

EARTHQUAKE RESISTANCE OF A STEEL FRAME APARTMENT
HOUSE WITH PRECAST CONCRETE PANEL

by

M. Murakami¹⁾, R. Tamura²⁾, Y. Tanaka²⁾, K. Oami³⁾,
Y. Osawa⁴⁾, H. Umemura⁵⁾

SYNOPSIS

This paper discusses the earthquake response and aseismic resistivity of a steel frame apartment house with precast concrete wall panels, which is an important new type of construction in Japan.

On an existing sample building, the forced vibration test is carried out and the long term earthquake observation has been carried out since 1970 at the inside of the building and also in the soil of the site. The response analysis of the building is made to the accelerograms recorded at the base of the building.

INTRODUCTION

The structural soundness of a new-type building under the earthquake excitations has been required to be checked by means of dynamic response analyses recently developed, because the present seismic provisions of building codes can not be applied to the new-type building directly as in the case of conventional type one.

Many new-type apartment houses with precast concrete wall panels were constructed at Kimitsu City in Chiba Prefecture in south part of Kanto and one of these was selected as the sample building. Large amount of observations of earthquakes and experimental and analytical investigations have been carried out using this sample building.

The main purpose of this study is firstly to know the dynamic behavior of the sample building and its surrounding soil during actual earthquakes, secondly to establish an appropriate dynamical model for the soil-building system and finally to check the structural soundness of this kind of building under severe earthquake excitations. In this paper only a part of these studies will be described.

OUTLINE OF THE BUILDING AND SUBSOIL

Building

The sample building is a 11-story steel structure with fire proof covering by concrete. It is 8.5m x 92.7m in plan and rises 30.4m above the ground level as shown in Fig.1. It is supported by steel piles which

1) Assoc. Prof., Chiba Univ., Faculty of Eng., Dr. Eng.

2) Nippon Steel Corporation

3) Assistant, Chiba Univ., Faculty of Eng.

4) Prof., Univ. of Tokyo, Earthquake Research Institute, Dr. Eng.

5) Prof., Univ. of Tokyo, Faculty of Eng., Dr. Eng.

rest in the sand layer (N>50) at about 12m below the ground level.

In the antiseismic design of the building against 0.2 design seismic coefficient, the lateral resisting elements are assumed to compose of steel frames line A and C in the longitudinal direction and steel braced frames in the transverse direction as shown in Fig.1. In the construction the beams line A and C and the bracings incased in reinforced concrete and prefabricated together with reinforced concrete panels including the beams line B. The columns and slabs are concreted in site after the precast beams and panels are built-up in each story.

Subsoil

The subsoil below the sample building consists of several layers. The subsoil profile and the penetration test results are shown in Fig.2 together with the location of the instruments.

FORCED VIBRATION TEST

The forced vibration test is carried out to know the dynamic characteristics of the building. Steady state sinusoidal vibration are induced in the structure by the three vibration generators placed on the roof floor as indicated in Fig.2. The generators can be synchronized in phase or 180 degrees out of phase. The vertical and horizontal displacement response are simultaneously measured at points of interest with 6 channel electro-magnetic seismometer-amplifier-visicorder system.

Results

The natural periods and dampings are determined from the resonance curves and tabulated in Table 1, together with the mode shapes and the shaking types. In the transverse direction the rocking and swaying displacements account for the main portion of the total deflection at the top of the building and their percentage is listed in Table 2. It is noticed that the mode shapes accompanied with horizontal slab deformation are apparently observed until 4th modes and these periods are longer than that of the second vertical frame mode shape. In the longitudinal direction, the displacement due to rocking is observed to be very small, while the ratio of the swaying response about 20%.

Analysis

An analysis is made to obtain a mathematical model representing the dynamic characteristics of the building in the transverse direction where the vibration modes of in-plane slab deformation are observed. The analytical model is represented by a 25-degree-of-freedom structure like Fig.5 in accordance with the usual assumptions for the dynamic response of linear, viscously damped, multi-degree-of-freedom system. The differential equation to an earthquake excitation is;

$$[M]\{\ddot{Y}+\ddot{y}_g\}+r[K_R]\{\dot{Y}\}+r[K_S]\{\dot{Y}+\dot{y}_g\}+[K_R]\{Y\}+[K_S]\{Y+y_g\}=\{0\} \quad (1)$$

where

[M] : Mass and inertia mass matrix

$[K_R]$: Stiffness matrix of frame and rocking

$[K_S]$: Stiffness matrix of slab considering restriction of frames in longitudinal direction

$\{Y\}$: Lateral displacement relative to the base including rocking displacement

$\{y_0\}$: Ground displacement

r : Damping, $r=2\zeta\omega_n$, ζ ; fraction of critical damping, ω_n ; fundamental circular frequency

The eigenvectors and eigenvalues are calculated from the equation putting $\{y_0\}=0$ and $r=0$ in eq.(1). The natural periods thus obtained are listed in Table 1. Close agreement between the test and calculated values are seen.

