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Response Control Systems by Tuned Dynamic Mass System for a 200-meter-tall tower-supported steel stack structure

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Abstract

In the wake of major earthquakes such as the 1995 Hyogo-ken Nanbu Earthquake, the design of towering structures like tower-supported steel stacks of thermal power plants has become dominated more by earthquake loads than wind loads. There is a growing need, therefore, for seismic reinforcement. This paper introduces a case of seismic renovation of a 200-meter-tall tower-supported steel stack structure.

The requirements of this work were as follows:
(1) Construct without putting the stack out of service.
(2) Attach seismic response control systems to the lower part of the steel tower if possible.
(3) Achieve seismic performance within the elastic limit against assumed ground motion (peak ground acceleration: about 300 cm/s^2, peak ground velocity: about 70 cm/s).

As a result, the target performances has been achieved by attaching a pantograph-type response control system with a toggle mechanism and a dynamic mass (DM) damper system to the lower part of the steel stack designed to control the bending mode of vibration.

The main improvements are as follows:
(1) Complex natural value analysis has shown that the damping ratio of first mode h_1 of the original structure was 1% and that of the response-controlled structure was 14%. Although it is difficult to provide high damping to a slender, easily bendable structure, the response control system ensures high damping.
(2) Earthquake response achieved by attaching the response control system was 40 percent smaller than the response of the original structure.
(3) The stress ratio of main column members of the original structure was 1.36 and that of the response-controlled structure was 0.99. The stress ratio of the other members of the original structure was 1.20 and that of the response-controlled structure was 0.93. All members were within the elastic limit.

Keywords : response control retrofit, dynamic mass damper, pantograph-type response control system, steel stack

1. Introduction

Tower structures such as tower-supported steel stacks of thermal power plants are often designed in view of wind loads. In view of the experiences of earthquakes such as the 1995 Hyogo-ken Nanbu Earthquake and the 2007 Niigata-ken Chuetsu-oki Earthquake, the intensity of earthquake ground motion to be considered has increased. Consequently, in some cases seismic load is deemed to be a governing factor necessitating some kind of seismic retrofit. This paper introduces an example of a seismic response control retrofit of a 200-meter-tall tower-supported steel stack structure.

The requirements for this improvement work included the following:

(1) It is desirable that the improvement work be carried out without putting the stack out of service and without interrupting power generation.

(2) It is therefore necessary to explore possibilities for achieving the seismic retrofit goal by installing some kind of device to the lower part of the steel tower.

(3) The improved tower structure must be capable of staying within the elastic limit (plasticity coefficient *F* × 1.1) under the assumed ground motion (peak ground acceleration: about 300 cm/s^2, peak ground velocity: about 70 cm/s).

As a result of the seismic response control retrofit, these requirements were met by installing a pantograph-type response control system consisting mainly of toggle mechanisms and tuned dynamic mass (DM) dampers capable of resisting acceleration and controlling vibration modes. Table 1 compares the two approaches considered, namely, seismic strengthening and response control retrofit.

Table 1 – Seismic strengthening vs. response control retrofit

Category	Seismic strengthening	Response control retrofit
Safety	・Work at height Strengthening of main column members , diagonal members and horizontal members at a height of more than 150m	・Work at low height Installation of response control units at GL+0-28.5m of the first segment ; no need for structural reinforcement
Supply reliability	・Large-scale protection of existing equipment ・Interruption of power generation	・Small-scale protection of existing equipment ・No need to interrupt power generation
Cost	100%	About 50%

2. Overview of Structure

The structure of interest here, which was designed in 1974 and completed in 1977, is a tower-supported multi-cylinder steel stack having a stack cylinder height of 200 m above ground, a tower height of 192 m above ground, a top width of about 14 m and a base width of 50 m. Information on this structure is shown below.

