RESEARCH ON RESPONSE CONTROL SYSTEM FOR STEEL TOWER STRUCTURES

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SUMMARY:
This paper presents a simple design method for tuned dynamic mass system, which is a kind of tuned mass damper system having a heavy auxiliary mass consisting of the "Tuned Dynamic Mass System (TDMS)". The tuned dynamic mass system is a structural system constructed by a mechanism which has a rotating inertial mass called the dynamic mass (D.M.) and a viscous damper in parallel and its connecting spring in series. This paper shows an example of the response control design which applied TDMS to steel tower structure using the tuning design method. This paper is organized as follows.

First, it describes the tuned dynamic mass system is effective by the eigenvalue analysis of a three-dimensional frame model. Finally, it is shown that this system is effective using a shaking table and a 1/20 scale model of steel tower structure.

Keywords: Steel tower structure, Tuned dynamic mass system, Invariant point theory

1. INTRODUCTION

In the design of steel tower structures such as steel-tower-supported chimneys at thermal power plants and steel-tower-supported exhaust stacks at nuclear power plants, the design load is generally wind load and little emphasis has been placed on seismic loads. As a result, input earthquake motions assumed for an earthquake-resistant design have been underestimated. Based on the knowledge obtained in the 1995 Southern Hyogo Prefecture Earthquake, the 2007 Chuetsu offshore Earthquake and the 2011 off the Pacific coast of Tohoku Earthquake, however, assuming large input earthquake motions for an earthquake-resistant design has recently been gaining momentum. Thus, seismic loads have greater influence and existing steel tower structures have been found to have few earthquake resistance margins and now require seismic retrofitting of structures quickly.

Conceivable seismic improvement methods are a "seismic strengthening" type with the cross section of members increased or a "seismic response control" type with a viscous damper attached to the structure. Either one or both of the two types are adopted according to the input earthquake motion assumed. This study proposes a method of improving steel-tower-supported structures using pantograph-type response control devices [1] as an optimum "seismic response control" type seismic improvement method. Conventional seismic response control dampers are intended for shear-type structures and therefore are not so effective for bending-type structures with predominant longitudinal deformation of lower columns that can hardly be expected to provide high seismic response control effects. The response control system proposed in this study works following the axial deformation of columns and accordingly can provide high response control effects in bending-type structures including steel tower structures.

The proposed method of improvement for seismic response control adopts a response control system that combines a viscous damper and a dynamic mass damper. The system is designed to use the mass control by a dynamic mass damper. For the specific design method, the concept of a "tuned dynamic mass system based on the auxiliary stiffness factor" proposed by Ishimaru et al. [2],[3],[4] was employed. The method is adopted for response control design using a tuned system based on the invariant point theory for optimizing a tuned mass damper.

This paper describes the effectiveness of the retrofitted response control system for steel tower structures. The components of the paper are described below.

First, the response control performance of the response control system proposed in this study is described based on the results of complex eigenvalue analysis using a three-dimensional frame model. Next, it is shown that the system is effective not only for response in a single horizontal direction but also for response in two horizontal directions using the amplification of relative displacement response and through response analysis. Finally, an outline and the results of shaking table tests conducted to verify the performance of the response control system are provided. Thus, the usefulness of the method for improving steel-tower-supported structures for response control that is proposed in this study is shown.

2. OUTLINE OF THE STUDY STEEL TOWER STRUCTURE AND METHOD OF IMPROVEMENT FOR RESPONSE CONTROL

2.1. Outline of the study steel tower structure

Analysis was conducted for an ordinary square steel-tower-supported steel-pipe structure. The aerial model is 100.0 m high and a side of the foundation is 18.0 m long. Horizontal loads such as seismic and wind loads are carried mainly by the steel tower section surrounding the structure and external forces by main column members and diagonal members. A three-dimensional frame model was developed for analysis as shown in Figure 2. The results of complex eigenvalue analysis in the x direction and at an angle of 45 degrees are listed in Table 1. The primary viscous damping coefficient is assumed to be an initial stiffness proportional damping and set at 2%. The study steel tower structure is a regular structure and uses only steel pipes as members. Thus, neither the natural period nor viscous damping coefficient vary according to the direction but are the same either in the x direction or at an angle of 45 degrees.

