SIMULATION ANALYSIS OF A HIGHRISE REINFORCED CONCRETE BUILDING IN TWO DIFFERENT EARTHQUAKES

by Kiyoshi Muto[I] Tsunehisa Tsugawa[II]

Masanori Niwa[III] Hiromichi Shimizu[IV]

SYNOPSIS

Past legal prohibition in Japan that pure reinforced concrete buildings can not exceed 20 meters and/or 6 stories in height was broken by the new R & D and consequent design of a 18-story Shiinamachi Building of Kajima Corporation. The building was constructed as an unprecedented case of the Construction Minister's special approval. Following its completion, earthquake observations have been continued as a post-construction study. Of the many recorded, two different earthquakes which accelerations registered remarkably with the building have been simulated. Using an idealized vibration model, excited motions are computed and compared with the observed behaviors. It was found that they are quite identical, which corroborates that the aseismic design procedure of the building was fully reliable.

I. Earthquake Observation

After obtaining the Construction Minister's special approval for actual construction, in accordance with Article 38 of Japanese Building Standard Law, a 18-story reinforced concrete Shiinamachi Building was constructed.(1) Since the completion of the building in 1974, an earthquake observation network has been established within the building. Earthquake motions are registered by servo-type accelerographs placed in the basement, at the 9th and the 19th (roof) floor respectively. For an appropriate data processing, successive earthquake signals at respective floors are preserved in the magnetic tapes of a data recorder in the basement. Among the observed records, the following two earthquakes are remarkable.

<u>Izu Peninsula Coastal Earthquake</u>

Occurred on May 9, 1974
Magnitude 6.9
Location of Focus 138°48'E 34°34'N
Focal Depth 20 kilometers
Epicentral Distance 150 kilometers

<u>Eastern Saitama Earthquake</u>

Occurred on August 4, 1974
Magnitude 5.8
Location of Focus 139°55'E 36°01'N
Focal Depth 20 kilometers
Epicentral Distance 40 kilometers

I. Member of Japan Academy, Professor Emeritus, University of Tokyo, Vice President of Kajima Corporation
II. Senior Research Engineer, Muto Institute of Structural Mechanics
III. Research Engineer, Kajima Institute of Construction Technology
IV. Research Engineer, Muto Institute of Structural Mechanics

Although epicentral distances of these earthquakes are 150 kilometers and 40 kilometers from the building site, local intensities are both reported to be III in Japan meterological intensity scale, which corresponds to IV or V in modified Mercalli scale. Acceleration time histories observed in the transverse direction at the basement are shown in Fig. 1. Acceleration response spectra obtained by numerical integration through 90 seconds time duration are also shown in Fig. 2.

In the Izu earthquake, the major movements of the building occur about 20 seconds after the beginning of the earthquake. Principal component of the major movement is estimated to have a period of 1 second. In the Saitama earthquake, on the contrary, the shorter period components are dominant in the major movements which appear 7 seconds after the beginning. Large accelerations are registered only at the fore part of the ground motion. The periodic characteristics appearing in the two earthquake records are so distinguished that the building's excited vibrations look quite different from each other. That is, the fundamental vibration with the first mode is prominent in the Izu earthquake, while the second and third modes are excited in the Saitama earthquake.

II. Vibration Model

As shown in Fig. 3, the vibration model has multi-lumped masses with equivalent bending and shearing stiffness. A horizontal motion observed at the basement is applied as the input earthquake ground motion. Equivalent stiffness is estimated by the results of frame analysis using FAPP computer program, where flexural, axial and shearing deformations of each building members are taken into consideration. Member rigidities are evaluated on the assumption that concrete works against tension as well as compression in a minor stress level. Elastic modulus of concrete is introduced directly from the test results obtained by on-site testing. Lumped mass at each floor level is evaluated mostly from the dead load previously defined at the time of design. Live load of 15 kg/m^2, although in small amount, is also included in the lumped mass.

A freedom of base rotational motion is additionally considered in this model. Rotational stiffness is assumed considering the reactions of soil and piers. Virtual mass effects due to subsoil are also introduced, which results in that moment of rotational inertia pertaining to base movements is 3 times as much as that of the foundation structure itself. Therefore, fundamental vibration periods of the model in transverse direction are to be estimated 1.02 second and 0.32 second.

