RESEARCH ON ASEISMIC DESIGN OF STEEL TOWER CONSTRUCTED ON THE TOP OF BUILDING

by

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SYNOPSIS

Earthquake response observation, examination of observed records and simulation analysis for the 200-meter high steel tower constructed on the top of a 15-story building were performed. General characteristics of seismic response and aseismic design data for such type structure were obtained by using the case study vibration analysis.

INTRODUCTION

It is known that a tower atop a building is subject to the influence of vibration of the building and shows peculiar response during earthquakes. The authors, taking an aim at obtaining data for aseismic design of steel towers of such kind, have carried out earthquake observations on a 200-meter high steel tower built atop a 15-story office building. The observation work began in 1973 and records of 39 earthquakes have been obtained up to 1978.

OUTLINE OF TOWER, BUILDING AND SOIL PROFILE

The outline of the steel tower, building and soil profile are as follows: The steel tower is composed of two elements, inner and outer towers. The inner tower is of a rigid frame structure composed of tripod steel-pipe columns, and the outer tower is of braced rigid frame structure which supports the inner tower at its upper and lower portions. The building is a 70.0-meter long, 32.4-meter wide, 15-story building of steel-framed reinforced concrete (SRC) structure with a two-story penthouse and a five-story basement. The steel tower stands on wall girders placed on the penthouse. The building site located in Shinbashi, Tokyo, where alluvial silt layers are accumulated to the depth of 15 to 22 meters from the ground level, and beneath them is diluvial formation made up of the " Tokyo gravel layer " and the " lower Tokyo layer ". The building is supported by the mat foundation on this diluvial formation. The general view of the steel tower and the building is shown in Photo. 1, and their structural outline and soil profile are shown in Fig. 1.

EARTHQUAKE OBSERVATION SYSTEM AND OBSERVED RECORDS

The observation was made at six points as follows; GL+200m position and GL+101.4m position on the tower, the penthouse, the 13th floor, the 1st floor, and the 5th basement floor of the building. Specifications of the observation instruments are shown in Table 1.

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The epicenter locations of observed earthquakes in the Kanto area are shown in Fig. 2, and the relationships between epicentral distance and focal depth are shown in Fig. 3.

EARTHQUAKE RESPONSE OF TOWER AND BUILDING

Natural Periods of Tower and Building: Spectrum analyses were conducted on the observed seismic waves in order to confirm the natural periods of the steel tower and the building. As a result, it was recognized that the periods for each vibration mode were changed according to the acceleration intensity. It was also found that this change appears conspicuously in the modes of excitation of the building. Table 2 shows the mean value of these periods.

Magnification of Acceleration Response: As for the response of 39 earthquakes observed, the maximum acceleration values of each point were normalized with those observed at the first floor, and their vertical distributions are shown in Fig. 4. The mean and the standard deviations of the normalized values are shown in Fig. 5. It is seen that the response magnification of the longitudinal component (X) differs from the transversal one (Y), and that this difference varies considerably between earthquakes. It is also seen in the figure that the mean of normalized values at the top of building is about two times larger, and about 20 times larger at the top of tower than that at the first floor.

Variance of Response Magnifications by Epicenter Locations: In case of the steel tower, the seismic response characteristics vary according to the frequency characteristics of seismic waves. The response magnifications were classified into the following two categories whose frequency characteristics of seismic waves are considered different from each other:

i) Classification by epicentral distance;
   0 - 40km, 40 - 150km, 150km and longer (See Fig. 2)

ii) Classification by incidence angle
   The incidence angle is defined by the angle between the horizontal plane and a connecting straight line of the observation site to hypocenter; 0°-30°, 30°-60°, 60°-90° (See Fig. 3)

The distribution of the maximum acceleration response magnifications as classified by the epicentral distance is shown in Fig. 6, and that as classified by the incidence angle is shown in Fig. 7. The main earthquake records obtained at the base of the building (38 records) were classified likewise, and response spectra were calculated. The whole spectra are shown in Fig. 8, their mean values and standard deviations in Fig. 9, the mean response spectra as classified by the epicentral distance in Fig. 10, and the mean response spectra classified by the incidence angle in Fig. 11.

It was revealed that through the result of this earthquake observation, the tower response tends to be large by the seismic waves which include long period components. The tower is apt to vibrate with the third vibration mode of the whole coupled system, which corresponds to the second vibration mode of the tower alone and this vibration period is close to the natural period of the building.

