# Seismic design and verification of seismic efficiency of a highrise reinforced concrete building

# SEISMIC DESIGN AND VERIFICATION OF SEISMIC EFFICIENCY OF A HIGHRISE REINFORCED CONCRETE BUILDING

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Summary

The authors designed a successful construction of the first highrise reinforced concrete building to be built in Japan after obtaining a special approval from the Minister of Construction. The structural design was carried out based on structural experiments and nonlinear dynamic analysis. After completion of the building, earthquake observations have been continued at the basement, 9th and 19th floors. The two largest earthquake motions were simulated. The computed accelerations showed good agreement with the observed ones. This proves that our dynamic analysis is sufficiently accurate.

Resume

Les auteurs ont étudié la construction du premier grand immeuble en beton arme, erigé au japon, après avoir obtenu l'approbation spéciale par le Ministre de Construction. L'étude structuralle a été effectuée suivant les base de l'essais de structure ainsi que de l'analyse dynamique non-linéaire. Après l'achèvement de l'immeuble, les observations sismiques ont été continuées au sous-sol, à 9$^{e}$ étage, et à 19$^{e}$ étage. Les deux plus grands mouvements sismique ont été simulés. Les accélérations calculées se montraient bonne concordance avec celles observées. Ceci prouve que notre analyse dynamique est suffisamment exacte.

Zusummenfassung

Die Verfasser projektierten ein in Japan erst ausgeführtes stahlbetonhochgebäude mit der Sondergenehmigung vom Minister des Aufbaus. Die Konstruktionsprojektierung wurde durch die konstruktiven Versuche und unlienierte dynamische Analysis gemacht. Nach der Baufertigstellung ist die Erdbenmessung im KG., 9. OG. u. 19. OG. durchgeführt. Die mit Komputer ausgerechneten Beschleunigungen hatten gute Stimmung mit den Messungsergebnissen bei zwei größten Erdbeben. Das zeigt, daß unsere dynamische Analysis genügende Genauigkeit hat.

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## 1. INTRODUCTION

It was believed that the construction of highrise buildings using reinforced concrete would be difficult in Japan due to the problems of earthquakes. As a common practice, reinforced concrete (RC) structures have traditionally been forbidden for buildings of 7 stories (20m) or more. Taller buildings than 31 meters were composed of either combined structural steel and reinforced concrete, or of structural steel.

The year following the Tokachioki Earthquake of 1968, the authors began to perform experiments for the improvement of the aseismic properties of reinforced concrete members. And it was found that, with adequate arrangement of reinforcing bars, brittle failure of the member is completely prevented and sufficient deformability and ductility are secured.

With the support of these experimental findings and dynamic computer analysis, the authors succeeded in designing an 18-story RC apartment building. It was confirmed that stresses in all of the building members remained in allowable values during the severe earthquake (maximum acceleration 0.3g) and the maximum ductility factor remained at 1.66 against worst earthquake (maximum acceleration 0.5g). Since this building was unconventional in terms of both height and structure, Article 38 of the Japanese Building Standard Law required that special approval be obtained from the Minister of Construction.

In September 1972, soon after applying and receiving of a special permit, construction work of the building was started, and in January 1974 it was brought to a successful completion. This paper describes the preliminary studies, the earthquake resistant design and post construction studies of dynamic behavior of this building.

The second highrise building (Makomanai Apartment, 11 stories) was designed by the same method and now under construction at Sapporo. Building G which is to be 25 stories apartment is now being designed.

## 2. PRELIMINARY STUDIES

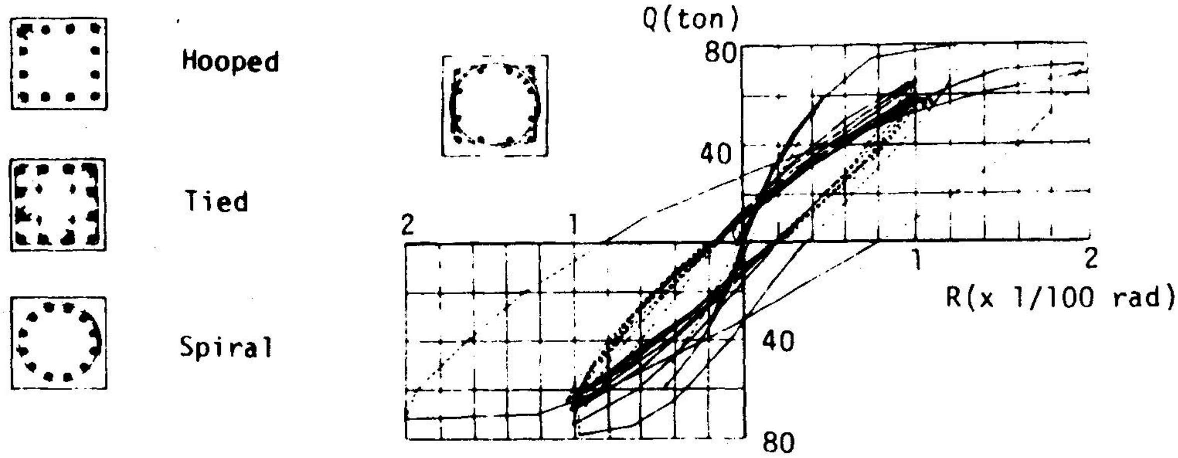
Various experiments were carried out to confirm the practicability of each of new construction methods. Among them the following three series of tests contributed much to the realization of the highrise RC buildings.

