DYNAMIC PROPERTIES OF UHV POWER TRANSMISSION TOWERS
- FULL-SCALE TESTS AND NUMERICAL INVESTIGATION -

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SUMMARY

This paper presents results of the full-scale dynamic tests performed on the UHV(1000kV) power transmission test line and associated numerical study. Natural frequency, mode shape and damping coefficient were obtained for the foundation, the tower and the tower-conductor coupled system. Simplified numerical models for each of them are developed which give accurate dynamic properties of them. Using these numerical models and the test results, foundation-tower interaction and the conductor-stringing effect are discussed. Stability of the test line against seismic and wind loads is briefly discussed and the current design method is indicated to be valid.

INTRODUCTION

The increasing power demand in Japan will require UHV(1000kV) power transmission in the near future. As UHV transmission towers are to be of considerable height and to have long arms as well as heavy conductors and insulator assemblies, their dynamic properties may be different from those of existing smaller towers. After the preliminary research on the conceptual design of UHV power lines and the development of various associated devices, a full-scale test line for UHV transmission was built at the Akagi Test Center, CRIEPI, during 1979-1980. Forced vibration tests and numerical studies were performed on this test line for the following objectives: (1) Characterize dynamic properties of the tower foundation, the tower and the tower-conductor coupled system, respectively. (2) Establish and verify their numerical models. (3) Confirm the dynamic stability of the test line against seismic and wind loads.

OUTLINE OF THE UHV TRANSMISSION TEST LINE AND THE DYNAMIC TESTS

The Akagi UHV test line was built midway up the southern slope (about 400 m above sea level) of Mt. Akagi. It extends in west to east direction and consists of two spans (300 m each) and three steel pipe towers. The

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conductors are of 2 circuits, 3 phases, 10-conductor bundles of 810mm^2 ACSR. Tower foundations are reinforced concrete slabs with or without four concrete piles. The towers were designed in accordance with Ref. 1.
Main features and configuration of the test line are given in Table 1 and Fig. 1. Fig. 2 shows the shape and dimensions of Tower No. 2.

Forced vibration tests using an exciter (max. load 10 ton) were performed at three different times during the construction of the line: (1) first, on one of the four foundations of No. 2 tower before tower erection, then (2) upon the erection of No. 2 tower, and (3) after the stringing of the conductors. The exciter was mounted on the slab or at the top of the foundation in case (1), and near the top of No. 2 tower in case (2) and (3). Frequency sweep range was from 0.5 to 25 Hz in (1) and 0.15 to 10Hz in (2), (3). It was varied by 0.01 Hz step near the resonance peaks in order to obtain resonance curves as precisely as possible. Damping coefficient of the tower, however, was very small and it was determined from free vibration curves obtained by suddenly stopping the exciter when the tower was executing resonance vibration.

Responses such as acceleration and strain throughout the towers, tension fluctuation of conductors, earth pressures on the foundation, etc. were measured (max. channel number of simultaneous measurement is 56) and their resonance curves were obtained.

DYNAMIC TEST RESULTS AND NUMERICAL MODELS

Foundation Properties

Fig. 3 shows the displacement resonance curves of the foundation slab obtained from the vertical and horizontal vibration tests. Eigen frequency and damping coefficient are given in Table 2. The foundation has apparently one degree of freedom vertically and two degrees of freedom (sway and rocking) horizontally. Its horizontal properties are the same in both longitudinal and transverse directions. Thus the foundation can be modeled as a spring-mass system with the above mentioned degrees of freedom.

In modeling the foundation as a spring-mass system, restoring forces of the slab and the piles exerted by the surrounding ground were determined by the theory of elasticity and the horizontal restoring force given to the slab by the piles was evaluated by making use of Chang's equation and Vesic's earth reaction equation. Resonance curves of the numerical model thus established are also shown in Fig. 3 which assures the validity of this simplified spring-mass model.

According to the test result and the numerical model, four reinforced concrete piles have a great effect on the properties of the whole foundation system. Ninety percent of the vertical stiffness, fifty percent of horizontal stiffness and seventy percent of rocking stiffness are provided by these piles. The percentage for the damping coefficient is 50, 24 and 37, respectively. (Ref. 2, 3)

Tower Properties

Fig. 4 shows the mode shapes of No. 2 tower obtained from the before and after-stringing tests. Eigen frequency and damping coefficient are given in Table 3. Frequency and damping after the stringing should of course be interpreted as those of the whole tower-conductor coupled system.

