SIMULATION ANALYSIS ON HIGH-RISE BUILDING BEHAVIOR BY THE USE OF STRONG EARTHQUAKE MOTIONS ACTUALLY OBSERVED

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SUMMARY

An 18-storied, steel framed reinforced concrete building in Sendai City, Japan was subjected to strong ground motions during the Miyagi-Ken-Oki earthquakes of February 20 and June 12, 1978. This paper describes the results of the comparative study for the elastoplastic seismic analysis made on the basis of the strong earthquake motion recorded on the building and the actually measured values. Concurrently, conventional changes of pre- and post-earthquake natural period of the building are introduced.

INTRODUCTION

In Japan, structural design of high rise buildings is generally made by what is known as the earthquake response analysis method. It is highly important for seismic design of buildings to know accurately the behaviors of buildings under a strong earthquake and consider them in a theoretical manner. In Japan, however, there had been no case where the structures designed on the basis of dynamic analysis were actually subjected to the strong earthquake motion, and this building presented the first case where actual behaviors of the structure under strong earthquake motions were really studied. In this present paper, the structural behaviors of Sumitomo Sendai Building which we designed by the dynamic method and experienced two great earthquakes (Miyagi-Ken-Oki Earthquakes) are described from various view points.

DESCRIPTION OF THE BUILDING

This building is an 18-storied office building with two basements, and has the plan and section as shown in Fig. 1.

The building is nearly square in plan, measuring 32.4 m x 32.6 m, and rises to 66.0 m above the ground level. The structure is mainly of steel framed reinforced concrete construction with lightweight concrete. Main framework of the building consists, in both directions, of reinforced concrete shear walls. The foundation of the building is in the form of mat foundation which is directly supported on hardpan.

Structurally, the building was designed by a dynamic aseismatic process in which repetitive earthquake response analyses were made, with the basic principle that the structure should remain in an elastic region when subjected to a ground motion of 200 gals and that the ductility factor for each story

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should not exceed 2.0 when it was subjected to a strong ground motion of 400 gals.

DESCRIPTION OF EARTHQUAKES

The building experienced two earthquakes which are briefly described below.

	The First Earthquake	The Second Earthquake
Date of Occurrence	February 20, 1978	June 12, 1978
Epicenter	Lat. 142.04° E	Lat. 142.23° E
	Lon. 38.78° N	Lon. 38.17° N
	Depth 56 km	Depth 25.8 km
Magnitude	6.8	7.4

Wave patterns and Fourier spectra of these two earthquakes are shown in Figs. 2 and 3 respectively.

DAMAGE OF THE BUILDING

During the earthquake of February 20, 1978, the maximum acceleration of 100 gals and 360 gals were recorded at the second basement and the 18th floor respectively; however, the damage due to this earthquake was limited to some cracks in partition walls, and no cracks were observed in columns, beams, and shear walls. In the Miyagi-Ken-Oki Earthquake of June 12, 1978, the lateral motion of about 280 gals on the second basement and about 580 gals on the 18th floor were recorded. This latter earthquake caused cracks in concrete partitions and damage to some items of building service equipment. In addition, it caused such damage to structural elements as minor cracks due to shear in the east-west shear walls around the central core on almost all floors and also very slight cracks due to bending at the outermost ones among the adjoining beams at the center spans in the north-south direction on almost all floors.

STRUCTURAL ANALYSIS

Frame Model

The fact that some structural damage was caused to this building by the earthquake on June 12 makes it necessary to conduct elastoplastic response analysis in connection with this earthquake. For dynamic response analysis which covers elastoplastic range, a model in which the building is converted to the equivalent shear spring is often used. However, in the case of this specific building where main aseismatic framework is continuous through stories, the bending-shear type model can be a more realistic simulation of the original than the shear-type model in which each story reacts to earthquakes independently. For the purpose of this analysis, the main shear walls continuous through all the stories have been replaced by equivalent linear members consisting of wall-columns, and these linear members have been connected to adjoining beams to obtain the bending-shear spring for the wall-columned framework in order to make the elastoplastic earthquake response of a framework having specific bending-shear deflection characteristics like this one. As for the other framework, the shear spring has been obtained, and these two types of springs have been combined to establish the model of the whole framework. Then, by assuming respective elastoplastic restoring characteristics for the wall-columns, boundary beams and shear springs, the earthquake response analysis has been conducted.