### 2.1 Studies of shear reinforcement of columns

Many RC columns reinforced by poor hoops suffered from brittle shear failures by the Tokachioki earthquake of 1968. The main reinforcing bars had buckled and concrete burst out. This illustrated an important role of confinement of concrete. Immediately after the earthquake, the authors performed column tests with three different types of transverse reinforcements, as shown in Fig. 1. The effects of transverse reinforcing bars of respective hoop, tie and spiral type were examined under identical condition. Then it was recognized that the hoop column lost its load bearing capacity soon after repeated loading at a deflection angle of 1/100. By contrast, the tied and spiral columns with proper amount of reinforcements were capable of deforming up to large deflection without any decrease of load bearing capacity.

While seeking an arrangement of longitudinal bars which would allow the beam reinforcing bars to pass easily through the column and also searching for method of prefabricating the column and beam bars on the ground for lifting into place, a new arrangement of the reinforcing bars for columns was developed. The shear reinforcement of the column was combination of spiral and square hoop, which was named Kajima Spiral.

It was also ascertained by testing that the deformability and ductility of this column were as same as those of the spiral column or tied column as shown in Fig. 1.




a. Three Types of Reinforcement b. P-δ Curves of Kajima Spiral Column

Fig. 1 Experiments of Columns

## 2.2 New anchorage system

The newly developed anchoring method for the beam reinforcement was subjected to the Construction Minister's approval. In the U Anchor Method, the ends of the main reinforcing bars in beam are anchored by form of the letter U at the beam-column joint of exterior frame as shown in Fig. 4. By developing the U Anchor Method and Plate Anchor Method, it became possible to prefabricate the reinforcement and to place the concrete for columns separately from that for beams and floor slabs.

The experiments on the effects of new anchoring methods were performed. From the load-deflection curve of the transverse framing with continous anchorage as shown in Fig. 2, it is obvious that the test specimen is found to be stable enough even after 10 cycle repetition of loading with story drift of 1.5/100 up to 5/100. Conventional type of anchorage with the embedded reinforcing bars in the column was also tested under the same condition, and it was found that there were no significant differences between them.