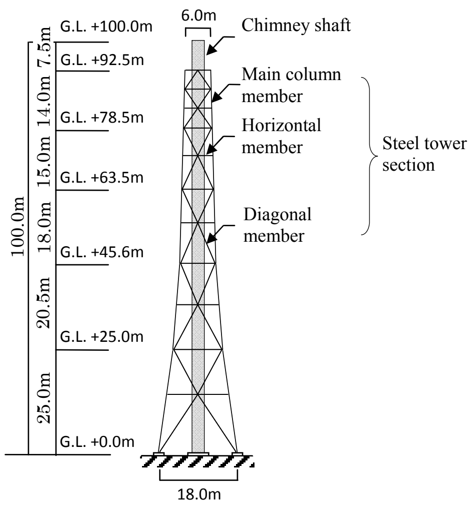


Figure 1 Study steel tower structure

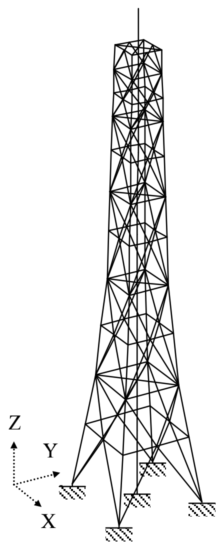


Figure 2 Three-dimensional frame model

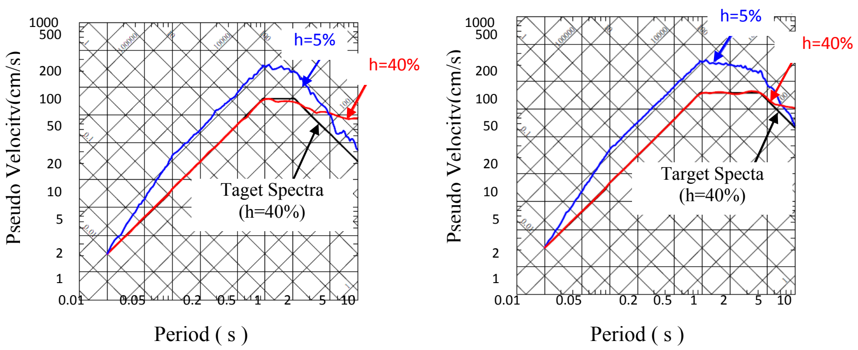
Table 1 Results of complex eigenvalue analysis in the x direction and at an angle of 45 degrees (no response is controlled)

Mode	Period T(s)	Viscous damping factor h^*
1st	1.194	0.020
2nd	0.406	0.059
3rd	0.201	0.119

*Stiffness proportional damping of 2% for the first mode is included

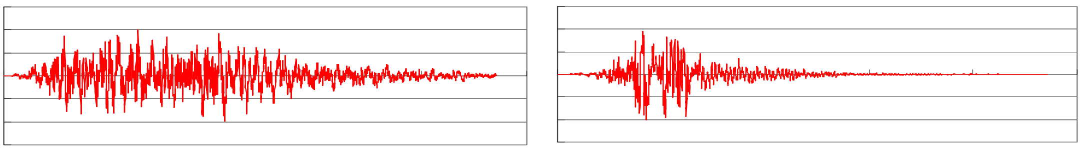
2.2 Input earthquake motions for study

As the input earthquake motions for study, "earthquake motion 1" with an acceleration of 10.0 m/s^2, a velocity of 1.5 m/s and a displacement of 0.5 m, and "earthquake motion 2" with an acceleration of 10.0 m/s^2, a velocity of 1.5 m/s and a displacement of 1.0 m were specified as the target response spectra with a viscous damping ratio of 40%. Simulated artificial earthquake motions with the respective characteristics were developed. As the characteristics of the phase of simulated artificial earthquake motion, the random number phase that is uniform at 0 to 2π and the phase of an actual seismic wave using the phase of JMA Kobe 1995 NS record were specified. The target response spectra and the response spectra of the simulated artificial earthquake motions produced are shown in Figures 3 a), b). The acceleration time histories for the artificial earthquake motions for study are shown in Figures 4 a), b). Simulated artificial earthquake motions are generally produced so as to be compatible with the target response spectrum with a viscous damping ratio of 5%. In order to prevent the response spectrum with a viscous damping ratio of 5% from fluctuating, two types of simulated artificial earthquake motions were produced compatible with a damping of 5% and another of 40% [5]. The relationship between 5% and 40% damping ratio is based on the method of Noda et al. (2002) [6].



a).Earthquake motion 1 b).Earthquake motion 2

Figure 3 Target response spectra and artificial earthquake motions



a).Earthquake motion 1-1,2-1 (random number phase) b). Earthquake motion 1-2,2-2 (JMA Kobe phase)

Figure 4 Acceleration time histories for artificial earthquake motions

2.3. Criterion of design for retrofitting of seismic response control

The criterion of design for retrofitting of seismic response control is listed in Table 2. The criterion of design is common to four types of earthquake motions for study. Ordinary "seismic response control" type improvement methods modify the existing frame by such means as the removal of horizontal or diagonal members to enhance the effect of response control by the devices installed. Modifying the existing frame enhances earthquake resistance but at the same time is likely to deteriorate wind resistance. The improvement method for response control proposed in this paper makes no modifications to the existing frame from a viewpoint of cost and the ease of construction.

Table 2 Criterion of design for retrofitting of seismic response control

Performance of viscous damping factor	First-mode viscous damping ratio h_1 should be 15.0% or higher.
Criterion of design	Inter-story drift is approximately 1/100

3. OUTLINE OF PANTOGRAPH-TYPE RESPONSE CONTROL DEVICES

Figure 5 is an enlarged view of the first mode in the lower levels of the steel tower at input of earthquake motions in the *x* direction. The values of participation functions at nodal point *a* of Figure 5 in the *x*, *y* and *z* directions and the axial direction of the main column member are shown in Table 3.

The values show that the steel tower structure, which is a bending-type structure, is subject to vertical (bending) deformation larger than horizontal (shear) deformation. It is accordingly more effective to use response control devices working against bending deformation instead of conventional devices working against shear deformation.