The model has been made by taking out a 1/4 of framework by supporting the ends of beams on rollers on the axes of symmetry of the building both in the N-S and the E-W directions (See Fig. 4).

Restoring Characteristics

Wall-columns: Bending rigidity is assumed as elastic, and the following bilinear proportion is assumed for shear-bending rigidity. That is, if the lateral, vertical and rotational deflection of the node i of the wall-columned frame is represented by u_i, v_i and θ_i respectively as shown in Fig. 5, then, the shear force Q_i acting on the wall-column on the i-th story may be expressed by the following equation:

$$Q_i = GA_s \cdot \frac{\alpha}{2(1+\alpha)} \{\frac{2}{h_i} \cdot (u_i - u_{i-1}) - \theta_i - \theta_{i-1}\} \qquad \text{----- (1)}$$

In which,

$$\alpha = \frac{12EI}{(\ell - \ell_i)^2 GA_s}$$

where,

- EI : Elastic bending rigidity of cross section of wall-column on the i-th story
- GA_s: Elastic shear rigidity of cross section of wall-column on the i-th story
- h_i : Story height of i-th story

In the right side of Equation (1), let it be expressed that

$$\{\frac{2}{h_i} \cdot (u_i - u_{i-1}) - \theta_i - \theta_{i-1}\} = w_i$$

where,

$$Q_i = GA_s \cdot \frac{\alpha}{2(1+\alpha)} \cdot w_i \qquad \text{----- (2)}$$

Then, assume that Q_i and w_i have the bilinear restoring characteristics as shown in Fig. 6. Q_{cr} is taken as the resistance against initial shear cracks in the wall-column and is obtained by the following formula:

$$Q_{cr} = \beta_{cr} \cdot GA_s \cdot R_{sc} \qquad \text{----- (3)}$$

where,

- β_{cr}: Rigidity reduction factor at the occurrence of initial shear cracking
- R_{sc}: Deflection angle use to shear at the occurrence of initial shear cracking

In this analysis, β_{cr} has been taken as 0.9 and R_{sc}, as 0.4×10^{-3} (radians). ${}_cK$ represents elastic rigidity, and the secondary gradient ${}_cK_2$ which occurs after initial cracking has been evaluated by taking elastic shear rigidity as 1/5.

Adjoining Beams: Shear rigidity has been considered elastic, and the following bilinear relationship has been assumed for the bending rigidity. As shown in Fig. 5, if the distance to the point of inflection of the boundary beam on the i-th story is taken as ℓ, and the length of rigidity zone, as ℓ_i, then, the rigidity zone end moment M_i of the boundary beam may be computed by the following equation:

$$M_i = \frac{12EI}{(\ell-\ell_i)^2} \cdot \frac{1}{(4+\alpha)} \cdot (\ell\theta_i - v_i) \qquad \text{----- (4)}$$

In which,

$$\alpha = \frac{12EI}{h_i^2 \cdot GA_s}$$

where,

EI : Elastic bending rigidity of the boundary beam on the i-th story
GA_s: Elastic shear rigidity of the boundary beam on the i-th story

By taking the term $(\ell\theta_i - v_i)$ on the right side of Equation (4) as g_i, the following formula can be obtained:

$$M_i = \frac{12EI}{(\ell-\ell_i)^2} \cdot \frac{1}{(4+\alpha)} \cdot g_i \qquad \text{----- (5)}$$

In the above equation, the bilinear restoring characteristics as shown in Fig. 7 are assumed for M_i and g_i. M_Y is the resistance against bending of the boundary beam and is taken as the sum of the full plastic moment of steel cross section and the ultimate bending strength of reinforced concrete cross section.

Shear Spring: All frames other than the wall-columned ones have been replaced with equivalent shear springs in the following manner using the same model and design loads as previously mentioned. Firstly, the elastic rigidity is computed as shear spring from the story shear force and story drift of each story when the simulation model for elastic analysis is subjected to the design load. Here, it should be noted that this building is designed as "beam yield type" structure; therefore, plasticization of beams and shear walls within the range of anticipated response only needs to be considered. Thus, the resistance of shear walls against initial cracking and bending strength of beams in each frame have been computed by the same methods as described in 'Wall-columns' and 'Adjoining Beams' above.

