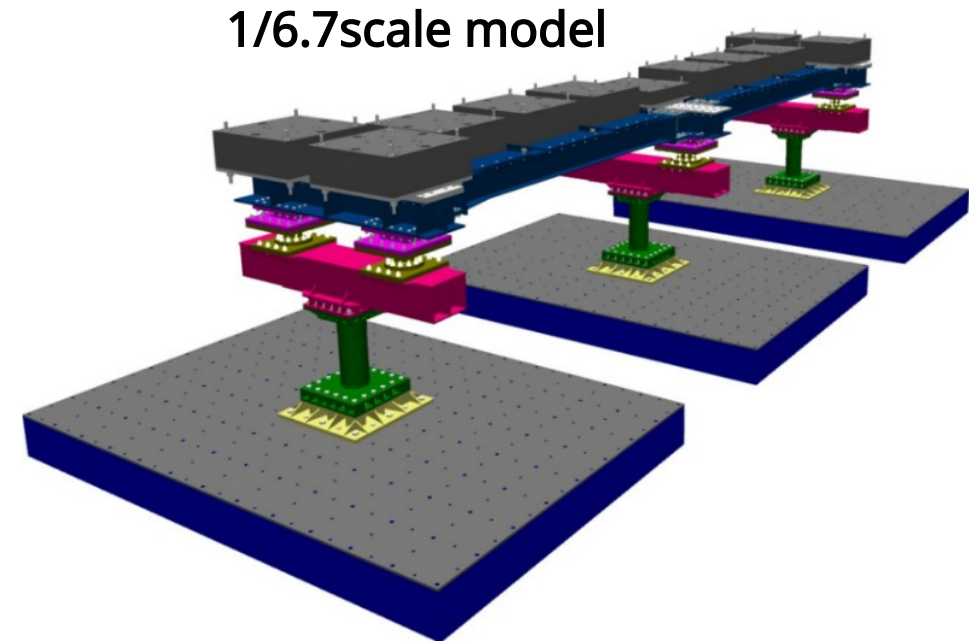
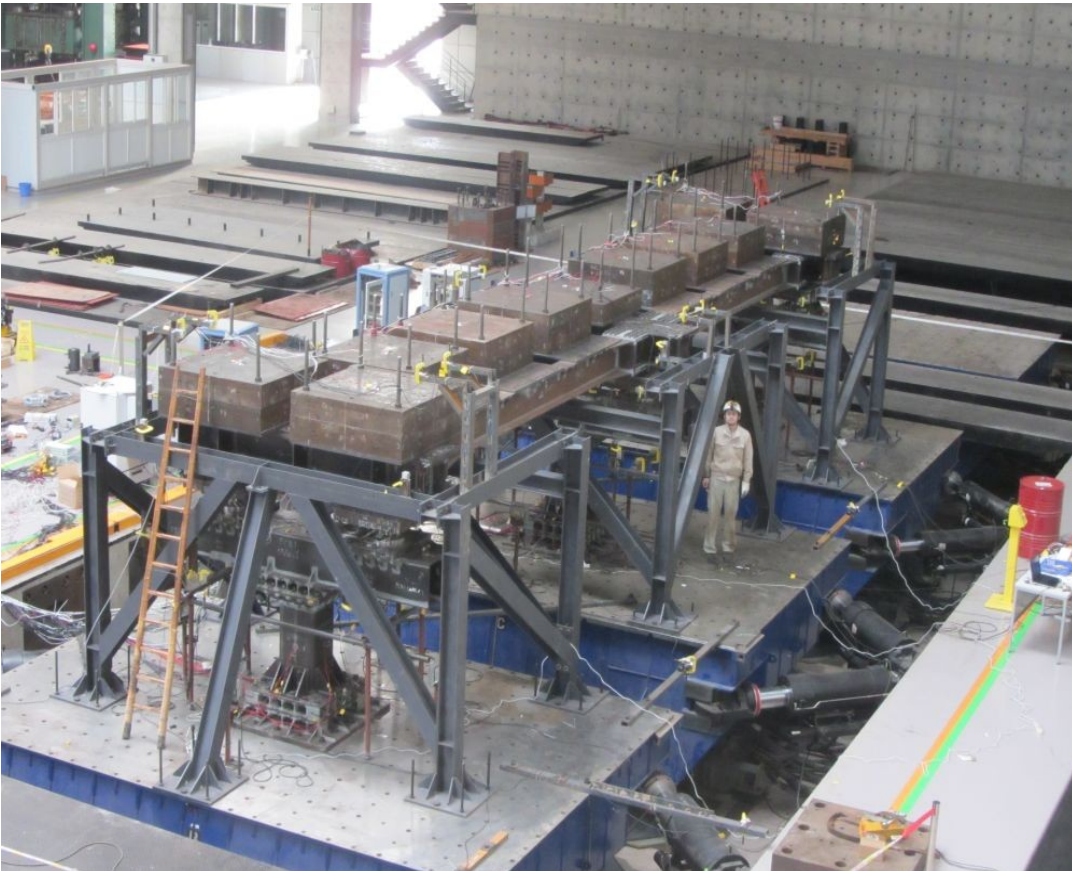
Example of the calculated collapse behavior of the entire elevated-girder bridge model.



This FE bridge model was extensively used to assess the collapse behavior of the piers in the elevated-girder bridges.

(2)-b Bi-directional shake table test on 1/6.7 scale two-span continuous elevated-girder bridge model with thin-walled hollow steel piers

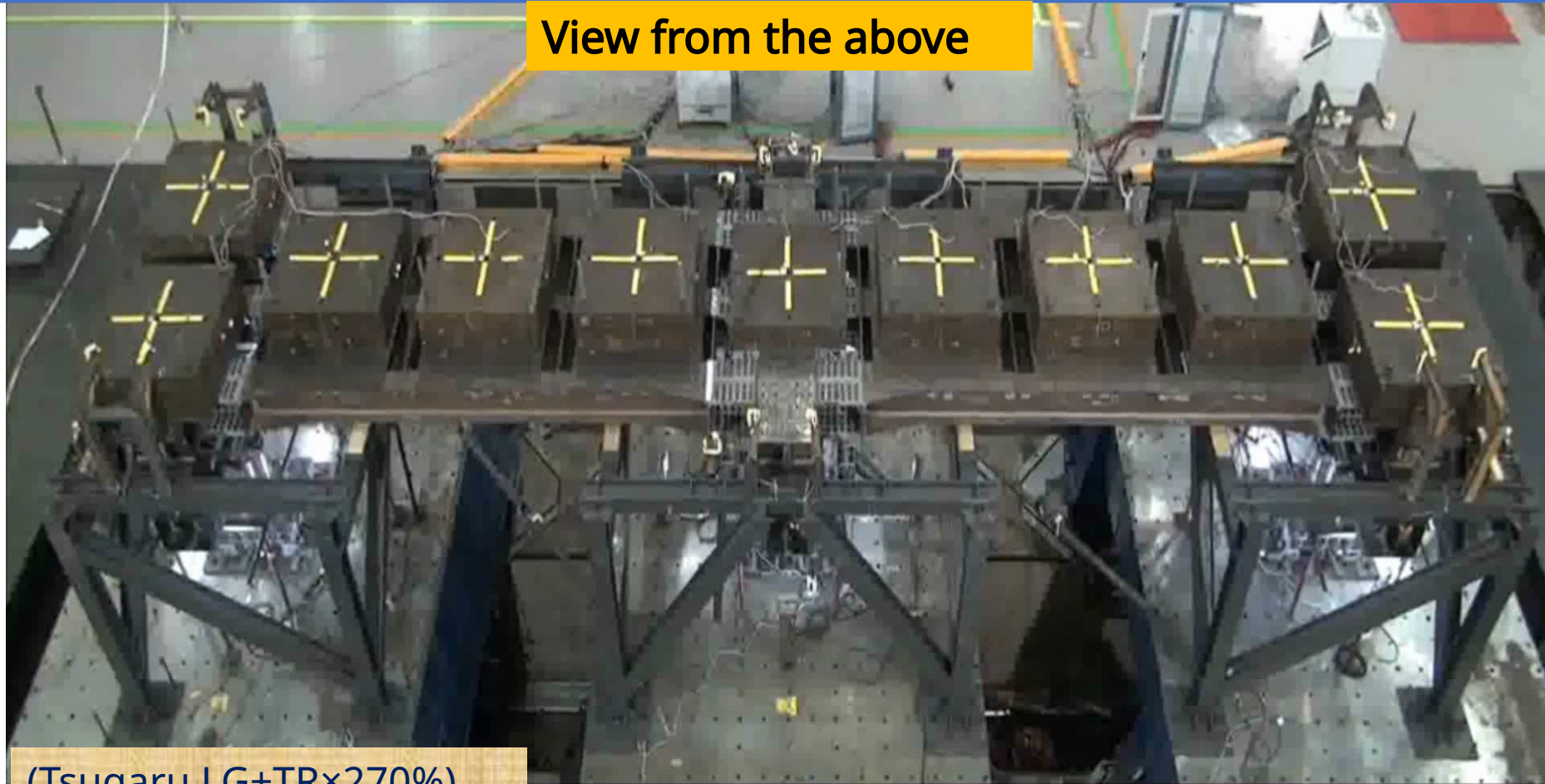
Examine the effect of the interaction between each pier, rubber bearings and the superstructure on collapse behavior of entire bridge system



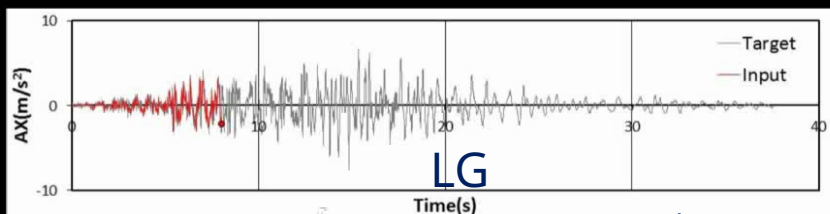
Shake table array at Tongji university

(2)-c Excitation of elevated-girder bridge model by shake table array (2014)

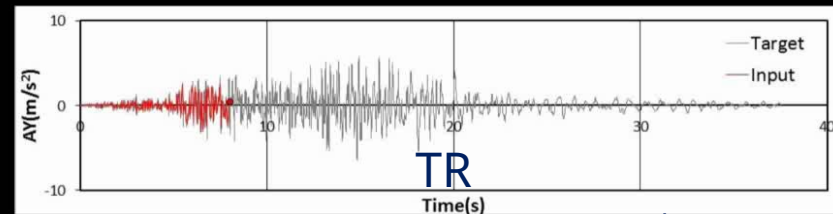
View from the above



(Tsugaru LG+TR×270%)

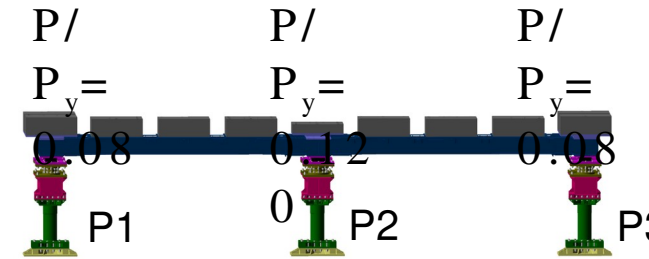
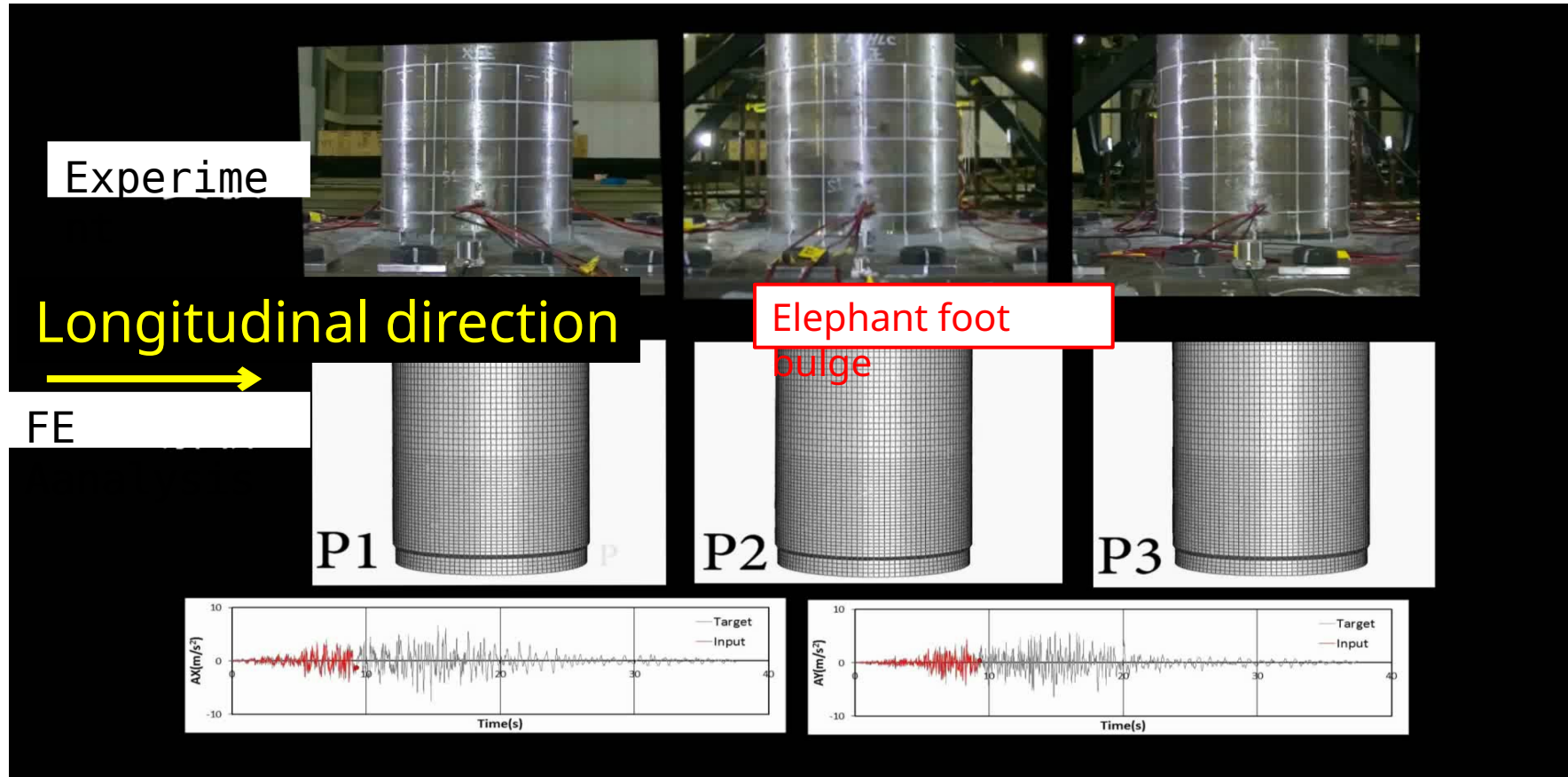


Time axis : $t/\sqrt{6.7}$ component



component

(2)-d Progress of local buckling bulge in each thin-walled hollow steel pier obtained by shake table test and analysis



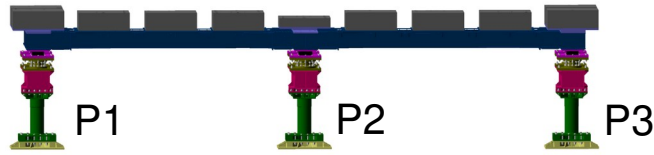
(Tsugaru wave(LG+TR) ×270%)

The axisymmetric local buckling bulge occurred in each pier.