The results listed in Table 3 indicate that installing the response control device so as to work against the deformation in the axial direction of the main column member is most effective. The pantograph-type response control device in Figure 6 has been developed to consider for this property. There are two characteristic. One is that the pantograph system, which is set to "pair" the toggle system, is to improve the efficiency of the damper by amplified the deformation of the damper. Another is using the D.M. damper in addition to the viscous damper. D.M. generates the inertial force by the relative acceleration of inter-damper, its force works in the opposite direction of the force by the deformation of the member. In other words, the inertial force works in the opposite direction of the axial force working on the main member. As a whole, it is possible to reduce the reaction force.

Figure 7 shows the dimensions of the pantograph-type response control device installed in the steel tower structure, and inter-damper displacement amplification ratio β for the axial deformation of the main column member. Table 4 lists the axial deformation of the main column member at the lowest level at input of earthquake motion. The axial deformation in the parenthesis shows the inter-damper displacement where amplification ratio β was taken into consideration.

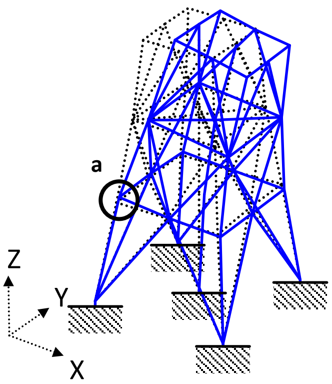


Figure 5 First mode at the lower levels of steel tower

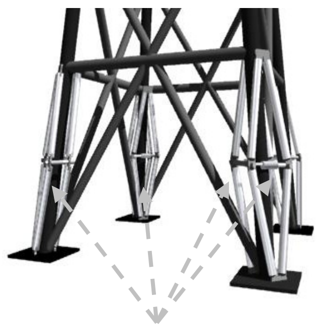


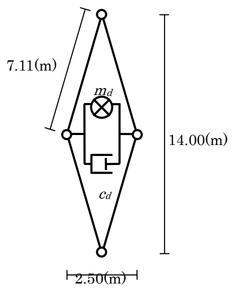
Figure 6 Pantograph-type tuned dynamic mass damper

Table 3 Values of participation functions at nodal point a (Figure 5)

Horizontal direction X direction	Horizontal direction Y direction	Vertical direction Z direction	Composite value The axial direction of main column member
0.0118	0.0000	0.0250	0.0261

Table 4 Axial deformation of the lowest level main column member

Target response spectra (m/s^2-m/s-m)	Phase characteristics of artificial earthquake motion	The axial deformation (mm)	Inter-damper deformation (mm)
10.0-1.5-0.5	1-1 : random number phase	21.0	118.9
10.0-1.5-0.5	1-2 : JMA KOBE phase	19.9	112.6
10.0-1.5-1.0	2-1 : random number phase	20.3	114.9
10.0-1.5-1.0	2-2 : JMA KOBE phase	18.4	104.1



Amplification ratio β of response control system
β=5.66
Deformation of response control system by 1.0 mm in the axial direction of the main column member means inter-damper deformation by 5.66 mm

○ : pin joint

Figure 7 Dimensions of pantograph-type response control system

4. CHECK OF THE RESPONSE CONTROL EFFECT USING A THREE-DIMENSIONAL FRAME MODEL

4.1 Response control design using a tuned dynamic mass system

The response control capacity of the response control system is examined here using a three-dimensional frame model shown in Figure 8. Optimum design is carried out for the response control system using equations (1) through (3) below presented in existing studies [1],[2]. The design is developed through complex eigenvalue analysis. The first-mode response of the structure is controlled.

$$\kappa_k = \left(\frac{T_0}{T_\infty}\right)^2 - 1 \tag{1}$$

$$T_\infty = \sqrt{T_{0,1}T_{0,2}} \tag{2}$$

$$h_1 = h_2 \approx (0.5 \sim 0.6)\sqrt{\frac{\kappa_k}{2+\kappa_k}} \tag{3}$$

κ_k : auxiliary stiffness factor

T_0 : natural period of the mode of the structure to be controlled where the damping coefficient of viscous damper C_d = 0.0

T_∞ : natural period of the mode of the structure to be controlled where the damping coefficient of viscous damper C_d = ∞

$T_{0,1}$: natural period of the mode of the structure to be controlled where a dynamic mass damper is additionally installed

$T_{0,2}$: natural period of dynamic mass mode (mode generated by the addition of the dynamic mass damper) where a dynamic mass damper is additionally installed

h_1 : optimum damping ratio of the mode of the structure to be controlled

h_2 : optimum damping ratio of the dynamic mass mode

The optimum design procedure by the design method is described below.
First, auxiliary stiffness factor κ_k is calculated using equation (1) from the first natural period T_0 where no response is controlled and the second natural period T_∞ where the damping coefficient of viscous damper C_d is set at ∞. Then, damping coefficient C_d is set to be 0.0 kNsec/m and dynamic mass is gradually increased. The optimum dynamic mass is determined when both first natural period $T_{0,1}$ and the second natural period $T_{0,2}$ satisfy the relationship expressed by equation (2). Finally, damping coefficient C_d is increased while dynamic mass is kept at an optimum level, and an optimum damping coefficient C_d that satisfies the relationship represented by equation (3) is determined. After several times of simple parametric studies as described above, the optimal quantities are determined for the response control device.