In the longitudinal direction, the periods of the steel frame structure line A and C are calculated to be 1.1 sec and 0.71 sec when the fire proof covering by concrete is considered. The observed period (0.31 sec) agrees with the calculated one considering the rigidity of the precast concrete wall panels line B.

OBSERVATION FOR ACTUAL EARTHQUAKE

Sixteen sets of the electro-magnetic seismometers are installed to measure the earthquake acceleration records under, around and inside the building as shown in Fig.2. The observation system inside the building is considered to be unique as it is intended to take measurement of the rocking displacement and the slab in-plane deformation.

Results

Until present, the accelerograms have been recorded of seven earthquakes, the data of which are shown in Table 3. The maximum acceleration amplitudes at some points during two earthquakes (No.1, No.2) and their ratios are listed in Table 4. Fourier spectra of the observed accelerograms are shown in Fig.3.

Although decisive conclusions can not be deduced from the analysis of an insufficient number of records, some important comments on the earthquake response of the subsoil-building system are summarized below.

The predominant periods of the building, the soil layers and the soil-building system are obtained by taking the ratio of spectra between two observation points as shown in Fig.4. The periods depending on the peak values of spectral ratios are listed in Table 5. The predominant periods of the building as obtained from the ratios of 11 to 1 are longer than those obtained from the forced vibration tests in Table 1. The significantly large interaction and combined effects of the subsoil and building seem to be apparent, judging from the spectra and spectral ratios.

The spectrum difference of the horizontal input waves at the center and end of the building base is shown in Fig.6. The phase and amplitude

difference is small in the range of the period which is longer than the fundamental natural period, but in small amplitude and short periods the phase difference reaches large values.

The time history of the acceleration response at the top of the building is shown in Fig.7. The higher slab modes are obvious but they disappear as the vibrational amplitude becomes large.

The amplitude and phase of the rocking motions at both side of the building in the transverse direction are same and in phase in the range of a long period than the fundamental natural period of the building, and are different and out of phase in the shorter period as shown in Fig.8. The ratio of the rocking displacement to the total displacement at the top agrees with that of the vibration test results.

Analysis

The response analysis is made using eq.(1) for transverse direction in such a way that the observed acceleration records at the different points of the building base are applied to a mathematical model with 2% of critical damping and the acceleration response wave at the top of the building are calculated. The calculated waves coincide well with the observed ones as shown in Fig.9.

SOUNDNESS OF THE BUILDING FOR SEVERE EARTHQUAKE

Assuming various hysteresis models of restoring force, response analyses of the building for severe earthquakes are made in the elastic-plastic regions and the earthquake resistance is synthetically examined together with the difference of the response due to the different hysteresis models. The results indicate that the antiseismic capacity in the transverse direction is larger than that in longitudinal direction because of a large amount of walls and rocking effects. Therefore, only the longitudinal direction is described in this paper.

Two earthquake motions observed at the center of the base of the building and EL CENTRO NS, 1940 Accelerogram are used for the response analysis, of which the maximum acceleration is modified to be 300 gal. The response is evaluated by stepwise integration process, using 0.00125 second time increment and assuming the acceleration to vary linearly during each increment. A 11 mass shear type system is considered, of which the fraction of critical damping is 2% in the fundamental mode.

Hysteresis Model

Nine different types of hysteresis curve, as shown in Fig.10, are selected and the earthquake response of each model is studied in detail. They are divided into two groups with different ultimate strength. Q_y is smaller than Q'_y considering the strength of frame line B.

Results

The story displacement responses are shown in Fig.11 and Fig.12, in which the points "smoothed" show the response of the building with the

smoothed distribution of the rigidity and strength as compared to the original ones. The response is influenced by the magnitude and distribution of the ultimate strength; the mean values and fluctuation of the response depend mainly on the magnitude and the distribution of the ultimate strength, respectively. The different responses due to the different restoring force characteristics, especially the area of the hysteresis loop, are shown in this figure. The structural soundness under the severe earthquake excitations seems to be ensured by considering the frame line B in the analysis.