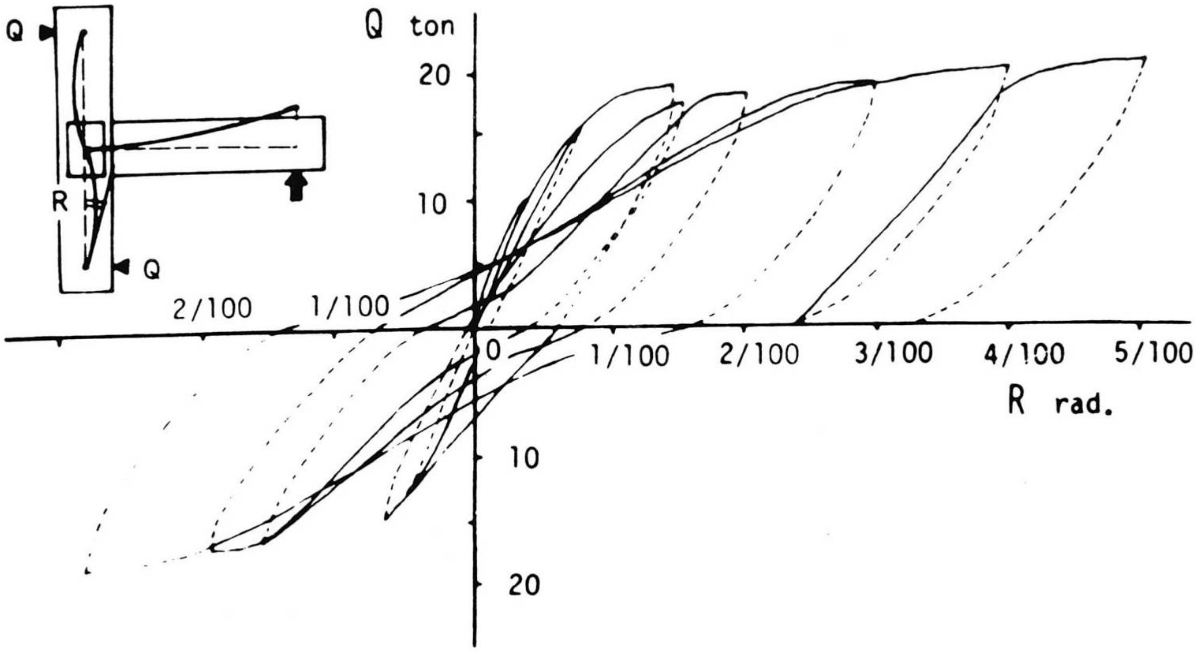


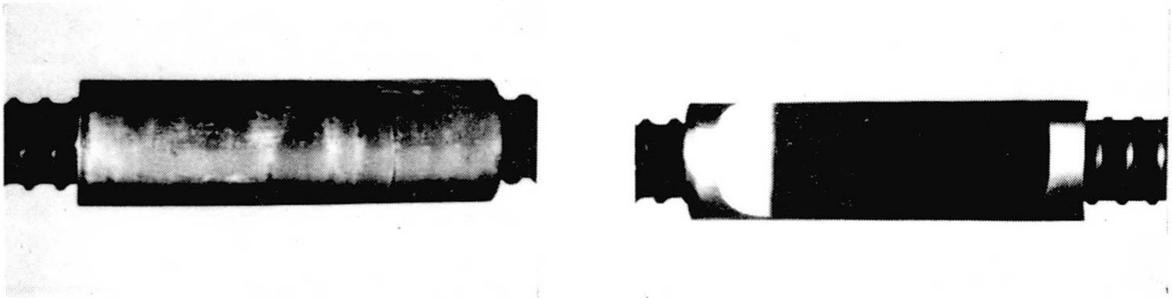

Fig. 2 Load-Deflection Curve of Exterior Framing including U Anchor Method

## 2.3 Joints of large-size reinforcing bars

When the stress in columns and beams increases in tall buildings, it becomes necessary to use large-size reinforcing bars. In such cases it is desirable to use a welded butt joint or sleeve joint rather than a lap joint. A welded butt joint is effective for the size of D32(#10) or less.

Various types of sleeve joints for heavy (D35 or more) bars were tested. Among them the Cadweld joint, mortar joint and squeeze joint was found to be highly reliable, satisfying all of the requirements for reinforced concrete specifications. In case of Cadweld or mortar sleeve joints, special moltend metal or strong mortar is used as joiner or connector between steel bar and sleeve. Regarding the squeeze type, the inside surface of the sleeve interlock with ribs of a specially developed reinforcing bar such as Rivercon as shown in Fig. 3.

For the building described herein, Cadweld joints were used but for the second tall building the squeezed joints were used after making cost studies of both joints.



a. Squeezed Joint b. Cut off Section

Fig. 3 Squeezed Sleeve Joint

# 3. SEISMIC DESIGN

## 3.1 Seismic design criteria

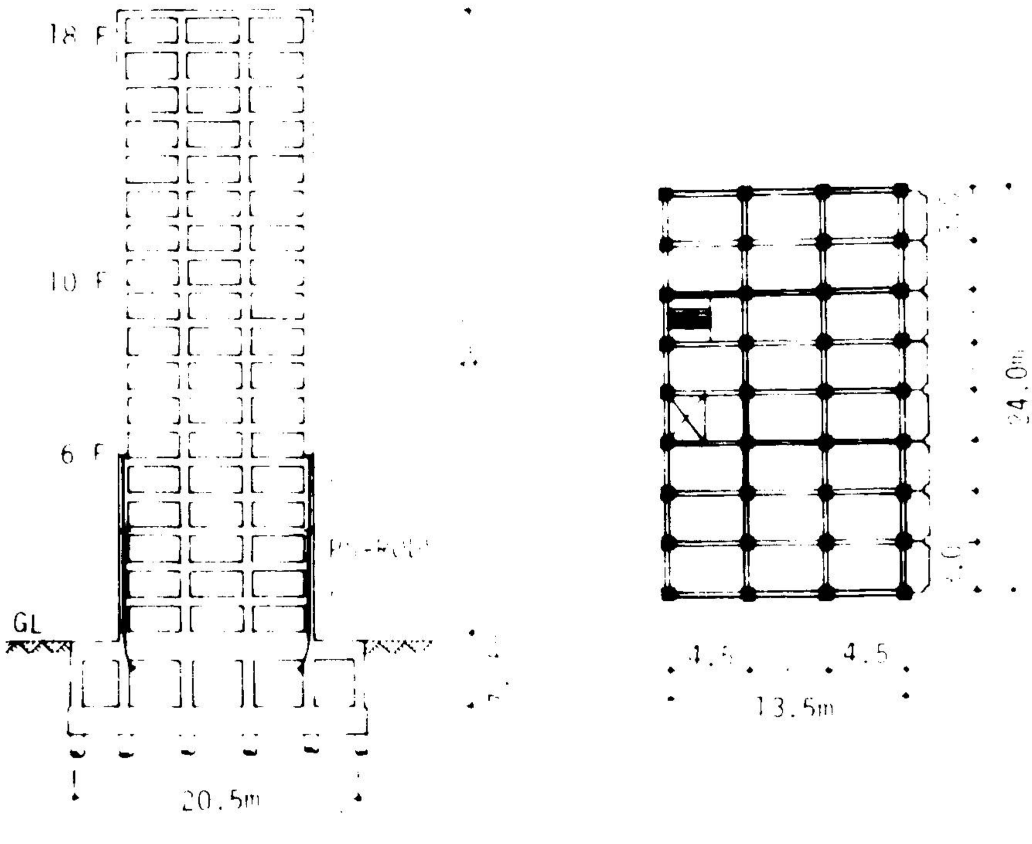
The author has established a basic criteria of earthquake resistant design. It is classifying earthquake intensities into three classes, and regulating the response of the building or degree of damages as follows:

| | | |
|---|---|---|
| Class I | Moderate earthquake (Max Acc. 0.1g) | No structural damages |
| Class II | Severe earthquake (Max Acc. 0.3g) | The stress in all members should be within allowable stress |
| Class III | Worst earthquake (Max Acc. 0.5g) | Building suffers some damages but never collapses |

Maximum accelerations of the earthquakes were increased appreciably, in consideration of the fact that this building was to be the first highrise reinforced concrete structure in Japan exceeding the prior legal height limit.

