Wind and Structures, Vol. 12, No. 4 (2009) 333-347 333
DOI: http://dx.doi.org/10.12989/was.2009.12.4.333

The damping efficiency of vortex-induced vibration by tuned-mass damper of a tower-supported steel stack

Shin Homma*

Structural Design Division, Hitachi Zosen Corporation, Osaka 559-8559, Japan

Junji Maeda

Faculty of Human-Environment Studies, Kyushu University, Fukuoka 812-8581, Japan

Naoya Hanada

Civil Engineering Dept., Kyushu Electric Power Co., Inc., Fukuoka 810-8720, Japan

(Received September 3, 2007, Accepted April 23, 2009)

Abstract. Many tower-supported steel stacks have been constructed in Japan, primarily for economic reasons. However the dynamic behavior of these stacks under a strong wind is not well known and the wind load design standard for this type of a stack has not yet been formulated. In light of this situation, we carried out wind response observation of an operating tower-supported steel stack with and without a tuned-mass damper. The observation revealed the performance of the tuned-mass damper installed on the stack in order to control the wind-induced vibration. Based on the observed data, we performed a wind tunnel test of a specimen of the stack. In this paper we report the results of the wind tunnel test and some comparisons with the results of observation. Our findings are as follows: 1) the tuned-mass damper installed on the specimen in the wind tunnel test worked as well as the one on the observed stack, 2) the amplitude of the vortex-induced vibration of the specimen corresponded approximately to that of the observed stack, and 3) correlation between Scruton number and reduced amplitude, y/d, (y is amplitude, d is diameter) was confirmed by both the wind tunnel test and the observed results.

Keywords: tower-supported steel stack; vortex-induced vibration; wind tunnel test; tuned-mass damper; scruton number; resonance.

1. Introduction

In Japan a tower-supported steel stack is often chosen as the structural form for a high stack when its height exceeds about 100 m. High stacks have become important for chemical plants or thermal power stations so as to diffuse exhaust gasses to the atmosphere more widely.

It is well known that the wind causes vortex-induced vibrations in slender structures with circular cross sections (Blevins 1990, Simiu and Scanlan 1996). This problem has become serious for many

* Corresponding Author, E-mail: homma_s@hitachizosen.co.jp

large stacks constructed throughout the world in the 1950's because of the lack of the research concerning the dynamic response of the cylindrical structure to the wind. To solve this problem, many observations of isolated steel stacks (Ciesielski, *et al.* 1992, Galemann and Ruscheweyh 1992, Ruscheweyh, *et al.* 1998) were performed, and several design standards for these isolated stacks were enacted (Eurocord 1995, CICIND 1999, The architectural Institute of Japan 2004). There have been many studies of the vibration characteristics of isolated stacks (Dyrbye and Hansen 1996, Koten 1998, Vickery 1998, Chmielewski and Górski 2007) with effective vibration-control techniques using helical strakes (Shimada, *et al.* 1986) or a damping device (Ueda, *et al.* 1992) were reported.

In Japan many stacks were built in the 1960's and the 1970's during the so-called "Japanese Post-War Economic Miracle". But, the Building Standard Law in Japan at that time required just static analyses of a stack of over 6 m in height under wind and seismic loads regardless of its structural form and scale. Though some dynamic observations of a tower-supported steel stack were reported (Kawamura, *et al.* 1992), the dynamic behaviors of tower-supported steel stacks under strong winds are not known due to a lack of site observations. Neither has the dynamic design standard for a tower-supported steel stack as yet been provided. Therefore, site measurements to understand the minute dynamic behavior of a tower-supported steel stack has become important in recent years in light of the increase in the amount of strong winds such as in large-scale typhoons.

We have made a three-year observation of a tower-supported steel stack that stands at northern Kyushu in Japan. The results of the observation have revealed that the vortex-induced vibration property of the tower-supported steel stack varies depending on the wind condition. A lumped-mass model was used for some vibration analyses of the observed stack. The structural properties of the lumped-mass model were adjusted based on the observed data. The information was very useful for making a specimen of a 3D elastic model used in wind tunnel tests. In addition, the damping performance of a tuned-mass damper (TMD) installed on the observed stack was also studied during the observation (Ikeda, *et al.* 2001, Susuki, *et al.* 2004, Susuki, *et al.* 2005, 2006, Homma, *et al.* 2007). In this paper we report the performance of the TMD on the observed stack, and discuss the wind tunnel test results based on the observations of the stack with and without a TMD.

2. Structural properties of the stack

The general structure of a tower-supported steel stack is shown in Fig. 1. The pipe-truss tower supports the stack shell and bears the horizontal loads such as those due to earthquakes or wind. The thickness of the stack shell is very thin, usually from 10 to 20 mm of steel plate. On the other hand, the vertical expansions are different between the steel stack shell heated by the inner exhaust gas and the steel support tower in the natural environment. If the support tower is connected tightly to the stack shell, the secondary heat stress from these differences appears in the whole stack. Therefore, the supporting points slide in the vertical direction so as not to constrain the expansion of the stack shell.

The specification of the stack in this experiment is shown in Table 1. The stack in this test is specified in simplest form with a single stack shell and tower of four posts, and 200 m height. The natural frequency of the stack was calculated from eigenvalue analysis of the lumped-mass model dividing the stack weight evenly between the top of the stack and each supporting point level. Concerning the natural frequency, good agreement was obtained between the observed frequency of