CONCLUDING REMARKS

Main results of this study are summarized as follows:

i) Forced vibration tests: In the transverse direction, the mode shapes accompanied with horizontal slab deformation is recognized up to 4th modes. The rocking displacement is measured to be one half of the total deflection at the top of the building. Satisfactory results have been obtained by analyzing a model with 25 degrees of freedom. In the longitudinal direction, it is found that the non-structural elements such as precast partition walls contribute significantly to the rigidity of the building.

ii) Earthquake measurements: The amplification factor of the building is about 5, while that of the soil-building system is about 10. The higher modal vibrations of the slabs which appear in the forced vibration tests are suppressed when the building vibrates in large amplitude. The amplitude and phase difference of the lateral input waves at the center and end of the building base is small in the range of longer period than the fundamental natural period, and also the vertical input waves at the both sides of the building in the transverse direction are similar. The ratio of the rocking displacement to the total deflection at the top agrees with the forced vibration test results.

iii) Response analysis: Assuming various hysteresis models of restoring force, response analyses of the building are made to the recorded accelerograms of the building base and also to the EL CENTRO, NS, 1940 Accelerogram with the maximum acceleration of 300 gal. The results indicate that this type of buildings of 11 story of which the ultimate strength is less than the strength corresponding to the 0.3 base shear coefficient and the hysteresis shape is not so ductile would be not warranted in severe earthquakes.

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Table 1 RESULTS OF FORCED VIBRATION TESTS

| MODE | EXPERIMENT | | | ANALYSIS/OBSERVATION | | |
|----------|------------|------|------------|----------------------|--------------|------------|
| | FRAME | SLAB | PERIOD sec | DAMPING % | SPACING TYPE | PERIOD sec |
| 1st MODE | NS | | 0.48 | 2-3 | | 0.48 |
| | | | 0.43 | 2-3 | | 0.42 |
| | | | 0.25 | 2-3 | | 0.29 |
| | | | 0.13 | --- | | --- |
| 2nd MODE | NS | | --- | --- | --- | 0.09 |
| | | | --- | --- | --- | 0.09 |
| 1st MODE | EW | | 0.31 | 3-4 | | 0.32 |
| 2nd MODE | | | --- | --- | --- | --- |

* Due To Eccentric Mass & Shaking Type

Table 2 RATIO OF ROCKING AND SWAYING DISPLACEMENTS (%)

| | 0.49 SEC | | | 0.43 SEC | | | 0.25 SEC | | |
|--------------------|----------|----------|-------------|----------|----------|-------------|----------|----------|-------------|
| | NO.15 | SWAY (%) | ROCKING (%) | NO.15 | SWAY (%) | ROCKING (%) | NO.15 | SWAY (%) | ROCKING (%) |
| FRAME | 11.0 | 45.0 | 10.0 | 10.0 | 37.0 | 41.0 | 10.0 | 41.0 | 52.8.μ |
| TOTAL DISPLACEMENT | 175. μ | 537. μ | 528. μ | 175. μ | 537. μ | 528. μ | 175. μ | 537. μ | 528. μ |

Table 3 EARTHQUAKE DATA

| NO | DATE | EPICENTER | DEPTH | MAGNITUDE | INTENSITY (J.M.A) | MAX. AMPLITUDE (GAL) OF BASE |
|----|-----------|----------------|-------|-----------|-------------------|------------------------------|
| 1 | 5,22,'70 | - | - | - | - | 3.4 |
| 2 | 12,08,'70 | 29.3°N,140.4°E | 180KM | - | II : CHIBA | 3.8 |
| 3 | 1,04,'72 | 35.8°N,140.4°E | 50KM | 5.0 | III : CHIBA | - |
| 4 | 2,29,'72 | 33.3°N,141.3°E | 70KM | 7.0 | IV : CHIBA | 20.0 |
| 5 | 10,18,'72 | 35.8°N,140.0°E | 70KM | 5.0 | III : CHIBA | 8.2 |
| 6 | 11,06,'72 | 36.3°N,139.4°E | 40KM | 4.0-5.0 | III : CHIBA | 2.6 |
| 7 | 12,04,'72 | 33.2°N,141.0°E | 60KM | 7.3 | IV : CHIBA | 16.8 |

Table 4 OBSERVED MAXIMUM ACCELERATIONS

| OBSERVED POINT | EARTHQUAKE NO. 1 | | EARTHQUAKE NO. 2 | |
|----------------|---------------------|-------|---------------------|-------|
| | MAXIMUM VALUE (gal) | RATIO | MAXIMUM VALUE (gal) | RATIO |
| NC 11 | 13.8 | 6.6 | 20.4 | 10.2 |
| NC 6 | 5.9 | 2.8 | 10.8 | 5.4 |
| NC 1 | 2.7 | 1.3 | 3.7 | 1.9 |
| NB 30 | 2.1 | 1.0 | 2.0 | 1.0 |
| NG 0 | 2.6 | 1.3 | 3.7 | 1.9 |
| EC 11 | 9.0 | 5.6 | 9.4 | 7.8 |
| EC 6 | 7.4 | 4.6 | 7.1 | 5.9 |
| EC 1 | 2.2 | 1.4 | 2.5 | 2.1 |
| EB 30 | 1.6 | 1.0 | 1.2 | 1.0 |
| EG 0 | 2.7 | 1.7 | 2.3 | 1.9 |