## 3.2 Outline of the building

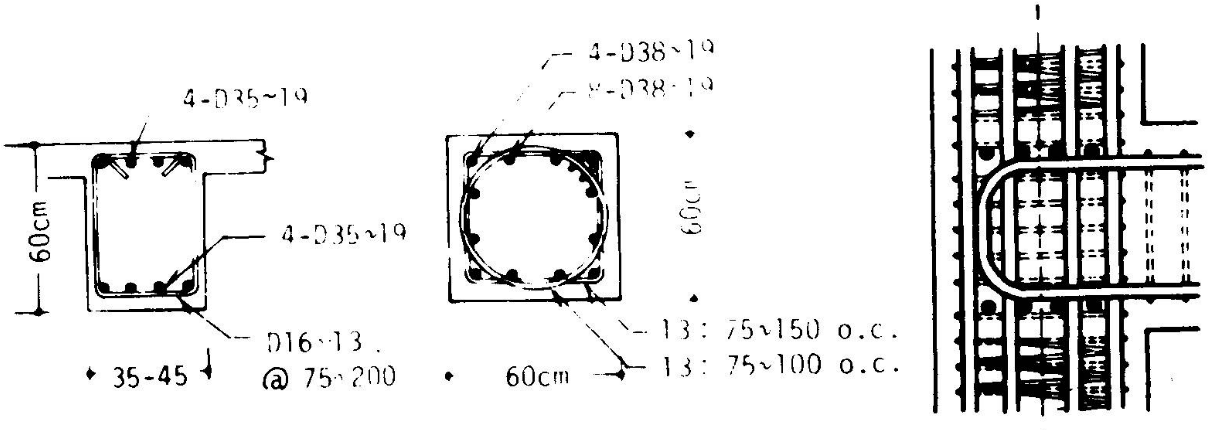
The building herein has 18 stories with one story basement and is 48.9 meters tall. The outline of the building structure is shown in Fig. 4 together with typical structural detials.




a. Framing Section

b. Framing Plan

Fig. 4 Outline of Structure




c. Beam d. Column e. U-Anchor Method

Fig. 4 Outline of Structure

Columns are all 60cm square which shear reinforcements consist of Kajima Spiral. Shear reinforcement ratios are 0.75 - 1.20% which was determined after various experiments. Beams are all 60cm in depth and 35, 40 or 45cm in width. The reinforcing bars of beams have newly developed U-shaped anchorage, thereby simplifying the task of prefabricating the reinforcement as mentioned before. Moreover, the exterior columns from the first to the sixth floor are prestressed with PS steel rods as a protection from the tensile forces which may be induced by overturning moments. This step exceeded the required safety standards and furthermore registered an important advancement in the design of still higher buildings.

## 3.3 Dynamic analysis

### a. Vibration model

In order to analyze dynamic behavior of the building accompnied by cracking and yielding, an idealized vibration model with 18 lumped mass is established. As shown in Fig. 5, the vibration model is assumed to have two kinds of stiffness. One is shearing stiffness which is derived from bending and shear deformation of beams and columns and shear deformation of beam-column joints, while the other is bending stiffness due to axial deformation of columns. Shear stiffness is assumed to have nonlinear degrading property and bending stiffness remains linear.

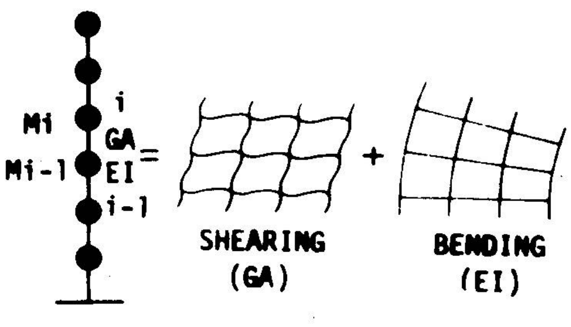



Fig. 5 Modeling

b. Shear stiffness and idealization of hysteresis loop

Static nonlinear frame analysis against the gradually increased lateral forces is carried out. Then the relations between story shears and shearing drifts are idealized by three straight lines (skeleton curve) in consideration of cracking and yielding of the members. In the nonlinear dynamic analysis shear stiffness property is defined by the skeleton curve and idealized hysteresis rule. Hysteresis loop with degrading stiffness property is idealized as shown in Fig.6.

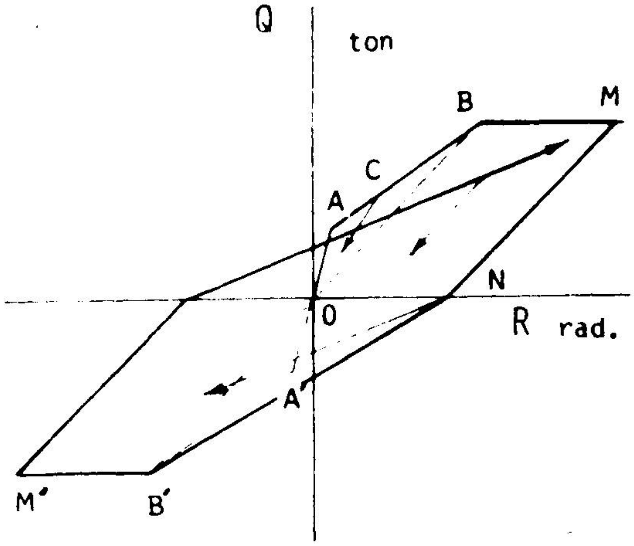



Fig. 6 Idealized Hysteresis Loop of Shear Stiffness

Point A in the figure indicates the first tensile crack. Up to this point, the structure is completely elastic. In the region between points A and B the curve tends to return to the origin (O) when unloading occurs. Point B means that yielding has occurred in the beam. In case where the load is decreasing at point M, the rigidity is the same as the gradient OB until the load becomes zero. Once zero is achieved, the curve points towards the opposite maximum deflection ever experienced.