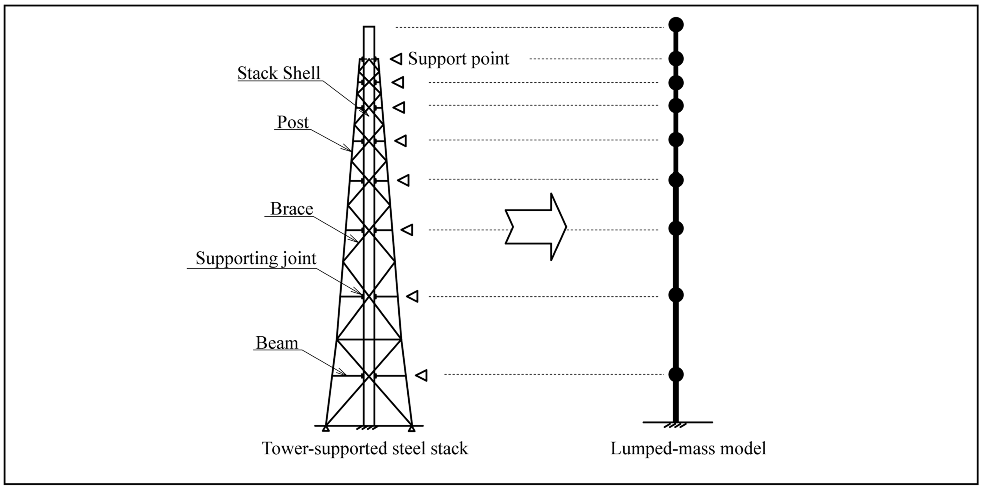


Fig. 1 General structure of tower-supported steel stack and lumped-mass model

Table 1 The specification of the stack condition

Parameter	Detail
Type	Single-stack shell, four-post, tower-supported type
Material	Steel (both the stack shell and support tower)
Stack shell diameter	ϕ 5.57 m (O.D)
Stack shell height	GL+200 m
Tower height	GL+185 m
Structure of stack shell	Accumulating and welding of unit ring, supported by the tower with a horizontal direction at eight points without constraint in the vertical direction
Structure of support tower	Pipe truss, weld bonding for tower post, high strength bolt connection for other members (Post-Beam, Post-Brace, Brace-Beam)
Location	Plain terrain, surrounded by large-scale buildings (industrial area)
Damping device	Tuned Mass Damper (TMD), set at the top of the support tower level
Others	Mortar type lining inside the stack shell

the actual stack and the results of eigenvalue analysis of the lumped-mass model (Ikeda, *et al*. 2001; See Table 2).

The standing area is assumed to be an industrial area, that is, plain terrain surrounded by large-scale buildings.

The following dynamic properties of the tower-supported steel stack were confirmed by the observation of the real stack (Susuki, *et al*. 2006).

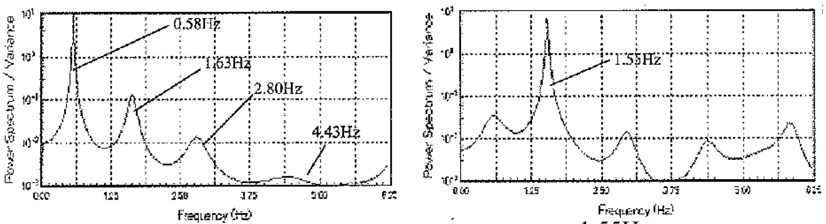
1) The damping ratio of the stack without the TMD is not large, being approximately 0.3%, though it has many members with high strength bolt connection.

2) The overall vortex-induced vibration of the stack can be generated even though the truss tower is surrounding the stack shell.

Table 2 The Comparison of the natural frequency between the observed frequency of the actual stack and the results of eigenvalue analysis of the lumped-mass model (Ikeda, *et al*. 2001)

Parameter		analysis	actual stack	
Vibration frequency (Hz)		-	0.58	1.55
Natural frequency (Hz)	first	0.58	0.58	-
	second	1.54	1.63	1.55
	third	2.78	2.80	-
	fourth	4.33	4.43	-

The actual stack was vibrated at the top of the tower by human power with 0.58/1.55 Hz and the observed data was analyzed by AR method.



0.58Hz 1.55Hz

Observed data of the Actual stack

3. The mechanism of the TMD

The method of control of vortex-induced vibration by the TMD is widely used not only in stacks but also in buildings. The performance of the TMD is determined by the fixed points theory. The optimum tuning and the damping ratio are basically calculated from the formulas as follows.

Optimum tuning: $$\mu = \frac{f_d}{f_s} = \frac{1}{1+R} \tag{1}$$

Optimum damping: $$\zeta = \sqrt{\frac{3R}{8(1+R)^3}} \tag{2}$$

$$R = \frac{m_d}{m_s} \tag{3}$$

with f_d: the natural frequency of the TMD, f_s: the natural frequency of the stack, m_d: the mass of the TMD, and m_s: the equivalent mass of the stack (here, the first mode).

On the other hand, an adjustable mechanism is necessary for an actual TMD because the observed performance of the real TMD usually differs from the theoretical. In our observation, the TMD installed on the stack was the pendulum type that is adjustable so as to increase or decrease the mass weight and contract or expand the suspender, as shown in Fig. 2. The length of the suspender influences the natural frequency of the TMD, and is defined as:

$$f_d = \frac{1}{2\pi}\sqrt{\frac{g}{\ell}} \tag{4}$$

with g: gravity acceleration and ℓ: the length of the suspender.

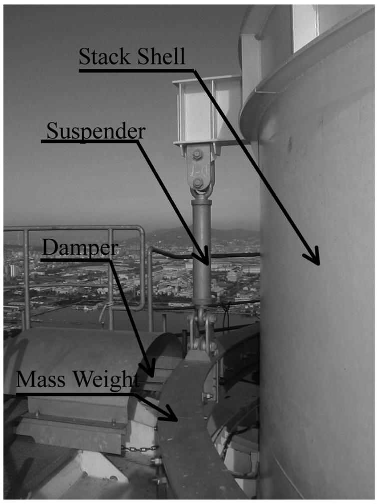


Fig. 2 The TMD

For the real stack the vortex-induced vibration was observed after the TMD was installed for two different damping ratios of the TMD. The performance after tuning was superior. Therefore, the results of the observation proved that the adjustment mechanism on the actual TMD is effective in improving the damping ratio of the TMD. In addition, we observed that the TMD is effective in not only controlling the vortex-induced vibration, but it also decreases the gust response (Susuki, *et al.* 2006).