The center pier exhibits the largest local buckling deformation because the largest axial force acts on the center pier.

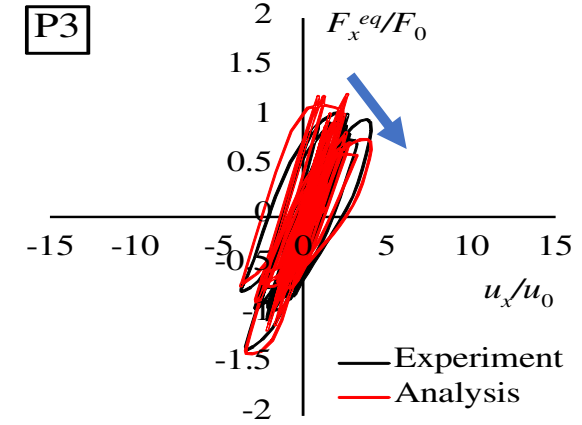
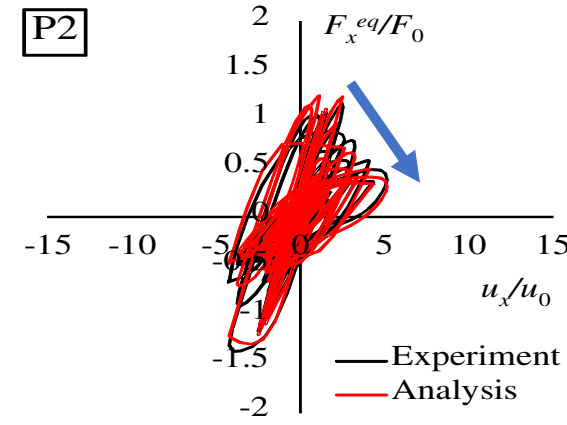
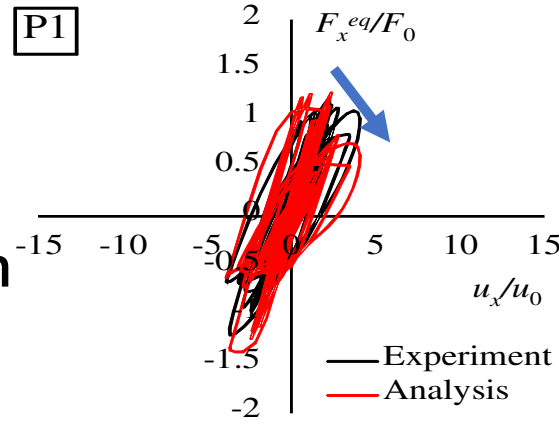
The FE analysis accurately simulates the local buckling behavior of each pier.

(2)-e Horizontal force- horizontal displacement relations (Tsugaru wave (LG+TR)×270%)

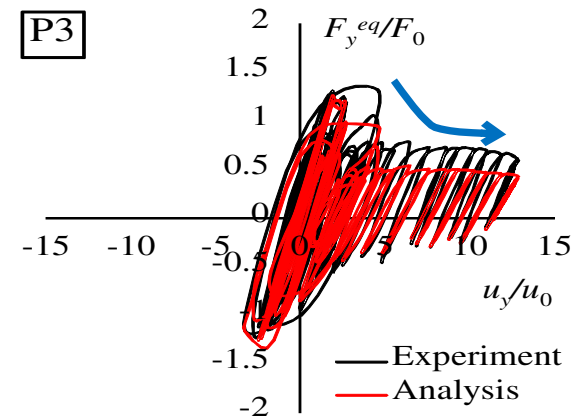
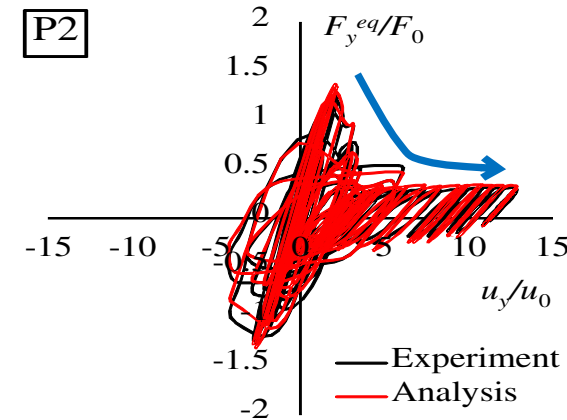
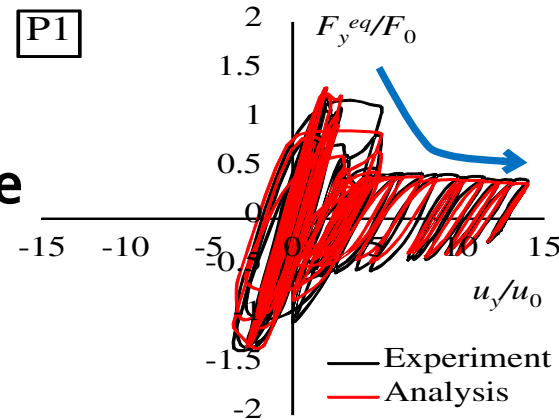


All the piers were almost similarly collapsed in the transverse direction due to the local buckling deformation

Axial
direction



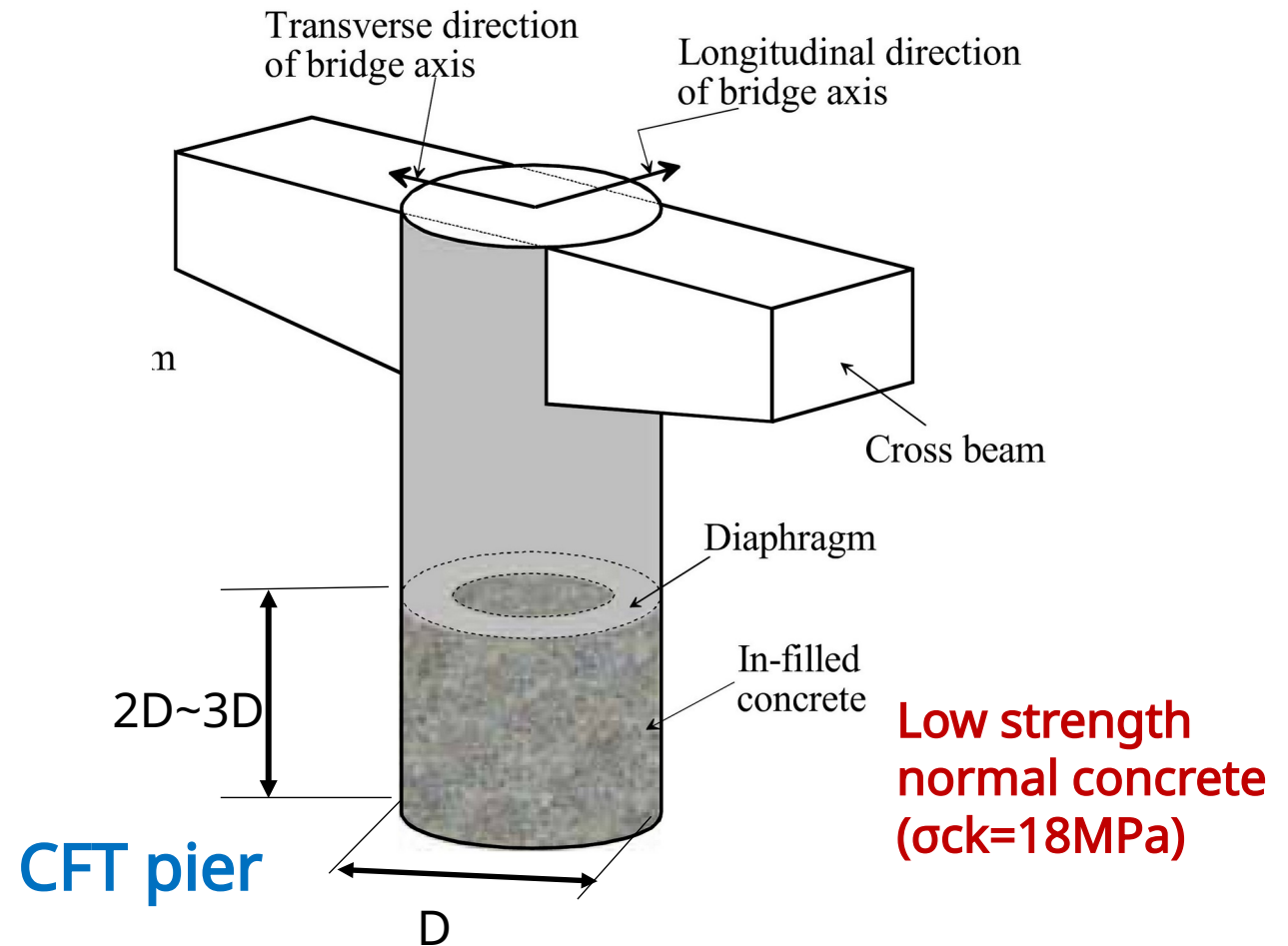
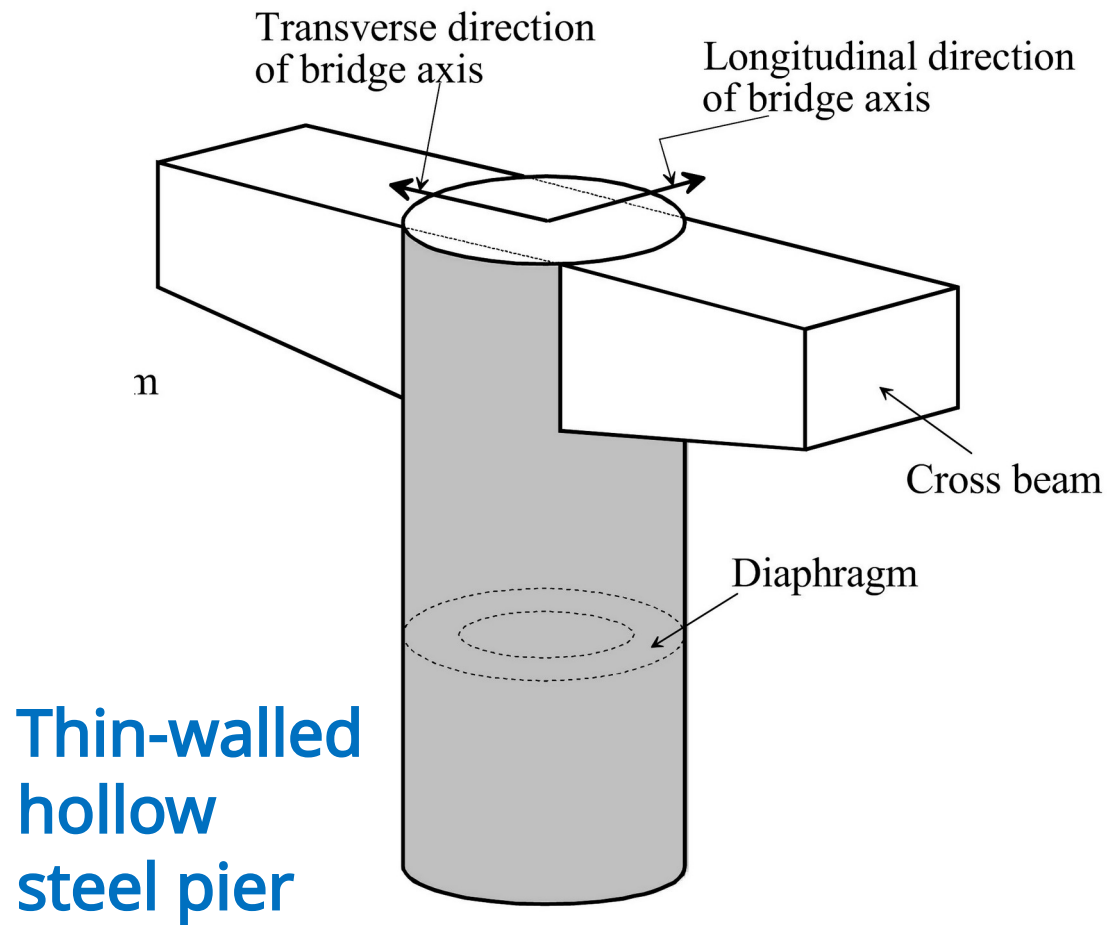
Transverse
direction



The hysteretic curves obtained for each piers are similar to that obtained from the single pier model test. These hysteretic curves imply that the piers exhibit the collapse behavior immediately after reaching the peak points.