Above mentioned modeling is established on the design basis assumptions which were adopted previously during the aseismic design procedure. Some modifications, of course, have been incorporated referring to studies on forced vibration test. For instance, it is decided that, damping for reinforced concrete differs from that for soil. Therefore, 2% of critical damping ratio in the fundamental mode is applied to the upper-structure while 10% is to base foundation. Equivalent damping factor for the 1st mode of this model is also computed to be 2.4%.(2)

III. Simulated Results

Case of Izu Earthquake

Computed acceleration time histories due to the Izu Earthquake at the 19th and 9th floors are compared with the observed earthquake waves as shown in Fig. 4. Triangle marks in the figures indicate the maximum accelerations which occurred through the time duration. In the observed acceleration records, it is recognized that maximum accelerations occur at about 30 seconds after the beginning of motion. Some pseudo beat phenomena are also clearly visible. A major motion in the input acceleration has a principal component of 1 second, which makes the upper floor accelerations extraordinarily amplified. For instance, the 19th floor acceleration is 6 times larger than that of the basement. Computed acceleration time histories coincide with the observed ones. The response spectra in Fig. 5 also show that both accelerations are quite identical.

Case of Saitama Earthquake

Fig. 6 shows the comparison of the observed and computed accelerations due to the Saitama Earthquake. Reflecting the fact that the basement record has prominent periods of 0.3 and 0.2 second at the time of major motion, the building is sharply excited at the fore part of the duration, then gradually changed to an oscillation with a longer fundamental period. Amplification ratios of acceleration both at the 19th and 9th floor are about twice that of basement acceleration. These tendancies are clarified by Fig. 7 showing the response spectra. It is also concluded that dynamic behaviors of the building are precisely reproduced by analytical simulations throughout long time duration of the earthquake.

IV. Concluding Remarks

The same assumptions as those used in the aseismic design of the tall reinforced concrete buildings are introduced in the simulation analysis in two different earthquakes. The results of the model computation fully coincide with the whole observed dynamic characteristics. It is therefore concluded that the aseismic design procedure of the Shiinamachi Building was further confirmed to be highly reliable through the simulation analysis.

References

(1) "Earthquake Resistant Design of a 20 story Reinforced Concrete Building" K. Muto, T. Hisada, T. Tsugawa and S. Bessho, Proceedings of the 5th World Conference on Earthquake Engineering, June, 1973.

(2) "Strong Motion Records and Simulation Analysis of KII Building in San Fernando Earthquake" K. Muto, Earthquake Engineering and Structural Dynamics, Vol. 1, No.1, 1972.

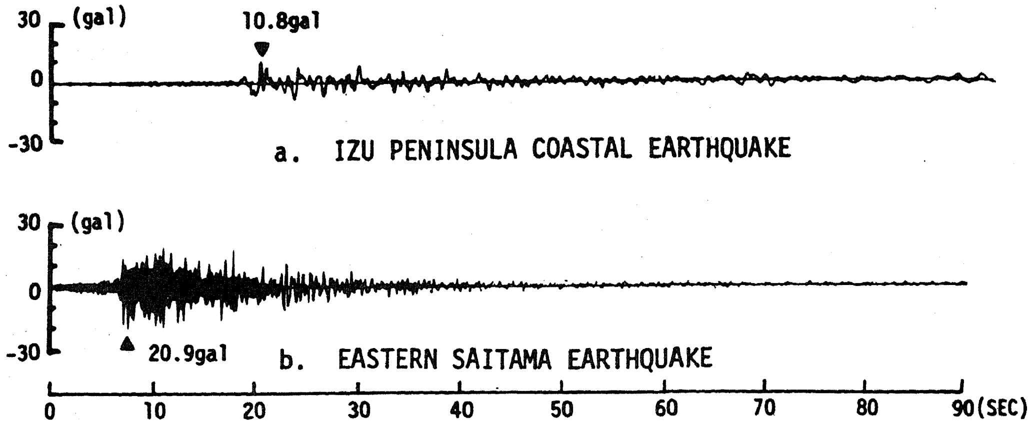


Fig. 1 OBSERVED ACCELERATION AT BASEMENT (TRANSVERSE)