4. Outline of the experiment

The stack model is shown in Fig. 3. The rigidity of the stack was modeled with a rigid bar inside a cylinder stack shell. The tower and the stack shell were divided into nine blocks and connected to the rigid bar at each support level. Trip wires were added on the surface of the model for simulation of aerodynamic properties. The specification of the TMD was determined so as to control the first (vibrational) mode of the stack. The ring-shaped mass weight (ring weight) was immersed in a viscous fluid. The natural frequency of the TMD was adjusted by changing the length of the suspender, and the damping ratio was adjusted by changing the viscosity of the viscous fluid.

The vibration properties of the model are shown in Table 3, and the outline of the experiment is shown in Table 4. It is necessary to keep the vibrations below the natural frequency of the model because there is the possibility of producing local vibration of the truss component at higher-order natural frequencies in the lightened aerodynamic elastic model. Therefore, the rigidity of the model was reduced by a factor of 7, and the first natural frequency was adjusted to 2.0 Hz in this experiment.

The model is made to simulate the prototype stack in Reynolds Number (here, supercritical range) using trip wires to reproduce the vortex-induced vibration of the prototype stack. Therefore, the vibration in the roughness Reynolds Number range (Ruscheweyh and Galemann 1996) is not assumed here. The influence of the vortex shedding of the lattice tower to that of the circular cylinder is not discussed here because the influence is not clear. (although the vortex has little influence because the frequency and the scale of the vortex shedding are different in the lattice tower and the circular cylinder.)

The experimental setup for the wind tunnel test is shown in Fig. 4. The airflow was for two conditions, uniform flow and turbulent boundary layer flow. In the case of the boundary layer flow, the spires and roughness blocks were set up to generate the boundary layer. The case of the boundary layer is described in this paper.

Vertical distribution of the turbulence properties in the wind tunnel is shown in Fig. 5 and the

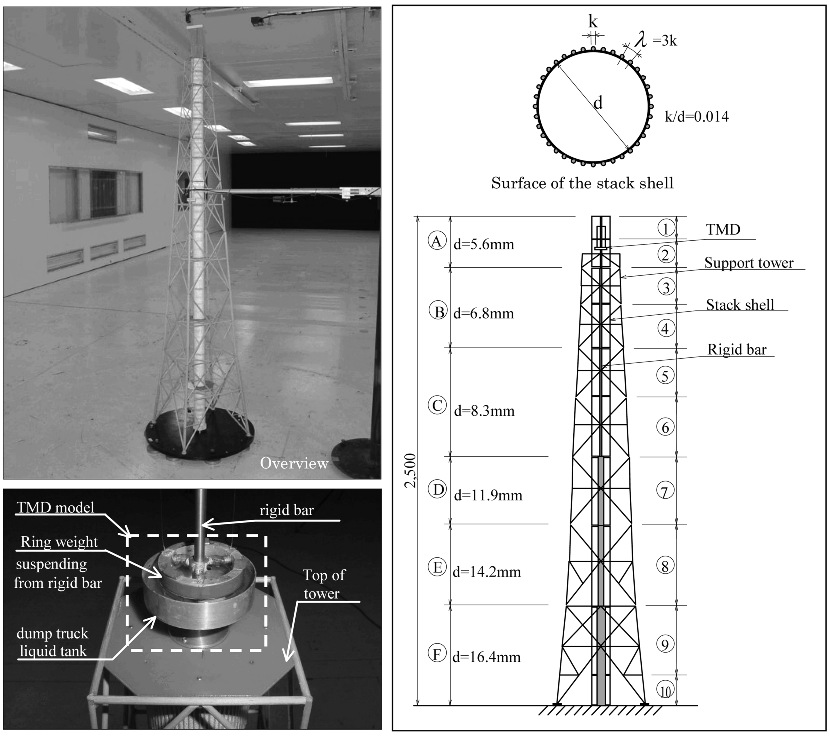


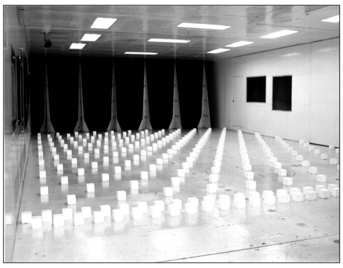
Fig. 3 The stack model

Table 3 The specifications of the mode

Parameter			Prototype stack	Wind tunnel model: Target	Wind tunnel model: Observed
Scale (m)			200	2.5 (1/80)	
Equivalent weight (first mode)			2514kN	4.91N	5.46N
Natural frequency (Hz)		first	0.58	1.96	1.71
		second	1.54	5.20	5.64
Mass ratio of TMD				2%	3%
Frequency ratio				0.95	0.97
Damping ratio	first	Without TMD	0.3%	-	0.2%
		With TMD	5%	-	2.3%
	second	Without TMD	-	-	0.3%
		With TMD	-	-	0.6%

Table 4 The outline of the experiment

Parameter	Detail
Model	3D elastic model
Similarity	Mass: $1/80^3$ Stiffness: $(1/80^5)/7$
Correspondence with Reynolds number	k/d = 0.014 (k: diameter of tripwire, d: diameter of stack shell)
Flow characteristic	1) Uniform flow 2) Boundary layer turbulent flow using spire and roughness block (described in this paper)
Magnification for real wind	x27.1 (first frequency) x21.8 (second frequency)
Sampling frequency	100 Hz
Sampling time	40 sec/unit



