ANTI-SEISMIC DESIGN OF A HIGH-RISE BUILDING IN DJAKARTA

by

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Synopsis

This paper presents a draft of a dynamical analysis method which was applied to the structural design of a high-rise building proposed to be built in Djakarta, Indonesia. In this method, the seismic design forces are assumed on the basis of the seismicity in respective region and of the subsoil characteristics of the site, and a primary structural design is carried out for these assumptive seismic forces. The framework designed primarily is then analysed by an electronic computor to detect how it will respond to some earthquake spectra recorded in the past, and is adjusted in accordance with results of the computor analysis.

1. Introduction

A 29 storied building is under project in Djakarta, Indonesia. At this moment, June 1964, various laboratory tests are being carried on to obtain back data for the assumptions employed in the antiseismic analysis and the design of structure that have already been nearly completed. In this paper we will report only the outline of dynamical analysis of the structure and its basic conceptions.

We are very much obliged to Dr. S. Omote, Dr. T. Hisada, Dr. K. Nakagawa and Mr. N. Nakajima of the International Institute of Seismology and Earthquake Engineering, Japanese Government, in the study of seismicity, introduction of seismic forces and micro tremor observations. We would like to express our deepest appreciation also to Dr. K. Muto who gave us many helpful advises in this study.

2. Outline of the Building and Design Conceptions

a. The building is composed of two blocks, one being the tower block and the other the flat. The former will be almost devoted to office spaces, except the uppermost floor being scheduled to accomodate some entertainment facilities. The roof deck is planned to be a heliport. In DjaKarta a 16 storied hotel, a reinforced concrete structure, has been completed and another reinforced concrete building, a 14 storied department store, is now under construction both by Japanese contractors. Construction of this building, which has 29 stories above and one below the ground level,

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will be much contributed to by the experiences gained in those two projects, but the fact that Indonesian Archipelago is one of noted earthquake regions in the world required our careful consideration concerning earthquake resistance.

b. Because of the requirement of Architects, rather to say owners, the plan and elevations of this building are not quite "simple", which is the ideal of high-rise building structures. Another condition which required our careful consideration was that almost all building materials would have to be procured and fabrication of structural steel and others would have to be made in Japan. Therefore, the design of structural details was made in such a manner that would be most suitable for prefabrication and transportation.

c. Followings are the outline of the building: (ref. Fig. 1)

Ground floor area	The Tower block approx.	1,150 sq.m
	The flat block approx.	5,300 sq.m
Gross floor area	The Tower block approx.	30,000 sq.m
	The Flat block approx.	10,000 sq.m

Structural scheme of the Tower block

Below G.L.	Steel framed reinforced concrete
Above G.L.	Steel framed (SS41 SS50)[6)
Floors	Reinforced concrete on metal decking
Columns	H shape steel, fireproofed with light-weight concrete block.
Beams	I shape steel, fireproofed with asbestic spray.

d. Foundations were designed to be supported directly by a tuffaceous stratum at the depth of 9.0 meter below the grade, of which another discription will be given later. It was judged that the stratum at this level would be sound enough to support the dead weight of the building estimated 20 - 30 tons/sq.m. upon the ground. It was also assumed that the underground structure had an extreme rigidity, because of its steel framework embedded in reinforced concrete, could be treated as a perfectly rigid body in the dynamical analysis hereinafter described.

3. Seismicity

Among considerable numbers of papers regarding the seismicity in Indonesian Archipelago, a map prepared by Gutenburg and Richter in 1955 is most noted. In "Seismic Zones in Indonesia", Geographical Note No. 2 presented by Mr. Suetadi also compiles extensive data related to the seismicity of the country. These data, however, have not yet been brought to practical application by means of such as statistical analysis.

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6) SS41 $F_t = 41 - 50$ kg/mm^2, $\sigma_y > 23$ kg/mm^2, in case $t > 38$ mm, $\sigma_y > \frac{F_t}{2}$
SS50 $F_t = 50 - 60$ kg/mm^2, $\sigma_y > 28$ kg/mm^2.

The estimation of seismic design force to be applied to the anti-seismic analysis of building structure is a matter of great importance, and it usually depends largely upon judgement of structural engineers. In this case, however, it was considered to be a practical way of estimation to introduce seismic design force on the basis of the severest earthquake ever occurred in the surroundings of the site.