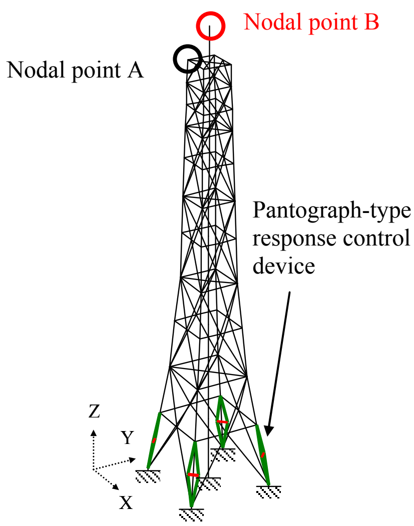


Figure 8 Analysis model

Table 5 Optimum quantities per response control device

m_d (ton)	c_d (kN·s/m)
250	950

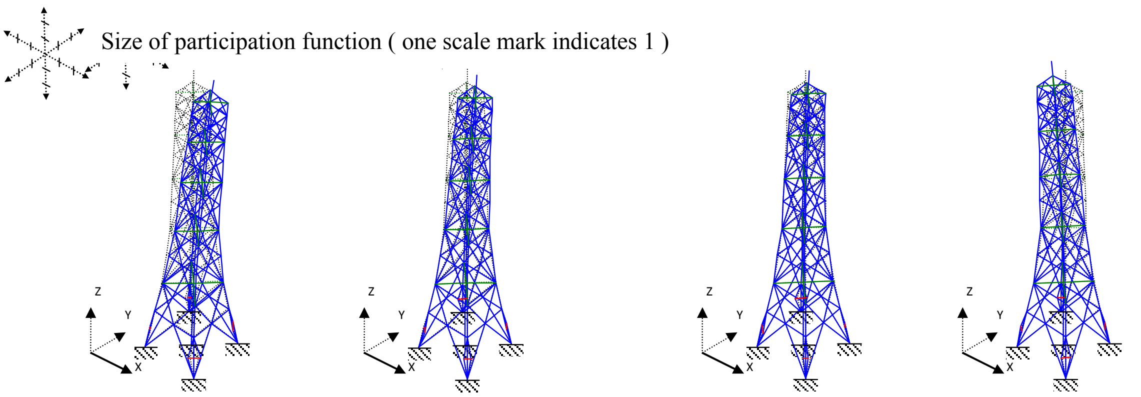
Table 6 Results of complex eigenvalue analysis in the x direction and at an angle of 45 degrees (optimum design)

Mode	Natural period T(s)	Viscous damping factor h*
1	1.285	0.185
2	0.924	0.187
3	0.393	0.062

* Stiff proportional damping of 2% for the first mode is included

The quantities determined in the above steps of optimum design are listed in Table 5. Table 6 lists the results of complex eigenvalue analysis by optimum design. Figure 9 presents images of participation functions. Figure 9 shows the results for the first and second modes, and for the real and imaginary parts.

The results show that the performance of viscous damping ratio h_1 of the first mode not falling below 15.0% is satisfied. In the design method, response is controlled by tuning the mode generated as a result of the addition of dynamic mass (dynamic mass mode below) to the mode unique to the structure. That is, the mode shown as the second mode at the time of optimum design is a dynamic mode not existent in the structure.



a) Real part b) Imaginary part a) Real part b) Imaginary part

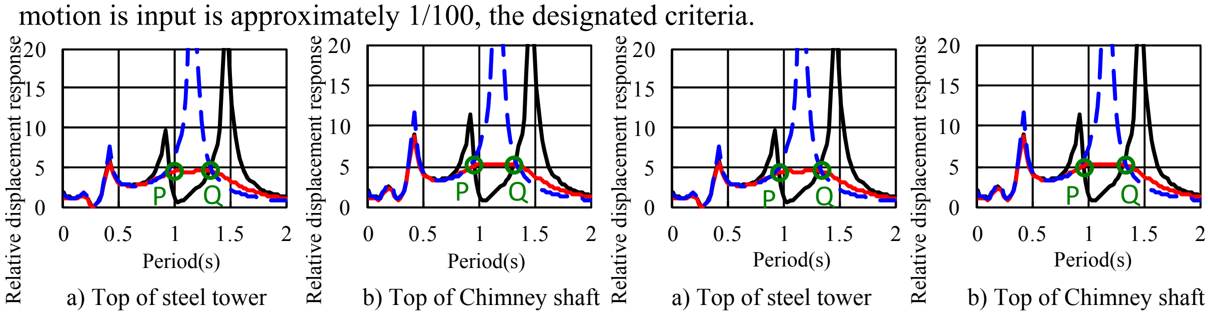
(i) First mode (ii) Second mode

Figure 9 Images of participation functions at the input in the x direction (optimum design)

4.2 Results of analysis

Figure 10 shows resonance curve of relative displacement response at the top of the steel tower (nodal point A in Figure 8) and at the top of the chimney shaft (nodal point B in Figure 8). Figure 11 shows the results of analysis of elastic response to each earthquake motion for study in cases where no response is controlled and where optimum design is carried out. Both Figures 10 and 11 show the (i) response in the x direction at the time of earthquake motion input in the same direction and (ii) response at an angle of 45 degrees at the time of earthquake motion input at the same angle for the steel tower and the chimney shaft.

