EXPERIMENTAL STUDY ON SEISMIC PERFORMANCE OF STEEL TOWERS FOR MICROWAVE ANTENNAS AND THE DEVELOPMENT OF SEISMIC PERFORMANCE EVALUATION METHOD

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SUMMARY

This paper describes research carried out in order to develop a seismic performance evaluation method for existing latticed steel antenna towers. Low cyclic loading tests on an existing steel tower and scale models, and vibration tests on an existing tower and for scale models were performed. Earthquake response characteristics of steel towers atop buildings was also studied by analysis. A seismic performance evaluation method and its application is also presented.

INTRODUCTION

Since 1954, NTT (Nippon Telegraph & Telephone Public Corporation) constructed more than 2,000 steel antenna towers throughout the Japanese islands. Almost all of these towers are of truss types consisting of shaped steel with bolted joints as shown in Fig.1. As steel towers are lighter in weight, compared to reinforced concrete buildings, their structural design was mainly concerned with withstanding wind loading acting on the tower. Seismic safety for steel towers was considered sufficient. However, the off-Tokachi earthquake (1968) caused some damage, such as slight deformation in structural members and framework joints. Since then, NTT has carried out extensive studies on the seismic performance of steel towers (Ref.1). As a result, it become clear that in some cases, seismic force exceeds the wind force according to the periodic ratio of the steel tower to its supporting building, and the tower weight distribution in the vertical direction. In 1972, NTT developed a structural design manual on the basis of these study results and since that time, structural design for steel towers has been carried out accordingly. However, approximately 1,000 towers were constructed prior to the application of this structural design manual. It is suspected that some of these might have insufficient aseismic ability. For this reason, NTT intends to evaluate the aseismic ability of these steel towers. If the towers do not have sufficient aseismic ability, NTT plans to reinforce them. In order to judge seismic safety, it is necessary to practically calculate the ultimate strength of the tower and the seismic force on it. However, there are few investigation on the load-deflection characteristics of steel towers and seismic force acting on it. Therefore NTT has carried out various loading tests, vibration tests, and a study of seismic response analysis. A seismic performance evaluation method for existing steel towers has been developed on the basis of the these results.

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LOADING TESTS AND VIBRATION TESTS ON STEEL TOWERS

Loading tests and vibration tests were carried out in order to clarify the following items; i) Ultimate strength for each component. ii) Load-deflection characteristics due to low cyclic loading. iii) Fracture behavior. iv) Vibration characteristics (Ref. 2)

Low Cyclic Loading Tests on Existing Tower

Test model and procedure

The existing twenty meter-high steel tower shown in Fig.3 was used. Tests were carried out by measuring the horizontal deflection and the axial strain on the structural members, as well as, slip on framework joints.

Test results

The relation between pulling load "P" and the resultant horizontal deflection " " is shown in Fig.4. The rigidity of steel tower degrades by the slip of the framework joints in the early loading steps (about 50%). With decreasing load, the rigidity becomes equal to the initial rigidity. As a result, the load-deflection relation depicts a hysteresis loop with cyclic loading. Axial strain exists in every member at each loading step in the elastic range. On the other hand, the slip of framework joints occurs with increasing load. The test was carried out, until the load value reached 1.5 times the allowable horizontal load, calculated according to the design standard for steel structures of AIJ (Architectural Institute of Japan). Buckling did not occur with such maximum load.

Low Cyclic Loading Tests on 1/2.5 Scale Models

Test models and procedure

Test models are partial models of standard latticed steel towers. Model dimensions are shown in Table 1. The model is set laterally, and secured to the reaction wall. Alternate loading was carried out using a loading truss in 45° direction (Fig. 5). Relative horizontal deflection, member strain, and the slip deformation of each joint were measured.