Fig. 2 shows the distribution of shallow earthquakes which are closely related to seismic design forces. This has been drawn up by Dr. S. Omote basing upon data in "Earthquakes in Indonesia up to 1960" by Ministry of Communication, Meteorological and Geophysical Institute of Djakarta, and referring to materials presented by Gutenberg and Richter.

According to this map, the largest earthquake around Djakarta is that in 1943, which had its epicenter at the west coast of Java Island at Sumatra Strait, 150 km away from Djakarta, and a Magnitude above 7.

Judging from above data and some other materials available, we have decided to begin with an assumption that an earthquake of magnitude 7.5 occurs at distance of 150 km from the site for figuring out the seismic design forces.

4. Subsoil Conditions of Djakarta and the Building Site

The plain of Djakarta is approximately 40 km wide in N-S direction and its subsoil layer consists of a tertiary stratum which occasionally outcrops as it folds, overlain with deluvial tuff and further with alluvial river deposit and volcanic mud from its hinterland.

Fig. 3 shows a modelized column of boring test. Under a comparatively shallow top layer of the alluvial deposit, there is a layer of tuffaceous deposit which becomes denser as it goes deeper, and it forms a very dense taffaceous stratum (or a sand stone stratum), ranging from a depth of 12 m to 18 m. This stratum is further underlain with a comparatively dense and deep tuffaceous clay deposit. Taking the result of micro-tremor observation as shown by Fig. 4 into our consideration, we assumed that the subsoil layer has its vibrational bed rock at a depth of 12 m below the grade.

As seen in Fig. 4, Frequency-Period diagram obtained from the micro-tremor observation, the predominant period appears at about 0.1 sec. forming a sharp peak. This tells us that the ground condition is very dense in general.

It is known, however, that inside Djakarta city area there are several deep alluvial valleys extending into the general subsoil strata described above. In fact, an alluvial valley with a depth not less than 30 m is observed at 500 m north to the proposed site of this building.

5. Introduction of Seismic Design Force

Seismic design forces of this building were introduced according to the following procedure.

a. Assuming

$$M = 7.5$$
$$\Delta = 150 \text{ km}$$

where M = Magnitude of Earthquake
Δ = Epicentral Distance

Acceleration α_o at the vibrational bed rock at the proposed site is obtained by the following formula by Dr. K. Kanai of Earthquake Research Institute, Tokyo University;

$$\alpha_o = \frac{10^{0.61M - 1.73\log\Delta + 0.13}}{T} = \frac{9}{T} \text{ cm/sec.}^2 \qquad 1)$$

where T = Period of earthquake (sec.)

b. Value of the acceleration increases as it goes up to the ground surface. According to Dr. Kanai, the factor of increase is given by following formula,

$$\gamma = 1 + \frac{1}{\sqrt{\left(\frac{1+\alpha}{1-\alpha}\left\{1-\left(\frac{T}{T_o}\right)^2\right\}\right)^2 + \left(\frac{0.3}{\sqrt{T_o}}\cdot\frac{T}{T_o}\right)^2}} \qquad 2)$$

The maximum value of γ is given when the period of earthquake T coincides with the predominant period of the ground surface T_o.

Now letting $T = T_o = 0.1$ (sec.), we have

$$\gamma = 1 + \frac{\sqrt{T_o}}{0.3} = 1 + 1.05 = 2.05 \qquad 3)$$

From above, the maximum probable acceleration α_g on the ground surface is given by

$$\alpha_g = \alpha_o \times \gamma = \frac{9}{T_o}\gamma = \frac{9}{0.1} \times 2.05 = 185 \text{ cm/sec.}^2 \qquad 4)$$

If the mean value of T_o of Djakarta city is assumed to be 0.2 sec., the mean value of α_g is about 80 cm/sec^2. According to an information of Regional Housing Center at Bandung which was given at the time of Second World Conference on Earthquake Engineering, the maximum intensity of earthquake that has been sensed at Djakarta during 1921-41 period is VIII in Rossie Forrel Scale. Upon comparison with this fact, we may consider that the maximum acceleration α_g as obtained above may be acceptable.

c. As the footing in this case is planned to rest directly upon the dense tuffaceous stratum, acceleration α_B imposed to the base of the footing was assessed on the assumption as shown by Fig. 5. Then we have

$$\alpha_B = \frac{9}{0.1} \times \left(1 + 1.05 \times \frac{3}{12}\right) = 112 \text{ cm/sec.}^2 \fallingdotseq 0.11 \text{ g} \qquad 5)$$