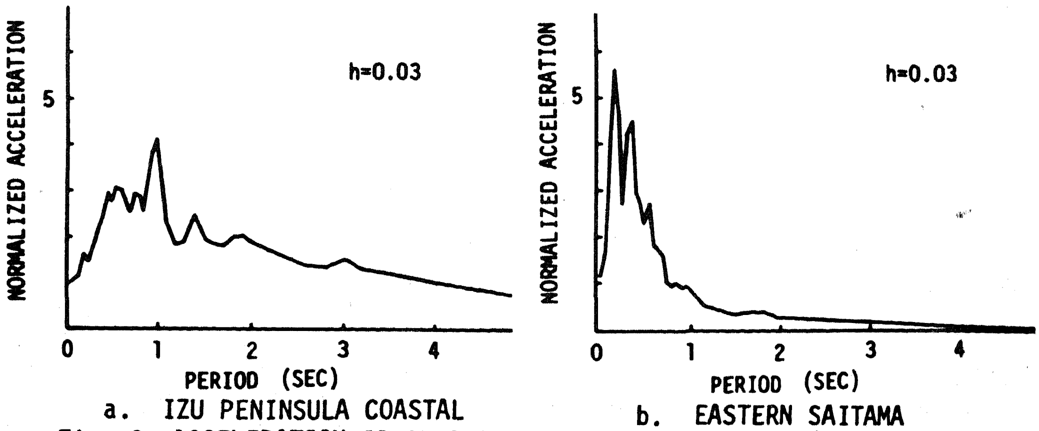


Fig. 2 ACCELERATION SPECTRA OF OBSERVED EARTHQUAKE (TRANSVERSE)