Test results

The rigidity of each model degrades by the slip of joints in the early loading steps. As a result, the load-deflection curves shown in Fig.6 were obtained. Varying the number of bolts does not bring about a difference in the rigidity. The maximum strength of each model was determined by the buckling of the main member. The buckling stress indicates 1.4 times allowable compressive stress according to the design standard for steel structures of AIJ. No difference in maximum strength can be seen corresponding to the number of bolts. After the occurrence of buckling, the load value degrades, and the horizontal deflection goes up. After the load settles to a certain value (about 70% for A1 and A2, 80% for B), the load value remains constant in spite of increasing the deflection. As a result, sudden collapse doesn't occur. A neutral axis shifts according to the buckling of the main member in the compression side and a new equibrium system is constructed. Fig.7 shows the relation in this process between the axial force of the main members and the load. It is confirmed that, in the ultimate fractural stage, the fracture of bolts at main member joints in the tension side for model A1 and fracture of main members for model A2 are in the same relative position. Such results are based on the difference in the number of bolts.

Monotric Loading Tests on Steel Tower Components

Test models and procedure

Scale (1/2.5), triangular pyramid truss models were used. Each model differs in its slenderness ratio of the main members and its section of stiffening members. Fig.8 shows test model dimension and the loading procedure. The model is secured at points B and C to fastners which can rotate round the D-D' axis. Test model was loaded in the direction of the main member axis at point A. Measurements were carried out for the relative displacement at point A in the loading direction, the deflection on each framework joint in the out-of-plane direction, and the axial strain in each member.

Test results

Table 2. shows the maximum load (Pmax) and the buckling portion. The Pcr value shown in Table 2 indicates the calculated maximum compressive strength based on the design standard for steel structures of AIJ. It is confirmed that Pmax values, except TYPE-III, exceeds the Pcr value. The value of Pmax/Pcr gradually degrades according to the increase in the main member section area. When the main member has the same section area, the Pmax value in the model with single-stiffening members is degrading 7% to 14%, as compared to the model with built-up-stiffening members.
The rigidity of stiffening-members comparatively degrades according to the increase in section area of main members, resulting in decreased buckling compressive strength.

Vibration Tests

Test procedure

Forced-vibration tests were carried out, with horizontal sinusoidal wave excitations, using a vibrator, placed on top of the antenna-deck of the tower shown in Fig.3. The resultant tower displacement was measured at the tower top and the tower base.

Test results

The resonant curves at the tower top are shown in Fig.9. Resonant frequency and the damping coefficient acquired using the resonant curve were respectively changed from 3.0 Hz to 2.6 Hz and 1.7% to 3.8% with the increase of the exciting force. Also a vibration test for 1/10 scaled model, using a shaking table was carried out. The buckling of the main member was achieved. It was assured that buckling in vibration test, corresponds to that in low cyclic loading tests.

EARTHQUAKE RESPONSE ANALYSIS FOR A STEEL TOWER ATOP A BUILDING

In order to determine the earthquake response characteristics for a tower atop a building, earthquake response analysis was studied by substituting the tower for an equivalent one mass model system.

Earthquake Motion on the Tower Base

Tower weight is considerably lighter than that of supporting building (1/50 - 1/1000). Therefore, it is practically feasible to analyze earthquake responses for towers atop buildings, using resultant response waves on the roof of the buildings. The response waves on the roof was acquired by plastic response analysis, using standard exchange building models and artificial earthquake waves. The artificial earthquake waves have velocity response spectra as shown in Fig.10. Examples of the acquired response waves on the roof are shown in Fig.11.

Applied Model System for Analysis

The tower was substituted for one mass dynamic system with the idealized hysteretic characteristics shown in Fig.12. Such characteristics were acquired by the resultant load-deflection curves by the cyclic loading test. This one mass dynamic system can be placed for a elastic one mass dynamic system with an equivalent elastic stiffness "Ke" and an equivalent coefficients "he". The maximum acceleration response value for the model system shown in Fig. 12 was calculated using the resultant waves on the roof. Fig. 13 shows the comparison between this maximum response value and the acceleration response spectra on the resultant waves. In Fig.13, an equivalent period "Te" calculated by Eq.1 is used as the period of the model system.

$$Te/T = 0.5 + (1/\gamma-1)/\pi + 1/2\sqrt{\gamma}, \quad T = 2\pi\sqrt{M/K} \qquad (1)$$

On the basis of this consideration, the earthquake response for the model system can be estimated by the response of the elastic model system with the equivalent period "Te" and the equivalent damping coefficient "he".