EARTHQUAKE RESPONSE SIMULATION ANALYSIS

In the Off-Miyagi Prefecture Earthquake (M=7.4) which occurred on June 12, 1978, it was also felt in Tokyo by the 4 JMA's intensity scale. At this coupled system of tower and building, the greatest acceleration value was recorded since the start of the observation. Then the earthquake response simulation analysis was conducted.

Vibration Models: The inner and the outer towers were represented by a model of 18 lumped masses and bending shear spring, and also a model of 16 lumped masses and shear spring was considered for the building part. In this model system, the inner and outer towers are linked at the upper and lower points of outer tower with horizontal springs, and rocking springs that resist overturning moment are put into the foot of the outer tower. As for the damping, strain energy type damping coefficient by part of h=0.005 is given to the tower, h=0.03 to the building and h=0.05 to the rocking spring. The vibration analysis model and the first to third vibration mode shapes are shown in Fig. 12.

Results of Analysis: The response analysis was conducted by using the seismic waves observed at the first floor of the building as input. The comparison of observed acceleration response waves and calculated ones are shown in Fig. 13, and that of response spectra is shown in Fig. 14. It can be said, from these figures, that a good agreement between calculation and observation are seen, and the above-mentioned modeling and the setting-up of damping coefficients are adequate. Consequently, it is confirmed that there is a 13cm single amplitude at the top of tower and a 3cm displacement at the top of building.

VIBRATION ANALYSIS OF TOWER-BUILDING SYSTEM BY CASE STUDY MODELS

Referring to the vibration model employed in the earthquake response analysis, case study models of the tower-building coupled system were made and their vibration analyses were performed. A three-story office building of reinforced concrete (RC) structure was plotted, upon which steel towers of seven different period ratios (Tower 1st period , T_t /Building 1st period ,T_b) ranging from 0.5 to 3.0 were installed. In this case, the weight ratio (Tower weight/Building weight) was fixed at 2 percent. The analysis models consisted of three lumped masses for the tower and also three lumped masses for the building, and a bending shear spring for the tower, a shear spring for the building were considered, and then a rocking spring was put at the basement of the tower. The vibration analysis model is shown in Fig. 15. The analysis of the model was conducted by giving damping coefficient of h=0.01 to the tower and h=0.05 to the building. The damping coefficient is of the same strain energy type as in the above-mentioned case.

Transfer Function: The transfer functions at the top of tower from the building basement are shown in Fig. 16.

Earthquake Response Analysis: Earthquake response analyses were performed by giving four seismic waves as shown in Table 3, to the case study models. The response spectra of input seismic waves are shown in Fig. 17. All of input waves were normalized by the maximum acceleration set by 100 gals.

The distribution of the maximum acceleration and the shear force coefficient for each model, obtained from the response analysis, are shown in Fig. 18 and Fig. 19, respectively.

Data for Aseismic Design: The four seismic waves used in the above-mentioned earthquake response analysis, when put together, have spectrum peaks in a fairly wide range of period. If the envelope of this maximum response value is used for an aseismic design, it can be tolerated to disregard the effects of the soil profile and the difference of period for the building. For instance, in case where elastic design is conducted at the input maximum acceleration value of 250 gals, the above-mentioned shear force coefficient multiplied by 2.5 can be used as earthquake load. The relation between shear force coefficient and the period ratio is as shown in Fig. 20.

CONCLUSION

The size of tower and building adopted in this analysis is limited to a certain extent, and furthermore, the analysis itself is submitted to only an elastic stage. However, by knowing the period ratio between the tower and the building, and then by considering the coupled vibration, it is thought possible to grasp the general feature of earthquake response for such a system.

ACKNOWLEDGEMENT

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2) Joseph Penzien and Anil K. Chopra : "Earthquake Response of Appendage on a Multi-Story Building" ; 3WCEE, 1965, New Zealand.