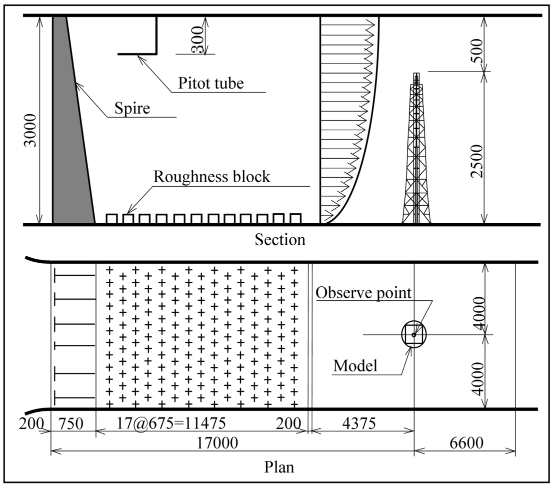


Fig. 4 The wind tunnel

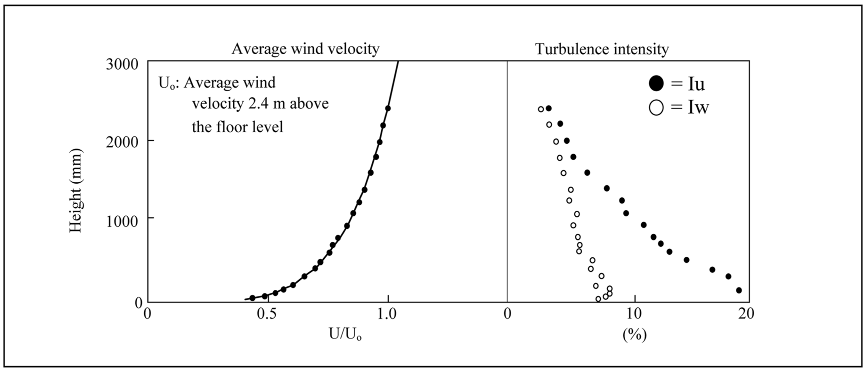


Fig. 5 The vertical distribution of the turbulence properties in the wind tunnel

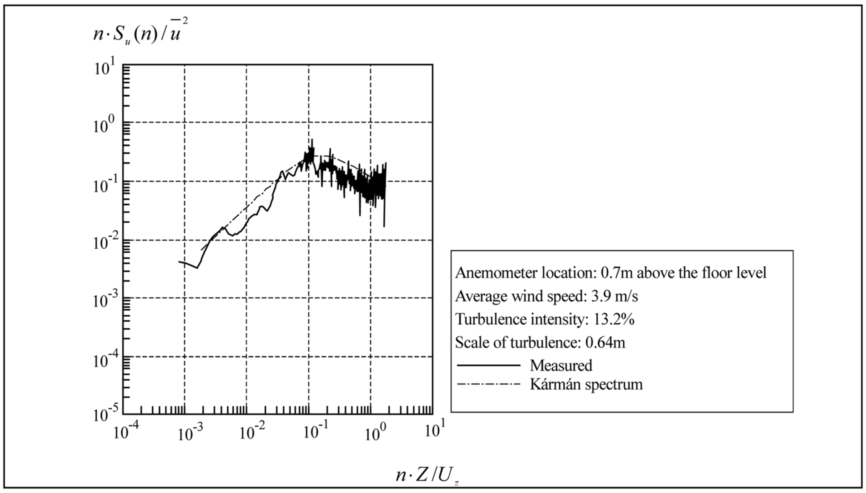


Fig. 6 Power spectrum of the wind tunnel

power spectrum of the wind tunnel is shown in the Fig. 6. The average wind velocity is measured at the 2/3 height level. The turbulence intensity at the 2/3 height is about 8%, and the thickness of the boundary layer is about 2700 mm. Moreover, the exponent of vertical distribution of the average wind velocity was adjusted to 1/5 because of the assumption of being surrounded of a large-scale plant building.

The testing wind velocity and the displacement amplitude were calculated as follows.

Ratio of prototype wind velocity to model: V_p/V_m

first mode: $V_{p1}/V_{m1} = (f_{p1}/f_{m1}) \times n = 27.1$

second mode: $V_{p2}/V_{m2} = (f_{p2}/f_{m2}) \times n = 21.8$

Displacement amplitude: A_p/A_m

$A_p/A_m = n = 80$

where n = 80, suffix p = prototype stack, and suffix m = wind tunnel model.

5. Experimental method

Before the wind tunnel test, a human-powered vibration test was performed to confirm the stack and TMD model properties. The natural frequency, vibration mode, and damping ratio for the wind direction and at a right angle to the wind direction were computed from this result. Next, the wind tunnel test in turbulent flow was performed. The response was observed for two conditions: with TMD operating and with TMD not operating. The observation points were at the top and at 2/3 height of the model stack.

The model stack was set up for two wind directions; with the truss wall inclined 90 degrees and 45 degrees to the wind direction as in Fig. 7. The natural frequency was calculated from 8192 waveform data using Fast Fourier Transform. The damping ratio was calculated from waveform data obtained with a digital band-pass filter. The sampling frequency was set to 100 Hz and sampling time to 40 sec/unit. And the response amplitude was measured at the maximum amplitude when the response indicated regular amplitude.

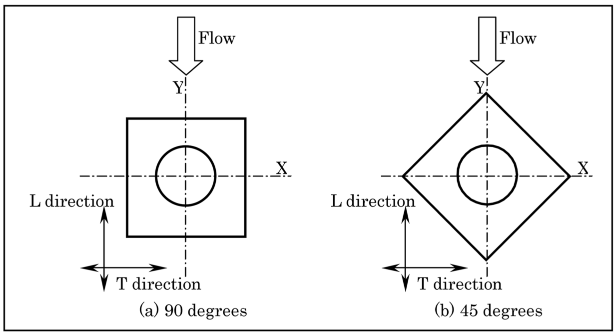


Fig. 7 The test wind direction

6. Results

6.1. The free vibration of the model

Fig. 8 shows comparison of the free vibration response between the human-powered vibration test and the eigenvalue analysis of the lumped-mass model. There is good agreement.

6.2. Dynamic wind response of the model

Fig. 9 shows the response curve in turbulent flow. In the figure for non-operating TMD, the theoretical resonant wind velocity (18.0 m/s) is shown by the dotted line. The resonant wind velocity is defined as:

$$U = D \cdot f / St \tag{5}$$

where U = resonant wind velocity (m/s), D = diameter of the stack shell (m), f = natural frequency of the stack (Hz), and St = Strouhal Number (here, St = 0.18).