Earthquake Response Characteristics for a Steel Tower

Based on above mentioned consideration results, it is possible to clarify the earthquake response characteristics for a steel tower, by the response using the equivalent elastic one mass model system. Fig.14 shows the magnification factor of the acceleration response, using the building models and the artificial waves as parameters, corresponding to the periodic ratio (Te/ T1, T1: Fundamental period for the building). It can be seen that the building models and the kinds of artificial earthquake waves have little influence on the magnification factor. Also it is possible to estimate the earthquake response characteristics for a steel tower, utilizing the equivalent period for the tower and the fundamental period for the building.

SEISMIC PERFORMANCE EVALUATION METHOD AND ITS APPLICATION

Seismic Performance Evaluation Method

The decision criteria for judging a tower's seismic safety is determined as follows: When subjected to very rare severe earthquakes (250-400 gal maximum ground acceleration), slight damage, if any, is caused to structural members and the joints. A safety judgement is reached based on a comparison between the ultimate strength of the tower and the seismic force. Fig.15 shows the seismic performance evaluation procedure. Strength index (C) is calculated on the basis of the ultimate strength of the structural members and their joints, and the vertical dead load distribution acting on the tower is calculated, using the seismic coefficient ratio shown in Fig.16. The buckling strength of structural members and the ultimate strength of framework joints are determined by the above mentioned loading test results. Seismic force index "Et" is calculated by Eq.2, according to the fundamental period of the tower and building, seismic activity in the construction area, and the importance degree.

$$Et = Z \cdot R \cdot I \qquad (2)$$

where, Z: Zone seismic activity index $Z = 0.7 \sim 1.0$
R: Response characteristics index (See. Fig.17)
I: Importance index, $I = 1.0$ for an ordinary steel tower

Response characteristics index of the tower atop a building is determined by the above mentioned earthquake response analysis results. The acceleration response magnification factors, corresponding to the value of R1 or R2 (response characteristics index) are shown with dotted lines in Fig.14. The tower whose strength index is in the range of ($Et1 < C < Et2$) is judged by a consideration of its ultimate strength degree of the framework joints, the strength degree of the stiffening members, and the tower shape. Seismic countermeasures are applied to the towers classified into class III or IV.

Application

Fig.18 shows actual towers damaged by the off-Tokachi earthquake in 1968. Damage to each tower was identical in regard to the structural members of the first unit on the tower base. Fig.19 shows the evaluation results for the above mentioned tower. It can be seen that the strength index value for the first unit is under the seismic force index. Good correlation between the evaluation results and the actual damage degree was obtained.

CONCLUSIONS

i) Buckling stress of steel tower component is proved to be 1.2 to 1.4 times allowable compressive stress according to the design standard for steel structures of AIJ.
ii) The buckling of main member does not bring about sudden collapse in 45° direction.
iii) The rigidity of steel towers with bolted joints degrades by the slip of joints in the early loading steps. As a result, the load-deflection relation depicts a hysteresis loop with cyclic loading.
iv) It is possible to estimate the earthquake characteristics for a steel tower, utilizing the equivalent period for the tower and the fundamental period for the building.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to Prof. M. Fujimoto, Prof. T. Suzuki of Tokyo Institute of Technology and Assoc. Prof. H. Akiyama of Tokyo University for the suggestions and co-operation in this study.