Eigen frequencies of the tower alone in the transverse direction are smaller than the longitudinal ones owing to the rotatory inertia of cross-arms. First and second mode shapes are similar for both directions. But longitudinal third mode is accompanied by the torsional mode which is probably due to the anti-symmetric horizontal framework in the right and left cross-arms, whereas transverse vibration has pseudo-third mode as well as normal third mode. Long and heavy cross-arms of UHV towers are evidently the cause of these complicated modes.

The tower-conductor coupled system has far more resonant or eigen frequencies because of its complex structural nature. Of those frequencies given in Table 3, lower frequencies (below 0.66 Hz or 0.55 Hz) corresponds to the conductors' vibration with very little tower displacement, the only exception being 0.27 Hz. The towers vibrate longitudinaly in the first mode shape at 0.27, 0.72, 0.92, 1.96 and 2.14 Hz but the phase relationship among the three towers at these frequencies is different from one another. The same applies to the higher modes and transverse vibration.

Within the limits of the test in which displacement - restoring force relationship of the tower remained linear (maximum tower displacement at the top was 17 cm and 2.2 cm for the first and second mode respectively and the stress produced in the tower legs was below a quarter of allowable stress), damping coefficient was rather small and no definite dependency on the amplitude of vibration was observed. Besides, there is little, if any, difference in the damping coefficient between the tower alone system and the tower-conductor coupled system.

Resonant stress produced in the upper part (panel 3) and the lowest panel of No. 2 tower (panel 15) is shown in Table 4 where stress value is normalized for the unit displacement. Stress in the upper members becomes relatively high in the second and third modes. Towers are generally designed by substituting equivalent static loads for wind loads and cross-sections of members are determined based on the first mode stress distribution. Hence the response calculation would be necessary against earthquakes which may excite second or third mode of the tower.

Modal properties of the tower can be obtained accurately by space truss model as is shown in Table 3. Space truss model for the large tower has, however, too many degrees of freedom to be applied to multi-span line structures. In order to make their analysis feasible, an equivalent beam model was developed. This model is made up of the same number of uniform beam elements as the tower panels whose effective shear area AS or shear coefficient α_S is given in Ref. 5. Fig. 5 shows the comparison of calculated and experimental mode shapes and it is evident that this simplified model gives fairly exact dynamic properties of the tower. Shear coefficient α_S of each beam element is greater than that of circular or rectangular beam and shearing deformation cannot be neglected in obtaining the second and higher model properties. This equivalent beam model, with its reduced degrees of freedom and less computational time (less than 1/10 of space truss model), facilitates the modeling of multi-span tower conductor system. (Ref. 4, 5)

FOUNDATION'S EFFECT ON THE DYNAMIC PROPERTIES OF THE TOWER

Numerical eigen-value study on the foundation-tower system using an equivalent beam model together with a spring-mass foundation model was carried out to show that the foundation-tower system has practically the same modal properties as the base-fixed tower. Parametric study also shows (Table 5) that the tower can be considered rigidly fixed at the base for the wide range of ground rigidities. This is because the tower is very flexible as compared with the ground and energy dispersion through the foundation is negligibly small, which is again the cause of low damping coefficient of the tower. This property makes it possible for us to design tower and foundation independently provided the ground is not extremely soft. (Ref. 5)

CONDUCTORS' EFFECT ON THE DYNAMIC PROPERTIES OF THE TOWER

As is seen from Fig. 4 and Table 3, conductors influence on the tower mode shape and eigen-frequency is small in the transverse direction. In fact, resonant displacements of No. 1 and No.3 towers are below a quater of that of the directly excited No. 2 tower in every transverse mode, while they are of the same order of magnitude as that of No. 2 tower in some longitudinal modes. These imply that the transverse spring constant of the conductor is much smaller than the longitudinal one and conductors and adjacent towers should be appropriately taken into account in calculating the longitudinal dynamic properties of a coupled tower.