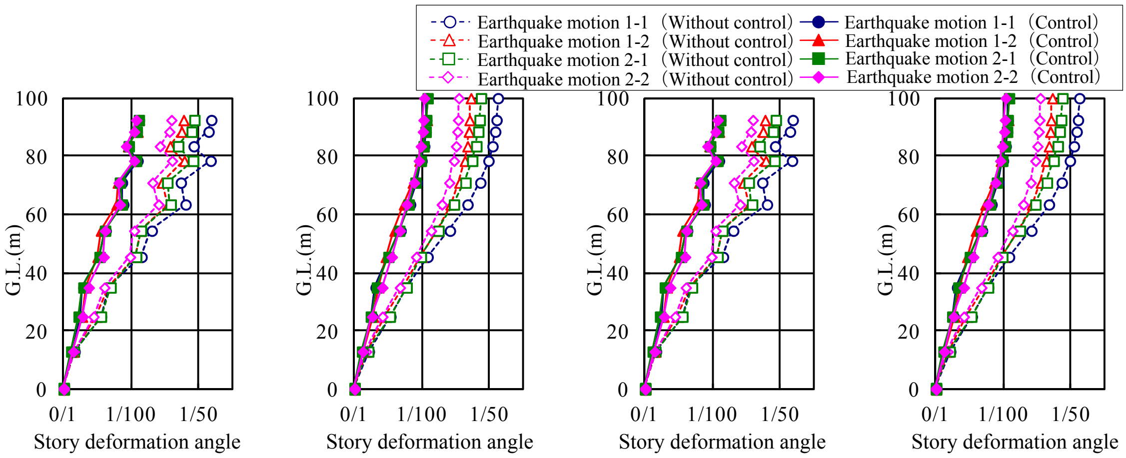
The results of analysis of the curves representing the amplification of relative displacement response (Figure 10) show that the optimum tuning condition in the invariant point theory (the height of invariant point P is identical to that of point Q) is satisfied and that the quantity of the designed dynamic mass is optimum. It is also evident that the optimum damping condition (response amplification curve takes the maximum value at invariant points P and Q at the optimum damping) is satisfied and that the damping coefficient C_d of the designed viscous damper has an optimum value. This indicates that optimum design equations (1) through (3), which are composed based on the relationship among the natural periods of structures, are applicable also to three-dimensional frame models. The results shown in Figure 11 indicate that the elastic response when the study earthquake motion is input is approximately 1/100, the designated criteria.



a) Top of steel tower (nodal point A) b) Top of Chimney shaft (nodal point B) a) Top of steel tower (nodal point A) b) Top of Chimney shaft (nodal point B)

(i) Response in the x direction when sine wave is input in the same direction (ii) Response at an angle of 45 degrees when sine wave is input at the same angle

Figure 10 Resonance curve of relative response ratio



a) Steel tower b) Chimney Shaft a) Steel tower b) Chimney Shaft

(i) Response in the x direction when earthquake motion is input in the same direction

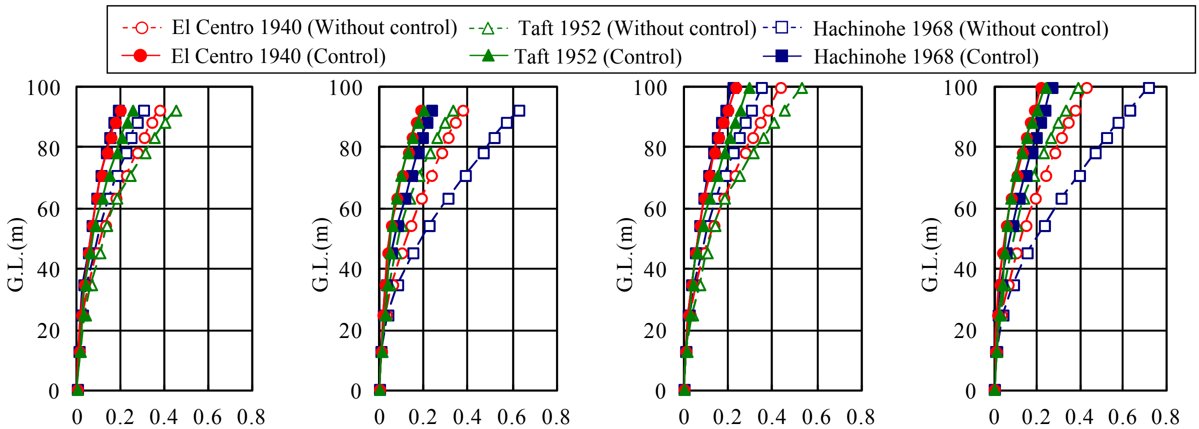
(ii) Response at an angle of 45 degrees when earthquake motion is input at the same angle

Figure 11 Maximum response (in the case of without response control or with response control)

Thus, it is evident that the response control system adequately controls the response of the structure. Both Figures 10 and 11 indicate that the results of analysis are identical either in the case of (i) input in the *x* direction or in the case of (ii) input at an angle of 45 degrees. This suggests that the pantograph-type response control device is effective for controlling response not only in one horizontal direction but also in two horizontal directions, and offers identical response control effects in various directions.