In computor analysis, it was decided to check by quake waves with maximum accerelation 100 gal and also 150 gal in the secondary design.

d. Taking into our consideration the result of research by Dr. K. Muto, we assumed that the maximum acceleration-spectrum, or base shear coefficient C_B , of a steel framed building constructed upon such a subsoil layer as described hereabove would be as Fig. 6.

e. For assessing the period of natural vibration of the building, we tentatively applied the following foruula 6) according to the preliminary recommendation of Architectural Institute of Japan (AIJ).

$$T = (0.06 \sim 0.1)N \qquad 6)$$

where : $0.06 \sim 0.1$ = coefficient, variable in accordance with structural system of bldg.
N = Number of floors.

Taking the condition that the building has comparatively small amount of resisting elements such as shear wall, the above coefficient was assumed to be 0.095. Number of floor N was assumed to be 29. (the basement was neglected because of its high stiffness)

$$T = 0.095 \times 29 = 2.7 \text{ sec.}$$

Therefore $C_B = \frac{0.1}{2.7} = 0.037$

Thus, $C_B = 0.037$ was taken tentatively as the base shear coefficient for this building.

The story shear coefficient was also assumed to be as Fig. 7 in compliance with the recommendation by AIJ.

f. The equivalent seismic factor, for reference, is approximately 0.05, if distributed uniformly to the whole height like in Fig. 7. While there is no building code at present in Indonesia to stipulate the seismic coefficient, there is a provision in the Indonesian Concrete Specification, which was originally enacted by Dutch in 1935 and later revised by Indonesian Standardisation Committee in 1955, that designates a seismic coefficient of 0.05 for the design of elevated water tanks and multi-storied buildings. It is understood that the seismic design forces we have introduced conform with the requirement of the Concrete Specification.

6. Primary Structural Design

A primary design of the framework was made on the base shear and the story shear coefficients as introduced in foregoing paragraphs. The framework was designed as a pure two-way framing system built with structural steel conforming to the requirements of Japan Industrial Standard SS41, of which the allowable stress was limited at 24 kg/mm^2 in case of quake.

Referring to Fig. 7, it can be seen that the story shear coefficient at the uppermost was assumed to be three times that of the ground level on account of whipping effects of a high-rise structure. In case of this building, however, the effects of setback of upper stories was taken into further account and the assumption of section of steel framing members was so made that it would give this portion a stiffer spring constant.

Furthermore, to minimize possible tortional phenomena due to the irregularity of plan, the arrangement of wide-flanged steel columns as well as the selection of section of columns were carefully considered and, in addition, concrete floor slabs were made comparatively thick. Fig. 8 illustrates typical structural members and connections.

7. <u>Check of the Primary Structural Framework through Electronic Computor Analysis.</u>

a. Fig. 9 shows a shear-displacement diagram of the framework determined by the primary design. The yielding of each story was assumed to start when the story shear reached the shear value in the primary design.

Judging from our experiences the damping coefficient was assumed to be h = 0.05. The electronic computor employed in this analysis was HIPAC 101. Earthquake spectra introduced in this case were that of El Centro (digitalised by Dr. G.V. Berg), which was recorded on a comparatively hard subsoil condition having a predominant frequency of 0.25 sec., that of Sendai Earthquake recorded on a gravel bed having a predominant frequency of 0.3 sec. and that of Tokyo Earthquake recorded on deluvial deposit of the uptown (both digitalised by SERAC Committee in Japan). Fig. 10 shows those spectra. For those earthquakes value of the maximum acceleration was set at 100 gal.

b. Results of response analysis in respect to story relative displacement δ, working factor $\bar{\mu} = \frac{\delta \text{ max.}}{\delta \text{ design}}$ and story shear coefficient are shown in Fig. 11, 12 and 13. Referring to Fig. 11, it can be seen that δ was smaller than 10 mm without a tendency of the higher stories having the larger displacement. A matter of particular interest was that the setback above the 23rd floor did not have remarkable effects in this respect. The reason is conceivably that the rigidity at the lower portion was comparatively low. The working factor never exceeded 1 like in Fig. 12, that is to say, every story stayed in the elastic range for these spectra with 100 gal. For El Centro spectrum, the maximum working factor was 0.94 that appeared at the 16th story and showed a remarkable peak. The story shear coefficient is as seen in Fig. 13, and at the upper portion it turned out about half the designed value.