Location	: Anesaki kaigan, Ichihara, Chiba, Japan
Maximum height	: GL+200 m
Type of structure	: Tower-supported steel stack (tower-supported 4-cylinder stack)
Structural members	: Steel pipes
Foundation structure	: Independent foundations with tie beams and steel pipe piles
Type of frame construction	: Double Warren truss
Owner	: TEPCO Fuel & Power, Inc.
Seismic retrofit design	: Tokyo Electric Power Services Co., Ltd., i2S2 Co., Ltd.
Seismic retrofitting	: Hitachi Zosen Corporation
Retrofit work period	: July 2013 to March 2014

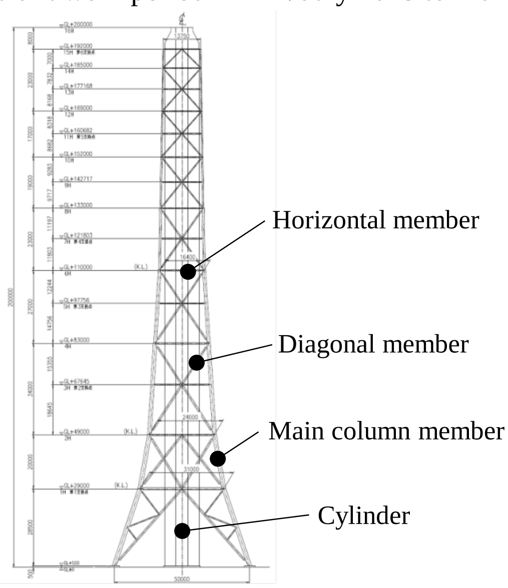

Fig. 1 – Tower structure

Photo 1 – General view of the tower-supported stack

3. Response Control System

As shown in Fig. 2, the tower structure has a bending-type vibration mode in which the lowermost members of the structure are deformed considerably in the longitudinal direction. It was decided, therefore, to use a seismic response control system designed to follow axial deformation of the main column members by use of pantograph-type response control devices having the amplification mechanism shown in Fig. 3 located at the bottom of the main column members. Pantograph configuration was determined so that dynamic mass damper deformation is greater than longitudinal tower-member deformation by a factor of about 8.5. As a result of the design study, it was decided to install two response control units to each tower leg. Thus, a total of eight response control units were installed.

