IV WCEE SPECIAL REPORT

Earthquake Resistant Design of 36-storied Kasumigaseki Building

Kiyoshi Muto

SYNOPSIS

In 1963 the Japanese Building Bylaws were revised to permit the building height exceeding the traditionally observed limit of 31 m. One of the features provided in the new regulations was that a free approach of earthquake resistant design as well as structural system layout can be adopted instead of following the conventional design criteria if the scheme and analysis are judged as appropriate and therefore approved by the agréments board authorized by the Minister of Construction. This building is the first example following this new setup and the author who is fully responsible on the structural design describes his approach on the earthquake resistant design of this building in this special report.

The author comes to conclude his report with following notes.

1. Highlighting the actual example of Kasumigaseki Building, his approach and general pattern on dynamic analysis presented in this report have been generally accepted and recognized as one of standard procedures and practices for high-rise building design in Japan to match with the large capacity computer age.

2. Ground investigation with its dynamic characteristic evaluation is very necessary. On this project besides the static tests of the soil, the natural micro-tremors and the ground motion due to actual earthquake were measured, confirming the stratum enough reliable statically and acquiring sufficient data to determine the input earthquake wave for the analysis.

3. Proper layout and distribution of stiffness of structural elements for the entire building are essential to improve the dynamic responses. In Kasumigaseki Building wide flange H shape sections were used at the maximum efficiency, i.e. uniform column size and castellated beam design. The merits of the slitted wall were discussed, i.e. modifying the vibration character of the structural system and providing favorable ductile resistive quality of the entire building. The bi-linear hysteresis for various stress conditions recognized by test results was feedbacked into the computer program to meet earthquake intensity level.

4. Dynamic responses for all stages against any anticipated earthquake should be investigated for high-rise building design in severe seismic regions. A complete rigorous generalized programing for totally computerized analysis has been developed for such tower-like structure with or without walls inserted to evaluate natural periods, vibration modes and responses with corresponding stresses and deformations including sequence of yielding phase as well as static stresses and deformations within reasonable time and expense.

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Kiyoshi Muto*

Preface

In Tokyo the construction of Kasumigaseki Building was successfully completed in April 1968 (Fig.1-1). This is the tallest building ever built so far which may be said to fit the real terminology as a high-rise building in Japan.

In 1963 the Japanese Building Bylaws were revised to permit the building height exceeding the traditionally observed limit of 31m holding the specified proto-ratio area preservation leaving more open ground space in certain zones of Tokyo city. One of the features provided in the new regulations was that a free approach of earthquake resistant design as well as structural system layout can be adopted instead of following the conventional design criteria if the scheme and analysis are judged as appropriate and therefore approved by the agréments board authorized by the Minister of Construction. The function of this board now has been shifted to Japan Building Center.

This building is the first example following this new setup and the author who is fully responsible on the structural design describes his approach on the earthquake resistant design of this building in this special report.

1. Introduction

1-1 Outline of building

The height of this building is 147m from the front street or 1st floor level with 36 stories above ground and 3 basement floors, comprising the upper part above the 2nd floor a tower structure. The typical floor area is 3750m^2 based on the overall structural system dimension of 84.0m x 42.4m and the total floor area has 153,226m^2 for the whole building. A penthouse is added on the roof level for a machinery room.

The structure of this building consists of steel framing for the tower part from 2nd to 36th floor and composite reinforced concrete

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structure with steel sections for the portion from the 1st basement to the podium level directly under the tower part. The other surrounding portion is constructed with ordinary reinforced concrete. See Fig.1-2, 3 & 4.

1-2 Progress of design concept

The progress sequence of design concept for this building is explained in this section. The author and his colleages carried numerous preliminary studies before the final design was derived. At first a 30 storied building of pure open framing with various member sections was analyzed by the SERAC analog computer on the elasto-plastic dynamic response against earthquake based on the pure shear model reduced by D-value method[1)2)]. From this analysis we have found our approach appropriate by which the design would be proceeded, feeding back and force the response analysis result on the assumption of the design shearing force, and studying the most favorable stiffness and strength distribution of the whole building which may produce the desirable response characteristics[3)4)]. Then the final architectural design features were decided as described in the above section for which the investigation of the structural system was carried. At this stage it was composed of rigid center core framing and less rigid exterior framings, and then the whole system was analyzed on various earthquake responses by use of digital computer[5)]. Meanwhile, the author has originated "slitted wall" or a reinforced concrete wall with slits which provides more ductility than solid reinforced concrete ordinary shear wall[6)]. Utilizing this slitted wall as earthquake resisting element in the core portion and developing the approximate analytical method based on the coupled system of pure open framing and infilled framing with the slitted wall, the final design employed in the actual construction was determined[7)].