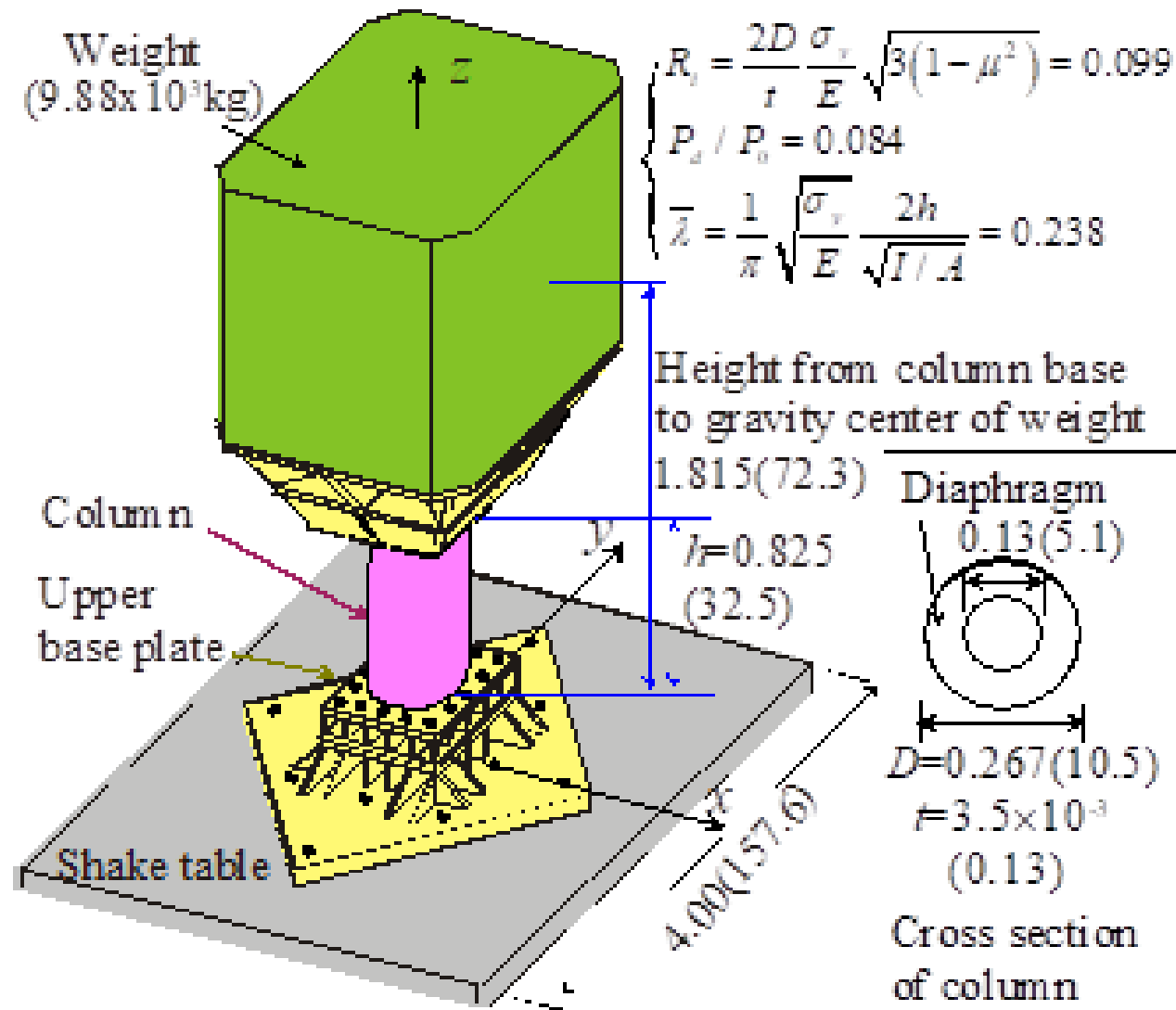
IV. Improve robustness of thin-walled hollow steel piers by filling their inside with low strength concrete

As a practical method to improve the robustness of existing thin-walled hollow steel piers, we examined the method of filling the inside of the hollow steel tube with low strength concrete and converting them to CFT piers. In the CFT pier, concrete is filled up to the location of the diaphragm installed inside of the hollow pier.



(1)-a

Shake table test on thin-walled hollow and CFT piers



Similar to the hollow steel pier, a shake table test was conducted on a single CFT pier model with a weight on the top.

(1)-a

Shake table test on thin-walled hollow and CFT piers

Hollow

$$R_t = 0.099$$
$$\lambda = 0.523$$

$P/P_y = 0.08$
225%Tsugaru LG+TR



CFT

$$R_t = 0.095$$
$$\lambda = 0.524$$

$P/P_y = 0.08$
300%Tsugaru LG+TR



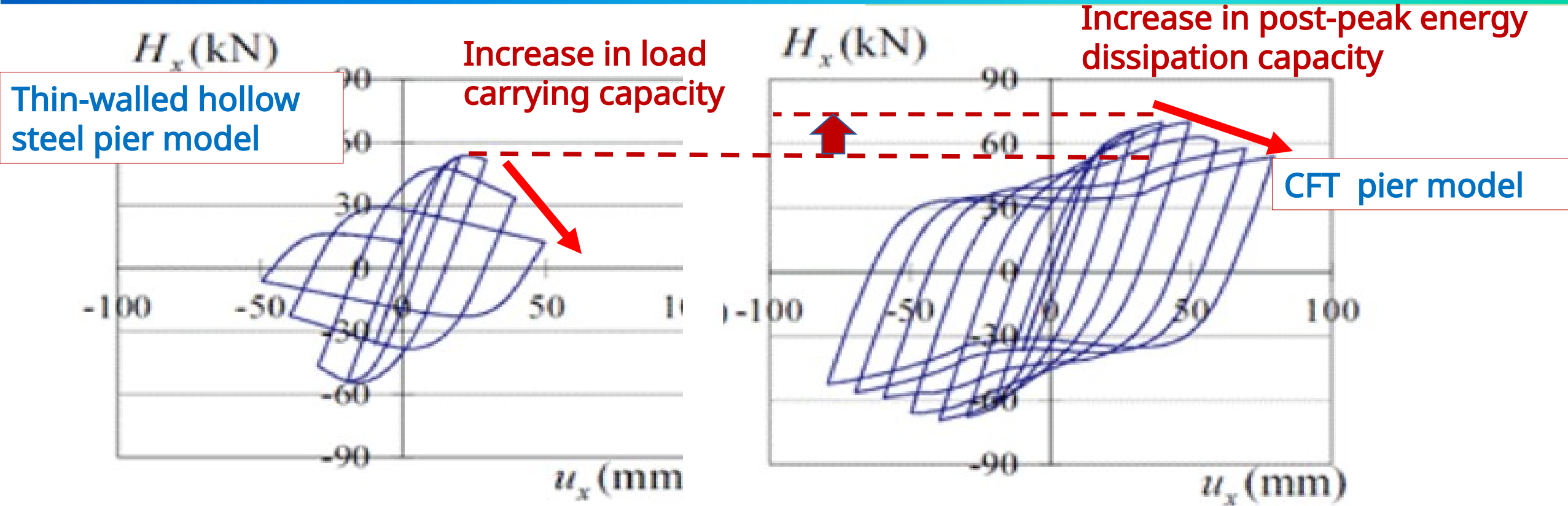
Experiment (2008~2010)



FE analysis

Local buckling deformation
at column base is
restrained in CFT piers.

(1)-b Hysteretic behaviors of thin-walled hollow pier and CFT pier models



The post-peak energy dissipation of the CFT pier model is much improved from that of the thin-walled hollow steel pier model. This improvement of the post-peak energy dissipation capacity in the CFT pier model is due to the local buckling restraining behavior of the in-filled concrete. However, there is a problem for the CFT pier model in that its load-carrying capacity increases, compared to that of the original thin-walled hollow steel pier model.

(2)-a Failure behavior of CFT piers in elevated-girder bridge model

Input acceleration wave : Tsugaru (LG+TR) $\times 350\%$, Time axis : $t/\sqrt{6.7}$



(2017)

Longitudinal
direction →

Longitudinal direction →

Longitudinal direction →



P
1



P
2

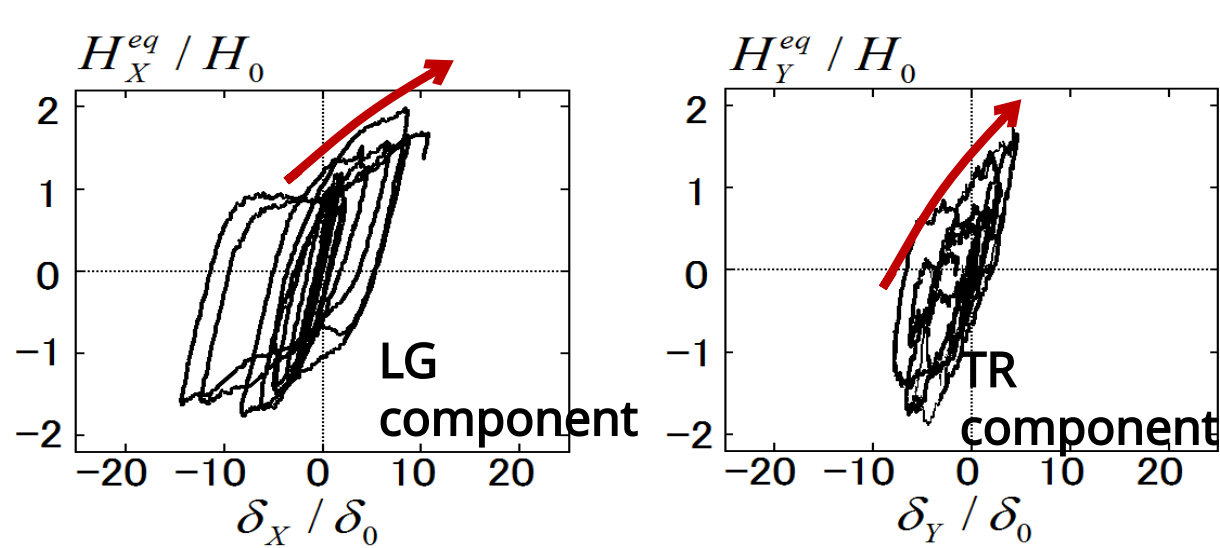


P
3

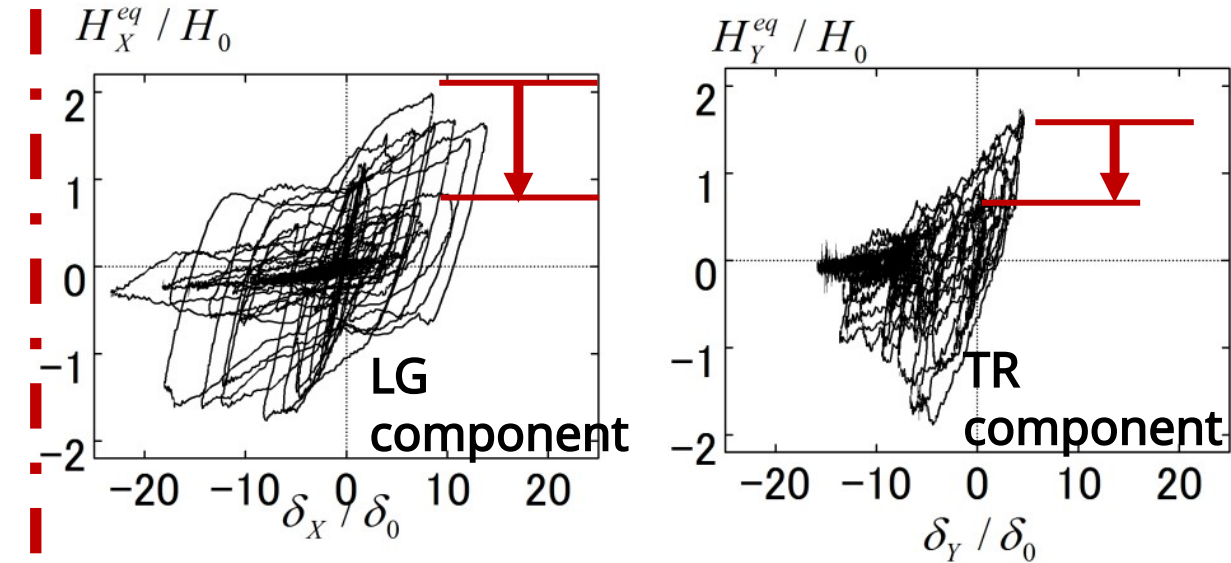
STK400
 $D = 262(\text{mm})$
 $t = 4.29(\text{mm})$
 $h_c = 529(\text{mm})$
 $R_t = 0.087$
 $\bar{\lambda} = 0.231$
 $P_d / P_0 = 0.119$

- Low-cycle fatigue crack occurred horizontally at the local buckling location of pier