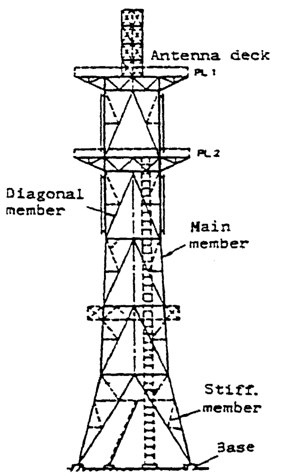

Fig.1 Steel Antenna Tower

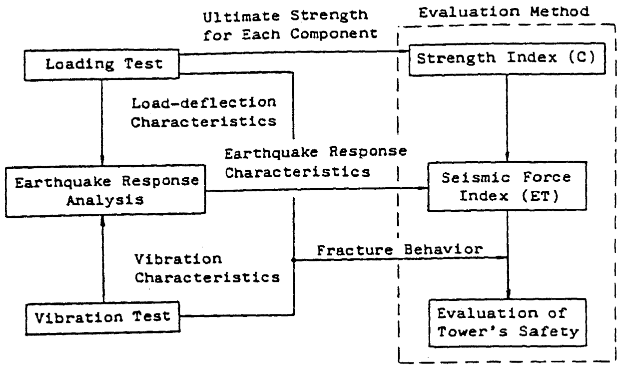

Fig.2 Relationship of test results,response analysis and seismic performance evaluation method

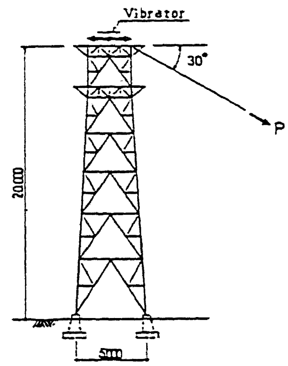

Fig.3 Tower Shape

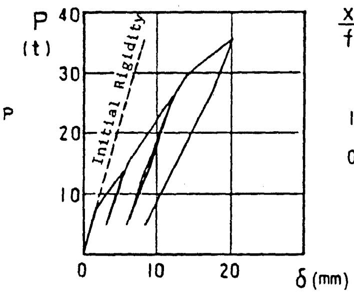

Fig.4 Load-deflection Relationship

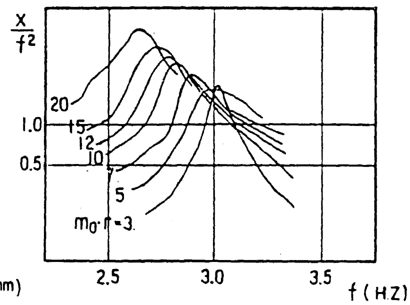

Fig.9 Resonant Curves

Table.1 Model Dimensions

MODEL TYPE		ST.	MEMBER	SECTION	λ	JOINT
A-1 (A-2)	(m)	3	a	L-100x100x10	52	22(14)-M12
			b	2L-60x60x4	64	7(5)
			c	2L-50x50x6	80	6(4)
		2	a	L-100x100x10	64	22(14)
			b	2L-60x60x4	80	7(5)
			c	2L-50x50x4	79	6(4)
		1	a	L-100x100x10	54	22(14)
			b	2L-60x60x4	69	7(5)
			c	2L-50x50x4	110	6(4)
B	(m)	3	a	L-100x100x10	52	22-M12
			b	2L-60x60x4	64	7
			c	2L-50x50x6	80	6
		2	a	L-100x100x10	62	22
			b	2L-60x60x4	73	7
			c	2L-50x50x4	79	6
		1	a	L-100x100x10	58	22
			b	2L-60x60x4	71	7
			c	2L-60x60x4	65	7