Furthermore, since we had the achievement of a rigorous generalized analytical method of framing system with or without walls inserted in 1967, we have carried again the static and dynamic analysis and checked the safety of the final design[8)9)].

In this report the author describes the procedure of this final actually adopted structural design and the earthquake responses following our newly developed method.

1-3 Systematized design process.

Our procedures are presented in the following steps.

1. At first the investigation of the ground of the site is held to check the strength and dynamic behavior of the soil, to select the input earthquake waves and the most feasible structural system together with the foundation level determination.

2. The structural system layout is selected, for which the design analysis and strength calculation are performed and the columns, beams and other frame elements are determined.

3. Then the earthquake response analysis with aid of computer for the proposed frame is investigated in the following manner.

When various types of earthquake waves with different intensities are applied, how the structure will vibrate or be safe in the strength is to be checked and the weak points are chased in stages which, if necessary, should be altered for improvement.

4. On the other hand, various kinds of tests are conducted to make sure whether the strength of the elements will meet the required value from design calculation and to check how the deformation character differs from the design assumption and the results will be feedbacked into the computer analysis.

5. Furthermore, we have carried a noticeable wind tunnel test. Since wind effect is governed by geographical features around the building, the models of a part of the city were constructed in 1:1000 and 1:500 scales with diameters of 1500m and 750m for the vicinity of Kasumigaseki Building which was set as the center of the model. Thus, the effective wind pressure has been determined experimentally and applied in the design.

6. All these investigations and results are to be judged collectively and evaluated synthetically. Necessary check and improvement by stages will be performed repeatedly and thus the design is proceeded.

Following the above described procedures the structural safety together with the cost study on each possible scheme is pursued to derive the most economical and rational solution which leads the final design.

2. Dynamic Characteristics and Anticipated Earthquake Waves of the Site

2-1 Dynamic and seismic characteristics of the site

The surface soil of the site is a loam and the bearing stratum is the firm layer called "Tokyo conglomerate" at G.L. −18m which has the bearing capacity over 300 t/m^2 with 4mm settlement. As a means to find the dynamic characteristics of the site the natural micro-tremors were measured in the site and the period-frequency curve was obtained. The result is shown in Fig.2-1, with the dominant period of 0.45 sec at G.L. −18m which is the foundation setting level and 0.38 sec at the ground surface.

Besides, the observation by the SMAC strong motion seismograph installed in the basement of the adjacent 6 storied reinforced concrete building was continued for 3 years but only minor earthquake records were obtained. The acceleration spectrum of the largest among them is shown in Fig.2-2 and the peak period is 0.35 sec, indicating the behavior of a hard stratum. Considering these ground characteristics, the anticipated earthquake waves were selected for the dynamic analysis.

2-2 Anticipated earthquake waves

As the input earthquake waves for the response analysis with the aid of computer we selected the following 3 waves among the observed waves for the hard soil stratum which was reasonably similar to the site ground.

1. EL CENTRO (U.S.A.) 1940 V 18 (NS)
2. TOKYO 101 (JAPAN) 1956 II 14 (NS)
3. SENDAI 501 (JAPAN) 1962 IV 30 (NS)

The response shear spectra of these waves are shown in Fig.2-3 when the input maximum acceleration or seismic intensity is set as a unity. For the adopted intensities of the input data 0.1g, 0.3g and 0.5g are used as the maximum accelerations.

3. Structural Details

3-1 Steel frame

The columns are made of wide and thick flange H shape section of 400mm series. In the longitudinal direction 4 lines of framing consisting of two exterior (A) and two interior (B) column lines are provided (Fig.1-3) with the column spacing of 3.2m. The total number of columns amounts 96 with 25 exterior and 23 interior columns for respective framing lines. The transverse direction is composed of 3 spans with 15.6m of exterior span and 11.2m of center one.

For the longitudinal exterior line spandrel beams of wide flange H shape run at two levels for each floor, while a single wide flange H section is used for the interior frame line. For the transverse direction a castellated beam cut and expanded from a wide flange H section is employed. This arrangement was adopted not only for enhancing the strength but also for improving the stiffness which would control the vibration behaviors.