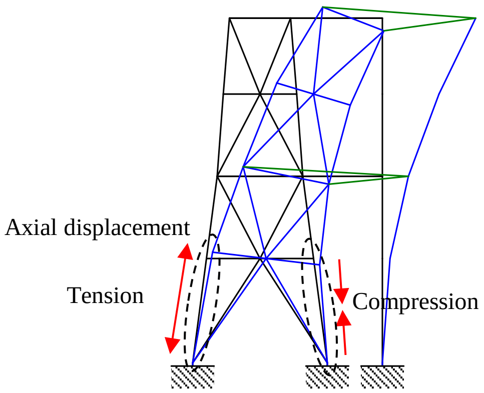

Fig.2 – Deformation mode of bending-prone structure

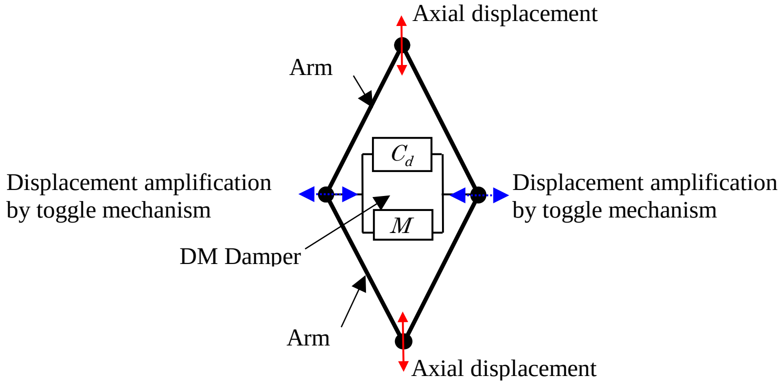

Fig.3 – Configuration of pantograph-type response control system

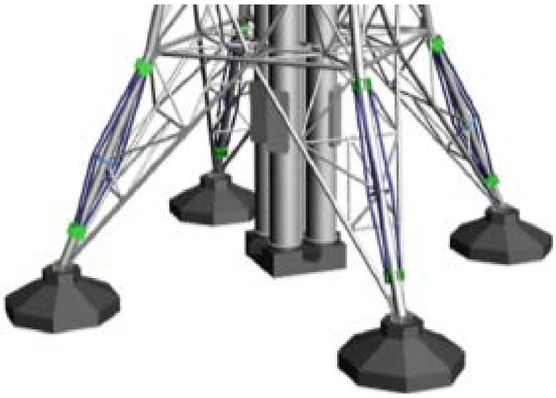

Fig. 4 – Installation of pantograph-type response control system

4. Ground Motions Considered

For the purposes of this study, three earthquake ground motions, selected taking ground motion amplification from the engineering bedrock into consideration, were considered: an Anesaki near-field earthquake, an Assumed South Kanto earthquake and a Northern Tokyo Bay earthquake. Table 2 shows data on these ground motions at the engineering bedrock. Figure 5 shows pseudo-velocity response spectra (h = 0.01, 0.4) determined in view of ground motion amplification from the engineering bedrock. Figure 5 also shows the horizontal first-mode period (2.64 s) and horizontal second-mode period (0.8 s) of the tower structure. As shown, the first-mode period overlaps with the dominant period of the Anesaki near-field earthquake ground motion.

Table 2 – Ground motions considered (engineering bedrock)

Ground motion	Peak ground acceleration (cm/s^2)	Peak ground velocity (cm/s)
Anesaki near-field earthquake	283	70
Assumed South Kanto earthquake	307	55
Northern Tokyo Bay earthquake	194	23

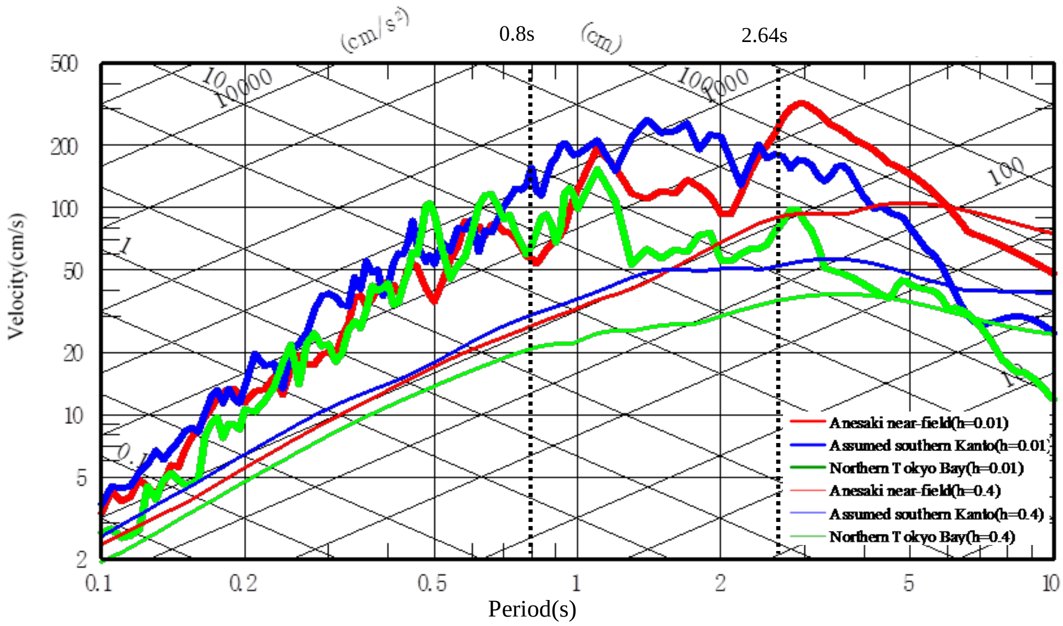

Fig. 5 – Pseudo velocity response spectrum of input ground motion

5. Response Control Model Study

For the purpose of response control study, four models were considered: (1) original structure (N), (2) installing viscous dampers to the tower (SC), (3) installing viscous dampers between the tower and the stack cylinders (CC) and (4) installing pantograph-type response control units to the lowest part of the tower (PT). Control response analyses were conducted by using the two-dimensional models shown in Fig. 6.