From the figure for non-operating TMD, two large responses were observed, and with TMD in operation, both of these responses were reduced. Moreover, the theoretical resonant wind velocity was in good agreement with the measured value for the primary peak.

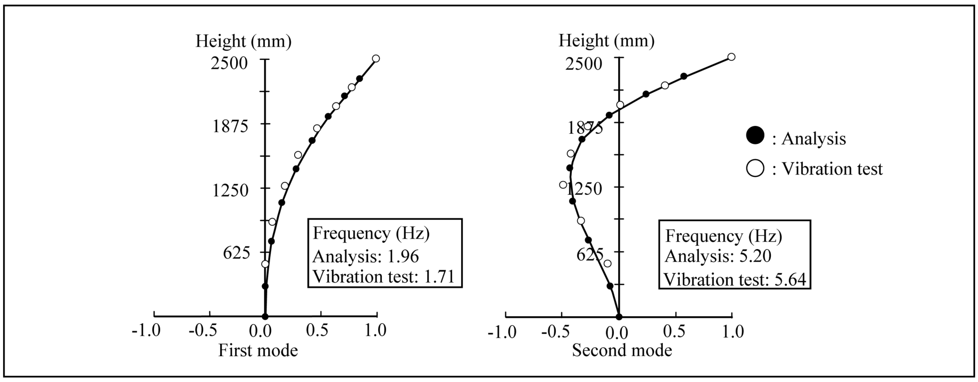


Fig. 8 The comparison of the response mode

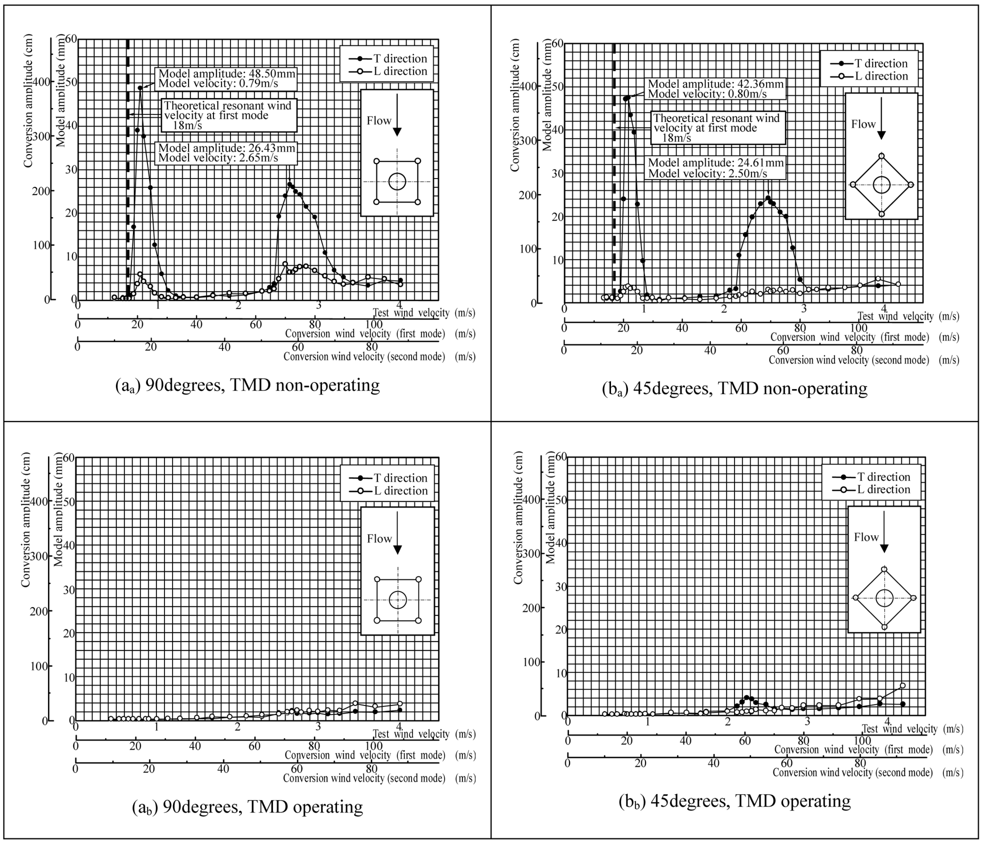
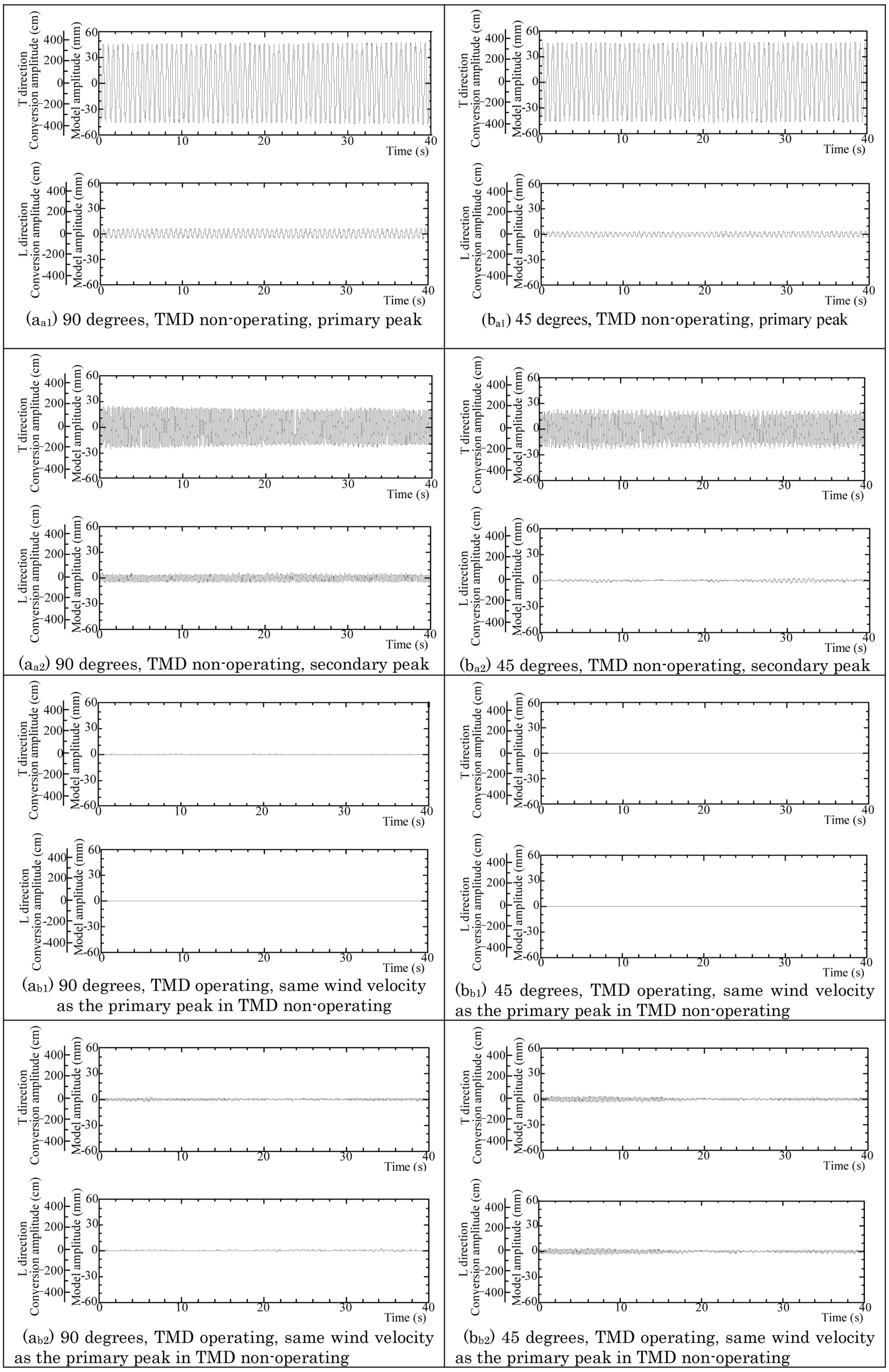