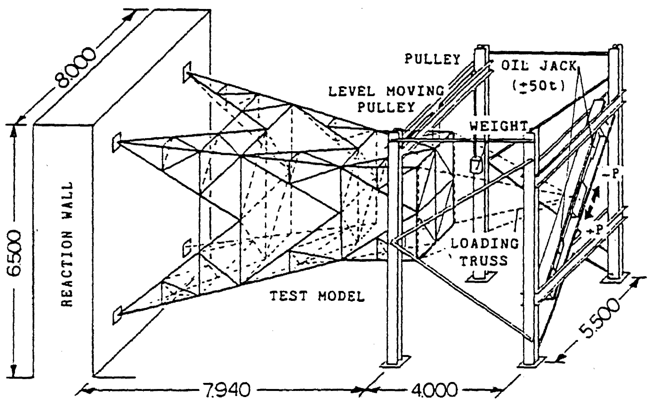

Fig.5 Loading Apparatus

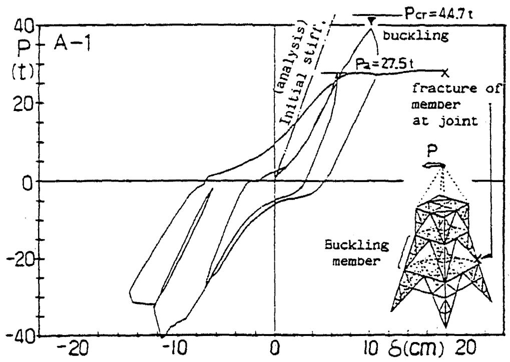

Fig.6 Load-deflection Curves

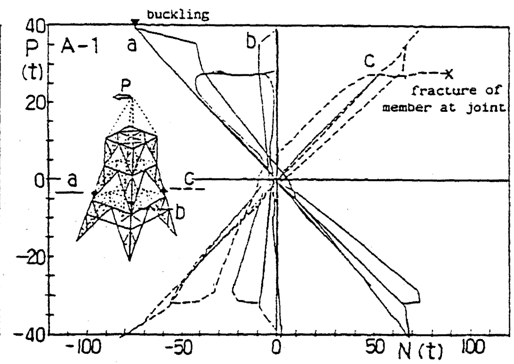

Fig.7 Axial Force of Main Members

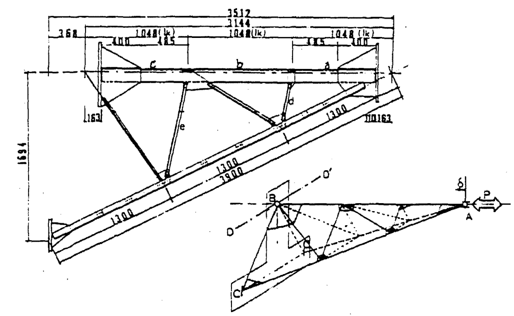

Fig.8 Test Model & Loading Apparatus

Table.2 Test Results

TYPE	MAIN MEMBER SECTION	MAIN MEMBER λ	DIA. MEMBER SECTION	DIA. MEMBER λ	P_{max} (t)	P_{cr} (t)	BUCKLING PORTION
I-S	L-90·90·7	59	2L-45·45·4	96	51.7	40.1	b
I-D	L-90·90·7	59	2L-45·45·4	96	55.5	40.1	b
II-S	L-100·100·10	54	2L-50·50·4	85	69.4	66.0	a
II-D	L-100·100·10	54	2L-50·50·4	85	80.5	66.0	a
III-S	L-130·130·15	41	2L-60·60·5	70	118.3	141.5	a
III-D	L-130·130·15	41	2L-60·60·5	70	129.3	141.5	a