Computation of Numerical Values

From the overall bending-shear spring $|K|$ having lateral, vertical and rotational freedoms obtained in the foregoing manner, the vertical and rotational freedoms are eliminated as they are only dependent variables which are irrelevant to vibration. Thus, the bending-shear spring $[k]$ for response computations is obtained with respect to the principal variable, i.e., the freedom in lateral direction only. From this and the mass concentrated on each floor position, 20 mass points fixed at the foundation are set up.

Then, the earthquake response analysis has been conducted by inputting the values observed and recorded at the second basement during this earthquake.

Response Acceleration Wave Patterns

The response acceleration wave patterns as obtained by this elastoplastic analysis for the 9th and 18th floors in the N-S and E-W directions are shown in Fig. 8 together with the corresponding measured record. The relative time relationships between the values measured on respective floors are unknown; therefore, the comparison of the wave patterns obtained by analysis with those obtained by the measured record will be made only in connection with the locations where the wave peak positions of the analytical patterns and those of the measured patterns are comparatively in good coincidence for the primary earthquake waves recorded within an initial few seconds of the measurement at each floor. According to such comparison, it is found that the measured wave patterns and the analized wave patterns are in very good coincidence at the 8th and 19th floors with respect to the N-S direction. In the E-W direction, some difference between the peak values and wave patterns obtained by analysis and those recorded is noticeable as to the both floors after a lapse of about 3.5 seconds from the beginning of the earthquake; however, fairly good coincidence is observed between the two types of peak values and wave patterns within an initial 3 seconds of the primary ground motion.

NATURAL PERIOD OF THE BUILDING

In this section are discussed the variation of natural period of the building and the comparison of its measured values and analytical values.

Upon completion of this building, the forced vibration tests were conducted by the use of the oscillator and micro tremors were measured in order to measure its natural period. Further, in early September 1978 or about three months after the second severe earthquake, micro tremors were measured again.

The primary and secondary natural periods as obtained from the peak values of the power spectra recorded by measurement are shown in Fig. 9. In this figure, the values obtained by other forced vibration tests as well as the values obtained from the Fourier spectra found in the measurement record for the 18th story of the building during the earthquakes on February 20 and June 12 (both in 1978) are also indicated.

From these table and figure, it is found that the natural period during the February 20th earthquake was 30 % longer than that under micro tremors at the time of completion of the building. The natural period became still longer during the strong earthquake motion on June 12th which caused partial plasticization of the building structure.

After the building underwent the two strong earthquakes, its natural period was lengthened by 20 % compared with the pre-earthquake natural period.

To compare the measured values with those obtained by analysis, the values obtained from the elastic rigidity and the measured values recorded during the February 20th earthquake in which the structure is believed to have behaved within the elastic limit are indicated. These two kinds of values are in good coincidence with respect to the primary natural period.

CONCLUSION

A simulation analysis was conducted for an 18-storied steel framed reinforced concrete building in Sendai City which was subjected to severe earthquakes twice in 1978. The frame composed of multi-storied shear walls and boundary beams connected to them was replaced by a bending-shear spring, and the frame of the other types was replaced by an equivalent shear type spring.

Analysis of these two types of springs as combined indicated that the measured values and the analytical ones were in good coincidence.

From the process of plasticization of a part of the building as indicated by the analysis, the conditions of damage to the building caused by the earthquakes is believed to have been fairly well ascertained.

This building was designed in such a way that the main aseismatic elements were composed of multi-storied shear walls with the boundary beams connected to them. It was the intent of the design that the earthquake energy input be prevented from being concentrated on a specific story but be absorbed by all the stories by letting the yielding of the boundary beams due to bending occur before the yielding of the shear walls. The results of the analysis endorse that such purpose of the structural design was met with sufficient success. The analysis results have led to the belief that it has now become possible to make structural evaluation of this type of buildings by means of elastoplastic analysis of bending-shear type.

As regards the natural period, the measured period under micro tremors at the time of completion of the building was considerably shorter than the analytical value. But it is found that the actual natural period was lengthened to equal the analytical value during the earthquake of February 20, 1978. The natural period became still longer during the strong earthquake of June 12, 1978 when a part of the structure was plasticized; however, the period under micro tremors after the strong earthquake was fairly short although it was not so short as the period observed at the time of completion of the building.