Photo. 1 SHINBASHI Building and Tower

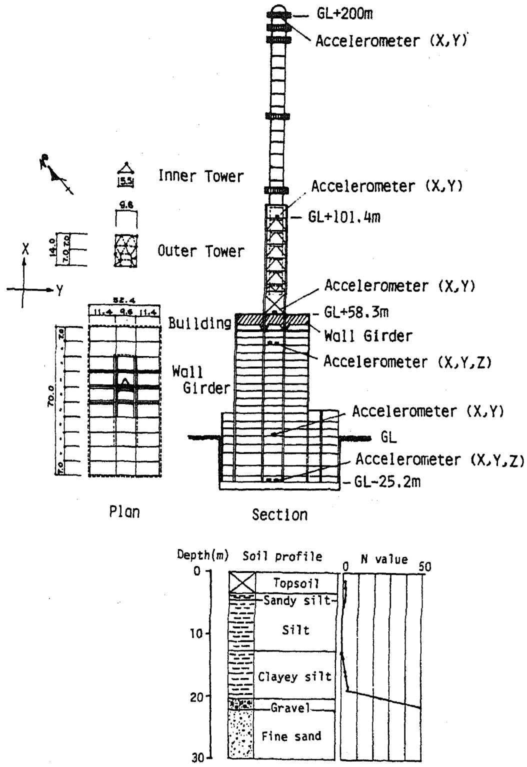


Fig. 1 Structural Outline, Accelerometer Position and Soil Profile

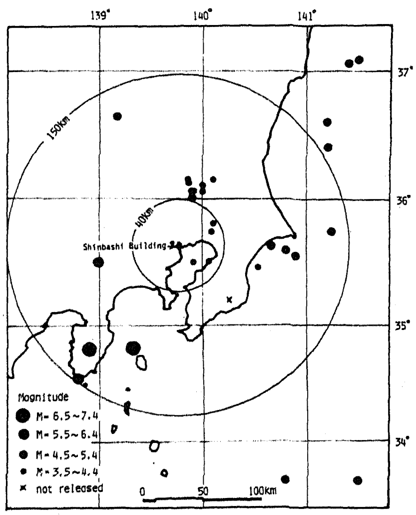


Fig. 2 Locations of Observed Earthquake Epicenters

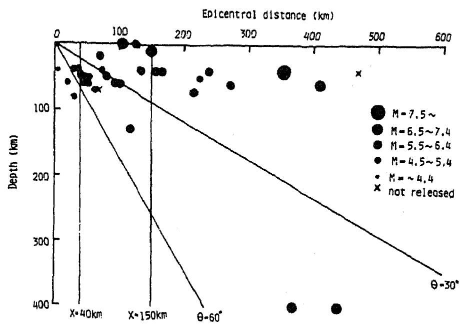


Fig. 3 Relationship between Epicenter Distance and Forcal Depth

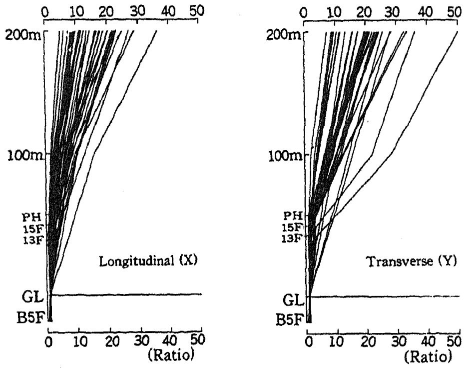


Fig. 4 Observed Max. Acceleration Ratio Relative to Building 1F

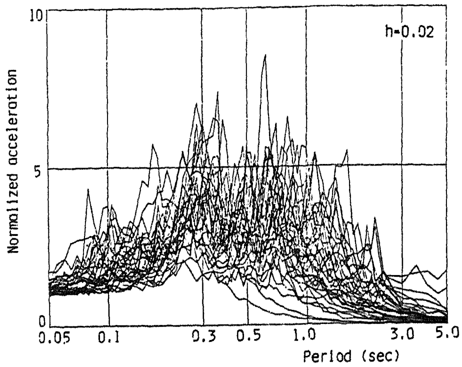


Fig. 8 Acceleration Response Spectra (B5F)

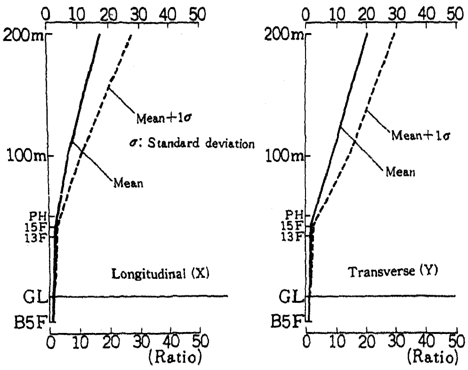


Fig. 5 Mean of Observed Max. Acceleration Ratio

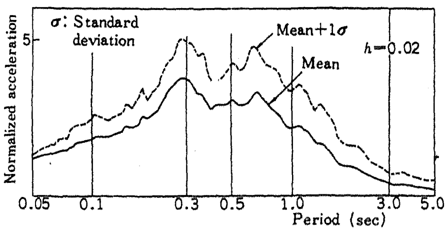
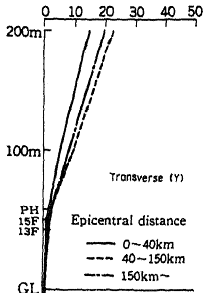