Next, the fabrication and erection of the steel framing are described in the following. The splice of the column was provided at every 2 or 3 floors height. For the most effective jointing the members the cold sawing machine to cut the material precisely and the multi-spindle boring machine to drill the holes to achieve full bearing contact with automatic controlling by electronic tapes were used. The beams in the longitudinal directions were divided into the two parts which were attached to the column assembly unit like a dragonfly's wings at the fabrication shop. All fasteners are the high strength bolts with splice or cover plates. For the connection of the transverse beams to the columns the split tee type with bolting was adopted with a thick plate welded on the beam end in order to regulate the deformation. The framing detail of the transverse direction and its actual construction are shown in Fig.3-1 and Fig.3-2 respectively. The full sized testing was performed for the columns and beams so that the strength and stiffness were confirmed to the requirements. Fig.3-3 shows one of the tests conducted in Kajima Institute of Construction Technology and Fig.3-4 illustrates one of the load and deformation curve observed from the testing.

3-2 Reinforced concrete slitted walls which the author originated were installed in the core part of this building.

Since the solid reinforced concrete shear wall used in a tall building has an unfavorable tendency of causing large cracking in early stage when subjected to a severe horizontal force so that the strength would be lost rapidly and the steel framing might not be effective due to time-lag resistance, the concept of this slitted wall was exploitted and developed to improve the interaction and collaboration between wall and framing.

As Fig.3-5 & 6 show the detail of the slitted wall, a number of slits are provided vertically at approximately 90cm spacing in the wall plane and the system could be compared to a continuum of walls or a multi-span continuous framing for its structural behavior. Though the initial strength will be reduced to some amount when subjected to a horizontal force, ductility is provided to keep the resisting capacity even in plastic range and the capability to follow the large deformation by the effective interaction with steel framing, being the cracks distributed finely and widely which would be closed again after the loading removes.

By the proper layout of these walls this building provides the structural property to keep sufficient human comfort and functional usefulness as an office building controlling the vibration due to strong wind loading or medium earthquakes and even for severer earthquakes these walls will work still effectively without serious damage. All the characteristics of the slitted wall have been investigated by various tests. In Fig.3-7 the shearing stress and strain curve obtained from the test result is shown with the estimated story displacement corresponding to the story height of 3.8m. The crack development of the specimen by testing is shown in Fig.3-8.

4. Design Analysis

4-1 Structural composition

In the transverse direction there are 18 open framings of 3 spans and 7 framings with slitted walls as shown in Fig.4-1, while in the longitudinal direction 2 exterior framings are open and 2 interior ones are infilled with walls as indicated in Fig.4-2.

On analysis the floor was assumed as completely rigid but not restraining the elongation and shrinkage of columns and walls. The structure was analyzed by the rigorous generalized method as a coupled system of open framing and infilled framing with slitted walls.

4-2 Analysis concept

Considering that the framing is composed of columns, beams and joint panels (Fig.4-3), we have developed a generalized programming taking account of bending, shearing and axial deformation on columns, bending and shearing deformation on beams and shearing deformation on joint panels as illustrated in Fig.4-5.

For treatment of wall the composite action with surrounding steel framing was considered and converting to an equivalent column the effective sectional properties were figured (Fig.4-4). On shearing deformation of wall the elasto-plastic characteristic combining elastic resistance of steel framing and the elasto-plastic resistance of slitted wall was taken into account. From the test results as described before the two kinds of shearing deformation were considered, i.e. one for low stress and another for high stress condition. In low stress stage the wall remains as elastic with high rigidity having the average shearing modulus $G = 60t/cm^2$, while in high stress condition the wall is assumed to have the elasto-plastic bilinear characteristic as shown in Fig.4-6 in which low rigidity $G = 15t/cm^2$ is taken for range (0) and ($\pm$ 2) and in plastic range ($\pm$ 1) the rigidity is reduced to one quarter of the above value. The average shearing yield stress $\bar{\tau}_y$ = 20 kg/cm^2 is assumed.

4-3 Equation development

The stiffness matrix of each element, i.e. column (including wall), beam and joint panel for its relative deformation (Fig.4-5) can be expressed as follows for the constant member sections though our programs are prepared for the varying section.