(2)-b Hysteretic behavior of CFT piers before and after low cycle fatigue



A-1. **CFT** before occurrence of fatigue crack

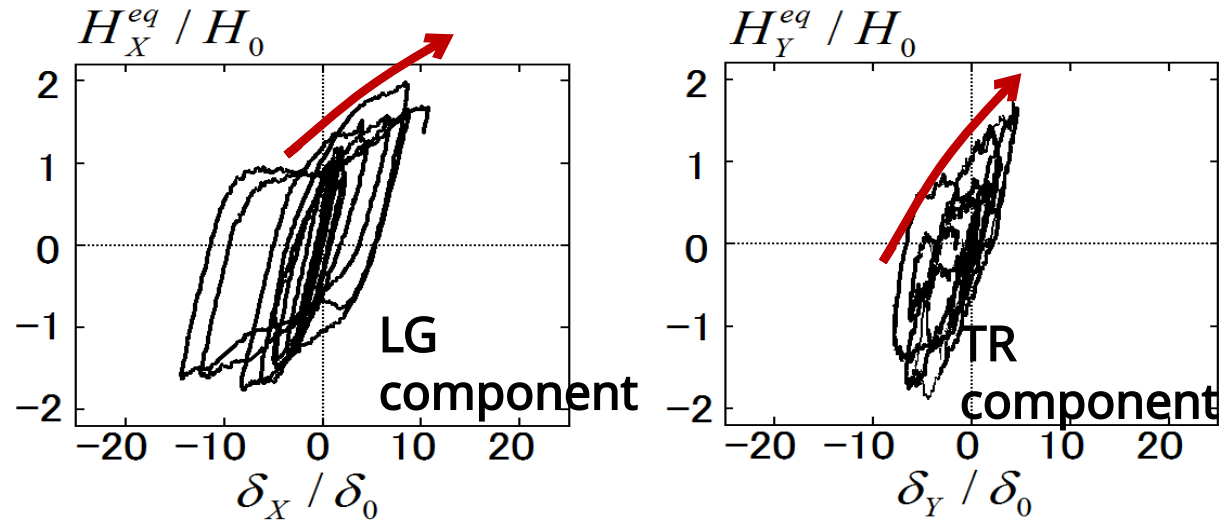


A-2 . **CFT** after occurrence of fatigue crack

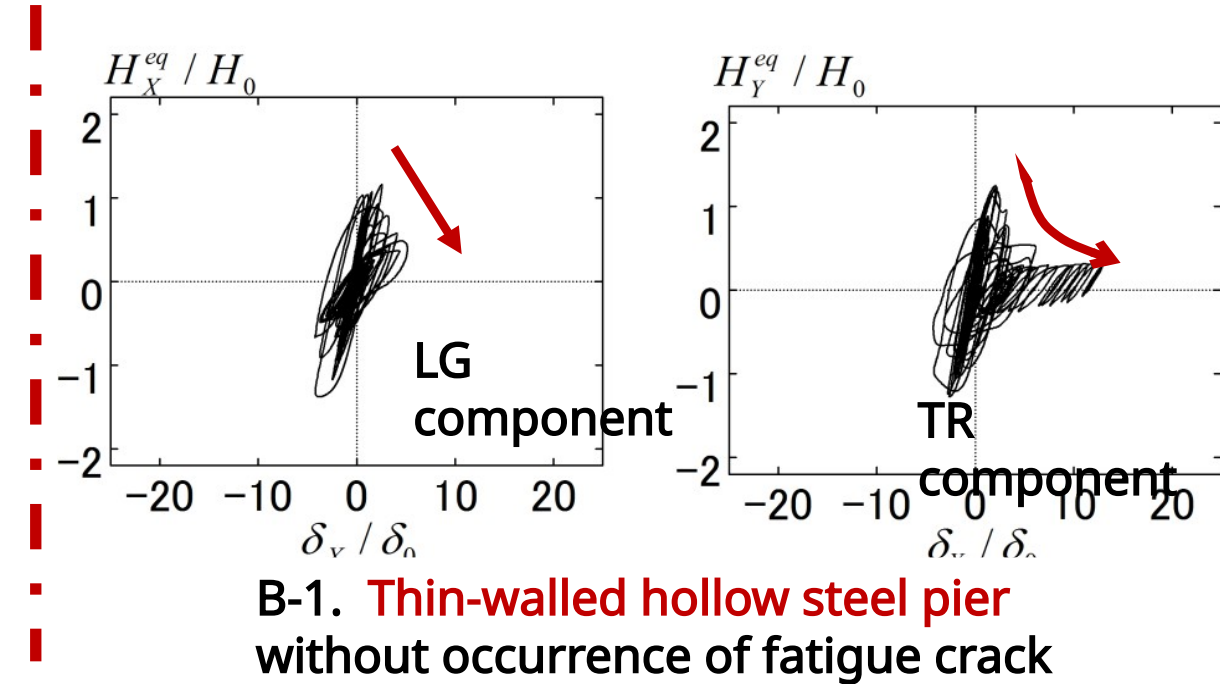
The horizontal restoring force of CFT pier monotonically increases even if the local buckling occurs in steel pipe. Therefore, the energy dissipation of the CFT pier is much larger than that of the hollow steel pier.

However, the increased horizontal restoring force caused low cycle fatigue cracks. The horizontal restoring force suddenly dropped, as soon as the fatigue crack occurred.

(2)-b Hysteretic behavior of CFT piers before and after low cycle fatigue



A-1. **CFT** before occurrence of fatigue crack



B-1. **Thin-walled hollow steel pier** without occurrence of fatigue crack

In the case of the hollow steel pier, the peak point is reached by the occurrence of the local buckling of the steel tube and the pier collapsed laterally with the progress of the local buckling deformation.

In this case, no fatigue cracks occurred in the outer steel pipe.

Trial use of multi-CFT column piers in Hanshin expressway



Ebie JCT 2012(1 pier) , Osaka,
Japan



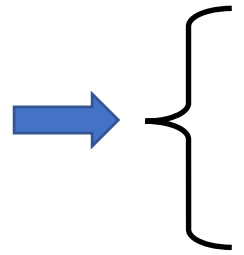
Nishisenba JCT (12 piers) 2018, Osaka,
Japan

© Hanshin Expressway

V. Recently developed multi-CFT column piers with shear dampers (Hanshin expressway, Osaka, Japan)

For L2 design earthquake (1000~2000 gal)

- Shear dampers mainly dissipate seismic energy.



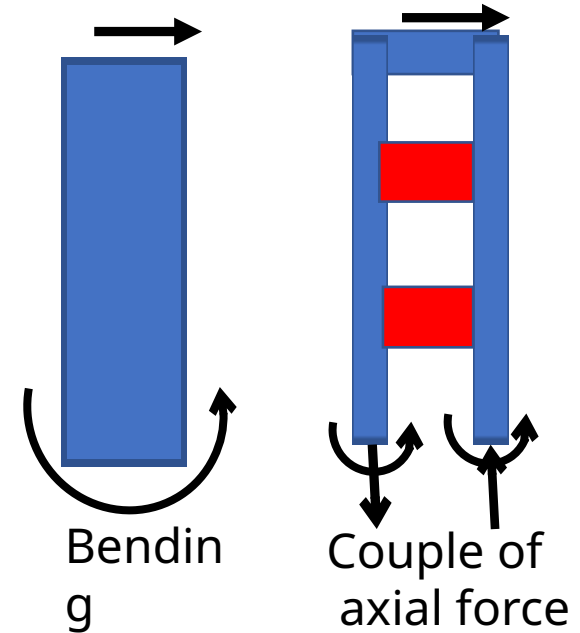
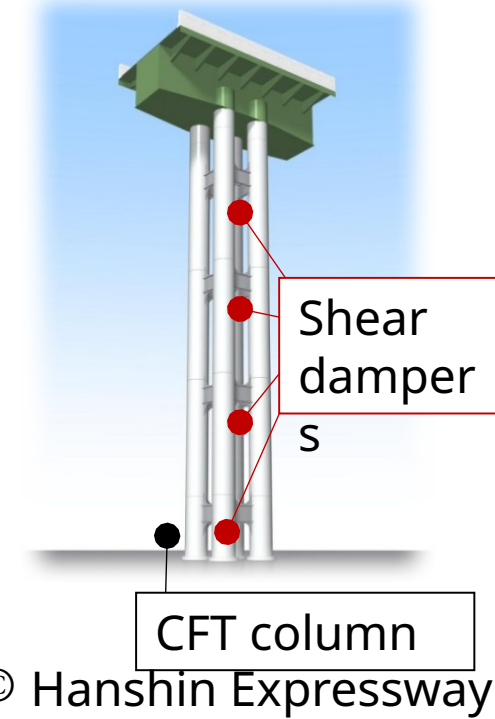
- Immediate restoration of serviceability
- Short term reparability
- Minimal reduction in load carrying capacity

capacity



For extreme earthquakes beyond L2 design earthquake

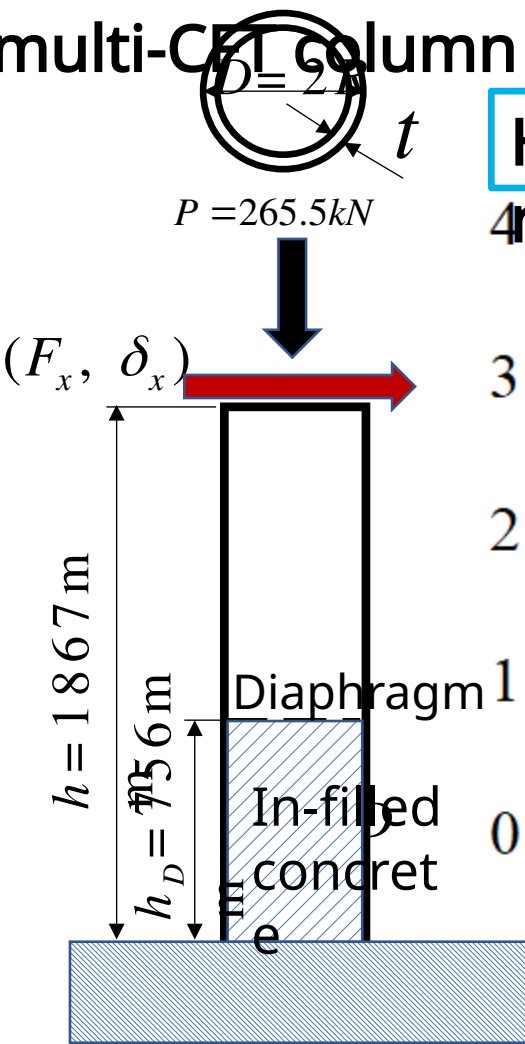
- CFT columns dissipate large part of seismic energy.
- Bending moment at pier base is resisted by the couple of axial force of the respective columns. Reduce the stress of steel pipes



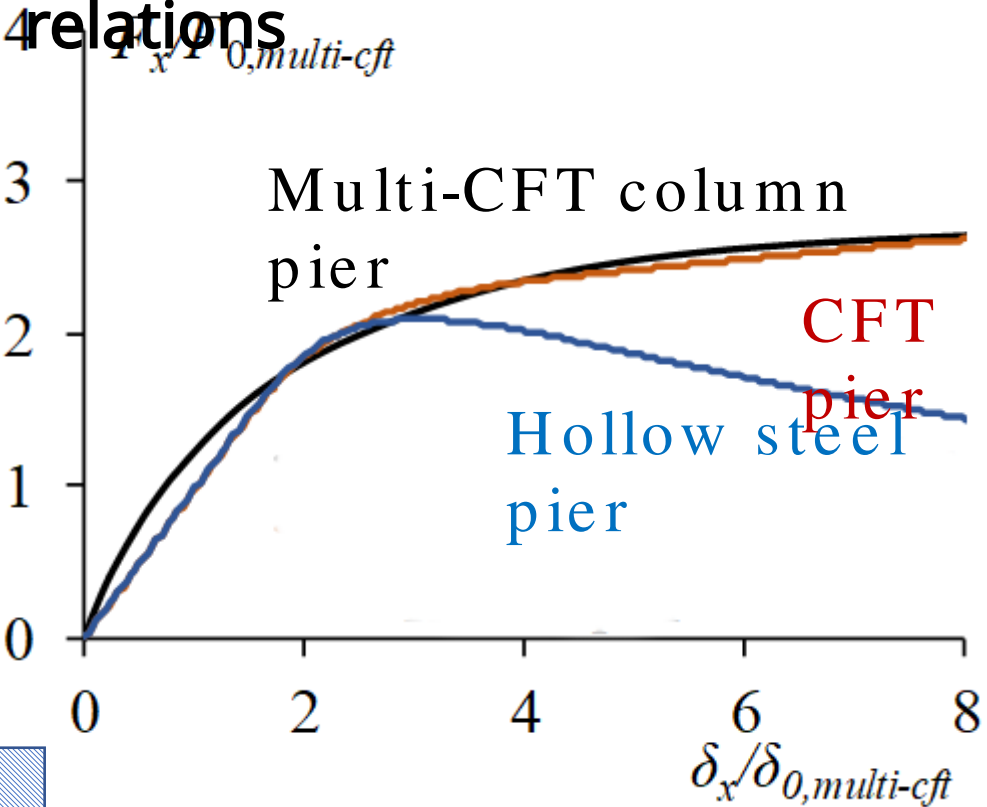
Comparison of multi-CFT column pier and CFT pier regarding seismic robustness

a. Equivalent CFT pier model to compare with multi-CFT column pier specimen

Equivalent CFT pier model was determined by push-over analysis such that the horizontal force-displacement relation of CFT pier model coincides with that of the multi-CFT column pier.