In the analysis performed by using Model SC, it was found that the amount of increase in the first-mode damping factor h_1 was only about 0.06 even in the case where viscous dampers were provided at all support points. It was decided, therefore, to use shear linking and remove diagonal members, and use a total of twelve viscous dampers (damping coefficient of C_d = 5.0 kN·s/mm) so as to achieve a large damping factor (h_1 = 0.19).

Model CC is configured so that the support points in the rigid connections between the tower structure and the stack cylinders are removed and a total of eight viscous dampers (C_d = 1.5 kN·s/mm) are attached between them.

Model PT is the model adopted for the tower-supported stack reported in this paper. In this model, a pantograph-type response control system is installed to the lowest part of the tower, and two response control units are attached to each leg so that a total of eight response control units are installed. The pantograph arms are assumed to be made of cylindrical members 426 mm in diameter and 35 mm in wall thickness. In accordance with the optimum design method (see Appendix), the dynamic mass damper is assumed to have a dynamic mass of 560 tons and a viscosity C_d of 0.7 kN·s/mm.

Table 3 shows the complex natural vibration analysis results obtained from the four models. Figures 7 to 10 show the maximum response story drift angles of the tower and the stress check ratios of the main column members. As shown, Model N (original structure) shows a maximum response story drift angle of 1/60 and stress check ratios to the elastic limit of 1.40 for the main column members, 1.03 for the diagonal members and 1.17 for the cylinder members. Model SC (shear link type) shows a maximum response story drift angle of 1/98. Since, however, the diagonal members have been removed, the stress check ratio of the main column members has increased to 1.56 (i.e. decrease in stiffness) so that the main column members have to carry larger loads. Model CC (rigid connection type) has a maximum response story drift angle of 1/72 and a stress check ratio of 1.10. Model PT has a maximum response story drift angle of 1/92 and a stress check ratio of the main column members of 0.98.

Table 4 shows the stress check results for each model. In view of these results, it was decided to use Model PT (h_1 = 0.14), which has a pantograph-type response control system that meets the seismic performance requirements.