<u>Column (wall)</u>

$$\begin{Bmatrix} M_U^c \\ M_D^c \\ Q^c \\ N^c \end{Bmatrix}_{ij} = 2EK_o k_{ij}^c \begin{bmatrix} a^c & b^c & c^c & 0 \\ b^c & a^c & c^c & 0 \\ c^c & c^c & d^c & 0 \\ 0 & 0 & 0 & e^c \end{bmatrix}_{ij} \begin{Bmatrix} \theta_i^c \\ \theta_{i+1}^c \\ u_i \\ \varepsilon_i \end{Bmatrix}_j \quad \cdots\cdots\cdots \quad (4\text{-}1)$$

<u>Beam</u>

$$\begin{Bmatrix} M_L^g \\ M_R^g \\ Q^g \end{Bmatrix}_{ij} = 2EK_o k_{ij}^g \begin{bmatrix} a^g & b^g & c^g \\ b^g & a^g & c^g \\ c^g & c^g & d^g \end{bmatrix}_{ij} \begin{Bmatrix} \theta_j^g \\ \theta_{j+1}^g \\ \lambda_j \end{Bmatrix}_i \quad \cdots\cdots\cdots \quad (4\text{-}2)$$

Panel

$$\{M^p\}_{ij} = [G^p]_{ij}\{\gamma\}_{ij} \qquad \text{(4-3)}$$

where

$$a = \frac{2+\gamma'}{1+2\gamma'},\ b = \frac{1-\gamma'}{1+2\gamma'},\ c = -\frac{a+b}{l},\ d = -\frac{2c}{l},\ e = \frac{S}{2K_o kl}$$

$$G^P = 4BDtG$$

γ' : shearing deformation angle due to a unit moment applied at the end in terms of 6EKok

K_o : standard bending stiffness

k : bending stiffness coefficient for the member

l : member length

S : sectional area

$2B$, $2D$, t : width, depth and thickness of joint panel respectively

E, G : elastic moduli of member

Since the elasto-plastic condition for shearing in wall is to be considered, γ' and the matrix components in Eq.4-1 should be changed according to the stress excursion.

The relative deformations in Eq.4-1 and 2 are expressed in the terms of the absolute displacement of joint panel as follows.

$$\theta^c_{i,j} = \theta_{i,j} - \frac{1}{2}\gamma_{i,j} \qquad \theta^g_{i,j} = \theta_{i,j} + \frac{1}{2}\gamma_{i,j}$$

$$u_{i,j} = (U_i - D_{i,j}\theta^g_{i,j}) - (U_{i+1} + D_{i+1,j}\theta^g_{i+1,j})$$

$$\varepsilon_{i,j} = V_{i+1,j} - V_{i,j} \qquad \text{(4-4)}$$

$$\lambda_{i,j} = (V_{i,j+1} - B_{i,j+1}\theta^c_{i,j+1}) - (V_{i,j} + B_{i,j}\theta^c_{i,j})$$

where

θ_{ij} : rotation angle of joint panel

γ_{ij} : shearing deformation angle of joint panel

U_{ij} : vertical displacement of the center of joint panel

V_{ij} : horizontal displacement of the center of joint panel

Then the desired equation of complete structure can be derived by the standard matrix analysis techniques[10)] in the following.

$$\begin{Bmatrix} R_1 \\ R_2 \\ \vdots \\ R_i \\ R_{i+1} \\ \vdots \\ R_{N-1} \\ R_N \end{Bmatrix} = \begin{bmatrix} K_1 & C_1 & & & & & \\ C_1^T & K_2 & C_2 & & & & \\ & \ddots & \ddots & \ddots & & & \\ & & C_{i-1}^T & K_i & C_i & & \\ & & & C_i^T & K_{i+1} & C_{i+1} & \\ & & & & \ddots & \ddots & \ddots \\ & & & & C_{N-2}^T & K_{N-1} & C_{N-1} \\ & & & & & C_{N-1}^T & K_N \end{bmatrix} \begin{Bmatrix} r_1 \\ r_2 \\ \vdots \\ r_i \\ r_{i+1} \\ \vdots \\ r_{N-1} \\ r_N \end{Bmatrix} \quad \cdots (4\text{-}5)$$

where $\mathbf{R}$: vector of external force

$\mathbf{r}$: vector of displacement

Statical equation

When external forces are given, the force terms $\mathbf{R}$ in Eq.4-5 should be determined accordingly and stresses and strains can be solved.

Dynamical equation

The vibration equation can be obtained in the following putting horizontal inertia forces into the external forces in Eq.4-5.