Then verification is made whether response can be controlled or not in the case where different earthquake motions are input in two horizontal directions. As the input earthquake motions, 1940 El Centro, 1952 Taft and 1968 Hachinohe records are used. In all input earthquake motions, the maximum velocity is normalized to be 0.50 m/s. The NS and EW components of each record are input in the *x* and *y* directions, respectively. It is evident that the response either in the *x* or *y* direction when response is controlled is half the response in the case where no response is controlled regardless of the input earthquake motion (Figure 12). Input of earthquake motions in two horizontal directions causes the response of the structure to exhibit a path represented by the orbit of deformation for the top of the chimney shaft (nodal point B in Figure 8) (Figure 13). The orbit of deformation also shows that response was held to half either in the *x* or in the *y* direction.

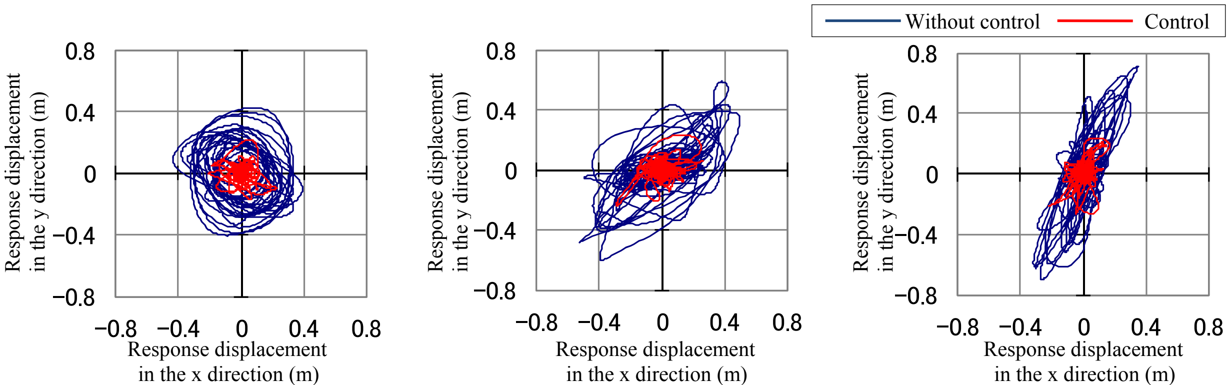
As a result of the above discussions, the response control system is highly effective also for controlling response to input earthquake motions in two horizontal directions. In the proposed response control system, multiple response control devices are installed only at the lower levels of the steel tower. The proposed method is an excellent method of improvement for response control in various terms such as cost performance, ease of construction and maintainability.



a) Response in the x direction b) Response in the y direction a) Response in the x direction b) Response in the y direction

(i) Steel tower (ii) Chimney shaft

Figure 12 Maximum response (in the case of no response control or optimum design)



a) El Centro 1940 b) Taft 1952 c) Hachinohe 1968

Figure 13 Orbit for response displacement (at the top of the chimney shaft)

5. PERFORMANCE VERIFICATION TESTS FOR A MODEL OF IMPROVEMENT FOR RESPONSE CONTROL

5.1 Outlines of specimen and test

In order to verify the dynamic characteristics of the response control system, shaking-table tests were conducted using earthquake motions in one and two directions. A two-layer specimen was used in the tests that simulated the lower levels of the steel tower structure (original structure below) discussed in this paper (Figure 14). The large specimen has a side of the foundation of 3.5 m and a height of 5.0 m. Three hundred and sixty degree free rotating connectors were installed at the foundation of the shaft. Coil springs were used in the main column member on the first layer. Thus, shaking induced rocking around the foundation of the shaft. The deformation of coil springs during shaking simulated the deformation in the axial direction at the lower levels of the original structure.

Table7 lists the parameters of the response control device used. Table 8 shows the natural periods and viscous damping ratios in the x direction and at 45 degrees in the case of no response control or optimum design. It is evident that the first natural period in the case of no response control was 1.201 seconds, nearly identical to 1.194 seconds, the first natural period of the original structure shown in Table 1. Thus, the specimen accurately simulated the original structure. Table 9 lists the details of the steel members, coil springs and weights used in the specimen.

In the tests, sine and seismic waves were used for shaking. In the tests using sine waves, the amplification of displacement response (ratio of response to the displacement of the shaking table at each layer) was calculated and its agreement with the curve representing the amplification of relative displacement response obtained in analysis was verified. Sine waves were input either (i) in one horizontal direction (x direction) or (ii) in two horizontal directions (at an angle of 45 degrees).

In the shaking-table tests using seismic waves, two records, 1968 Hachinohe and 1995 JMA Kobe, were used. The waves were set at 10% of 1968 Hachinohe and 5% of 1995 JMA Kobe. The NS and EW components of each record were input in the x and y directions, respectively for shaking. The shaking-table tests proved that the method of improvement for response control using pantograph-type response control devices was effective for response not only in one horizontal direction but also in two horizontal directions.