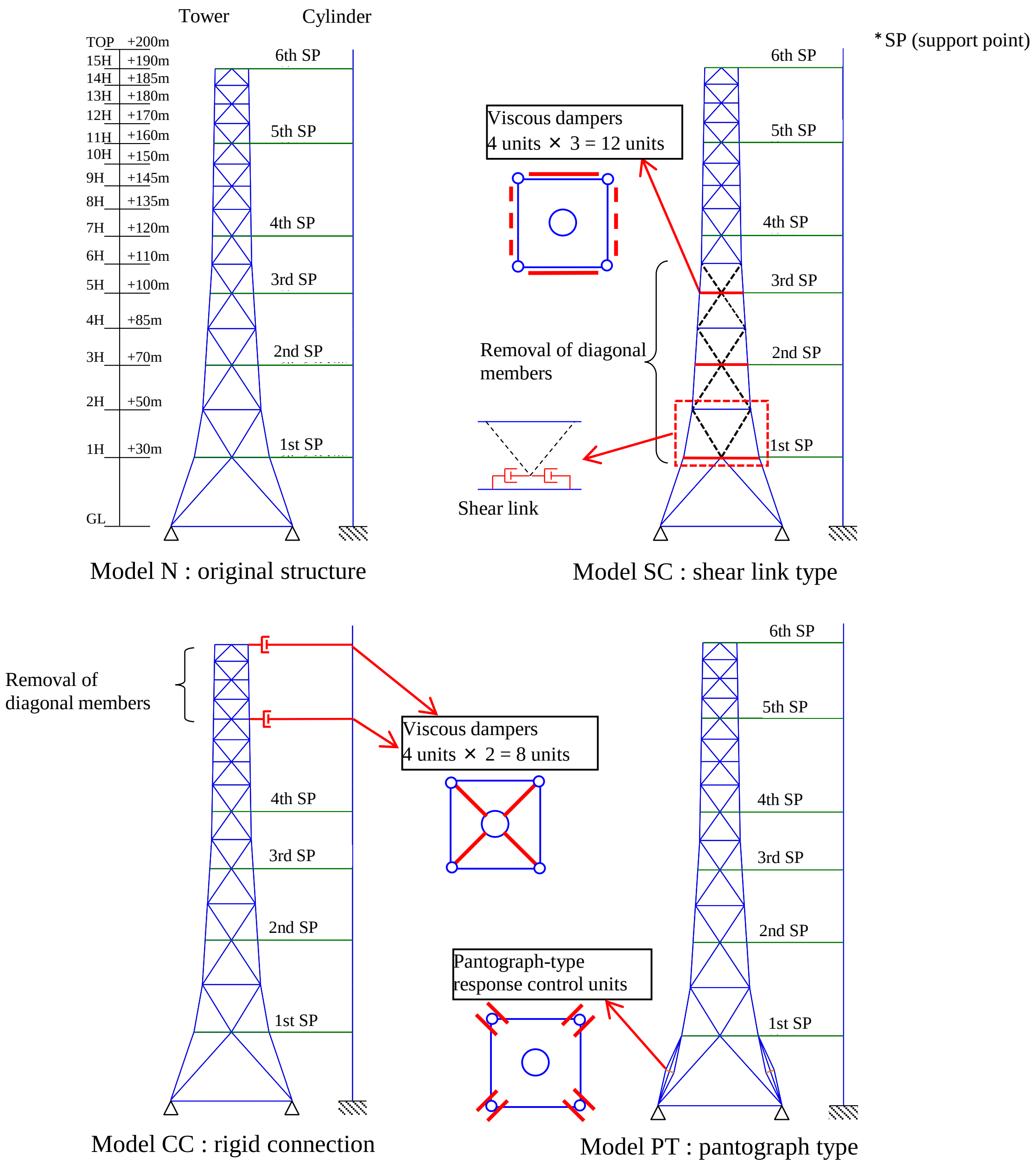

Fig. 6 – Plane analysis model

Table 3 – Complex natural vibration analysis results

Model		N	SC	CC	PT
First	T_1(s)	2.64	2.99	2.66	2.78
	h_1	*0.01*	*0.19*	*0.04*	*0.14*
Second	T_2(s)	0.80	1.23	0.80	0.77
	h_2	*0.01*	*0.73*	*0.03*	*0.02*
Third	T_3(s)	0.45	0.46	0.45	0.45
	h_3	*0.02*	*0.30*	*0.04*	*0.03*