Fig. 9 The response curve in turbulent flow

Fig. 10 shows the time series at the top of the model when these two large responses were observed. This suggests that the amplitude was a steady response. And the Lissajous figure of the top of the model with non-operating TMD is for the same time as in Fig.10 and is shown in Fig. 11. From these figures, it is clear that the direction of the response amplitude is at a right angle to the wind direction. And also, the amplitudes at both 90 degrees and 45 degrees are approximately the same.

Fig. 12 shows the results of the frequency analysis for the same time as in Fig. 10. From the figure, apparently one predominant frequency existed in the primary peak, and two frequencies in the secondary peak corresponding to the values obtained from the human-vibration test (as in Fig. 8).

From these results, it is clear that the response is vortex-induced vibration because the vibration direction is at a right angle to the wind direction and the response frequency corresponds to the natural frequency of the model stack. And also the amplitude is approximately constant regardless of the projective width or solidity ratio of the support tower, and regardless of changes in the wind direction. Moreover, in these wind tunnel tests, we observed that the vortex-induced vibration was significant and that this vibration was effectively reduced by TMD. In addition, the reason why the secondary peak decreases for TMD is possibly because 1) the first and second frequency components corresponding to the model frequency are superior in the second peak, 2) the mass ratio of TMD is so large that the damping tuned to the first frequency is effective not only for the first but also the second frequency.



(a_{a1}) 90 degrees, TMD non-operating, primary peak

(b_{a1}) 45 degrees, TMD non-operating, primary peak

(a_{a2}) 90 degrees, TMD non-operating, secondary peak

(b_{a2}) 45 degrees, TMD non-operating, secondary peak

(a_{b1}) 90 degrees, TMD operating, same wind velocity as the primary peak in TMD non-operating

(b_{b1}) 45 degrees, TMD operating, same wind velocity as the primary peak in TMD non-operating

(a_{b2}) 90 degrees, TMD operating, same wind velocity as the primary peak in TMD non-operating

(b_{b2}) 45 degrees, TMD operating, same wind velocity as the primary peak in TMD non-operating