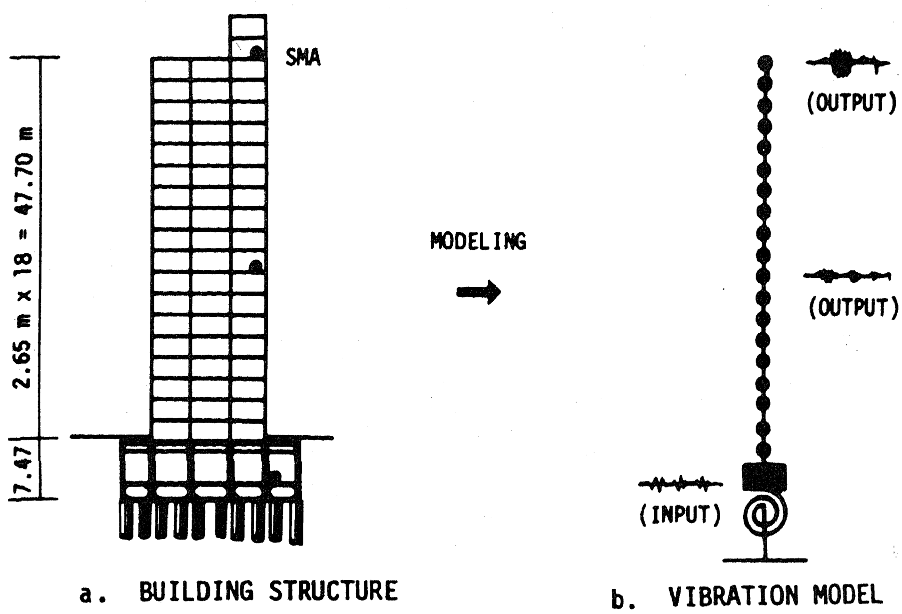
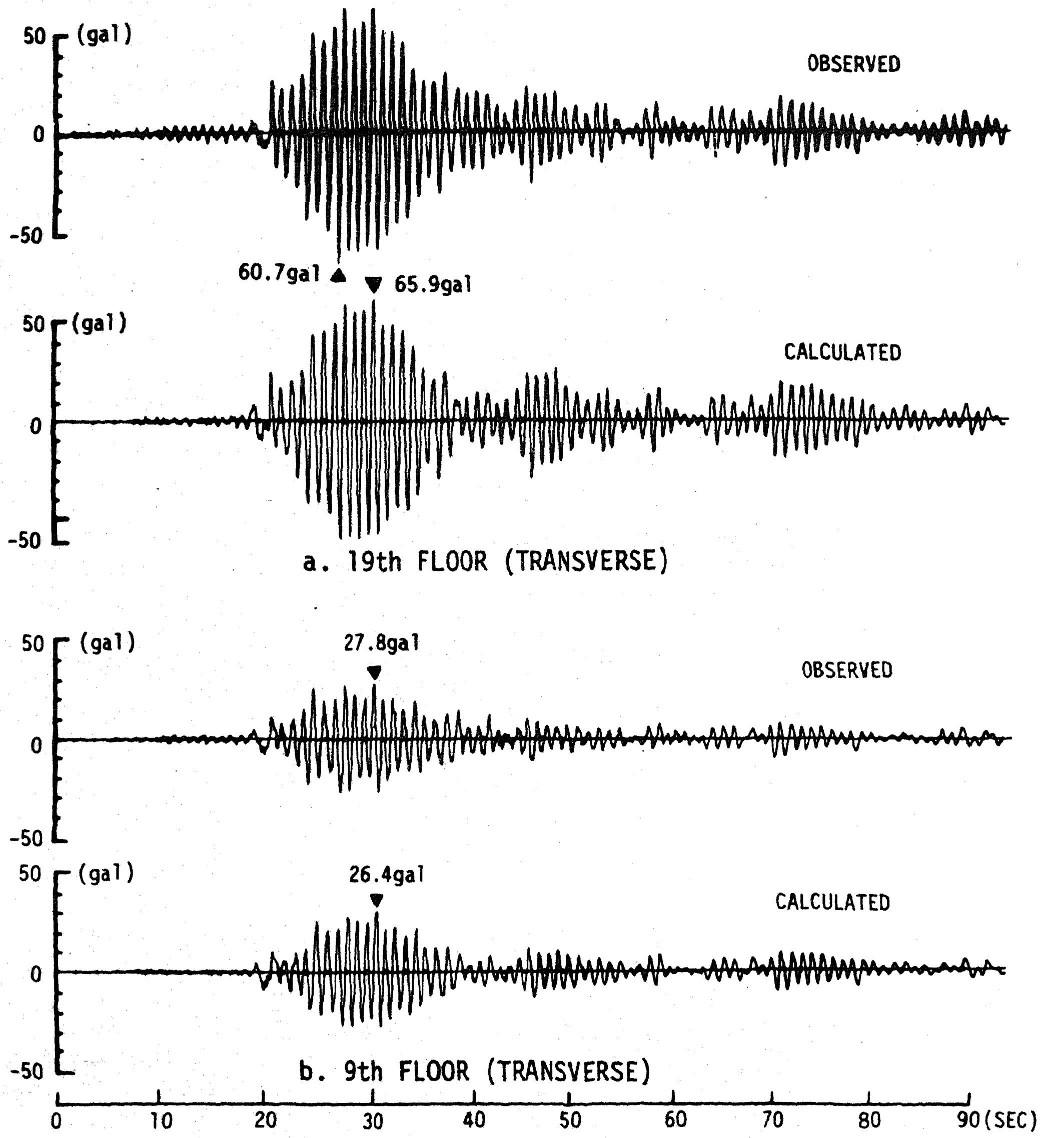