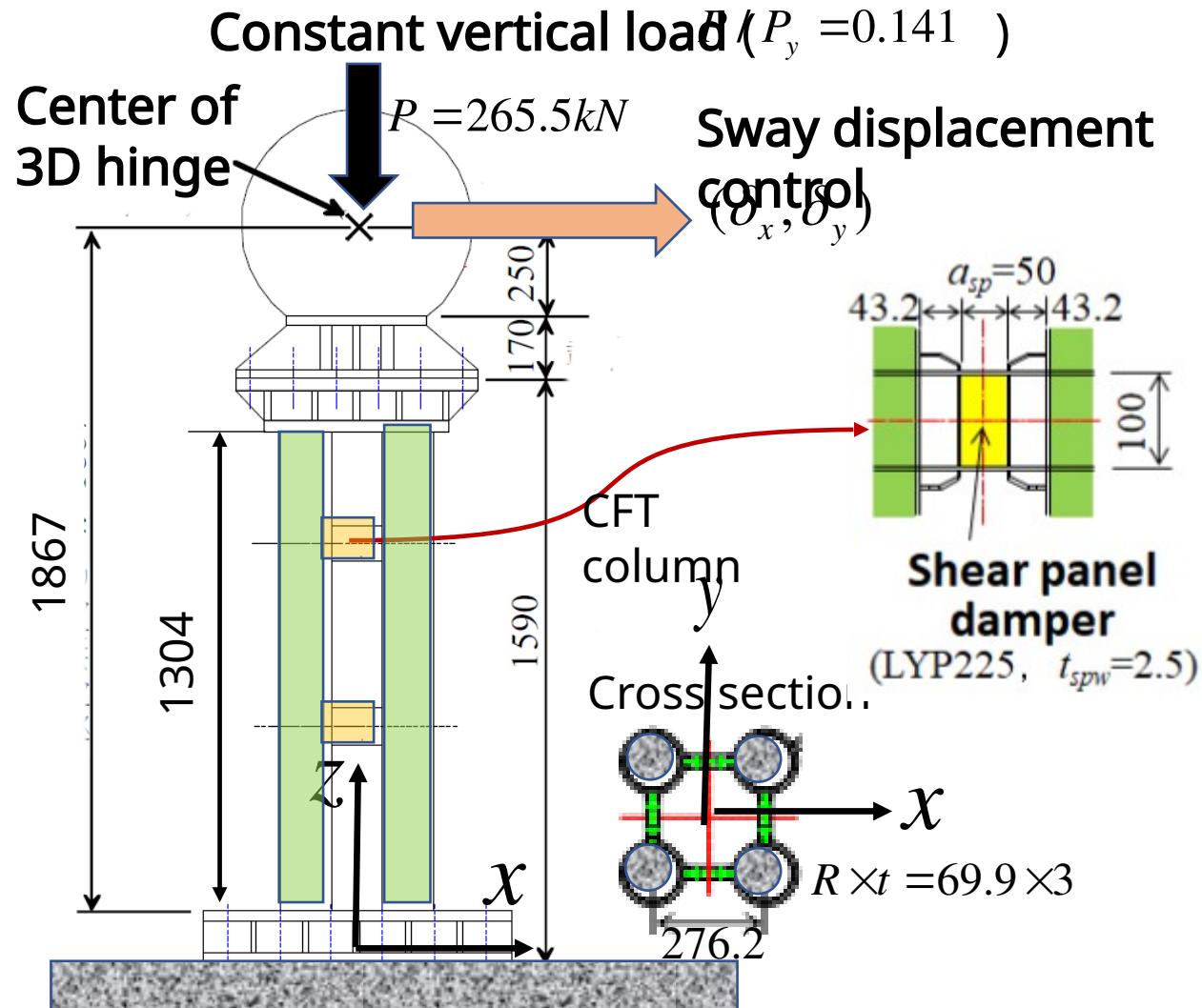
Horizontal force-displacement relations



	CFT pier	Hollow pier
R (mm)	182.5	182.5
t (mm)	5	5
R_t	0.12	0.12
$\sigma_{y,steel}$ (MPa)	396	396
σ_{ck} (MPa)	26	-

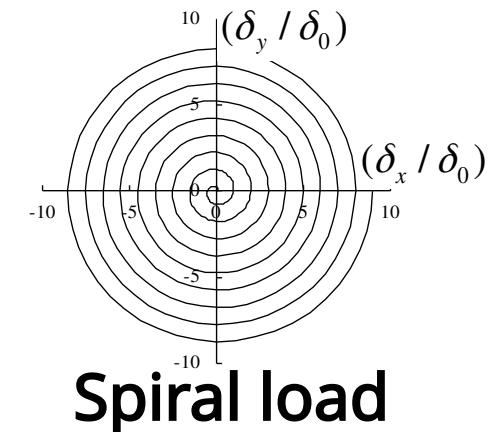
• **Multi-CFT column pier**
Horizontal yield force:
 $F_{0,multi-cft} = 63.1(\text{kN})$
Horizontal yield displacement:
 $\delta_{0,multi-cft} = 7.7(\text{mm})$

b. Static cyclic loading tests of multi-CFT column pier(1/10 scale)



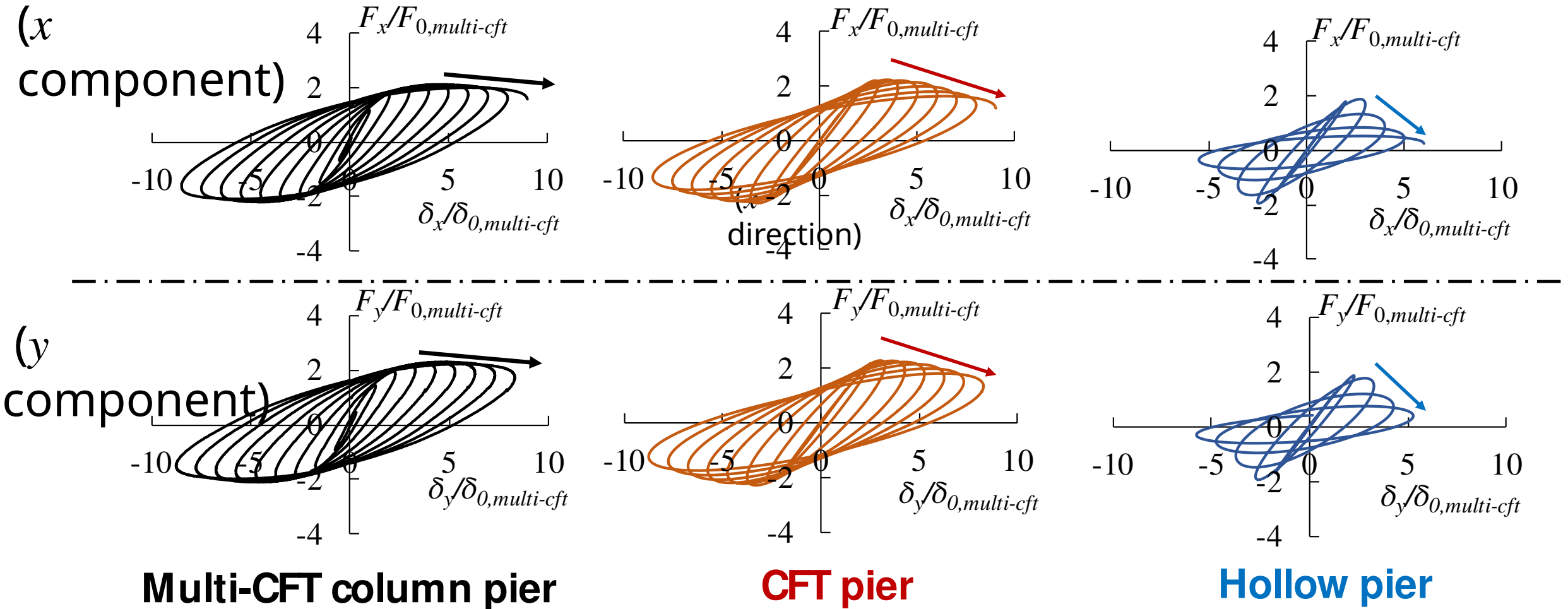
Multi-CFT columns pier consists of 4 CFT columns connected with 8 shear dampers .

The horizontal displacement pattern applied to the pier models is a displacement-controlled spiral load to examine the bi-directional behavior of the pier.



c. Behavior of piers under spiral load

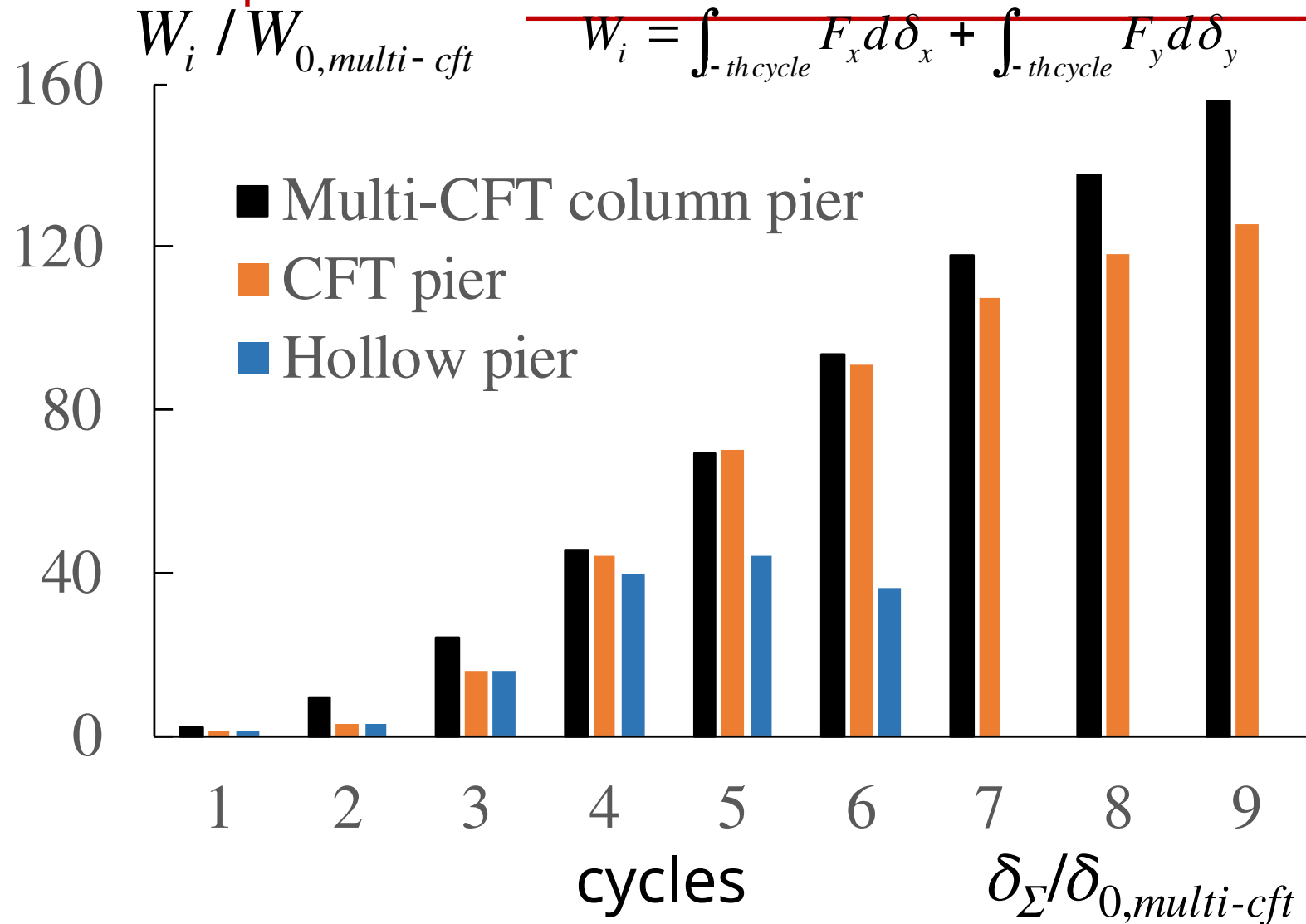
Horizontal force-displacement relations (x, y)



The load drop of the multi-CFT column pier in the post-peak range is further mitigated. Among the three types of piers, the multi-CFT column pier exhibits the most stable and largest post-peak energy dissipation capacity.

d. Energy dissipation of piers under spiral load

Dissipated energy for each load cycle

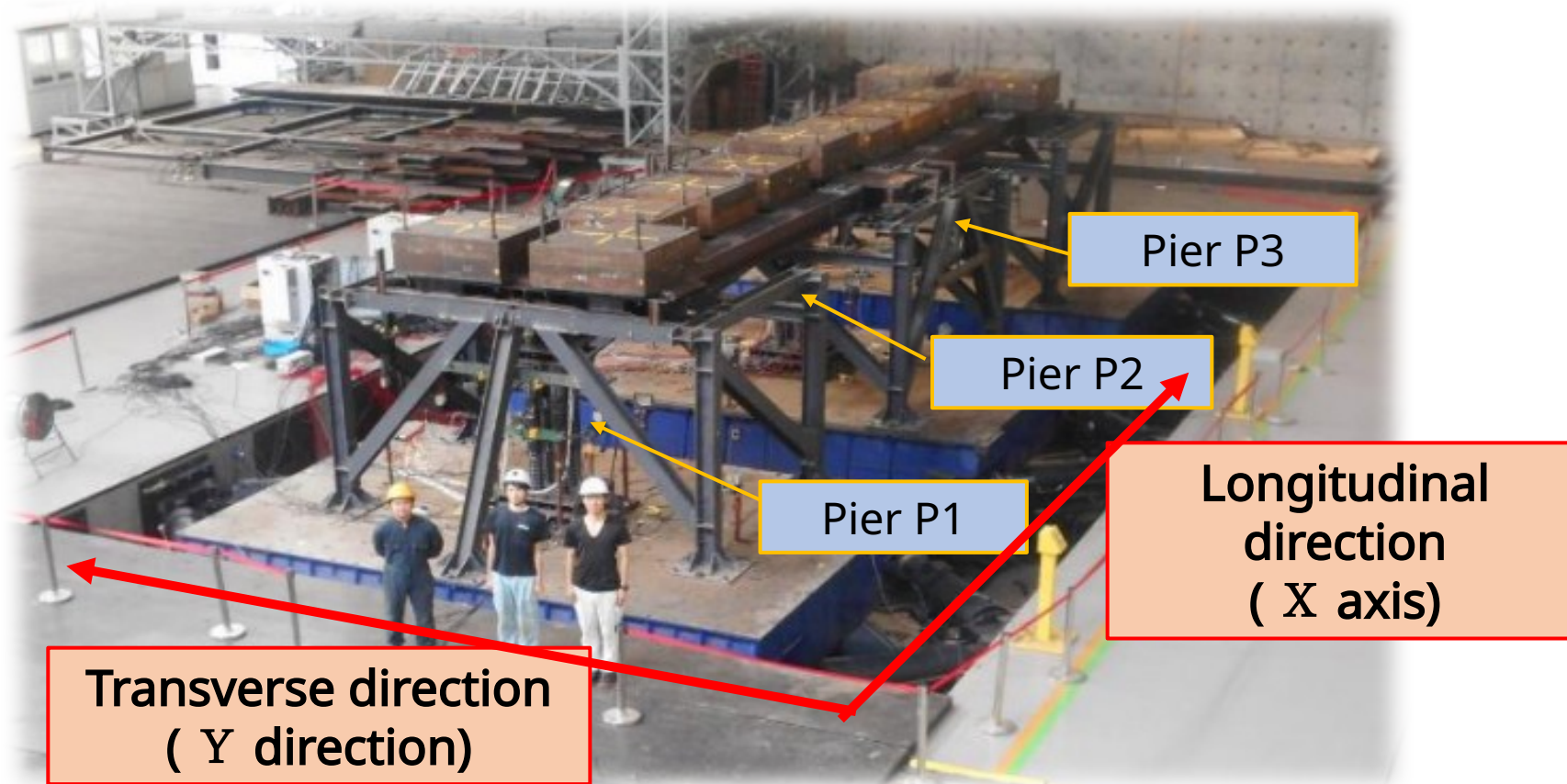


The multi-CFT column pier exhibits the most improved energy dissipation among the three-piers.