Table 5 PREDOMINANT PERIODS DETERMINED FROM SPECTRAL RATIOS

| DIRECTION | MODE | 11/1 (SEC) | 11/B30 (SEC) | O/G30 (SEC) |
|-----------|-----------|------------|--------------|-------------|
| N-S | SLAB 0 th | 0.50-0.52 | 0.52-0.56 | - |
| | 1 st | 0.42-0.43 | 0.43-0.44 | 0.34-0.38 |
| | 2 nd | 0.28-0.29 | 0.28-0.29 | - |
| E-W | 1 st | 0.32 | 0.38-0.42 | 0.34-0.38 |

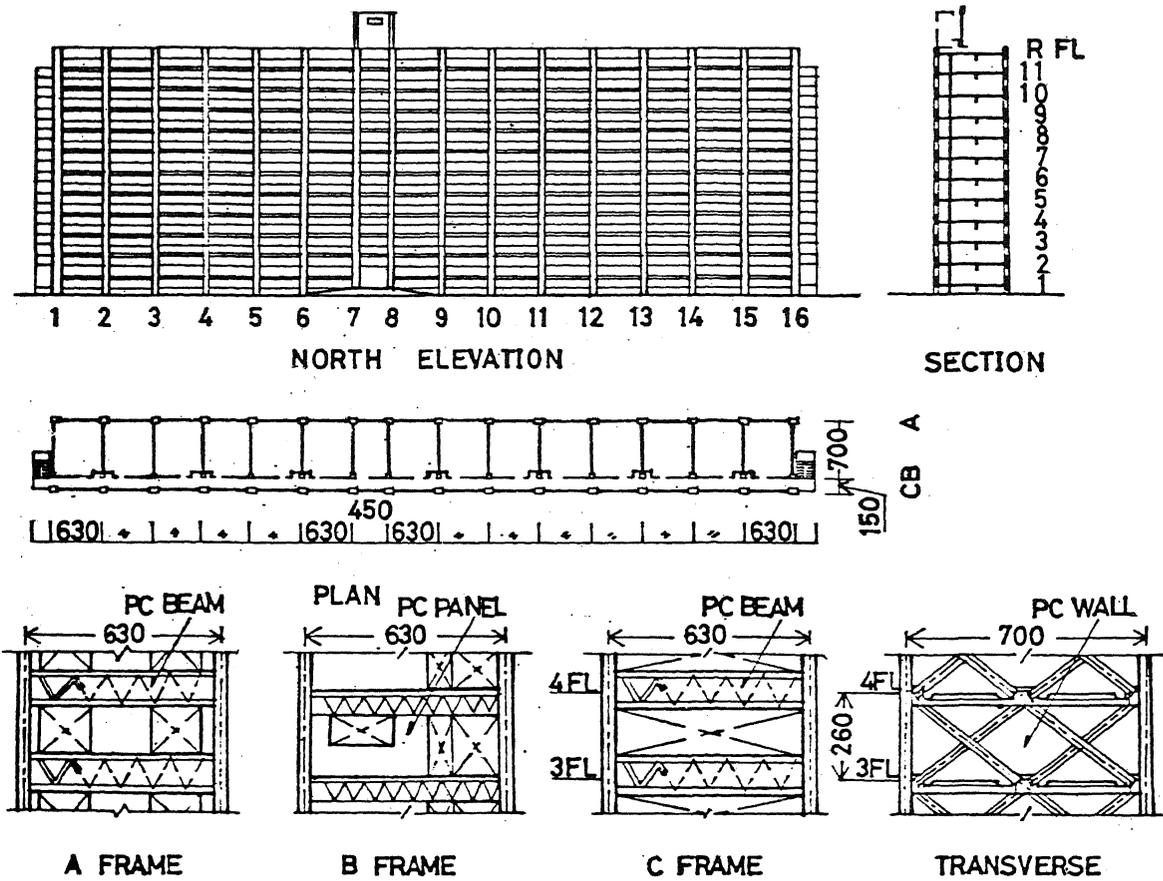


Fig.1 OUTLINE OF BUILDING AND DETAILS OF FRAMES.