Table 7 Specifications for response control device

m_d (ton)	c_d (kN・s/m)
1.0	4.4

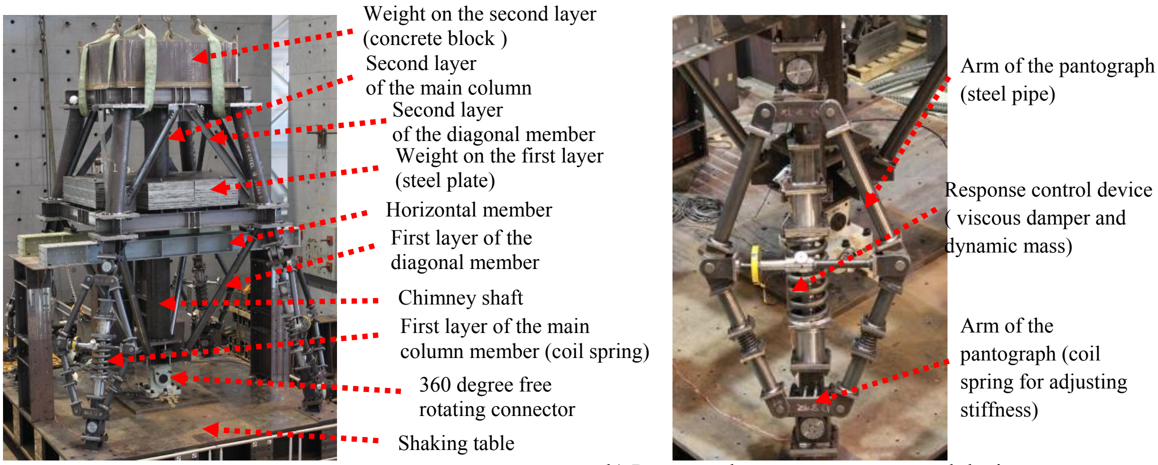
Table 8 Result of complex eigenvalue analysis (in the case with or without response control)

Without control

Mode	natural period T(s)
1	1.201
2	0.032

With control

Mode	natural period T(s)	Viscous damping factor h
1	1.292	0.181
2	0.855	0.180



a) Steel structure test model b) Pantograph-type response control device

Figure 14 Outline of the specimen

Table 9 List of members of the specimen

Steel members

Position	Cross section	Dimensions(mm)
Chimney Shaft	Steel pipe	φ318.5 - t 9.0
Second layer of the main column	Steel pipe	φ216.3 - t 8.0
First layer of the diagonal member	Steel pipe	φ101.6 - t 5.0
Second layer of the diagonal member	Gauge channel	100 - 50 - 5.0 - 7.5
Horizontal member	H section	200 - 100 - 5.5 - 8.0
Arm of the pantograph	Steel pipe	φ76.3 - *t* 4.0

Coil spring

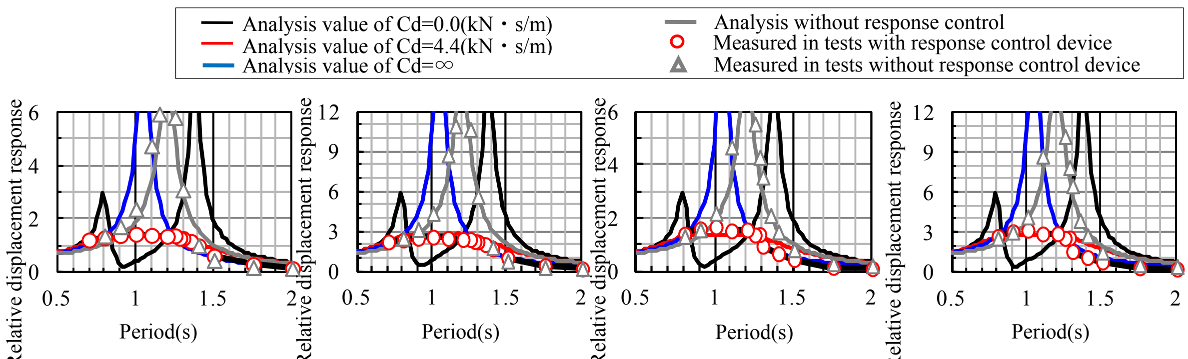
Position	Axial stiffness(kN/m)
First layer of the main column member	520.0
Arm of the pantograph	90.0

Weight

Story	Mass(ton)
2	7.0
1	11.2

5.2 Test results

Figure 15 shows the amplifications of relative displacement response obtained in analysis and the amplifications of displacement response measured in tests. Shaking by sine waves (i) resulted in the amplification of response in the *x* direction (response in the *x* direction/displacement of the shaking table in the *x* direction on each layer). Shaking by sine waves (ii) resulted in the amplification of response at an angle of 45 degrees (response displacement at 45 degrees/displacement of the shaking table at 45 degrees on each layer). The results shown in Figure 15 confirm that the response amplifications obtained in the tests are in good agreement with the curves representing optimum amplifications of relative displacement response (represented by black solid lines in the figures) regardless of whether sine waves (i) or (ii) is used for shaking. It is therefore evident that having the device work against the axial deformation of the main column member is expected to control the response of bending-type structures.



a) On the first layer b) On the second layer a) On the first layer b) On the second layer

(i) In the x direction (ii) At an angle of 45 degrees

Figure 15 Resonance curve of relative displacement response

It was also verified that the measurements taken in the tests using sine waves (i) and (ii) are identical to each other. It is thus evident that the method of improvement for response control using pantograph-type response control devices is equally effective in two horizontal directions.