ACKNOWLEDGEMENT

Valuable help was given authors by the concerned personnel of Sumitomo Life Insurance Company for publication of this report. Dr. Toshio Shiga, professor at Tohoku University and Dr. Akinori Shibata, associate professor at the same university offered to the authors not only the data on the forced vibration and on the normal micro tremor measurement, and the records of the strong earthquake on February 20, 1978 but extended to them very beneficial guidance. The micro tremor measurement after the strong earthquake was conducted by Toda Construction Co., Ltd.'s Technical Research and Development Center. Intensive help was given to the authors also by Messrs. Taira Yoshikawa, Shigezo Furui, Osamu Nishida, Hideaki Kirihara and Koichi Matsunaga, all of Nikken Sekkei Ltd and Mr. Fumiya Ezaki of Kyushu University. To all them, the authors express their heartfelt gratefulness.

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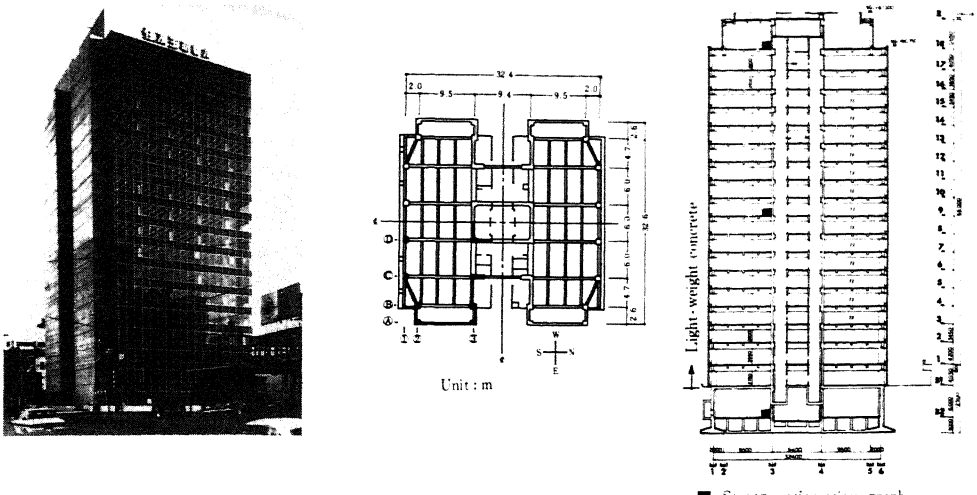

Fig. 1 Plan and Section

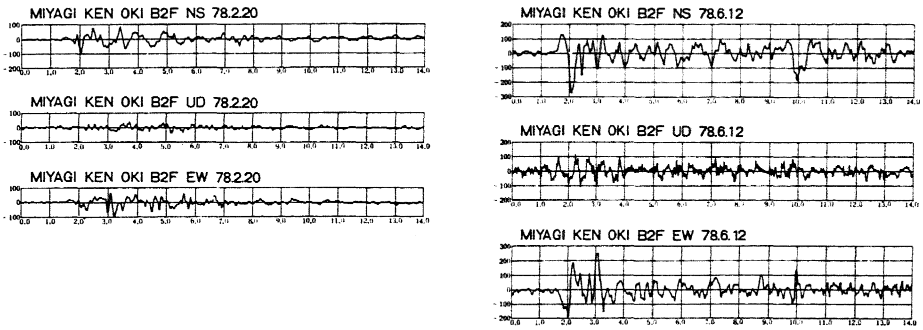

Fig. 2 Miyagiken-oki earthquake waves recorded on the 2nd basement Unit : gal, second