8. <u>Secondary Design and Its Computor Analysis</u>

a. According to results of the computor analysis of the primary design, there was a wide deviation of working factors in various heights. In order to make this as even as possible and to get a unformity of strength, the distribution of the story shear coefficient which was originally as seen in Fig. 7 was modified to be as the dotted line in the same figure. Sections of framing members were redesigned under revised criteria in the secondary design.

Revised criteria are as follows:

From 1st story to 10th	C_B = 0.037
At 29th story	C_T = 0.037 x 2.5 = 0.093
Intermediate stories	parabolic adjustment from 0.037 to 0.093

In the secondary design, considering the difficulty in hot-rolling and welding, it was decided to limit thickness of flange not larger than 45 mm. As a consequence, all column members up to the 7th floor were changed from SS41 to SS50, of which the allowable stress was taken at 30 kg/mm^2. (See Fig. 14)

b. The framework thus redesigned was again analysed by the computor applying El Centro and Sendai Spectra of 100 gal. Results are shown by Fig. 15, 16 and 17 in which a considerable improvement is seen such as that δ , and $\bar{\mu}$ were made even in general.

c. The natural frequency of vibration of the modified framework was examined. We found that it would have a natural period of 4.28 sec. which might look to be too far from 2.7 sec., the assumptive value of natural period. This in turn will mean that its actual base shear coefficient might be away from the basic assumption. However, considering that this figure is in the safety side, re-adjustment of the basic shear coefficient and consequent re-designing were not repeated anymore.

d. Further analysis was made by introducting 150 gal as the maximum acceleration into El Centro and Sendai spectra. In the former case, it showed a result that the maximum working factor around 1.0 with the relative displacement of 9.0 mm would be induced at the 14th floor, which meant it would still stay within the elastic range, and no whipping would occur. From this analysis we judged that the secondary structural design was sufficiently safe. (The maximum acceleration of 150 gal at the foundation of this building is roughly estimated to be equivalent to that induced by an earthquake of Magnitude 7.5 with its epicenter at a distance of 100 km.)

9. Conclusion

This paper has covered briefly the procedure of dynamical analysis and its basic conceptions introduced to an actual design of a highrise building. However, this sort of dynamical analysis still involves a number of assumptions, and therefor, we are not of opinion that it has been proved to be perfect by those results as described in this paper.

Needless to say, it will be most essential for us to endeavor to finalize our design by combining these results of analysis with our engineering judgement based on empirical data of damages caused by earthquakes in the past.

Bibliography

1. Kanai K., "An Empirical Formula for Spectrum of Strong Earthquake Motions". Bulletine of the Earthquake Research Institute, vol. 39 (1961).

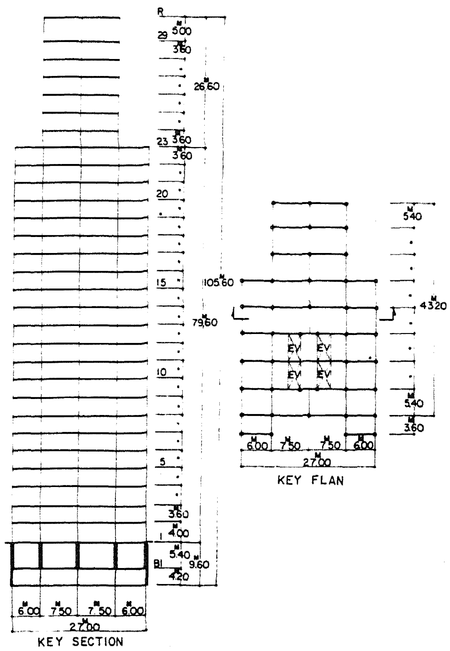

Fig. I KEY PLAN AND SECTION OF THE TOWER BUILDING

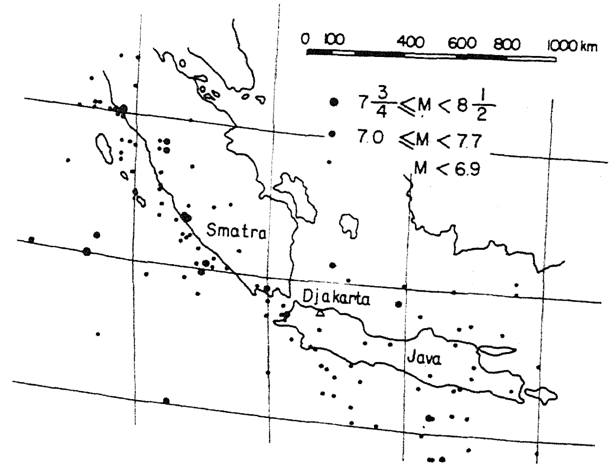

Fig. 2 MAP OF EPICENTERS OF SHALLOW EARTHQUAKES UP TO 1960 (Omote)