This tendency is very significant for large displacement cycles after the 7th cycle.

The Multi-CFT column pier is expected have an excellent robustness against extreme earthquakes.

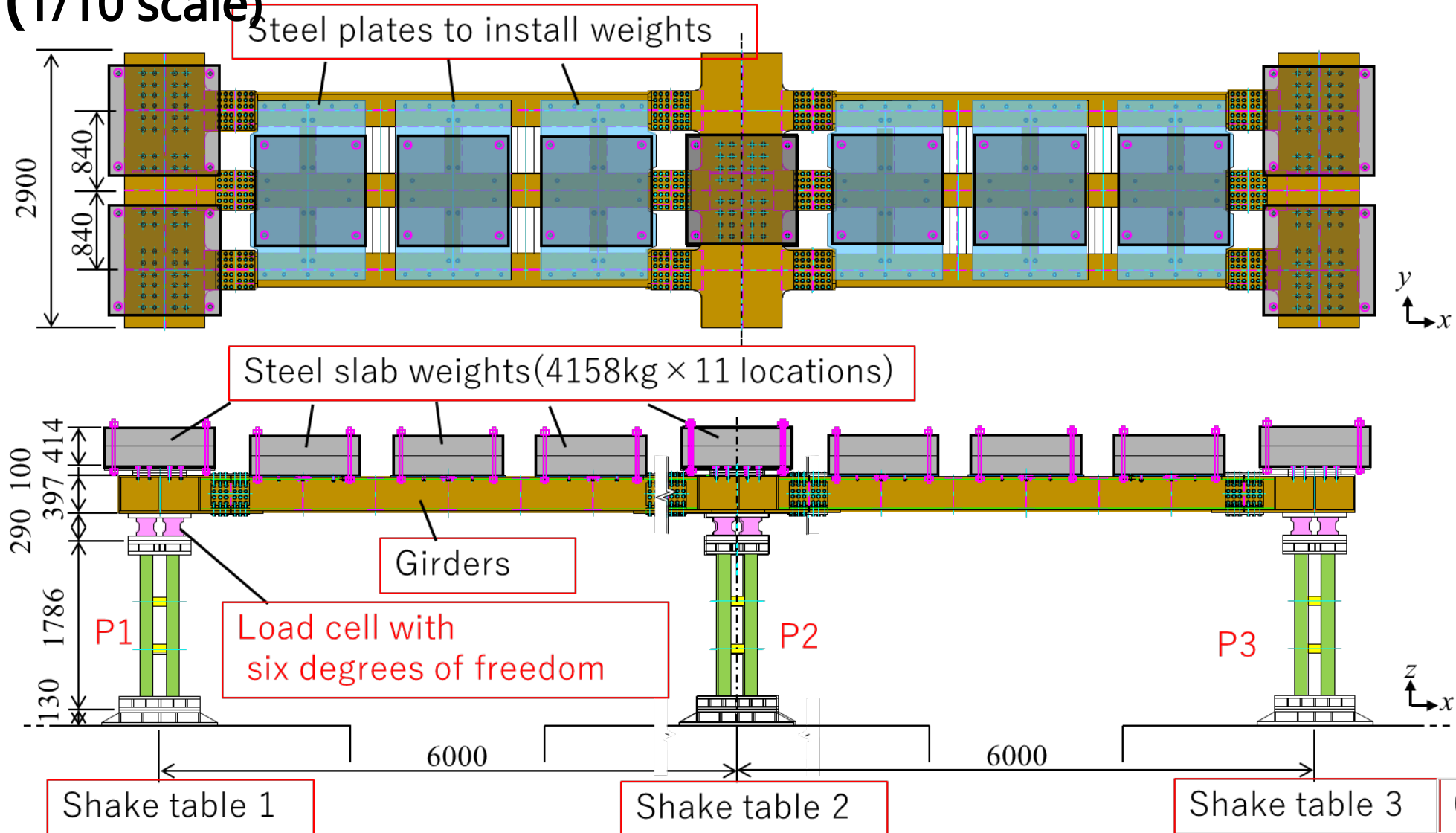
(2). Shake table test on multi-CFT column piers in continuous elevated-girder bridges under bi-directional seismic acceleration



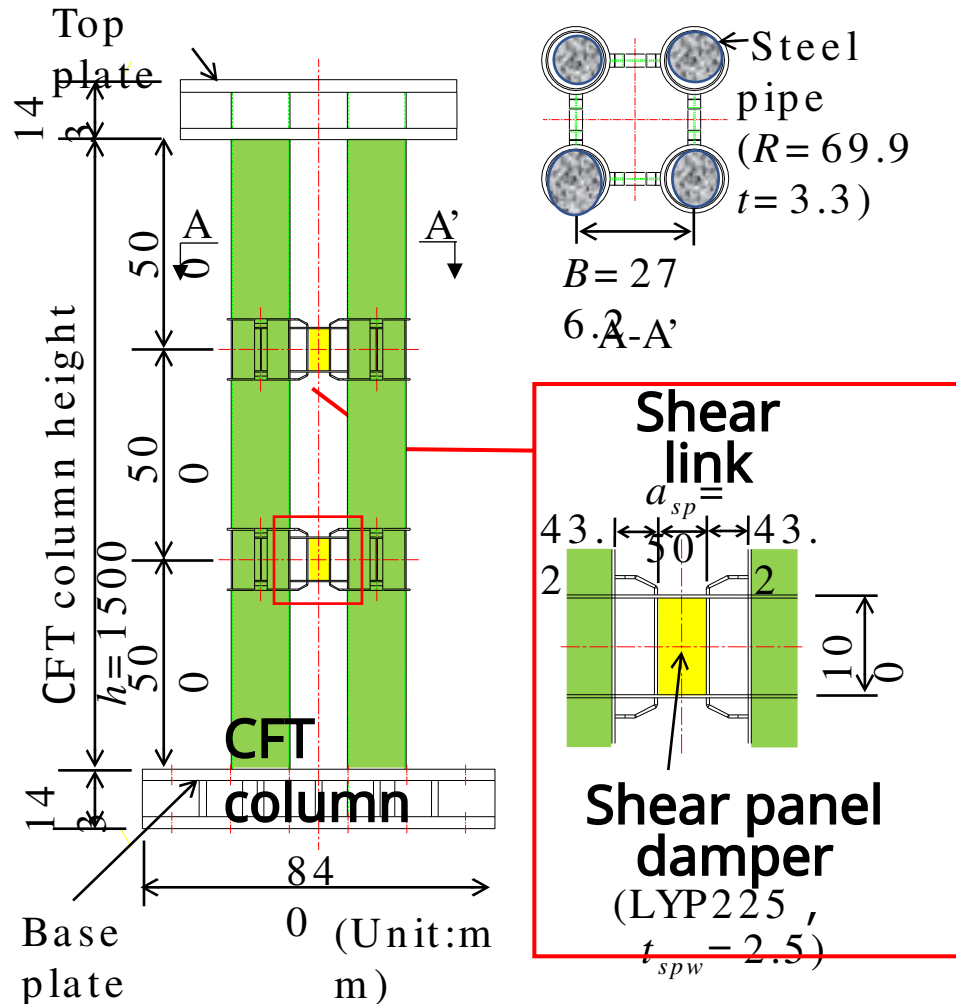
2-span continuous elevated-girder bridge model (1/10 scale)
on the shake table array of Tongji University (2018)

a. 2-span elevated-girder bridge model with multi-CFT column piers

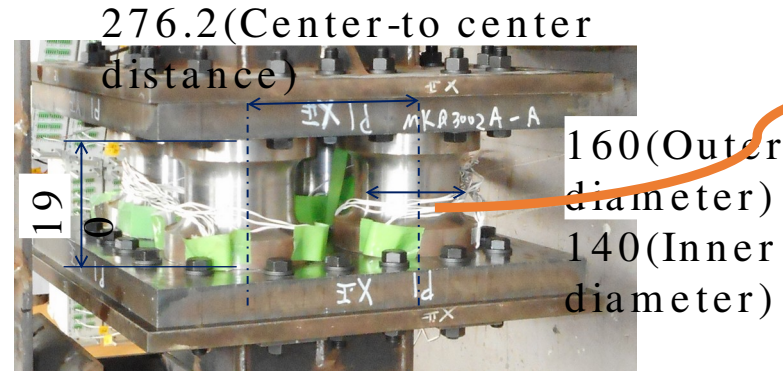
The pier top is rigidly connected to the girders
(1/10 scale)



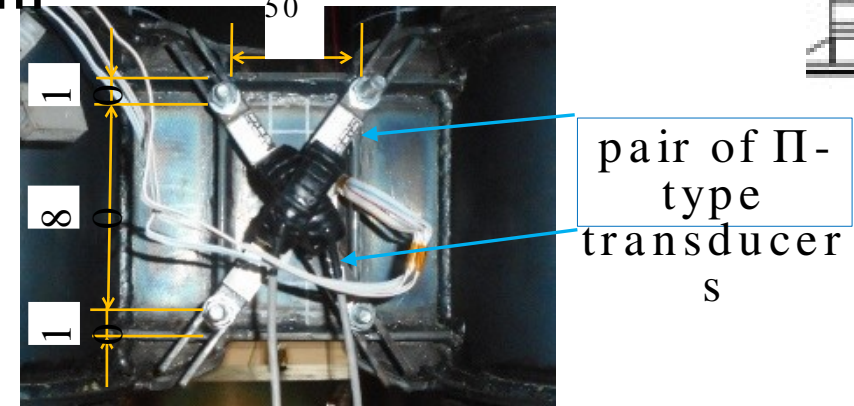
Multi-CFT column pier specimen and measurement



Multi-CFT column pier specimen



Load cell with six degrees of freedom



Transducers to measure deformation of shear dampers

b. In-put seismic waves used for shake table test

L2 design earthquake wave

< L2 design earthquake wave

L2 was determined based on JRT wave during the 1995 Kobe earthquake

- EW component of JRT → Longitudinal axis
- NS component of JRT → Transverse axis

Time axis is shortened by $\sqrt{s}$ from similarity law
: scale factor of the specimen.

< Beyond L2 wave >

2×L2

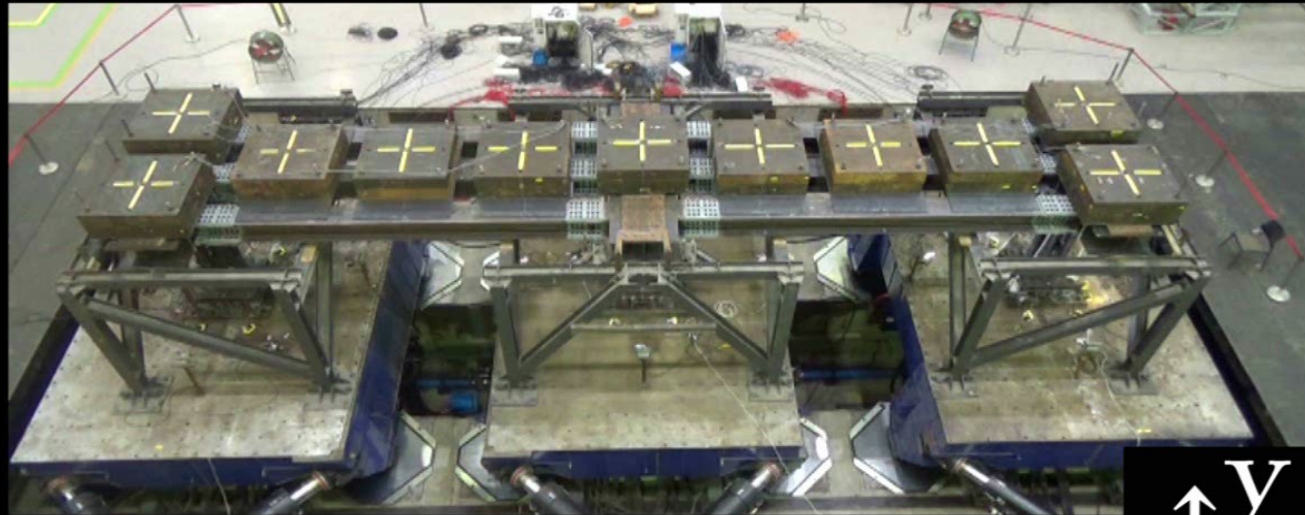
3×L2

4×L2

As the input earthquake waves beyond the L2 wave, three waves were created by magnifying the amplitude of the L2 design wave.