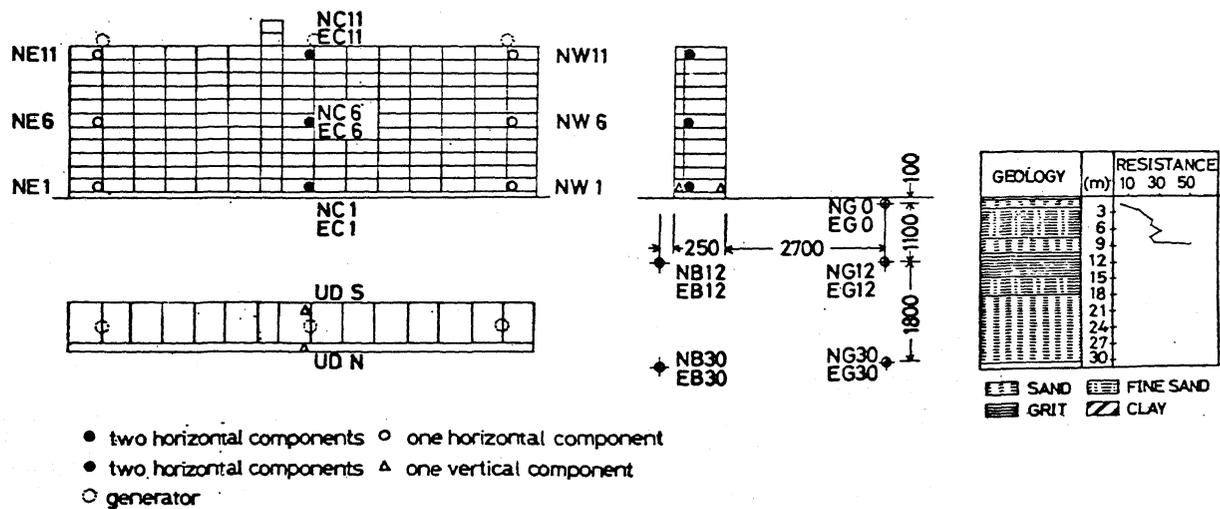


Fig.2 LOCATION OF INSTRUMENTS AND SOIL PROFILE OF BUILDING SITE

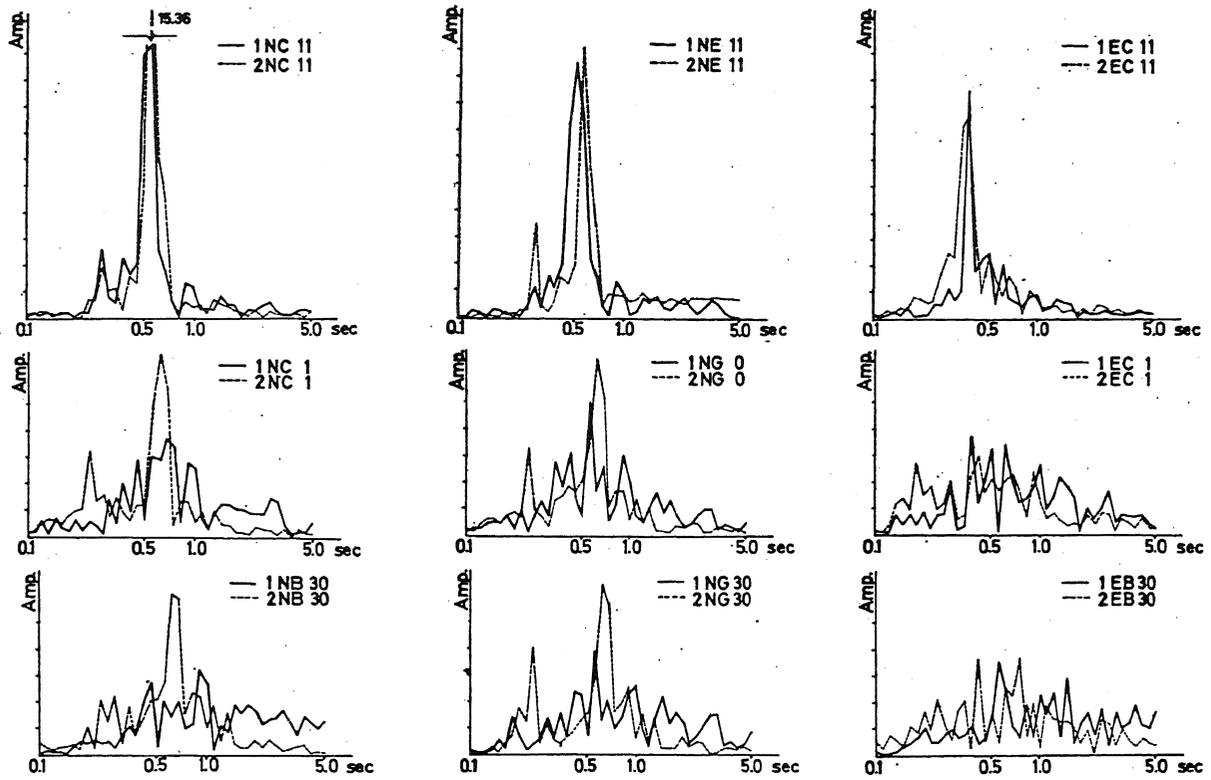