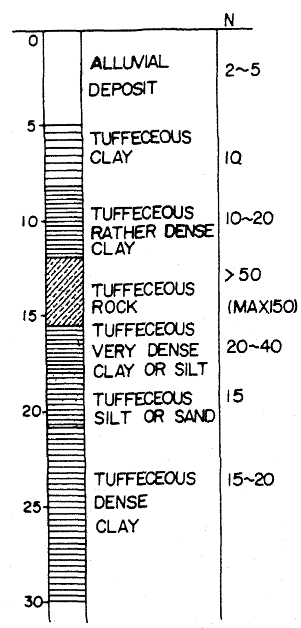

Fig. 3 MODELISED BORING COLUMN

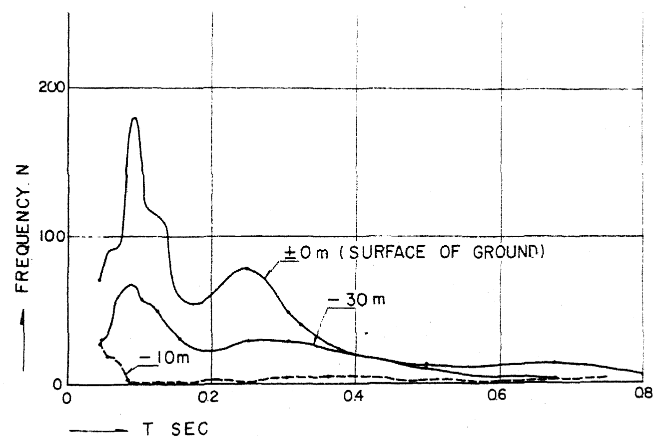

Fig. 4 MICRO TREMOR OBSERVATION (N.NAKAJIMA)