Fig. 10 The time series of the amplitude of the top of the stack shell

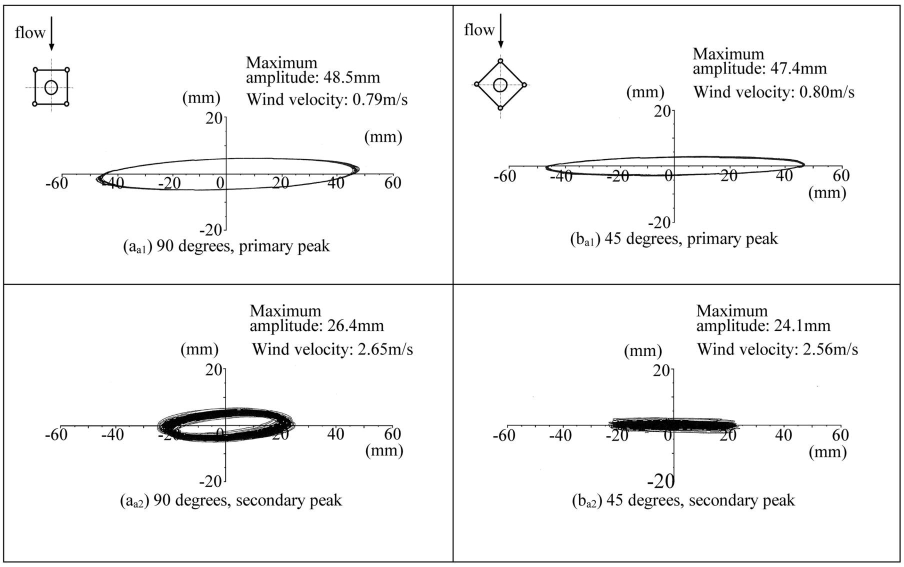


Fig. 11 The Lissajous figure of the top of tower in response

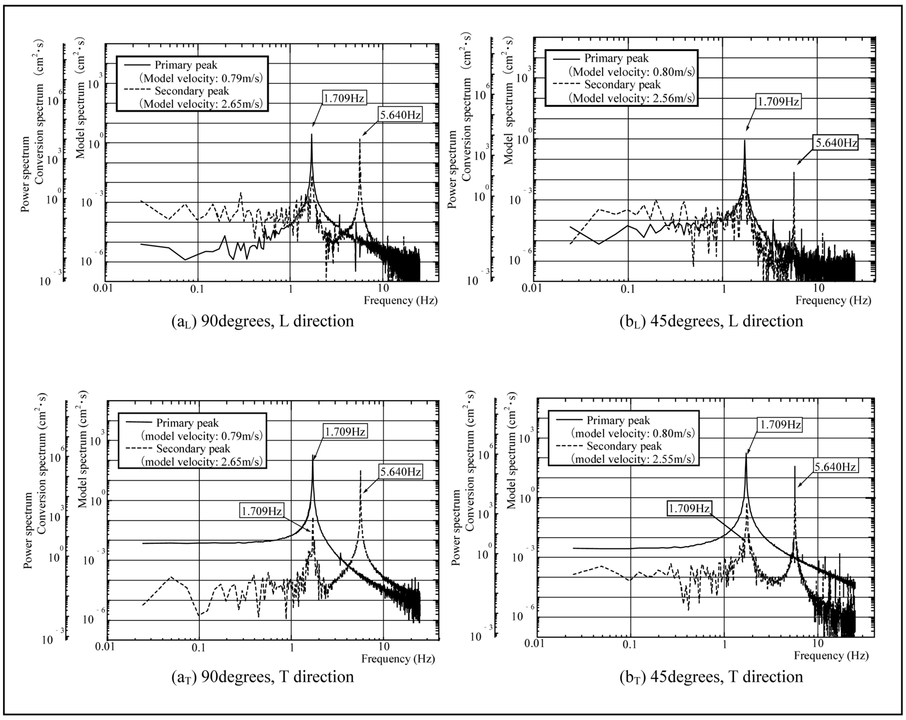


Fig. 12 The frequency analysis in the time when the large response was observed

7. Conclusions

We compared the result of the wind tunnel test to the dynamic response of the observed stack before and after setting up the TMD. In comparing the data between model and observed stack, we correlated the ratios of response amplitude (*y*), stack shell diameter (*d*), relative amplitude (*y/d*), and Scruton number (*Sc*), defined as:

$$S_c = \frac{4 \cdot \pi \cdot m_0 \cdot c/c_r}{\rho_a \cdot d^2} \tag{6}$$

$$m_0 = \frac{\int_0^h m(z) \cdot u^2(z)dz}{\int_0^h u^2(z)dz} \tag{7}$$

where ρ_a = air density (kg/m^3), d = the diameter of the shell stack (m), u = the mode shape of the first resonance frequency, c/cr = damping ratio, and h = the height of the stack (m).

Fig. 13 shows the result. The experimental data for a rocking model of a tower-supported steel stack (Ueda, *et al*. 2000) and Hansen's results (Hansen 1998) concerning some correlations between *y/d* and *Sc* for some actual measurement of the isolated stack are also presented in Fig. 13. The relation between the response amplitude and *Sc* are nearly the same. We confirmed that the response amplitude near the real stack was reproduced in the wind tunnel test, and that the vibration properties of the real stack were very similar to those produced in the wind tunnel test.

The following conclusions were obtained from the wind tunnel test of the tower supported steel stack.

1) Both primary and secondary peak responses of the tower supported steel stack to vortex-induced vibration are reproduced by wind tunnel test.

2) Calculated resonant wind velocity with St = 0.18 is in good agreement with the measured value

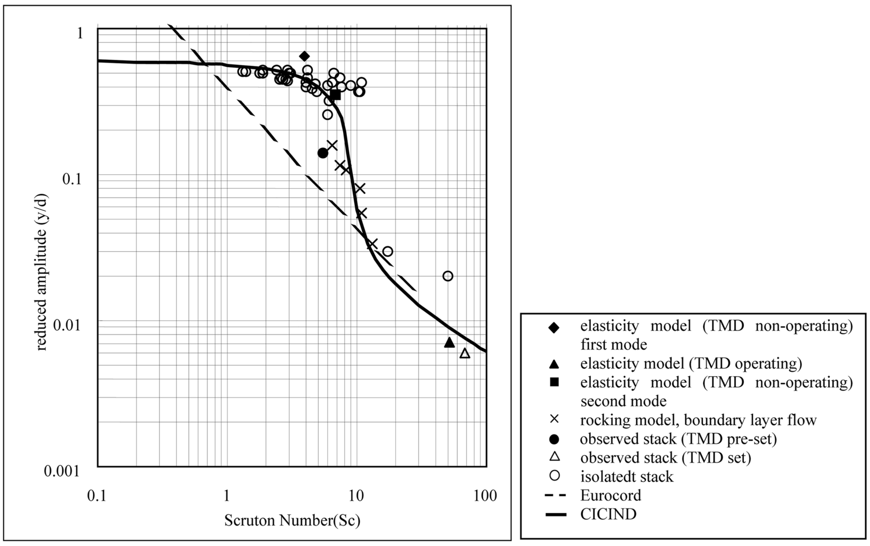


Fig. 13 The reduced amplitude (*y/d*) and Scruton number (*y* = response amplitude (m), *d* = stack shell diameter (m))

for the primary peak.

3) In these wind tunnel tests, the vortex-induced vibration was effectively reduced by TMD.

4) Between the wind tunnel test on one duct tower supported type elastic model and the observed stack (and the independent stack), the relation to Scruton number compared to the amplitude is almost the same curve.