c. Analytical conditions

Assuming that the building is fixed on the 1st floor, four types of input earthquake waves, El Centro 1940 NS, Taft 1952 EW, Tokyo 1956 NS and Sendai 1962 NS are adopted. Maximum acceleration of ground motion are selected 0.1g, 0.3g and 0.5g corresponding to the design criteria. Viscous damping is assumed and 3% of critical damping for the elastic 1st mode is adopted.

d. Results of dynamic analysis and seismic safety

The 1st vibration periods in the elastic range are 0.81 sec. in longitudinal direction, and 0.95 sec. in transverse. Responses of the dynamic analyses show similar results for longitudinal and transverse directions. Hence the results of longitudinal direction only will be described. Fig. 7 shows the maximum story shear and story drift resulting from severe earthquake (0.3g). It is noticed that stresses in all members remain in allowable values and maximum story drift is only 1.22cm at 11th story, which correspond to 0.45/100 deflection angle (see Table 1). Under the worst earthquake of 0.5g, yielding occurs but the maximum ductility factor remains at 1.66 at 12th story. These response values satisfy entirely the seismic design criteria established at the outset.

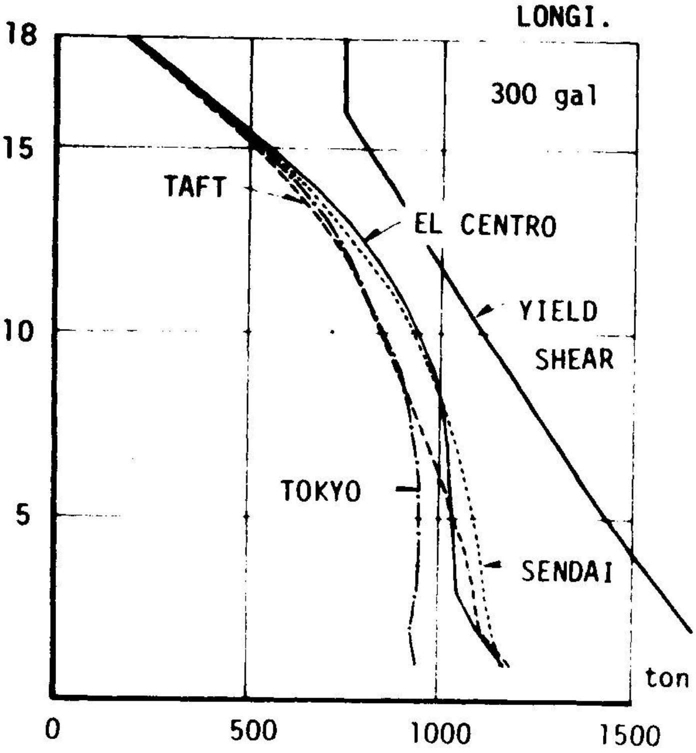



Fig. 7 Maximum Story Shear (0.3g)

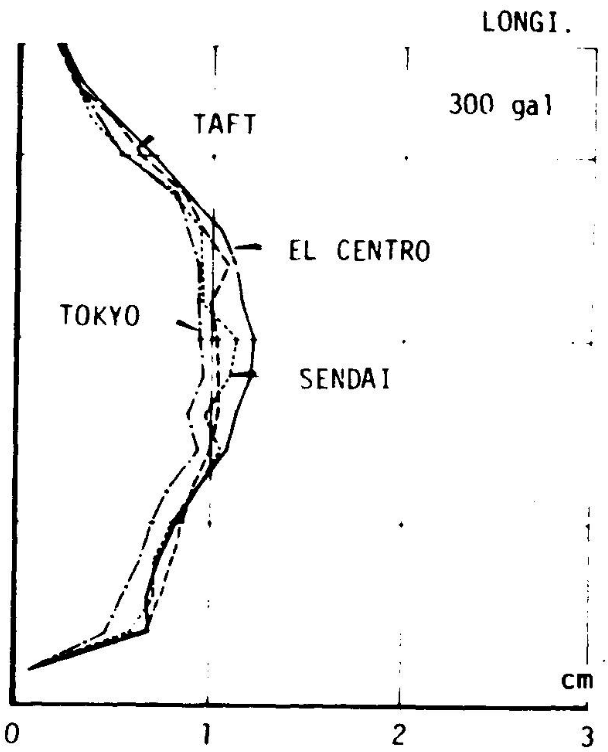



Fig. 8 Maximum Story Drift (0.3g)