c. Shake table test on elevated-girder bridge model with multi-CFT column piers

Shake
table



y
x



y

(200%×L2)



x

Tongji University
(2018)

d. Response of shear panel dampers and CFT columns

L2 earthquake

L2×4 earthquake

L2

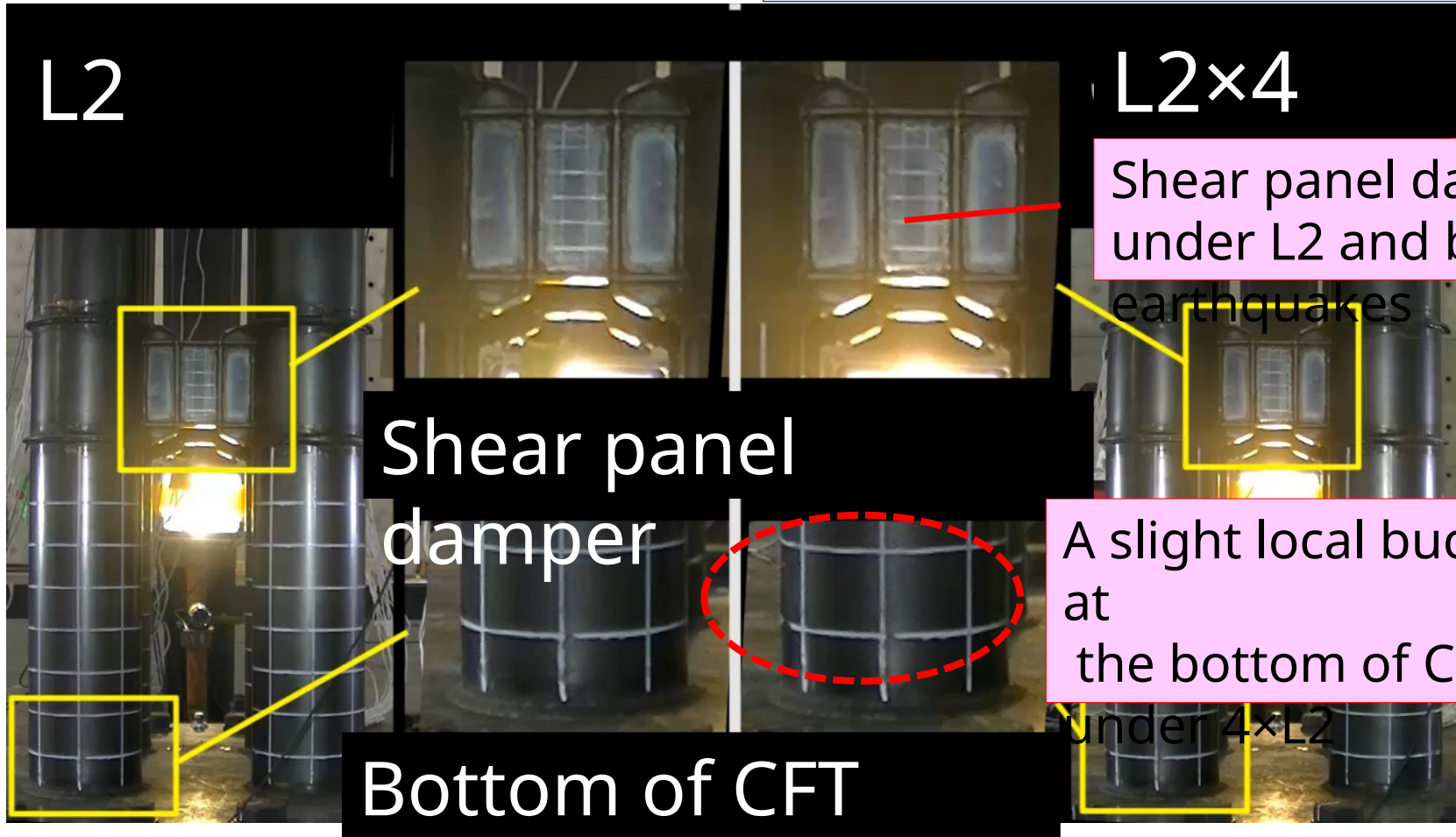
L2×4

Shear panel dampers work both under L2 and beyond L2 earthquakes

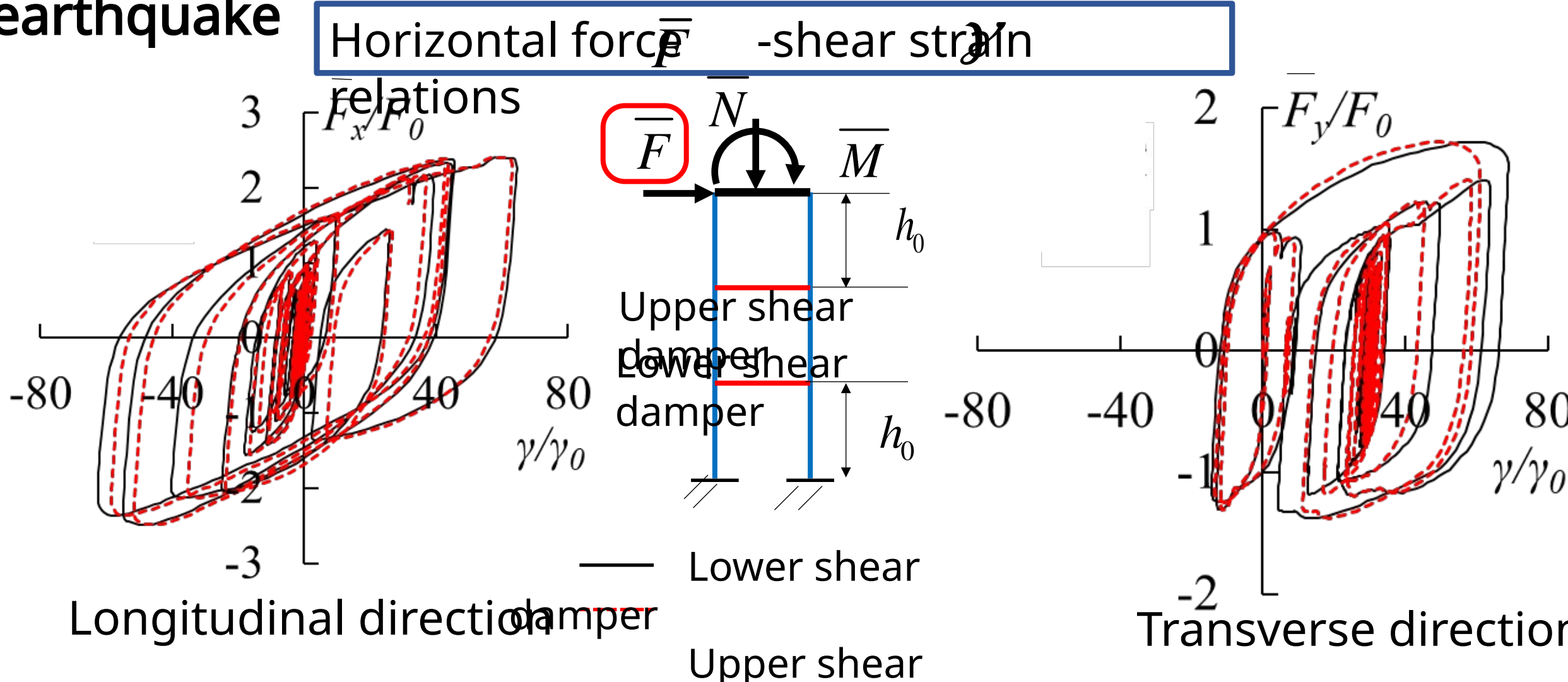
Shear panel damper

A slight local buckling occurs at the bottom of CFT column

Bottom of CFT

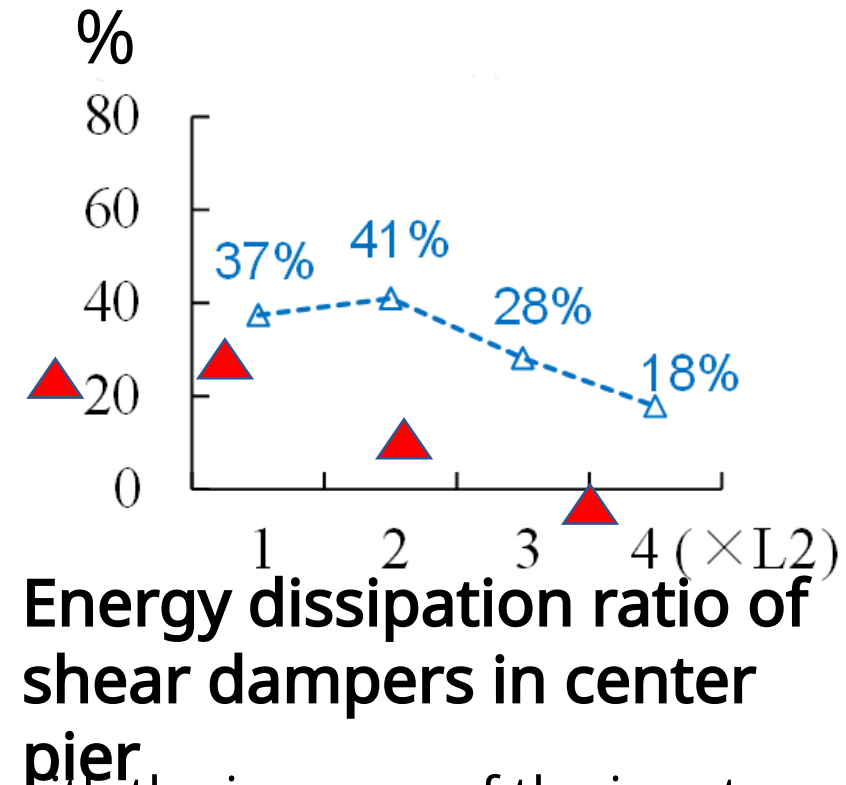
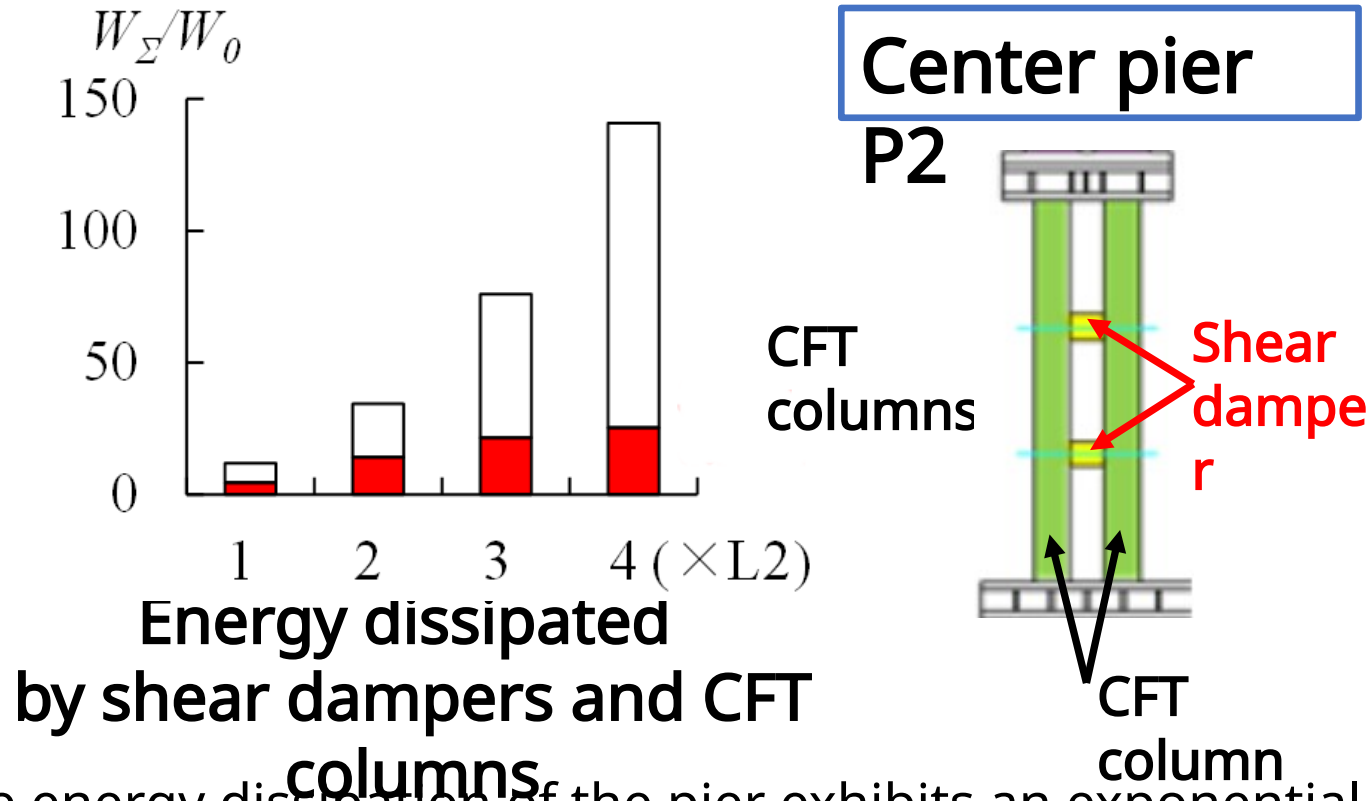


e. Hysteretic behavior of shear panel under 4×L2 earthquake



Both the upper- and lower shear dampers exhibit almost the similar stable response both in the longitudinal and transverse directions.

f. Energy dissipation characteristics of shear dampers and CFT columns in the multi-CFT column pier model

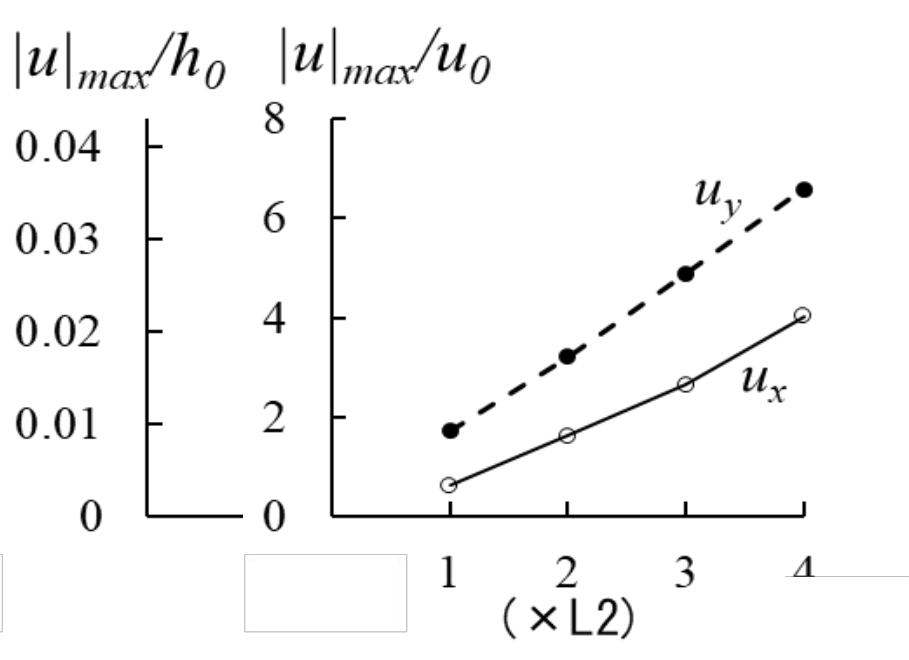


The energy dissipation of the pier exhibits an exponential growth with the increase of the input-wave amplitude beyond L2 design wave.