STIFFENING MEMBER : I-III-S , L-30·30·3
I~III-D , 2L-30·30·3

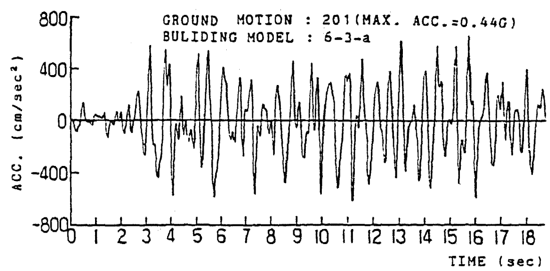

Fig.11 Response Wave at Roof Floor

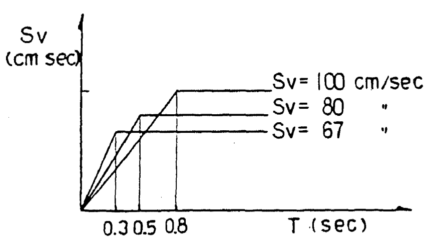

Fig.10 Velocity Response Spectrum

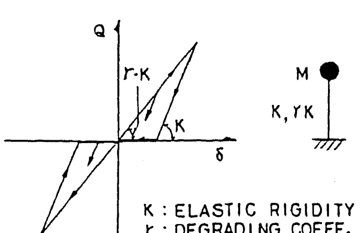

Fig.12 Applied Model System

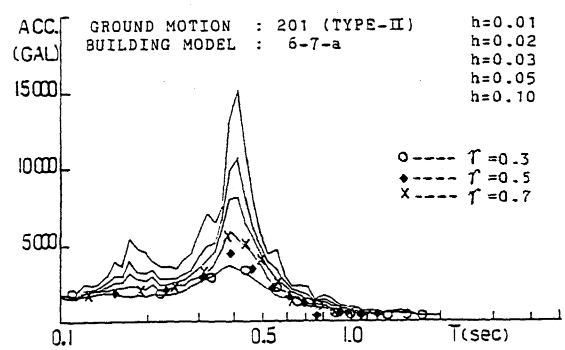

Fig.13 Maximum Acceleration Response

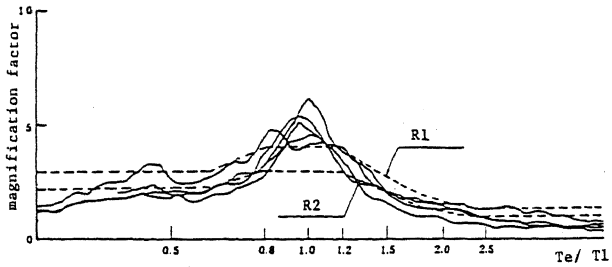

Fig.14 Magnification Factor of Acceleration Response

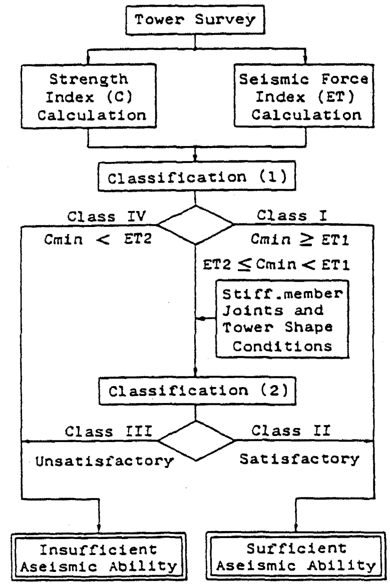

Fig.15 Seismic Performance Evaluation Procedure

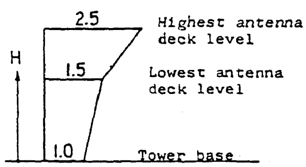

Fig.16 Shear Coefficient Ratio

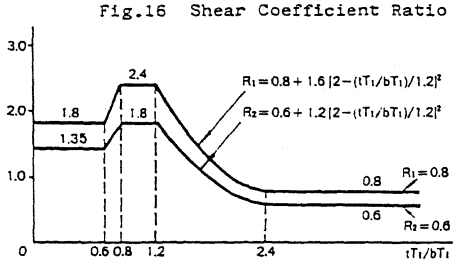

Fig.17 Response Characteristics Index

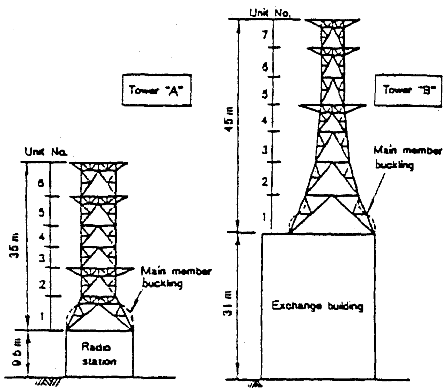

Fig.18 Damaged Steel Towers

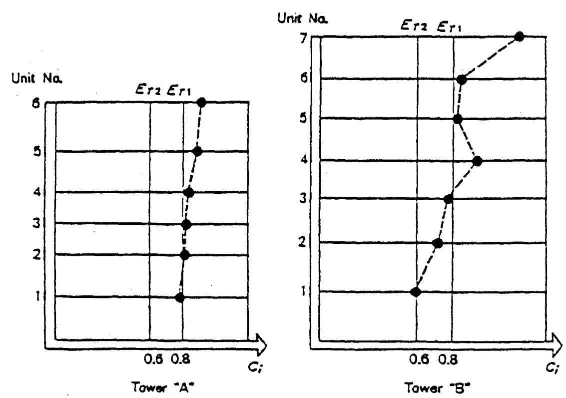

Fig.19 Evaluation Results

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