Table 1. Damage Evaluation due to Earthquake Response

| Intensity of earthquake | Base shear (B.S. coefficient) | Story drift | Yield (Steel) | | Ductility factor |
|---|---|---|---|---|---|
| | | | Column | Beam | |
| Class I (100 gal) | 950 ton (0.17) | 0.4cm (at 11th story) | None | None | - |
| Class II (100 gal) | 1180 ton (0.21) | 1.22cm (at 11th story) | " | " | - |
| Class III (500 gal) | 1910 ton (0.34) | (at 12th story) | " | 2-16th Floor | 1.66 (at 12th Floor) |

## 4. POST CONSTRUCTION STUDIES

### 4.1 Forced vibration test

Immediately before the completion of the building, vibration tests were carried out. Vibration exciter was installed at the 19th(roof) floor. The periods and damping factor in transverse direction were obtained as shown in Table 2. The periods obtained by the test were about 20% shorter than those estimated in designing due to the difference of stress level (1st mode 4 - 5 gal, 2nd mode 10 gal in test) and the absence of live loads. It is noteworthy that the displacement at the top in the 1st vibration mode includes 1.4% of sway and 7.6% of rocking due to the deformation of soil and piling.

Table 2 Periods and Damping in Transverse Direction

| | 1st | 2nd | 3rd |
|---|---|---|---|
| Period | 0.83 (sec) | 0.27 | 0.15 |
| Damping factor | 1.5 (%) | 3.1 | 6.5 |

## 4.2 Earthquake observation

Earthquake observations are being taken by servo-type accelerographs installed at the underground, basement, 9th and 19th floors. Among many observed records, the following two earthquakes are noteworthy.

| | | A Earthquake (Izu Peninsula Coast) | B Earthquake (Eastern Saitama) |
|---|---|---|---|
| Occurred | : | May 9, 1974 | August 4, 1974 |
| Magnitude | : | 6.9 | 5.8 |
| Location of Focus | : | 138°48'E 34°34'N | 139°55'E 36°01'N |
| Focal Depth | : | 20 kilometers | 20 kilometers |
| Epicentral Distance | : | 150 kilometers | 40 kilometers |

The local intensities of the site are both 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. 9. Acceleration response spectra are also shown in Fig. 10.

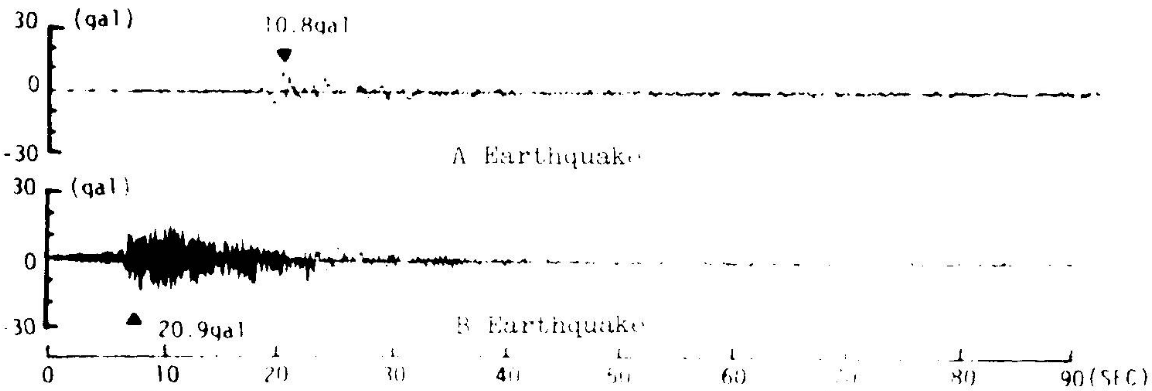
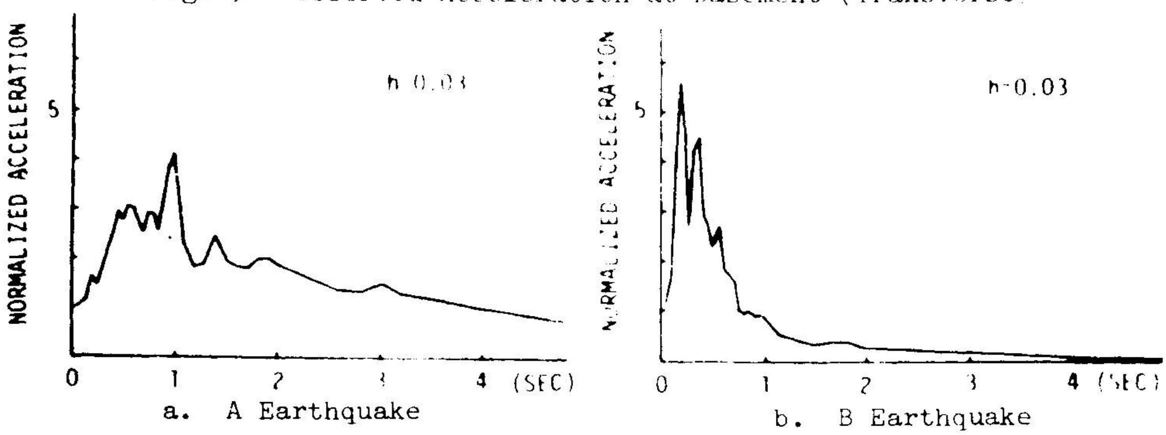



Fig. 9 Observed Acceleration at Basement (Transverse)




a. A Earthquake

b. B Earthquake

Fig. 10 Acceleration Spectra of Observed Earthquake (Transverse)

The frequency components of the two earthquakes were quite different that the motions of the building varied correspondingly. In the A earthquake, the first vibration modes of 0.8 seconds in the longitudinal direction and 1.0 second in the transverse were prominent. For both directions the amplifications of the accelerations were six times at the top floor relative to the basement. By contrast in the B earthquake, the second modes of 0.3 seconds were the most prominent in both directions and the amplification factors were only two.