The exponential growth of the energy dissipation in the multi-CFT column pier is chiefly due to the rapid increase of the energy dissipation in the CFT columns.

The CFT columns in the multi-CFT column pier effectively dissipate seismic energy under the sever earthquakes beyond the L2 earthquake.

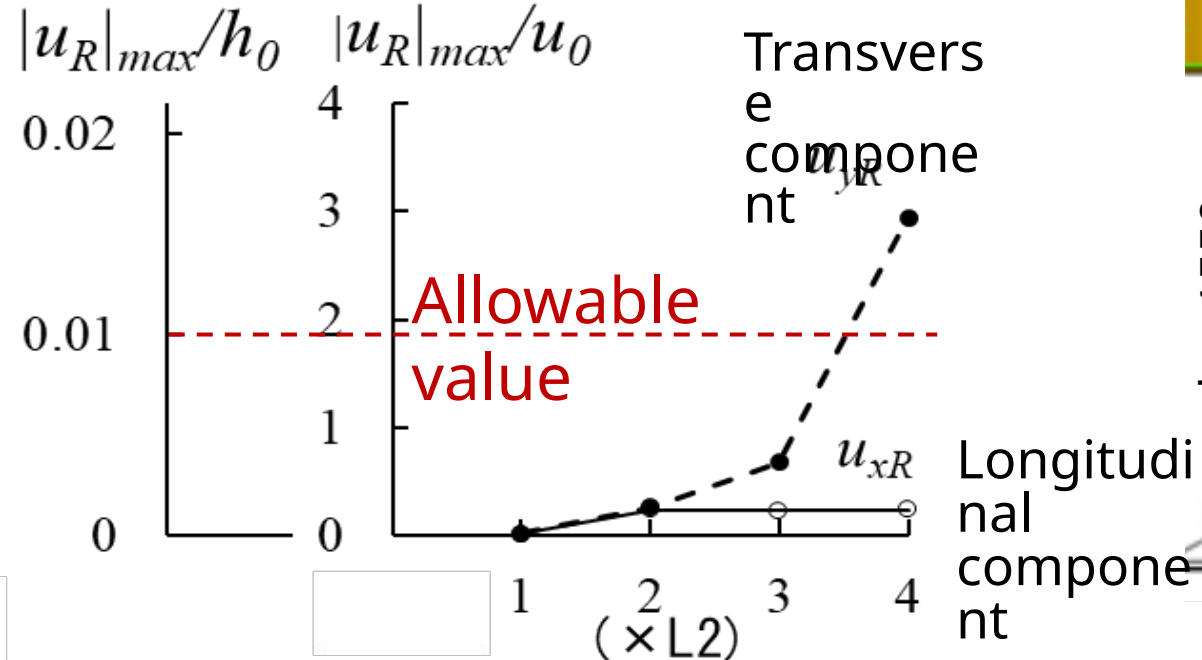
f. Maximum response sway displacement and residual sway displacement of the multi-CFT column pier model



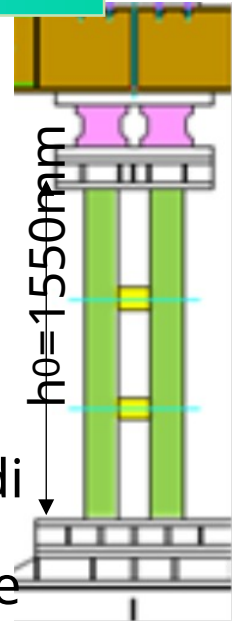
Maximum response sway displacement

The maximum response sway displacement increases and is proportional to the amplitude of the input waves.

The transverse residual sway displacement increases with the increase in the amplitude of the input waves. The residual sway displacement exceeds the allowable value when the amplitude of the L2 wave is magnified by 4 times. However, up to the triple amplitude of the L2 design wave, the residual displacement is sufficiently small.

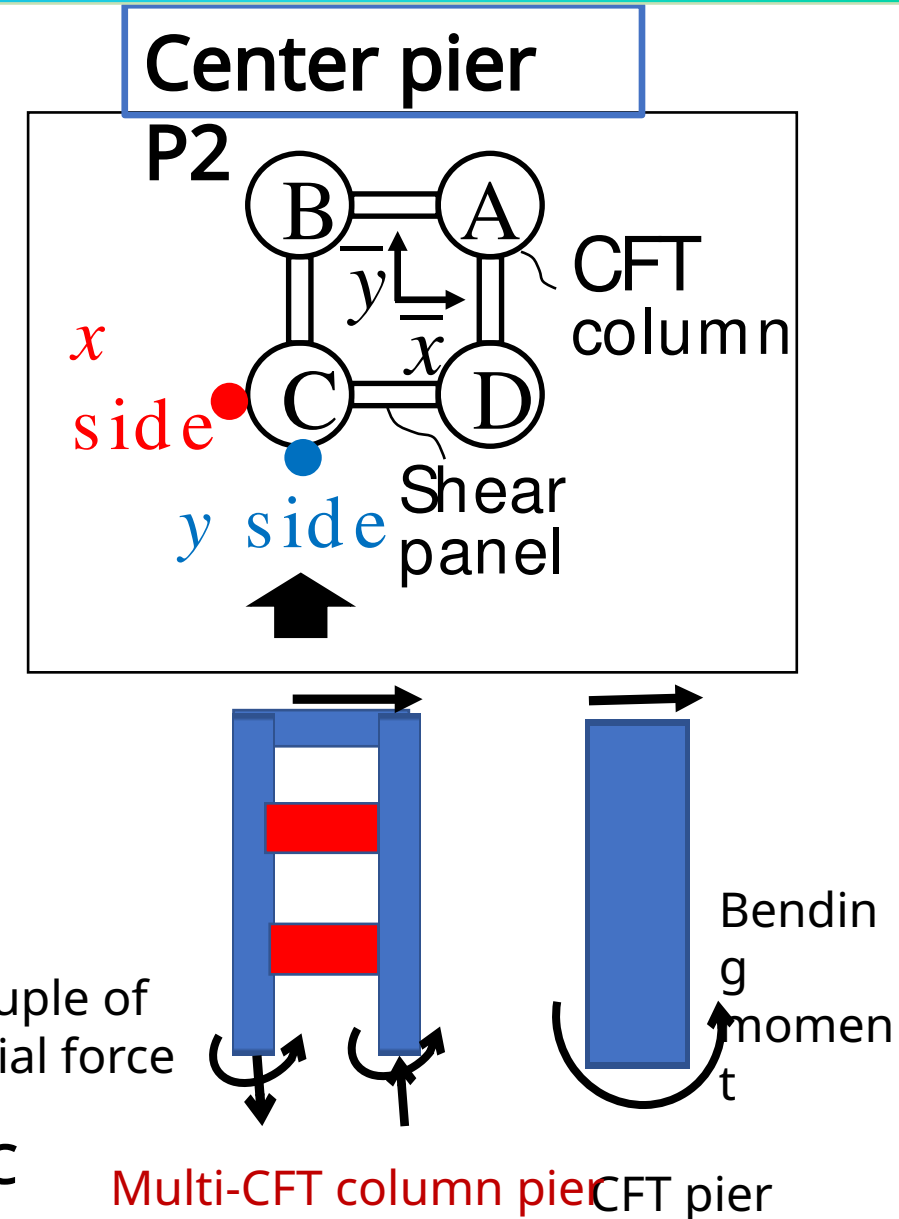
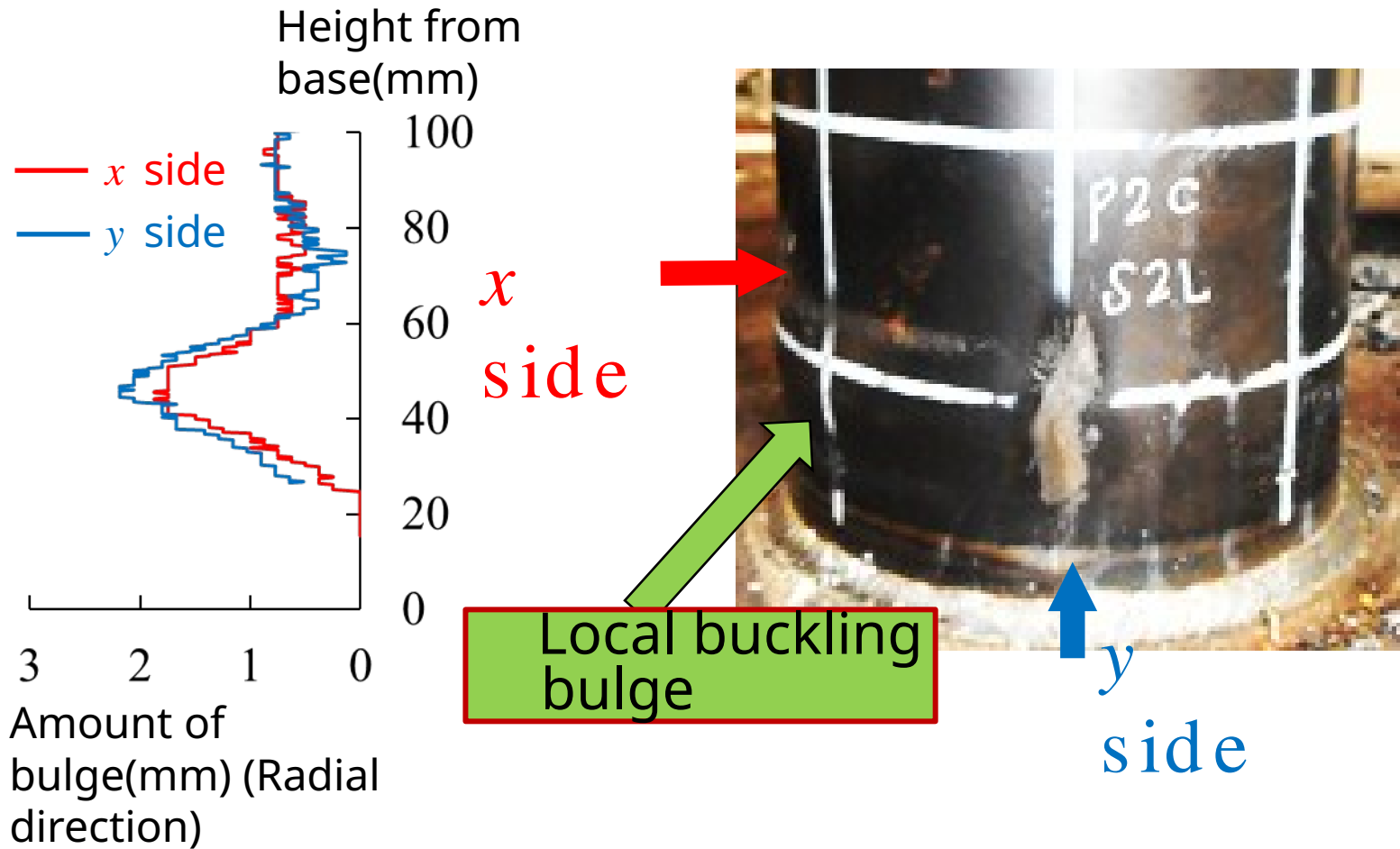


Residual sway displacement



The multi-CFT column piers in the continuous elevated girder bridge model exhibited some robust seismic

g. Damage of multi-CFT column pier after input of L2×4 wave



- Small local buckling deformation at the bottom of CFT column C
- No low-cycle fatigue cracks occurred.

Summary

- The concept of collapse control design is presented to prevent the disproportionate collapse of bridges under extreme earthquakes.
- As the collapse control design, the strategies to improve the robustness of thin-wall steel piers in elevated-girder continuous bridges are presented in the.
- The robustness of thin-walled hollow steel piers can be enhanced by filling their inside with low strength normal concrete and forming CFT piers.
- The deficiency of CFT piers is that the CFT piers may collapse in a brittle manner by low cycle fatigue.
- The robustness of the newly developed multi-CFT column piers with shear dampers was examined by shake table test and analysis.
- The multi-CFT column piers with shear dampers proved to be more robust than the current CFT piers, because the multi-CFT column piers have higher resistance to local buckling deformation and low cycle fatigue.

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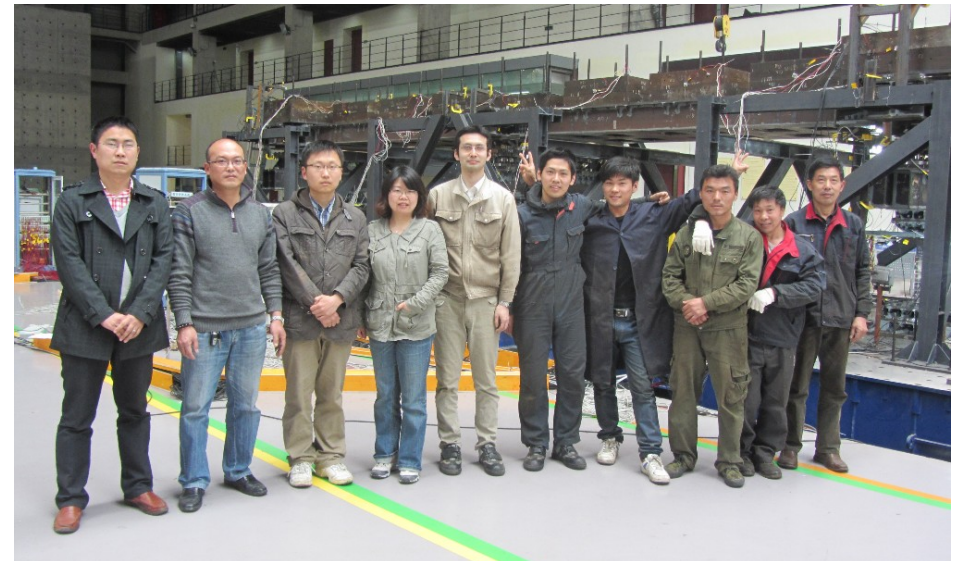
Elevated-girder bridge shake table tests

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Thank you for you attention