Although conductors are essentially geometric nonlinear structures, nonlinear behavior was scarcely perceived in the test. Taking these into account, ten-conductor bundle was modeled as an assemblage of many straight pin-connected bars whose geometric stiffness is linearized by neglecting its dependency on the fluctuating component of the axial force. In this model the out-of-plane vibration is decoupled from the in-plane vibration and coincides with the vibration of the taut string. Eigen-frequency given in Table 3 was obtained using this model. A calculated resonance curve of the test line is shown in Fig. 6 where experimental resonant peaks are also plotted. The curve was given by the pin-connected bar conductor model and equivalent beam tower models. A good agreement between the calculated and tested results is obvious.

Eigen-frequency and frequency response calculation have been made extensively using these numerical models and their accuracy has been confirmed. (Ref. 5, 6, 7)

DYNAMIC STABILITY AGAINST EARTHQUAKES

Response calculation of the test line for El Centro NS wave and an artificially simulated seismic wave shows that it can endure acceleration of 400 gal at the ground level if the instantaneous maximum stress produced in the tower members should be below the yielding points of them.(Ref. 6,7)

No definite earthquake loads are specified in Ref. 1 and these towers are designed so that they can withstand wind load of about 40 m/sec maximum instantaneous speed. This design wind load is of the lowest rank because the area where the towers are constructed is classified as such. Towers constructed in more windy areas are expected to withstand stronger earthquakes.

CONCLUSION

Dynamic properties of the Akagi UHV test line were determined through a series of forced vibration test and simplified but accurate numerical models were established for the foundation, tower and tower-conductor coupled system.

The test results and several numerical studies, including the earthquake and wind-response calculation which is only briefly mentioned in this paper, indicate that UHV transmission towers designed by the current design standard would be safe against seismic and wind loads.

Numerical models herein reported are general enough and applicable to transmission line structures in general. Reduction of degrees of freedom, however, especially of the conductor model without losing its accuracy and simplicity should be realized so that multi-span (more than 3 or 4 spans) coupled system can be analysed efficiently. Earthquake and wind-response records should also be accumulated.

ACKNOWLEDGEMENT

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Table 1. Outline of UHV Akagi Test Line

Item	Tower			Conductor	Foundation (No. 2)	
	No. 1	No.2	No. 3		Slab	Pile
Type or Shape	dead end	tension	dead end	810^{mm} ACSR	Rectangular	Cylinder
Height or Depth (m)	93	97	90		1.5	8
Weight (ton or kg/m)	326	294	285	2.7 kg/m	115	24.5
Area or Diameter (m)					5.2 x 5.2	1.3

Table 2. Eigen-frequency and Damping Coefficient of the Foundation

Mode	f (Hz)	h (%)	Rocking length (m)
Vertical	25.2 (25.5)	0.348 (0.348)	
Horizontal 1st	12.2 (13.9)	0.209 (0.256)	12.1 (10.4)
Horizontal 2nd	20.0 (20.7)	0.200 (0.316)	- 0.492(-0.613)

( ) = Calculated value where the foundation is modeled as a spring-mass system

Table 3. Eigen-frequency and Damping Coefficient of No.2 Tower

System	f, h / Mode	Longitudinal direction f (Hz)	Longitudinal direction h (%)	Transverse direction f (Hz)	Transverse direction h (%)
Tower alone prior to stringing	first	0.80 (0.76)	0.34	0.78 (0.75)	0.34
	second	2.68 (2.66)	1.18	2.47 (2.54)	0.63
	third	5.50 (5.33)		4.20 (4.84)	
Tower-conductor coupled system after stringing	Conductor mode	(0.27 (0.27))		0.17 (0.18)	
		0.33 (0.30)		0.34 (0.35)	
		0.43 (0.39)		0.37	
		0.56 (0.53)		0.51 (0.50)	
		0.66		0.55	
	Tower first	0.72 (0.75)		0.61 (0.57)	
		0.92 (0.94)	0.26 - 0.36	0.71	0.29
		1.96		0.83 (0.74)	0.70
		2.14 (2.02)		0.92 (0.99)	
	Tower second	2.54 (2.60)	0.31 - 1.75	2.39 (2.23)	0.75
		3.37 (3.15)		2.74 (2.85)	0.60 - 0.71
		3.54 (3.23)	0.41		
	Tower third	4.31			
		5.76			

( ) = Calculated value where Tower No.2 : Space truss model
Tower No.1 & No.3 : Equivalent beam model
Conductor : Divided into 30 parts along the span
Tower base : Fixed