Fig. 3 FOURIER SPECTRA OF OBSERVED ACCELEROGRAMS

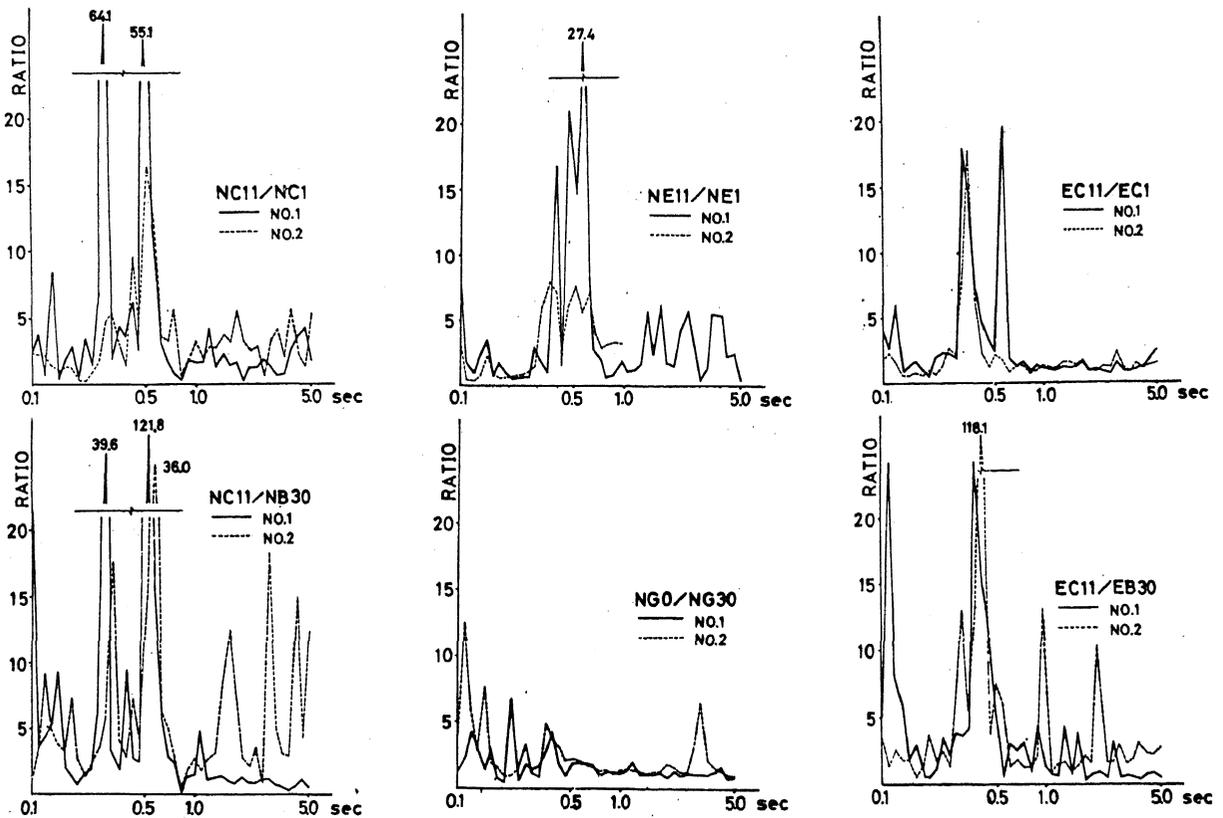


Fig. 4 SPECTRAL RATIOS

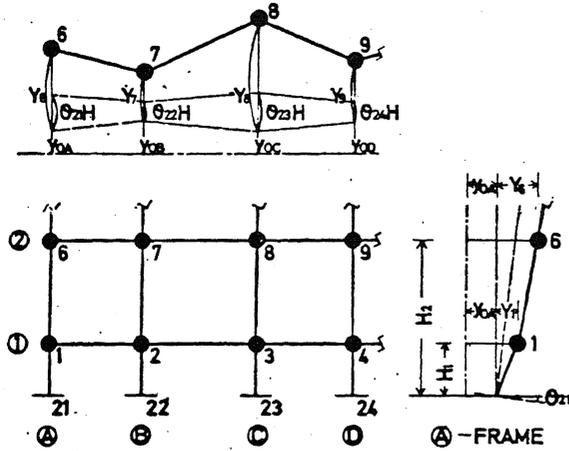
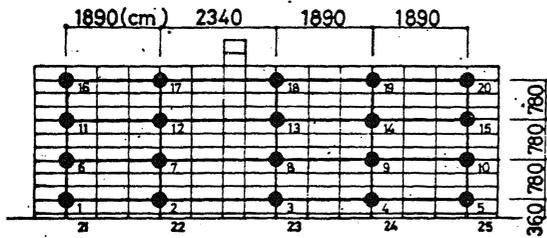