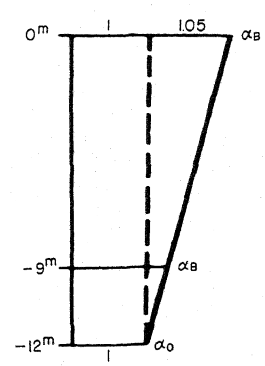

Fig. 5

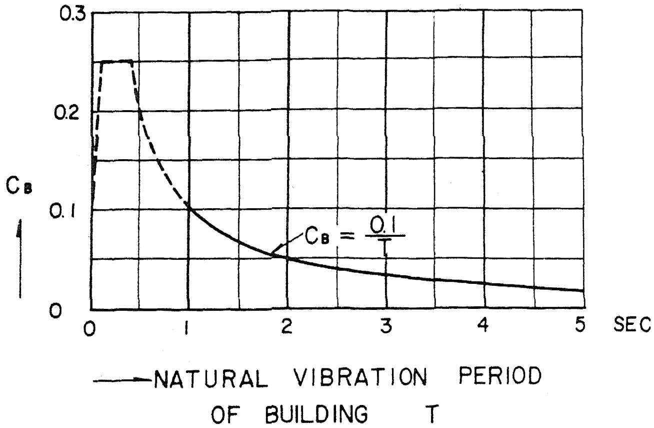

Fig. 6 BASE SHEAR COEFFICIENT C_B AND NATURAL VIBRATION PERIOD T

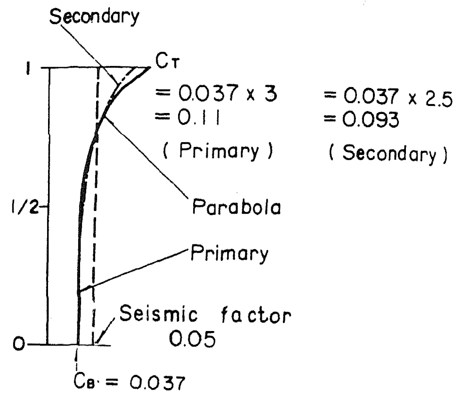

Fig. 7 DISTRIBUTION OF STORY SHEAR COEFFICIENT

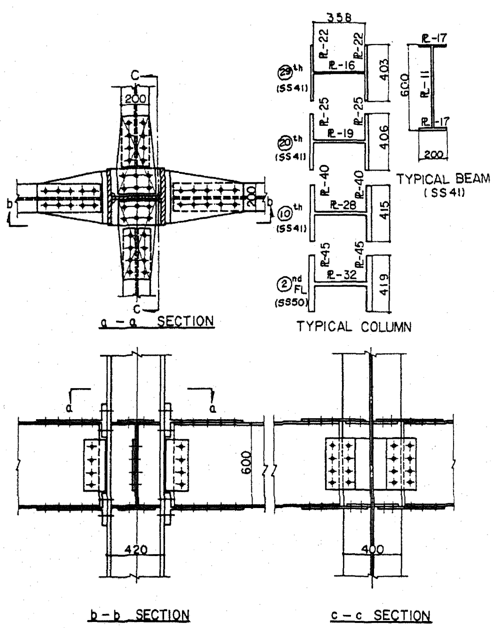

Fig. 8 TYPICAL STRUCTURAL MEMBERS AND CONNECTIONS

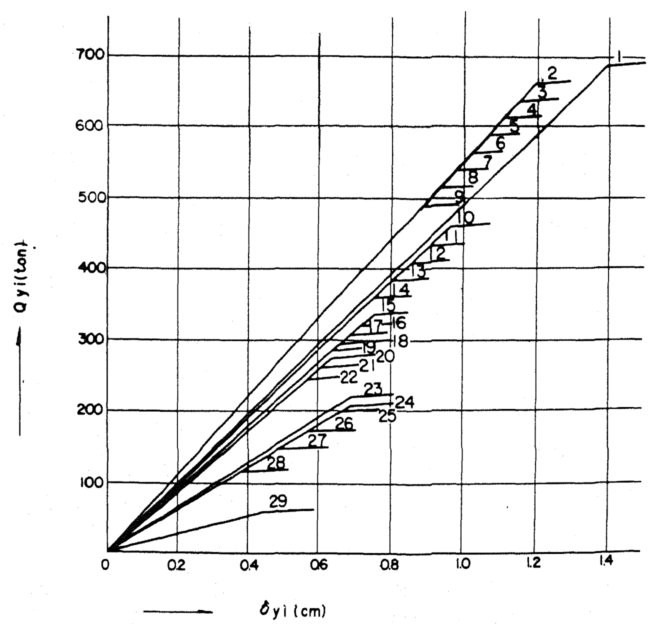

Fig. 9 SHEAR DISPLACEMENT DIAGRAM (Primary)

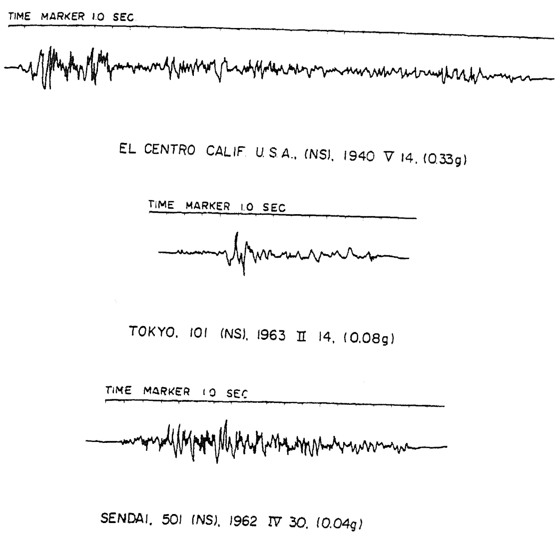

Fig 10 ACCELEROGRAMS OF EARTHQUAKE WAVES

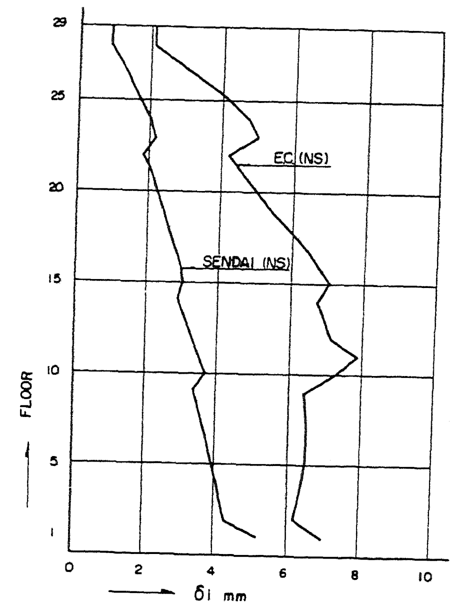

Fig. 11 MAXIMUM STORY DISPLACEMENT(Primary)