Table 4. Member Stress (kg/cm^2) by Unit (1cm) Displacement of the Excited Point

System		No.2 Tower Alone			No.2 Tower in the Coupled System	
Mode		1st Mode	2nd Mode	3rd Mode	1st Mode	2nd Mode
Fre-quency	longi	0.80 Hz	2.68 Hz	5.50 Hz	0.92 Hz	3.37 Hz
	trans.	0.78 Hz	2.45 Hz	5.35 Hz	0.71 Hz	2.39 Hz
upper panel (3)	main posts	9	70	396	8	143
		8	124	95	9	158
	brace	11	66	194	10	125
		11	155	1083	15	247
lowest panel (15)	main posts	31	84	83	22	82
		23	94	53	24	103

Upper and lower value correspond to longitudinal and transverse vibration, respectively.

Table 5. Eigen-value of No.2 Tower on the Various Ground

Mode / Ground	Shear Wave Velocity Vs (m/sec) 170	100	80	60	Fixed at the Base	Experimental Value
First	0.78	0.77	0.76	0.75	0.78	0.78
	0.34	0.39	0.45	0.58	-	0.34
Second	2.48	2.45	2.43	2.40	2.48	2.45
	0.63	0.73	0.83	1.13	-	0.63
Third	5.33	5.29	5.23	4.48	5.33	5.35
	3.40	3.67	4.65	13.53	-	3.40

Upper values mean eigen-frequency (Hz)
Lower values mean damping coefficient (%)
Vs = 170 m/sec corresponds to the test site at Akagi