Figure 16 shows maximum responses obtained in analysis and tests in the cases with and without response control where 1968 Hachinohe (10% of original wave) and 1995 JMA Kobe (5% of original wave) records of earthquake motions were used. The figure confirms that response was controlled more in the case with response control than in the case without response control either in the *x* or *y* direction.

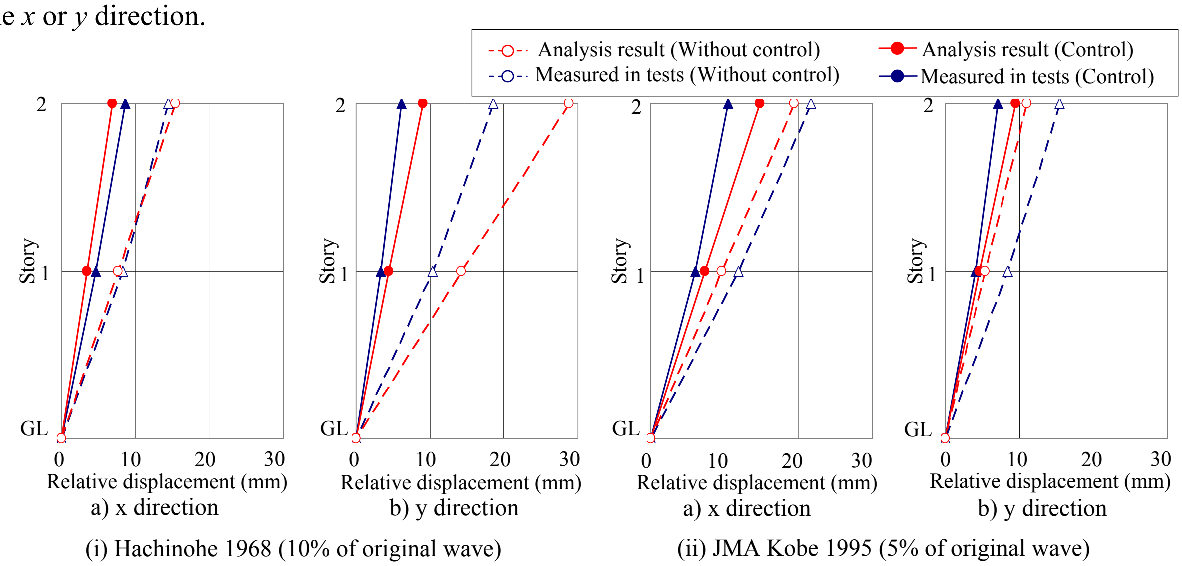


Figure 16 Maximum response (in the x or y direction)

6. CONCLUSION

As a result of analysis using a three-dimensional frame model, it was illustrated that the method proposed in this study for improving steel tower structures for response control is highly effective for controlling the seismic response of structures. It was shown that adopting the method satisfies performance requirements using a few response control devices installed at the lower levels of the steel tower without modifying the existing frame. Thus, it was presented that using the method is extremely effective in such terms as cost performance, ease of construction and maintainability. Performance verification tests were also conducted using a specimen that simulated the lower levels of the study steel tower structure. The results show that the response control system is highly effective for controlling response not only in one horizontal direction but also in two horizontal directions.

Based on the above, it was verified not only by analysis but also by shaking-table tests that improvement using pantograph-type response control devices is highly effective for improving steel tower structures for response control.

REFERENCES

1) Ishimaru,S. and others. (2010). Basic studies on tuned dynamic mass systems with twin toggle mechanism (Parts 1 through 4). *Proceedings of Conference of Architectural Institute of Japan (AIJ)*, pp. 477-484.

2) Ishimaru, S., Mikami, J., Hata, I. and Furuhashi, T. (2010). Simple method of designing tuned dynamic mass systems. *Journal of Structural and Construction Engineering of AIJ*, Vol. 75, No. 652.

3) Ishimaru, S., Hata, I., Mikami, J. and Kimiduka, M. (2010). Simple method of designing tuned dynamic mass systems using auxiliary stiffness factor. *Journal of Structural and Construction Engineering of AIJ*, Vol. 75, No. 654.

4) Ishimaru, S., Hata, I., Furuhashi, T., Kimiduka, M., Mikami J. and others. (2010). Simple method of designing tuned dynamic mass systems through complex eigenvalue analysis (Parts 1 through 6). *Proceedings of Conference of Architectural Institute of Japan (AIJ)*, pp. 465-476.

5) Ishimaru, S., Hata, I., Mashimo, M., Ogihara, M. and others. (2010). A study on the method of improving steel tower structures for response control (Parts 1 through 5). *Proceedings of Conference of Architectural Institute of Japan (AIJ)*, pp. 611-620.

6) Ishimaru, S. (2008). Seismic Design Method - Introduction to Dynamic Design. *Kenchikugijutsu*