Fig. 9 Mean of Response Spectra (B5F)



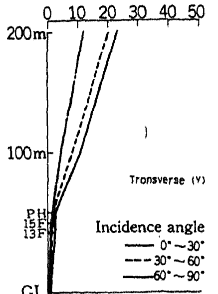


Fig. 6 Mean of Observed Max. Acceleration Ratio (Epicentral Distance)

Fig. 7 Mean of Observed Max. Acceleration Ratio (Incidence Angle)

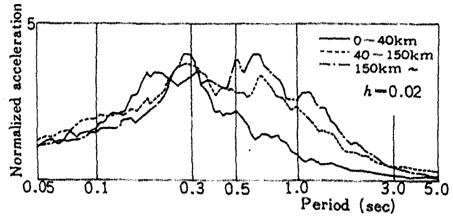


Fig. 10 Mean of Response Spectra (Epicentral Distance)

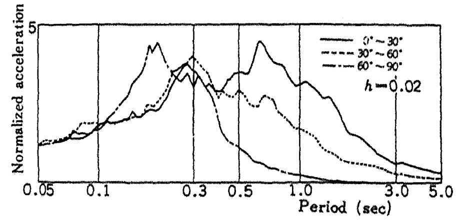


Fig. 11 Mean of Response Spectra (Incidence Angle)

Table 1 Specifications of Observation System

ACCELEROMETER	Velocity Feedback Type Servo Accelerometer Natural Frequency ; 3Hz Damping ; h=30. Bandwidth ; 0.3 - 30Hz Range ; Tower - 1 G(F.S) Building - 0.1 G(F.S) Building Base - 0.3 G(F.S)
RECORDER	FM Recording Type Analog Data Recorder : 14 ch'
TRIGER	Common with SMAC Type Accelerometer (BLDG.15F) Start Level ; 5 gal , UD

Table 2 List of Observed Natural Period T(sec)

Direction	1st	2nd	3rd	4th
Longitudinal (X)	2.55	1.11	0.71	0.43
Transverse (Y)	2.67	1.26	0.77	0.45
Remarks	Tower 1st	Bldg. 1st	Tower 2nd	Tower 3rd