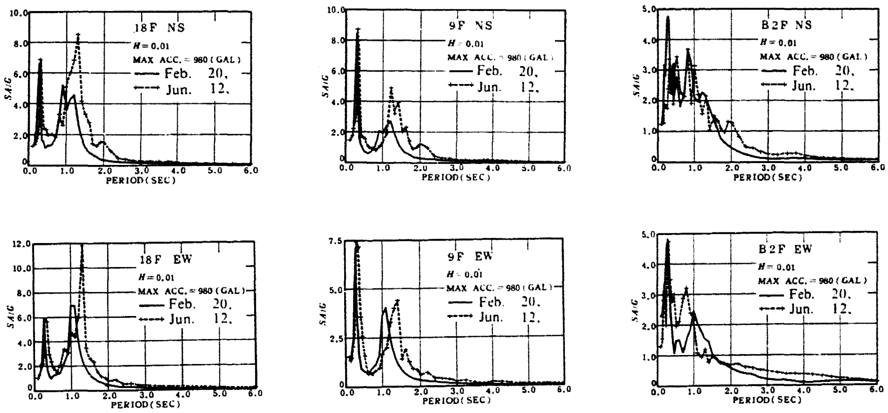

Fig. 3 Fourie Spectra

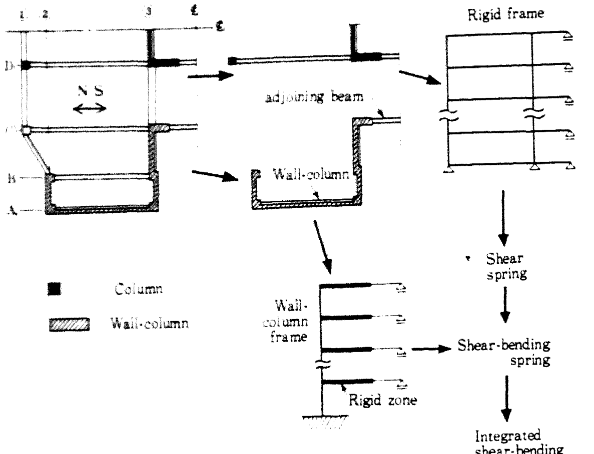

Fig. 4 Models of frames in N-S direction

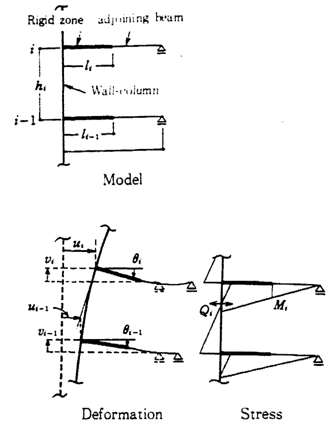

Fig. 5 Deflection and stress of Wall-column

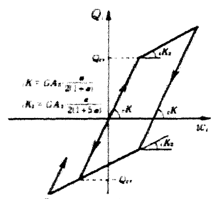

Fig. 6 Restoring characteristics of wall-column (shear)

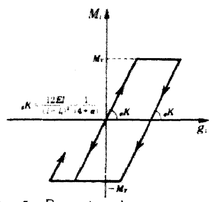

Fig. 7 Restoring characteristics of adjoining beam (bending moment)

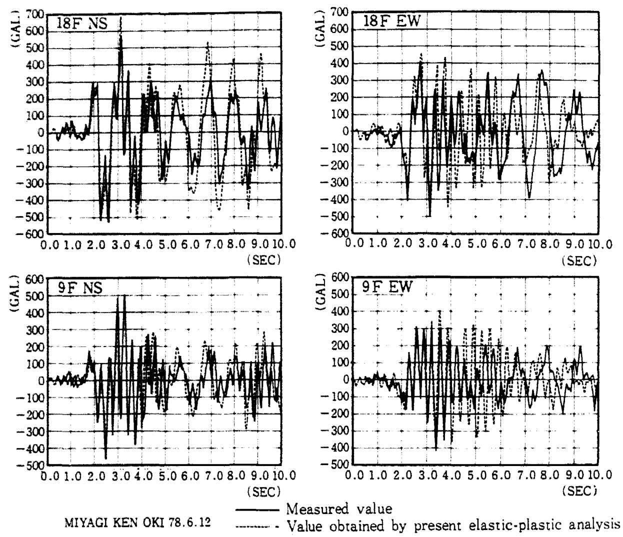

Fig. 8 Wave patterns of response acceleration

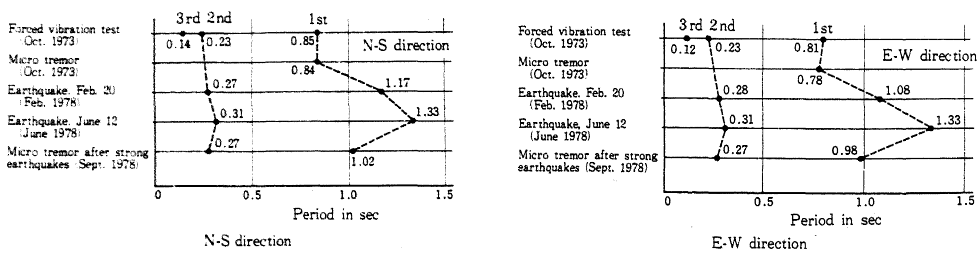

Fig. 9 Transition of natural period