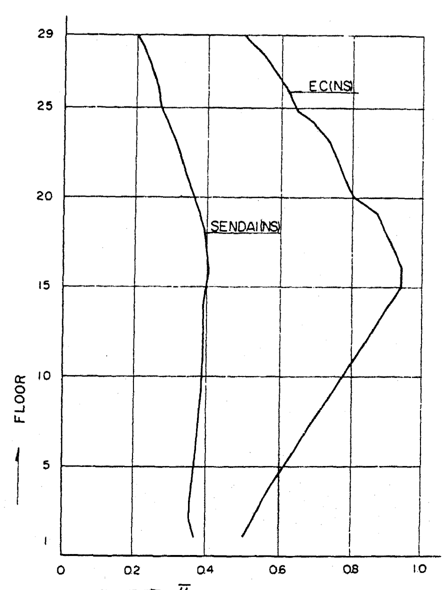

Fig. 12 MAXIMUM WORKING FACTOR (Primary)

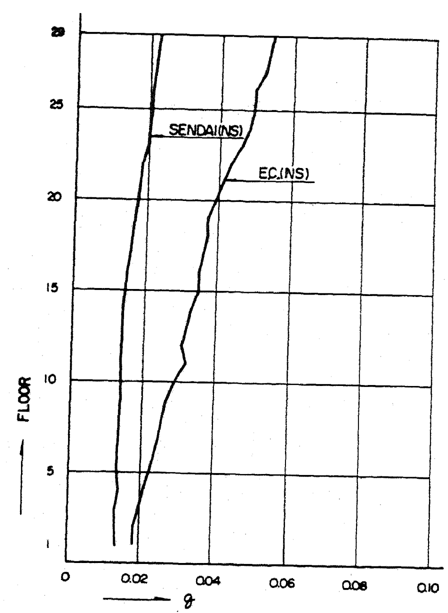

Fig. 13 MAXIMUM STORY SHEAR COEFFICIENT (Primary)

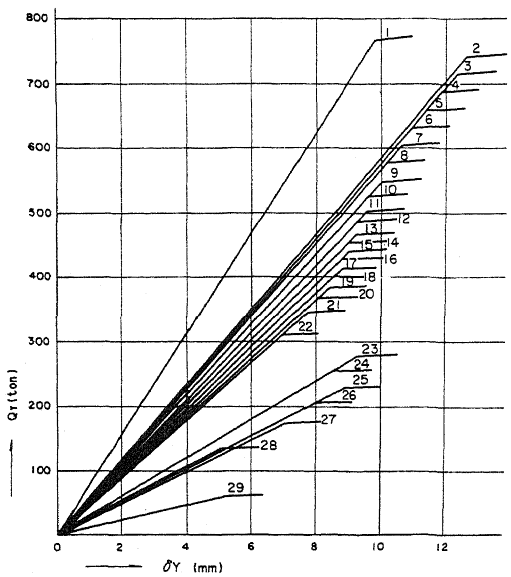

Fig. 14 SHEAR DISPLACEMENT DIAGRAM OF EACH STORY (Secondary)

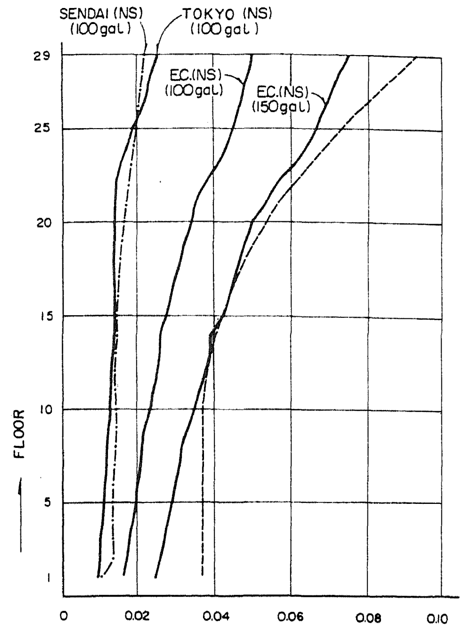

Fig. 15 MAXIMUM STORY SHEAR COEFFICIENT (Secondary)