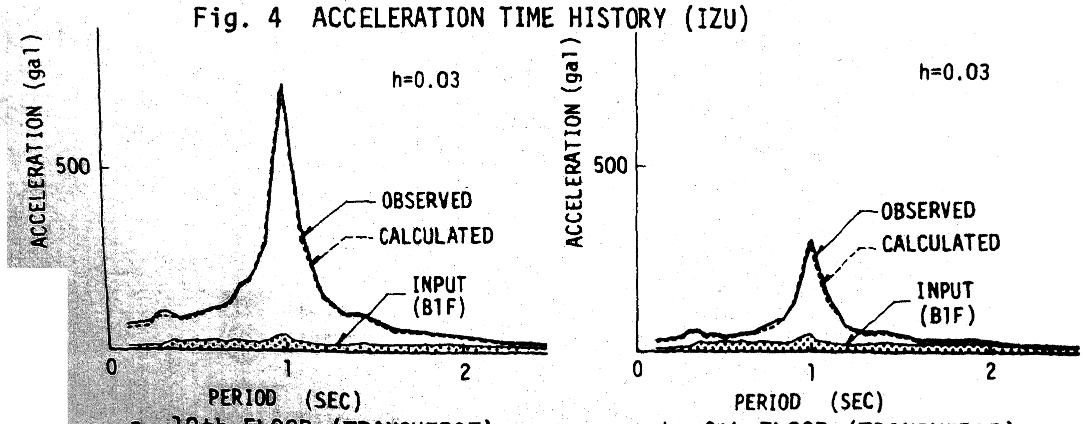
Fig. 3 VIBRATION MODEL



a. 19th FLOOR (TRANSVERSE)

b. 9th FLOOR (TRANSVERSE)

Fig. 4 ACCELERATION TIME HISTORY (IZU)



a. 19th FLOOR (TRANSVERSE)

b. 9th FLOOR (TRANSVERSE)

Fig. 5 COMPARISON OF ACCELERATION SPECTRA (IZU)