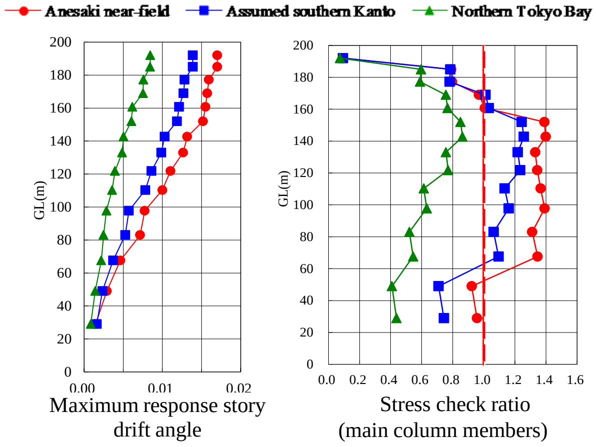

Fig. 7 – Model N response results

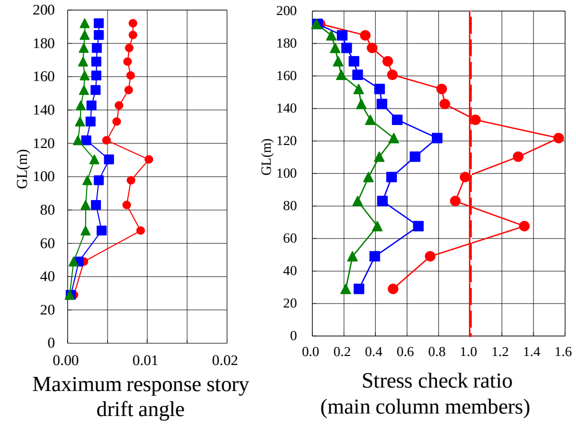

Fig. 8 – Model SC response results

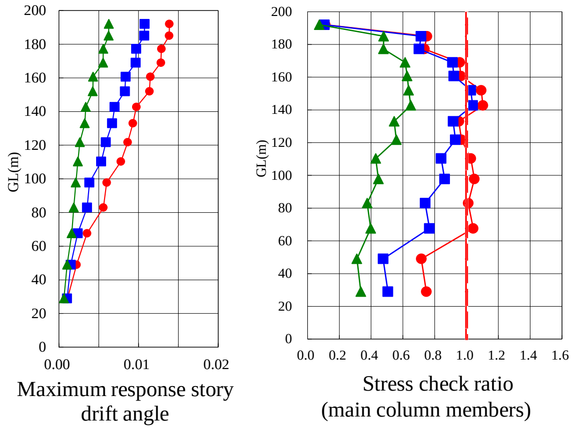

Fig. 9 – Model CC response results

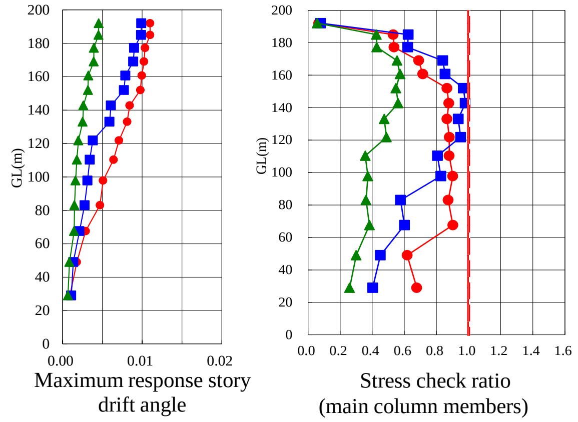

Fig. 10 – Model PT response results

Table 4 – Stress check ratios

Model	N	SC	CC	PT
Main column member	**1.40**	**1.56**	**1.10**	0.98
Diagonal member	**1.03**	0.67	0.85	0.74
Horizontal member	0.74	0.55	0.81	0.72
Stack cylinder	**1.17**	**1.14**	0.97	0.85

6. Three-Dimensional Response Analysis

To evaluate damping in the space frame vibration model, the horizontal first-mode and second-mode damping factors for Rayleigh damping were assumed to be h_1 = h_2 = 0.01. In the earthquake response analysis, validity of response results was checked by using two analysis codes.

The analysis was performed for three input directions, namely, 0, 45 and 90 degrees, taking into consideration dynamic mass damper characteristics and variations in conditions such as installation and construction accuracy variations. Specifications of the dynamic mass dampers were designed taking account of the ease of fabrication of their members and stress in existing members. In short, theoretical optimum values were slightly changed so as to make intermediate member stress smaller. The specifications thus determined are shown in Table 5.

Figure 11 shows the response analysis results in the standard case in which the angle of input direction is 45 degrees, dynamic mass damper characteristics are standard and construction accuracy error is assumed to be zero, in comparison with the results in the non-response-control and original-structure case (dotted lines).

Comparison with the original structure case reveals that the response values at the top of the structure decreased by about 40 percent.

Figure 12 shows the stress check results for the main column members, horizontal members, diagonal members and stack cylinders in the standard case. Table 6 shows the maximum values of the stress check ratio for different members adjusted by taking into consideration variations among the response control units. As shown, in the response-controlled structure with pantograph-type response control units, the responses of all members are within the elastic limit, indicating that the seismic performance goals are met.

Table 5 – Dynamic mass damper specifications (per unit)

	Optimum value	Design value
DM(ton)	560	450
Cd(kN･s/mm)	0.70	1.63

Table 6 – Maximum values of stress check ratio

Member	Original structure	Response controlled structure (maximum value of all cases)
Main column member	**1.36**	0.99
Diagonal member	**1.07**	0.82
Horizontal member	**1.20**	0.93
Stack cylinder	**1.03**	0.84

* The underlined values are for the southern Kanto earthquake, and the other values are for the Anesaki near-field earthquake.