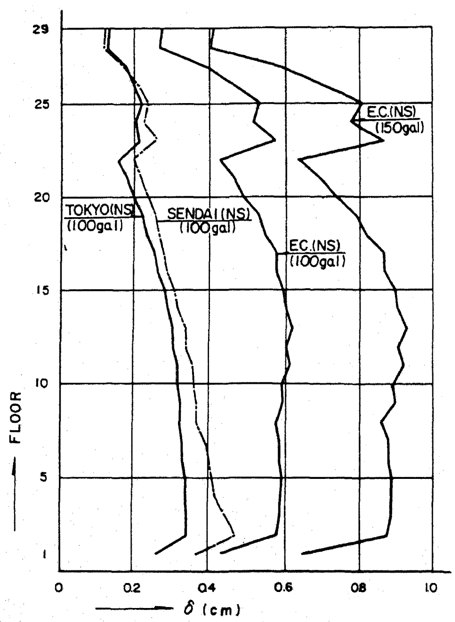

Fig. 16 MAXIMUM VALUE OF RELATIVE STORY DISPLACEMENT(Secondary)

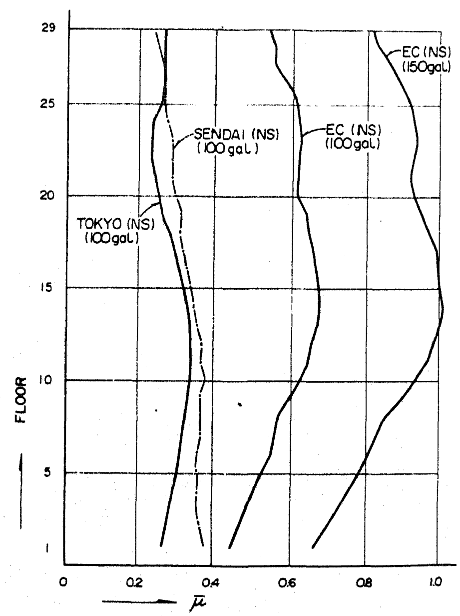

Fig. 17 MAXIMUM WORKING FACTOR (Secondary)

ANTI-SEISMIC DESIGN OF A HIGH-RISE BUILDING IN DJAKARTA

BY K. TAKEYAMA, T. OTA, Y. NAGATA, K. ATSUMI

QUESTION BY: M. IZUMI - JAPAN

The building has 30 storeys including basement, and the analysis was made for a 29 storied structure. What is the reason for one storey being neglected?

AUTHORS REPLY: The total number of storeys is 30, but the basement floor was neglected because of its high stiffness, so it was treated as a 29 storied building and earthquake intensity was assumed as the value expected at the foot of the basement.

QUESTION BY: J.E. AMRHEIN - U.S.A.

1. I see that you computed the ductility of the frame for the El Centro and Sincki Earthquakes - What is the ultimate ductility of the building frame?
2. Your definition of ductility is $M = \dfrac{\delta \text{ Max}}{\delta \text{ Design}}$

   Should it not be $M = \dfrac{\delta \text{ Max}}{\delta \text{ Yield}}$

AUTHORS' REPLY:

1. Considering the assumed earthquake intensity and the ground condition, we designed this building frame in which the ultimate working factor was to be below 1.0 or so for the response calculations, and the ability of deflection of this frame was approximately 8 to 10 times more than the designed value that was confirmed through full scale model experiments on the building frame.
2. Our definition of working factor is $\bar{\mu}$ = Max./Design and ductility factor is μ = Max./Yield. In this case we employed the working factor.

QUESTION BY: C.M. STRACHAN - NEW ZEALAND

Did overturning conditions govern the bearing capacity of the foundations? If so what increase was allowed in bearing over that for normal vertical loading?

AUTHORS' REPLY: No, they did not.
We calculated the bearing capacity of the foundations

as follows:

The permanent bearing capacity of the tuffaceous stratum was to be 30.0 T/sqM., and the maximum over-turning moment value, being applied El Centro Earth-quake of 150 gal., was M=47700 TM, consequently, we could calculate the fibre stress of the foundations as M/Z = 4.44 T/sqM.

The stress of the base for vertical load was σ_L = 21.3 T/sqM. and then the maximum stress was $\sigma_{Max.}$ = 25.74 T/sqM.