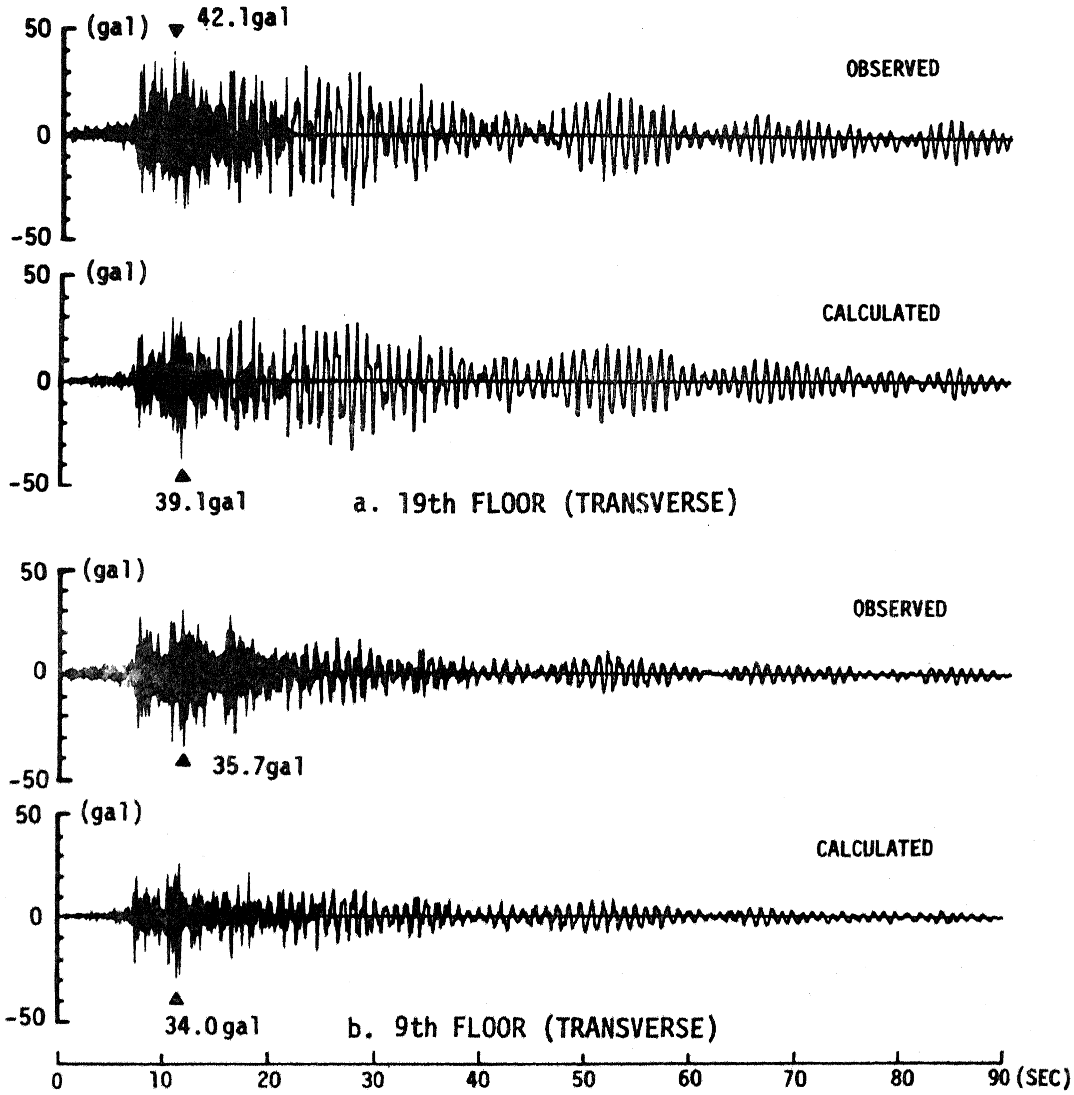


Fig. 6 ACCELERATION TIME HISTORY (SAITAMA)

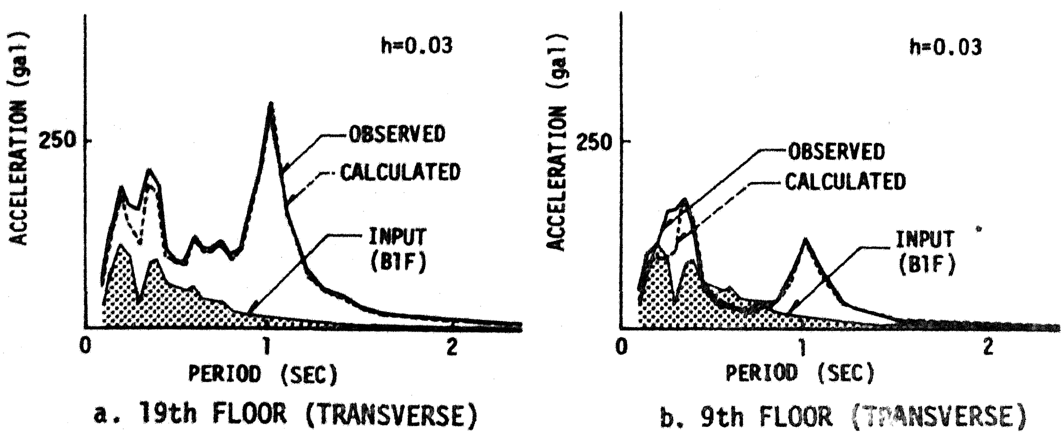


Fig. 7 COMPARISON OF ACCELERATION SPECTRA (SAITAMA)

DISCUSSION

K. Madhavan (India)

The authors have presented very important results which give us more confidence in the analytical methods now being adopted. Would the authors clarify whether the stress - strain characteristics of the concrete assumed was valid during the strong accelerations that occurred. The analysis appears to confirm that the elastic method can be used to predict the response of frame structure even though some of the sections may have undergone considerable nonlinear deformations. This may not apply to other structures like dams.

Author's Closure

The writers would like to thank Mr. Madhavan for his interest in the paper and for his valuable comments, and completely agree with his opinion that the nonlinear behavior of the structure should be taken into account in the analysis of the R/C structure subjected to major earthquakes.

The writers have to point out that, although the two earthquakes adopted in the simulation analysis were the largest among the observed ones, they were small compared to the major earthquakes assumed in the aseismic design of the structure. For instance, 0.03g was the maximum acceleration of observed earthquakes, while the maximum one of more than 0.3g was assumed in the design. The analytical results of the observed earthquake indicated that concrete strain reached to less than 0.01%. The behavior of the structure during those earthquakes was assured to be fully elastic.

The nonlinear dynamic analysis was performed in the aseismic design of the building as previously reported in the paper in the 5WCEE entitled "Earthquake Resistant Design of A 20-Story Reinforced Concrete Building", where degrading trilinear hysteretic characteristics were defined so that cracking and yielding of the members might be accurately taken into account.