Fig. 5 ANALYTICAL MODEL

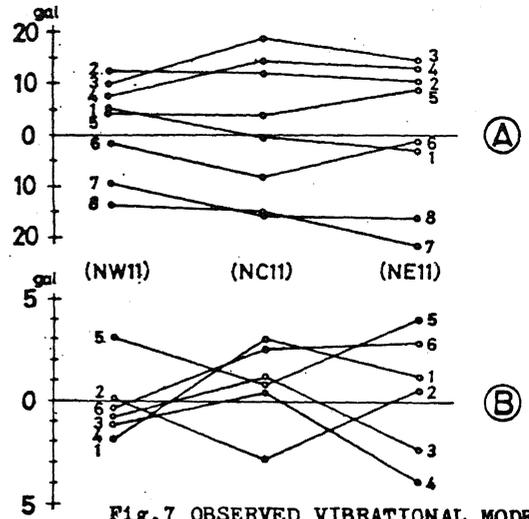
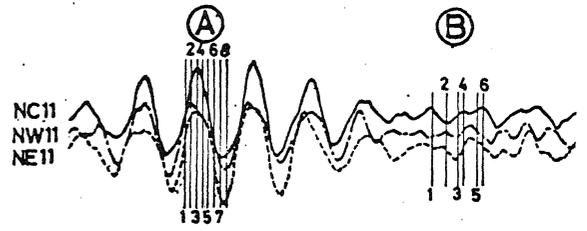


Fig. 7 OBSERVED VIBRATIONAL MODES OF 11TH FLOOR SLAB

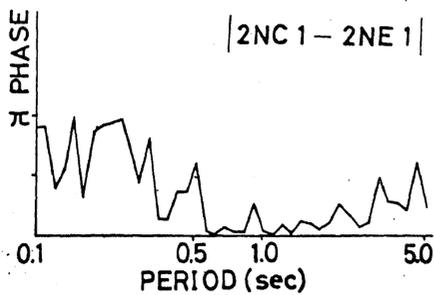
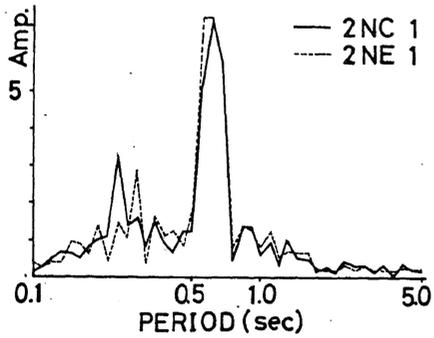


Fig. 6 AMPLITUDE AND PHASE DIFFERENCE OF SPECTRA AT TWO DISTANT POINTS (NC 1, NE 1) OF BUILDING BASE

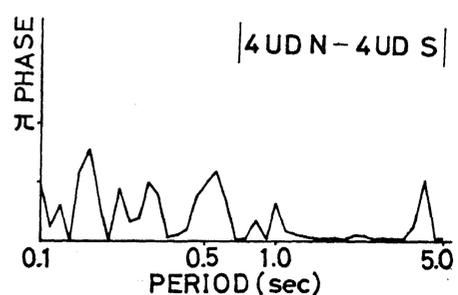
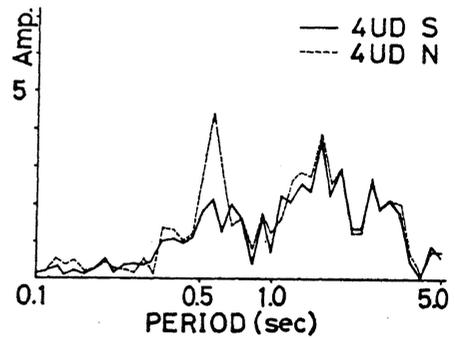


Fig. 8 AMPLITUDE AND PHASE DIFFERENCE OF SPECTRA OF TWO VIRTUAL EARTHQUAKE MOTIONS (UD 1, UD 2)