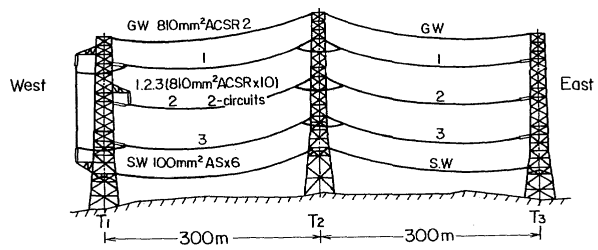


Fig. 1 Schematic Figure of UHV Akagi Test Line

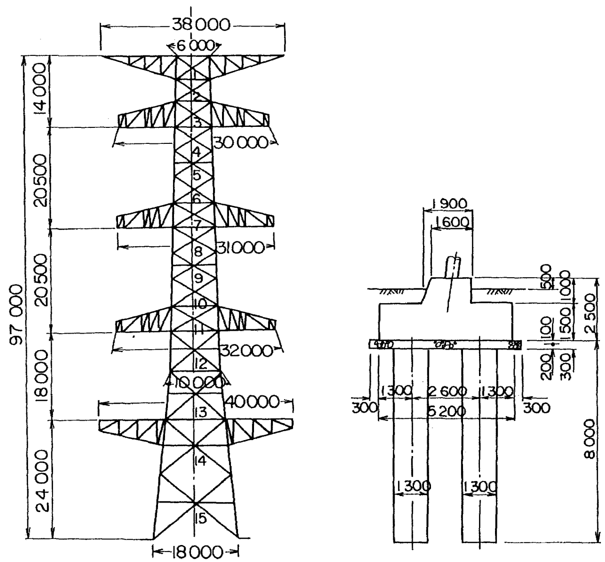


Fig. 2 Dimension of No. 2 Tower and its Foundation

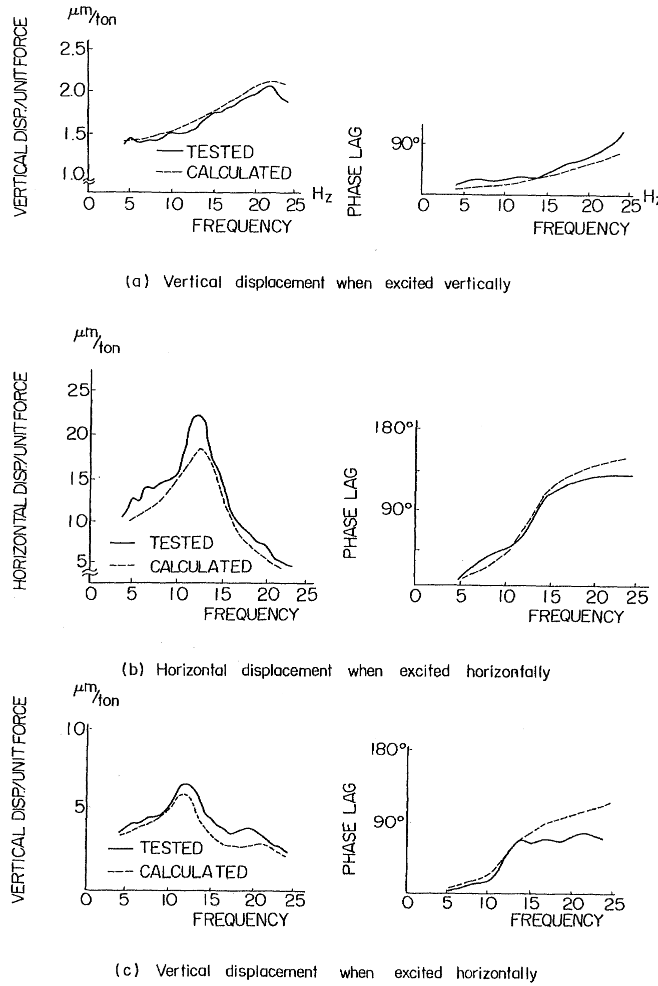


Fig. 3 Resonance curves of the foundation

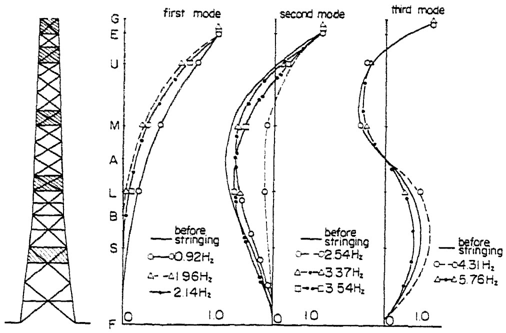


Fig. 4 (a) Longitudinal mode before and after stringing

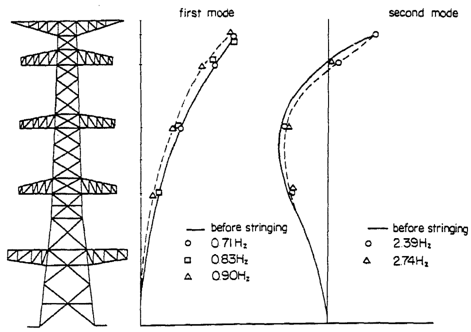


Fig. 4 (b) Transverse mode before and after stringing

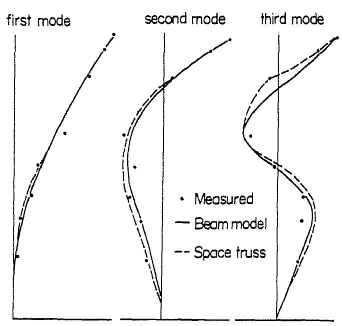
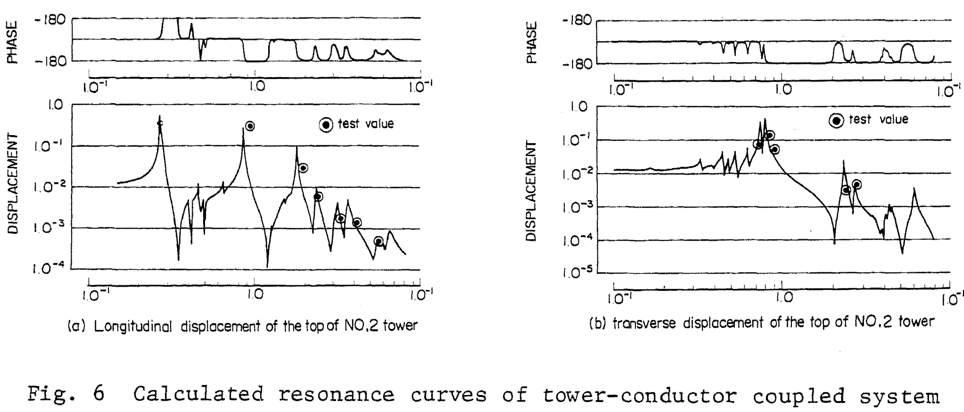


Fig. 5 Longitudinal mode shape



(a) Longitudinal displacement of the top of NO.2 tower

(b) transverse displacement of the top of NO.2 tower

Fig. 6 Calculated resonance curves of tower-conductor coupled system