(i) Free vibration equation

$$\mathbf{M}\ddot{\mathbf{Y}} = -\mathbf{K}\mathbf{Y} \quad \cdots\cdots\cdots (4\text{-}6)$$

(ii) Forced vibration equation

$$\mathbf{M}\ddot{\mathbf{Y}} = -\mathbf{K}(\mathbf{Y} - r\dot{\mathbf{Y}}) - \mathbf{M}\ddot{y}_0 \quad \cdots\cdots\cdots (4\text{-}7)$$

where

$\mathbf{Y}$: horizontal displacement

$\mathbf{M}$: mass matrix

$\mathbf{K}$: stiffness matrix for horizontal displacement

r : damping coefficient

$\ddot{y}_0$: ground motion acceleration

5. Natural Vibration

Since the two kinds of wall characteristics were considered as described before natural periods are tabulated for the transverse direction in the following.

	Low stress condition (minor earthquake)	High stress condition (severer earthquake)
1st mode	4.02 sec	4.24 sec
2nd mode	1.14 "	1.33 "
3rd mode	0.55 "	0.70 "

It is observed that reduction of rigidity of the wall in severer earthquakes prolongs the 1st mode natural period by approximately 5%.

Although the vibration tests were held to investigate the dynamic responses of the building while it was under construction, after completion of the building at the last TOKACHIOKI earthquake of 16th May 1968 the natural periods of this building were found as 3.9 sec for the 1st mode and 1.1 sec for 2nd mode according to the records measured by the SMAC seismograph installed on the roof of the building.

6. Responses for Earthquake Motions

From the result of the response calculation carried out on the anticipated earthquake waves mentioned before with the damping coefficient corresponding to 5% of the critical damping for the 1st mode natural vibration, the response magnitude for EL CENTRO 1940 (NS) was found the largest. In the following the responses on EL CENTRO earthquake are described for transverse direction. The calculation was performed for the maximum acceleration of 0.1g for elastic stage, and 0.3g and 0.5g for elasto-plastic condition.

The maximum story shear force for each story in the transverse direction is shown in Fig.6-1. The story shear capacity indicated in the figure means the collective shear force magnitude from the components in the story as an index showing the actual capacity provided in the structure. Comparing this with the response shear magnitude, there may be about twice margin at the lower part of the building for the maximum acceleration of 0.3g and still some allowance even for 0.5g. In Fig.6-2 the response ductility factor of the slitted wall is shown, which means how much in-elasticity with associated ductile strain develops. From this figure we may say that the wall will become inelastic but just a little from the bottom floor to 16th floor at 0.3g and may have the maximum strain of about 3 times as the one at yield point for 0.5g. In both figures the bold lines show the regions where the wall yielded.

The maximum story drift is given in Fig.6-3, from which we may observe 6mm displacement at 0.1g and 20mm even at 0.3g.

The distribution of the maximum acceleration is indicated in Fig.6-4 for 0.3g earthquake when the input maximum intensity was assumed as a unity. It is observed that the response acceleration is rather small and found less than 50% in the upper floors.

7. Conclusion

The author now comes to conclude his report with following notes.

1. Highlighting the actual example of Kasumigaseki Building, his approach and general pattern on dynamic analysis presented in this report have been generally accepted and recognized as one of standard procedures and practices for high-rise building design in Japan to match with the large capacity computer age.

2. Ground investigation with its dynamic characteristic evaluation is very necessary. On this project besides the static tests of the soil, the natural micro-tremors and the ground motion due to actual earthquake were measured, confirming the stratum enough reliable statically and acquiring sufficient data to determine the input earthquake wave for the analysis.

3. Proper layout and distribution of stiffness of structural elements for the entire building are essential to improve the dynamic responses. In Kasumigaseki Building wide flange H shape sections were used at the maximum efficiency, i.e. uniform column size and castellated beam design. The merits of the slitted wall were discussed, i.e. modifying the vibration character of the structural system and providing favorable ductile resistive quality of the entire building. The bi-linear hysteresis for various stress conditions recognized by test results was feedbacked into the computer program to meet earthquake intensity level.

4. Dynamic responses for all stages against any anticipated earthquake should be investigated for high-rise building design in severe seismic regions. A complete rigorous generalized programing for totally computerized analysis has been developed for such tower-like structure with or without walls inserted evaluate natural periods, vibration modes and responses with corresponding stresses and deformations including sequence of yielding phase as well as static stresses and deformations within reasonable time and expense.