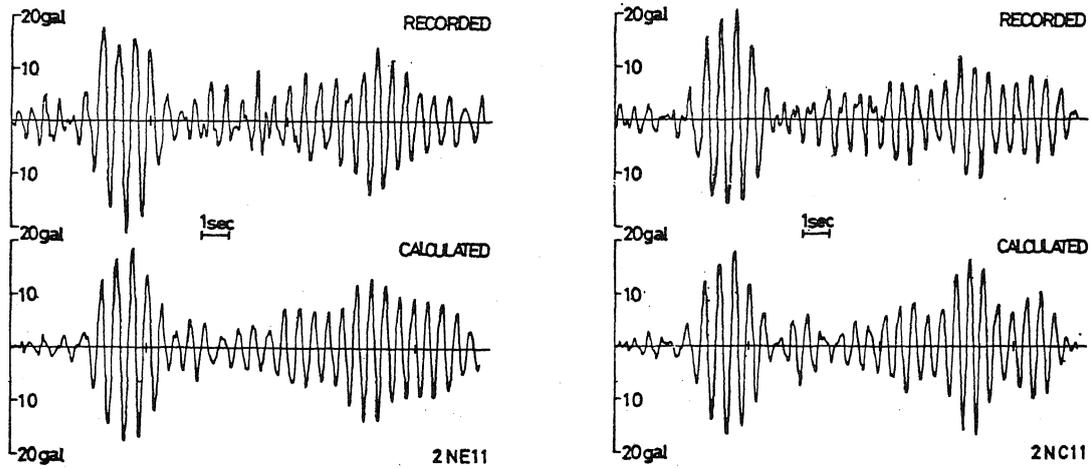


Fig. 9 COMPARATIVE TIME HISTORY OF RECORDED AND CALCULATED ACCELERATIONS

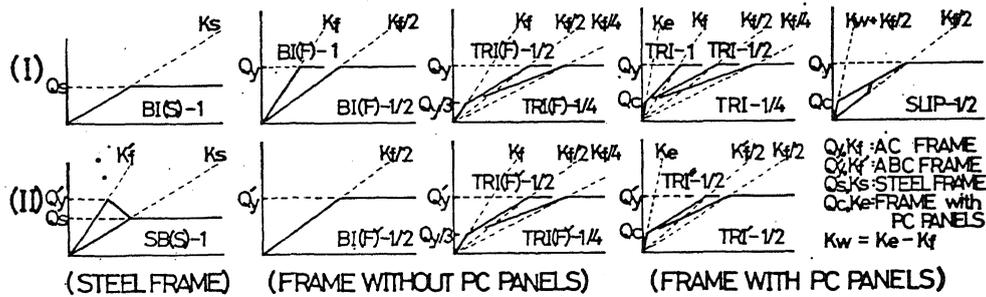


Fig. 10 HYSTERESIS MODELS

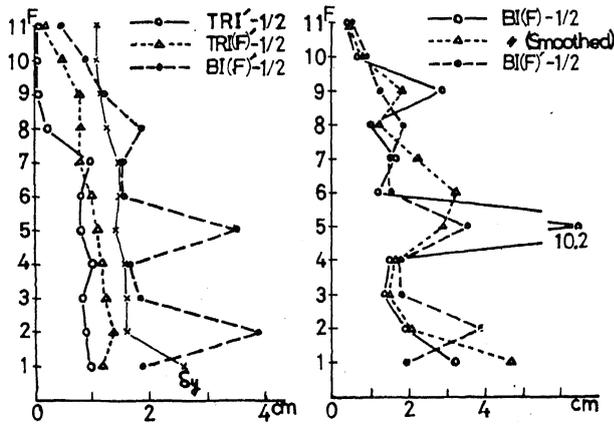


Fig. 11 MAXIMUM STORY DISPLACEMENT RESPONSE DUE TO IEC1

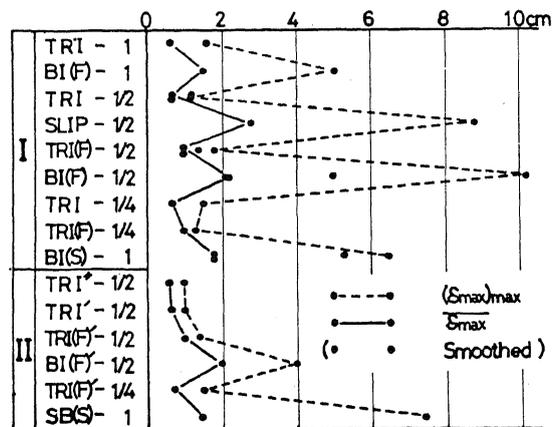


Fig. 12 MAXIMUM AND AVERAGE STORY DISPLACEMENTS IN EACH MODEL