## 4.3 Simulation of earthquake motions

### a. Vibration model

Essentially same model as that was adopted in the seismic design was used for the simulation of the earthquakes. Some modifications have been incorporated in reference to studies on forced vibration test. For instance, a freedom of base rotational motion is additionally considered in this model. Rotational stiffness is assumed considering the reactions of soil and piling. Therefore, the 1st and 2nd vibration periods of the model in transverse direction are to be estimated 1.02 seconds and 0.32 seconds. It was decided that, damping for reinforced concrete should differ 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%.

### b. Simulated results

#### Case of A Earthquake

Computed acceleration time history due to the A earthquake at the 19th floor is compared with the observed earthquake waves as shown in Fig. 11. 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. 12 also show that both accelerations are quite identical.

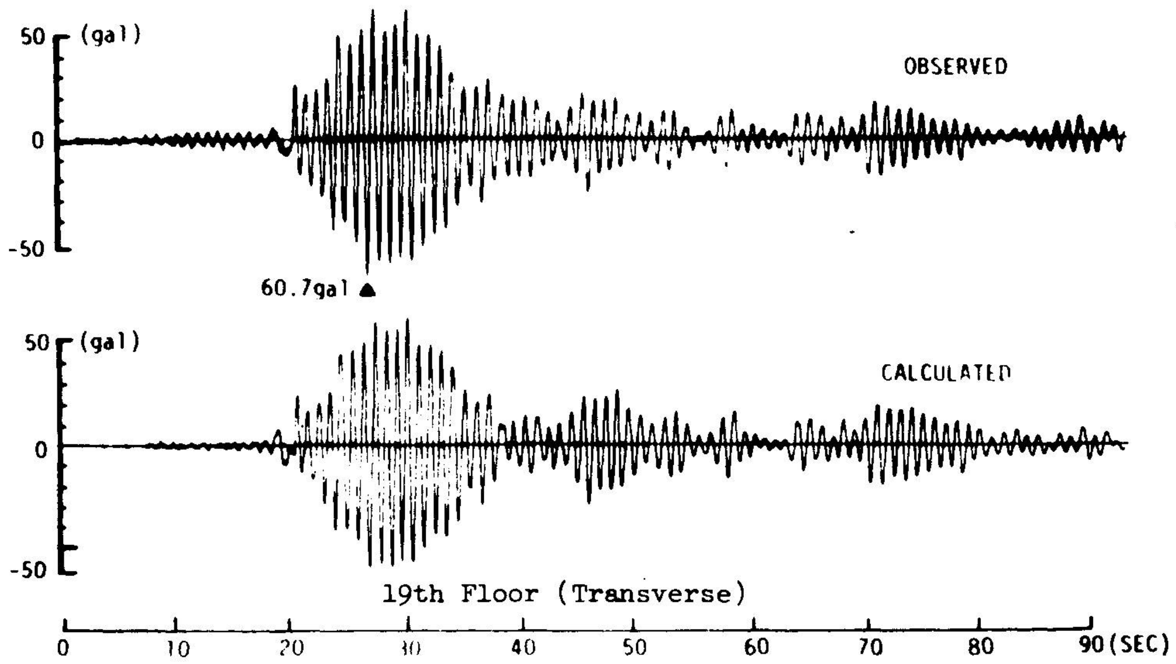



Fig. 11 Acceleration Time History (A)

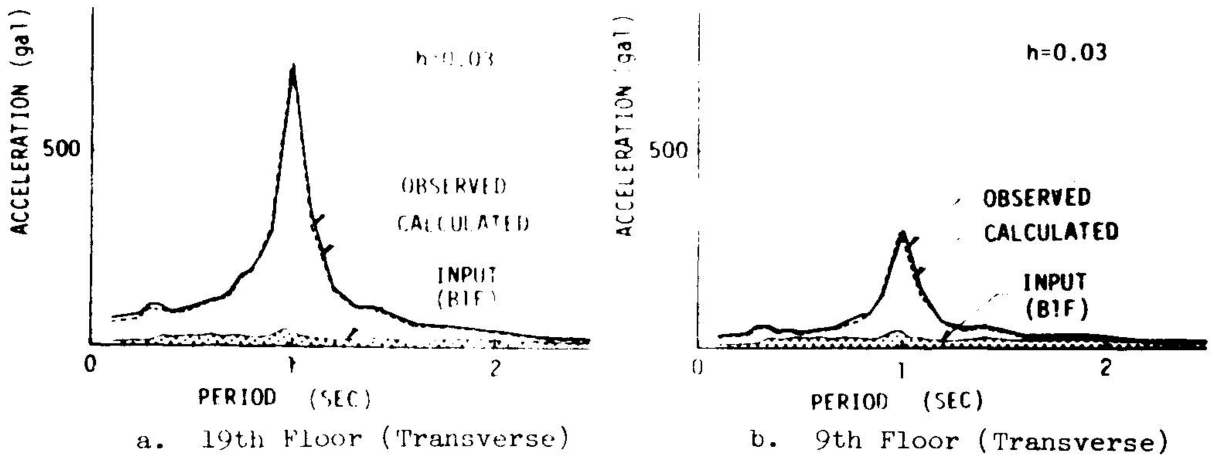



Fig. 12 Comparison of Acceleration Spectra (A)