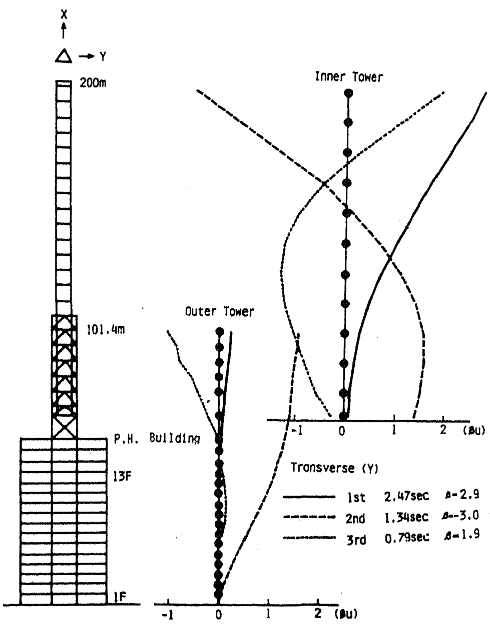


Fig. 12 Vibration Model and Vibration Modes

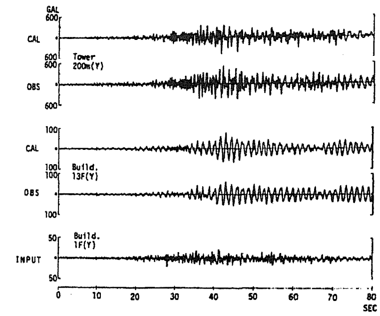


Fig. 13 Comparison of Calculated Acceleration Waveforms and Observed Waveforms

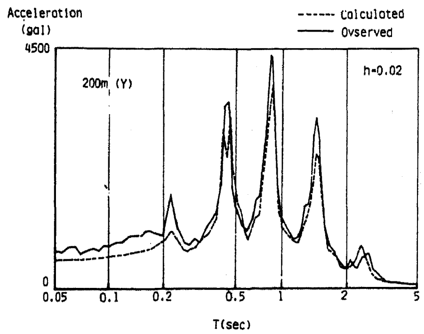


Fig. 14 Comparison of Calculated Response Spectrum and Observed Result

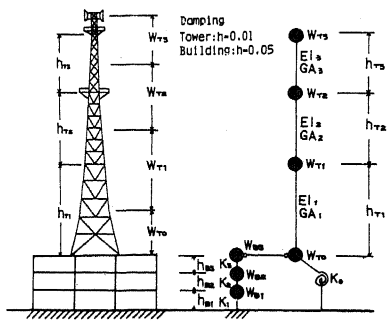


Fig. 15 Vibration Model of Case Study Analysis

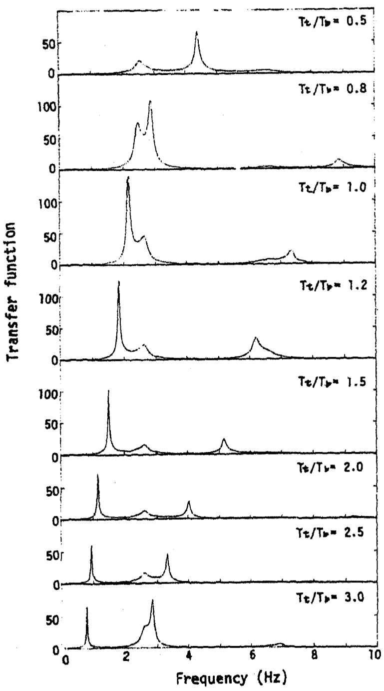


Fig. 16 Transfer Function from Building basement to Top of Tower

Table 3 List of Input Earthquake Waves for Case Study Analysis

No.	Location-Component	Date, Epicenter	Legend
1	El Centro-NS	May 18, 1940 Imperial Valley	————
2	Taft-EW	July 21, 1952 Kern Country	------------
3	Shinbashi Bldg.-EW	Aug. 4, 1974 SW Ibaraki Pref.	—-—
4	Shinbashi Bldg.-EW	June 4, 1978 Off Miyagi Pref.	—-—-—

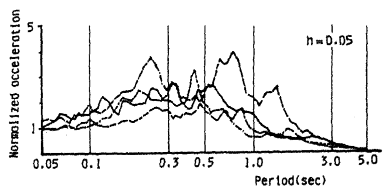


Fig. 17 Input Earthquake Acceleration Response Spectra

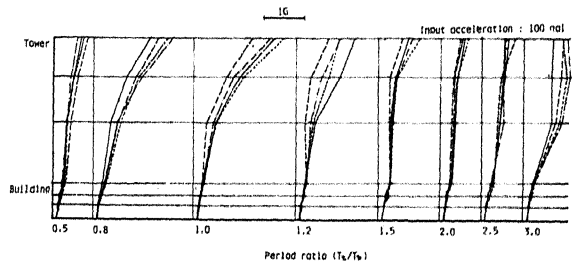


Fig. 18 Relationship between Distribution of Max. Acceleration and Period Ratio

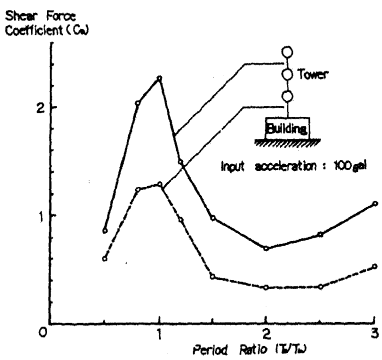


Fig. 20 Relationship between Shear Force Coefficient and Period Ratio

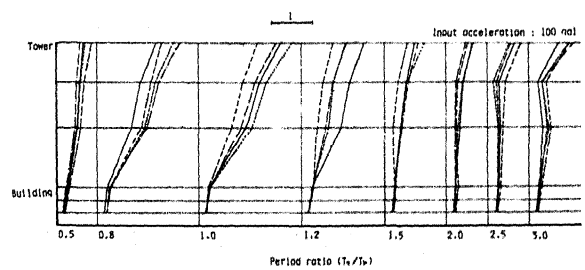


Fig. 19 Relationship between Distribution of Shear Force Coefficient and Period Ratio