The strength and ductility of steel structures are generally evaluated for strong unsteady forces such as those encountered in earthquakes. For continuous and steady forces to the structure typical of those produced by wind, the general understanding is that these forces produce fatigue damage. Especially in a steel truss structure, such as a tower supported steel stack, more safety is demanded than for general building design. In June 2007, the Building Standard Law in Japan was revised and a high stack over 60 m high is required to be designed to withstand vibration. However, concerning the vibration by the wind, neither the design method nor the evaluation method is yet clear. We think that the response amplitude can be evaluated by using a related chart of the Scruton number and the response amplitude, and further that evaluation due to the accumulated fatigue damage also is useful as a dynamic wind performance evaluation of the tower-supported steel stack.

References

Architectural Institute of Japan (2004), *AIJ Recommendations for Loads on Buildings (2004Edition)*.

Blevins, R.D. (1990), *Flow-Induced Vibration*, 2_{nd} Ed, Krieger Publishing, Florida.

Chmielewski, T. and Gorski, P. (2007), "A Comparative Study of Slender Structures Cross-wind Response for Ruscheweyh's and Vickery-Basu's Models", *12th Int. Conf. of Wind Engineering*, 991-998, Cairns.

Ciesielski, R., Gaczek, M. and Kawecki, J. (1992), "Observation results of cross-wind responses of towers and steel chimneys", **41-44**, 2205-2211.

Dyrbye, C. and Hansen, S.O. (1996), *Wind Loads on Structures*, John Wiley & Sons, Inc.

Eurocord 1 (1995), *Basis of design and actions on structures*.

Gaczek, M. and Kawecki, J. (1996), "Analysis of cross-wind response of steel chimneys with spoilers", *J. Wind Eng. Ind. Aerod*., **65**, 87-96.

Galemann, T. and Ruscheweyh, H. (1992), "Measurements of wind induced vibrations of a full-scale steel chimney", *J. Wind Eng. Ind. Aerod.*, **41-44**, 241-252.

Hansen, S.O. (1998), "Vortex-induced vibrations of line-like structures", *CICIND's 50th Meeting.*

Holmes, J.D. (2007), *Wind loading of structures,* 2_{nd} Edition, Taylor & Francis.

Homma, S., Hanada, N. and Maeda, J. (2007), "Evaluation of the Vortex-induced Vibration of a Tower-supported Steel Stack", *12th Int. Conf. of Wind Engineering*, 991-998, Cairns.

Ikeda, K., Homma, S. and Maeda, J. (2001), "Consideration of tower-supported steel stack's dynamic characteristics", Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan, B-1, 213-214 (in Japanese with English summary).

International Committee on Industrial Chimneys (CICIND) (1999), *Model Code for Steel Chimneys*.

Kawamura, S., Kikuchi, T. and Taniguchi, T. (1992), "Full scale measurement on a triangular tower-supported stack with two flues", *J. Wind Eng. Ind. Aerod*., **41-44**, 2177-2186.

Kawecki, J. and żurański, J.A. (2007), "Cross-wind vibrations of steel chimneys - A new case histry", *J. Wind Eng. Ind. Aerod*., **95**, 1166-1175.

Koten, H. (1998), "Vortex excitation of steel chimneys", *Wind effects on Buildings and Structures*, Riera & Davenport (eds.), Balkema, Rotterdom, 209-219.

Repetto, M.P. and Solari. G. (2007), "Wind-induced fatigue of structures under neutral and non-neutral atmospheric conditions", *J. Wind Eng. Ind. Aerod*., **95**, 1364-1383.

Ricciardelli, F. (2001), "On the amount of tuned mass to be added for the reduction of the shedding-induced response of chimneys", *J. Wind Eng. Ind. Aerod*., **89**, 1539-1551.

Ruscheweyh, H. and Galemann, T. (1996), "Full-scale measurements of wind-induced oscillations of chimneys", *J. Wind Eng. Ind. Aerod*., **65**, 55-62.

Ruscheweyh, H., Langer, W. and Verwiebe, C. (1998), "Long-term full-scale measurements of wind induced vibrations of steel stacks", *J. Wind Eng. Ind. Aerod*., **74-76**, 777-783.

Shimada, T., Hara, H. and Ishizaki, H. (1986), "The efficiency of Helical Strakes for the Suppression of Vortex Oscillation and its application of Steel Stack in Use", *J. Wind Eng., Japan Association for Wind Engineering,* No.27, 3-15, March (in Japanese with English summary).

Simiu, E. and Scanlan, R.H. (1996), *Wind Effects on Structures*, 3rd Ed, John Wiley & Sons, Inc.

Susuki, T., Hanada, N., Ohmori, M., Homma, S. and Maeda, J. (2004), "Wind-induced vibration of tower-supported steel stack", *Proc. of the 15th Conf. on Electric Power Supply Industry.*

Susuki, T., Hanada, N., Homma, S. and Maeda. J. (2005), "Wind-induced vibration control of a 200m-high tower-supported steel stack", *Proc. of the 6th Asia-Pacific Conf. on Wind Engineering (APCWE-VI)*, 2442-2449.

Susuki, T., Hanada, M., Homma, S. and Maeda, J. (2006), "Wind-induced vibration control of a 200m-high tower-supported steel stack", *Wind Struct*., **9**(5), 345-356.

The Building Center of Japan (2004), *The Building Standard Law of Japan*.

Tranvik, P. and Alpsten, G. (2005), "Structural behaviour under wind loading of a 90 m steel chimney", *Wind Struct*., **8**(1), 61-78.

Ueda, T., Nakagaki, R. and Koshida, K. (1992), "Suppression of wind-induced vibration by dynamic dampers in tower-like structures", *J. Wind Eng. Ind. Aerod*., **41-44**, 1907-1918.

Ueda, T., Yoguchi, M., Sunada, H., Yamaguchi, E., Makihata, T., Kashihara, M. and Tatsumi, T. (2000), "Wind Tunnel Tests on Vortex-induced Vibration of Steel Stack", Hitachi Zosen Technical Review, **60**(4), 82-87 (in Japanese with English summary).

Vickery, B.J. (1998), "Wind loads and design criteria for chimneys", *Wind effects on Buildings and Structures*, Riera & Davenport (eds.), Balkema, Rotterdom, 273-296.

CC