Case of B Earthquake

Fig. 13 shows the comparison of the observed and computed accelerations due to the B earthquake. Reflecting the fact that the basement record has prominent periods of 0.3 and 0.2 seconds 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. Amplication ratios of acceleration both at the 9th and 19th floors are about twice as that of basement acceleration. These tendancies are clarified by Fig. 14 showing the response spectra. It is also concluded that dynamic behaviors of the building are precisely reproduced by analytical simulations throughout lengthy duration of the earthquake.

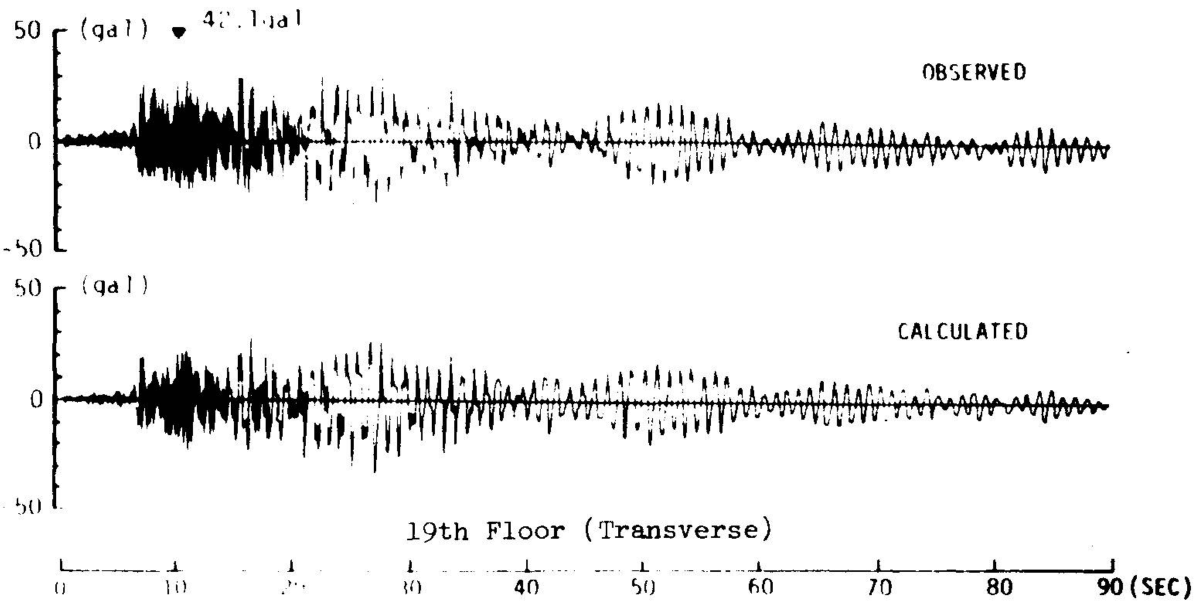



Fig. 13 Acceleration Time History (B)

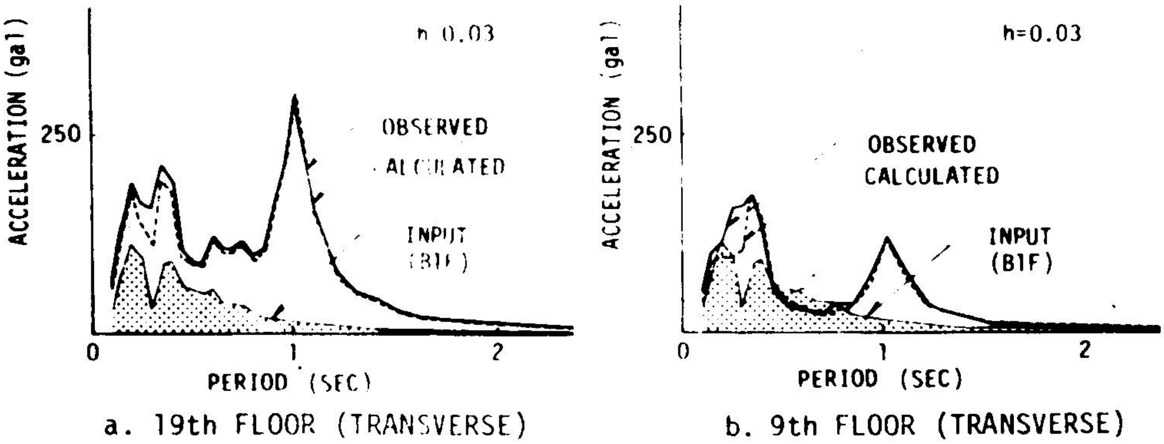



Fig. 14 Comparison of Acceleration Spectra (B)

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