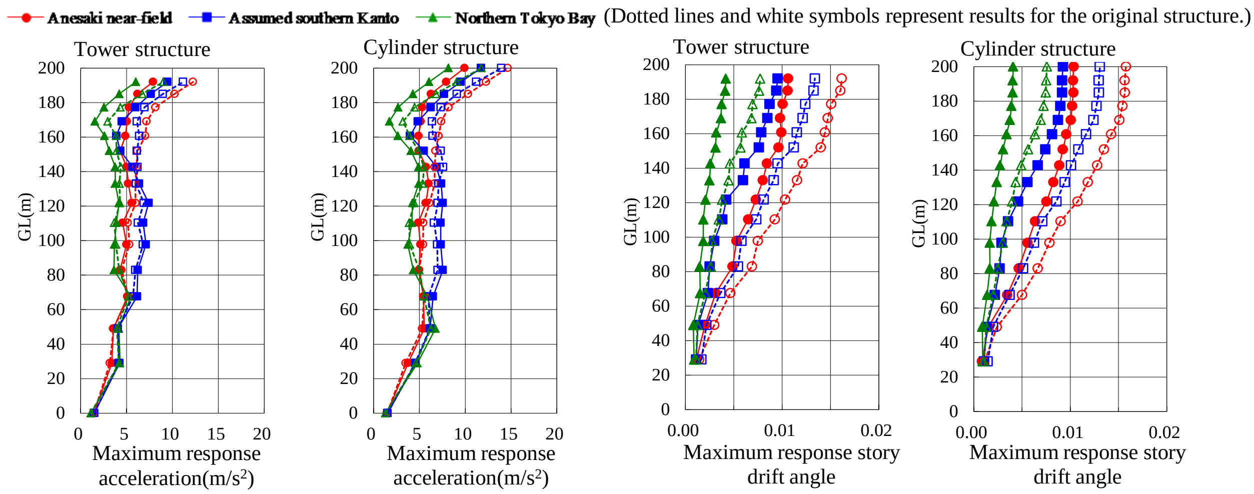

Fig. 11 – Results of 45-degree response analysis (standard case)

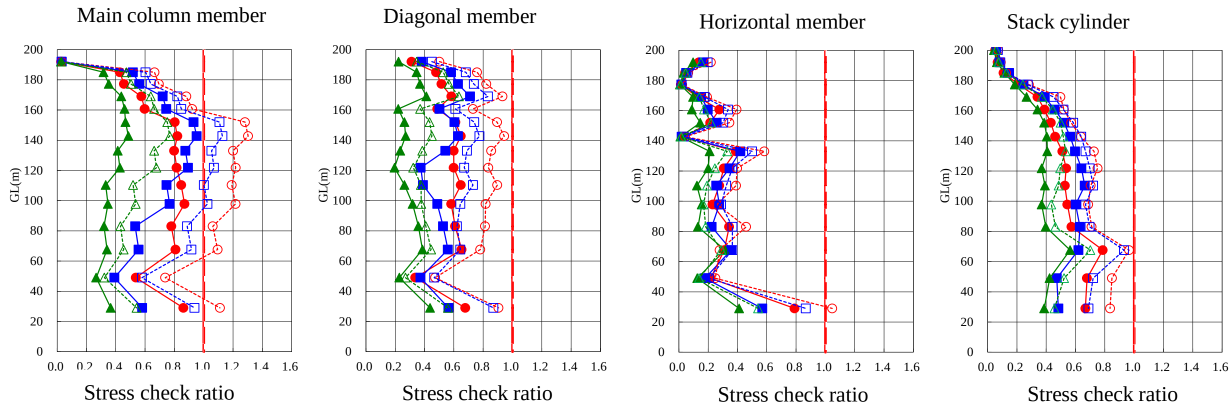

Fig. 12 – Stress check ratio (standard case)

7. Conclusion

This paper has reported on the response control retrofit of a tower structure carried out by using a pantograph-type response control system equipped with tuned dynamic mass systems so as to increase the viscous damping factor h_1 by 14%, which is difficult to achieve with a structure prone to bending. The first-mode seismic reduction ratio achieved was η = 0.68, and the first-mode effective mass ratio in that case was about 50%. Thus, it has been shown that seismic response control by use of a large mass comparable to the mass of the structure to be controlled may expand the scope of response control applications.

The retrofit work was carried out by erecting a scaffold about 30 m in height, as shown in Photo 2, and using a crane. As mentioned at the outset, the retrofit work was done without interrupting power plant operation.

Photo 2 – Pantograph-type response control system

8. Acknowledgements

The authors gratefully acknowledge the generous assistance provided by Dr. Shinji Ishimaru, former professor, at the Department of Architecture, College of Science and Technology, Nihon University, in connection with the design of the tower structure reported in this paper and the model shaking experiment conducted for the purposes of this study. The authors would also like to express their heartfelt appreciation to Hitachi Zosen Corporation, Miyaji Engineering Co., Ltd., Johetsu Industries Co., Ltd. and Aseismic Devices Co., Ltd. for their generous assistance provided in connection with this study.

9. Appendix

The procedure for design optimization (tuning optimization) of the tuned dynamic mass damper system is described below.

(1) Calculate the natural period of the original structure. $\rightarrow T_0$ = 2.64 s

(2) Calculate the natural period in the case where the damping coefficient is infinitely large. The tuned dynamic mass system does not work if $C_d = \infty$. Hence, $m' = 0$. $\rightarrow T_\infty$ = 2.51 s

(3) Calculate the auxiliary stiffness factor κ. $\rightarrow \kappa = (T_o / T_\infty)^2 - 1 = (2.64/2.51)^2 - 1 = 0.11$

(4) Calculate the optimum damping factor *h*. $\rightarrow\ h \approx 0.5\sim0.65\sqrt{\kappa/(2+\kappa)} \fallingdotseq 0.14$

(5) Assume a dynamic mass and perform natural frequency analysis using a damping coefficient of $C_d = 0$. In the case where the dynamic mass m' is assumed to be 560 tons, $T_{0,1} = 2.96$ s and $T_{0,2} = 2.13$ s, and the law of geometric mean, $T_\infty = \sqrt{T_{0,1} \times T_{0,2}} = 2.51$, is met. The assumed dynamic mass value ($m' = 560$ tons), therefore, is deemed to be an optimum value.

(6) Assume a damping coefficient (C_d) and continue tuning until the viscous damping factor determined through complex natural frequency analysis converges to $h = 0.14$ obtained in Step (4). Since $h = 0.14$ is obtained when C_d is assumed to be 0.7 kN·s/mm, it is taken as an optimum value.

(7) Figure 13 shows the resonance curve for the relative displacement at the top of the tower. Since the two fixed points (P and Q) are at similar heights (optimally tuned) and since their heights represent the peaks of the resonance curve (optimal damping), the design may be deemed to be optimal.

In this way, specifications of a pantograph-type response control system are optimized.

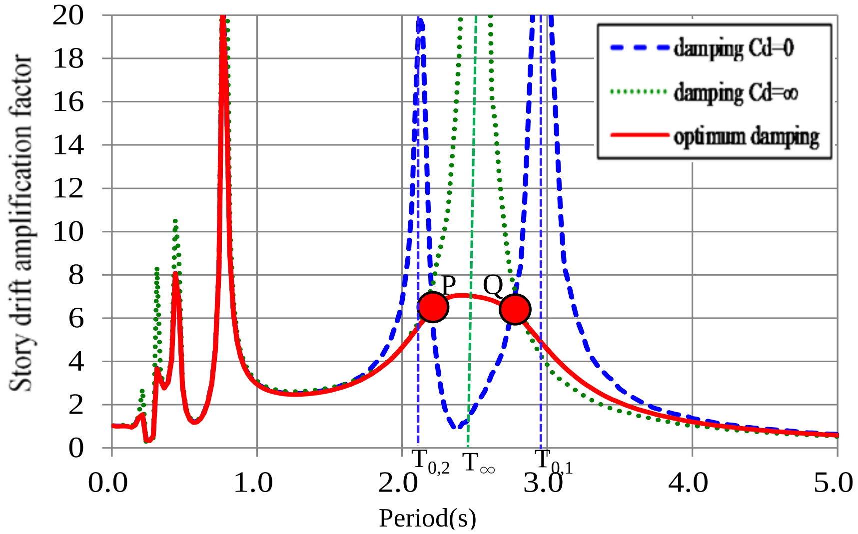

Fig. 13 – Resonance curve for the uppermost section of the tower

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