Since the author is still engaged to design many high-rise buildings in Japan after this first landmark experience, he has a sincere wish to check whether his leadership and development of procedures on this building design have been adequate or not by carrying continuous observation and investigation on it in a long run for further improvement and progress of Earthquake Resistant Design. As a means to do this, in this building 5 strong motion seismographs have been installed; namely, at the foundation level, 36th floor and 3 places at the intermediate floors. Beside, other seismographs are located in the bed rock (G.L. —90m) and at the foundation level. Though the natural vibration periods of the building were confirmed by the actual observation at the last TOKACHIOKI earthquake as mentioned above, the author would like to have another chance in WCEE to report how this building responds for any severer earthquake in the future.

Acknowledgement

The author likes to express his appreciation to Messrs. Terazaki, Koh, Uchida, Ujiie and Nagata of Muto Institute of Structural Mechanics Messrs. Takase, Tsugawa and Inoue of Kajima Computer Center and Messrs. Omori and Ota of Kajima Institute of Construction Technology and other engineers involved in this project for their kind help and collaborations to the author on the pursuit and development of the work.

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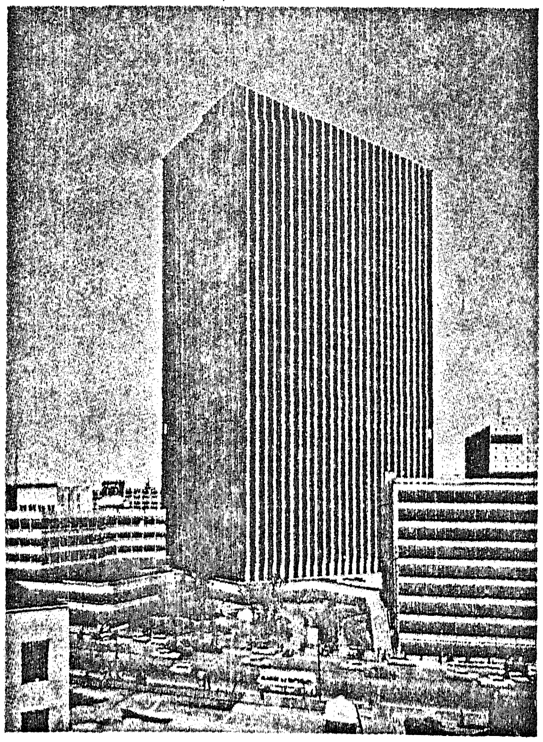

Fig.1-1. Completed appearance

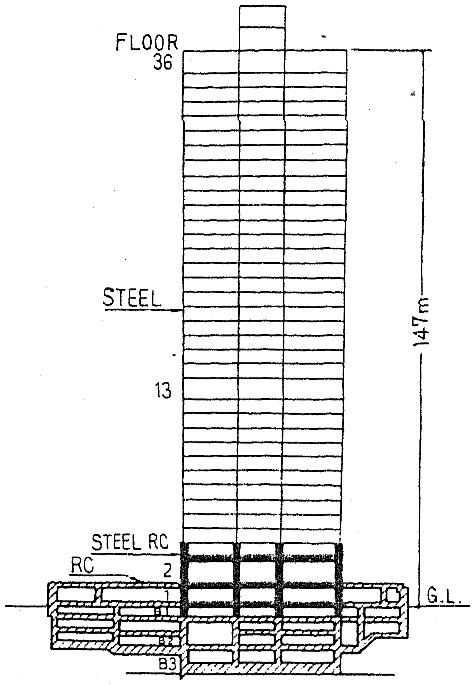

Fig.1-2. Transverse section

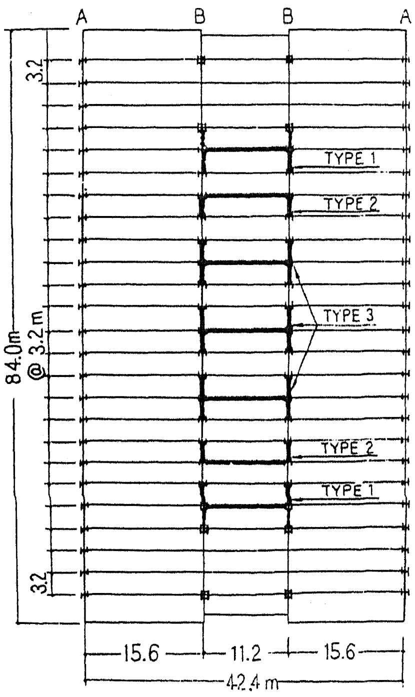

Fig.1-3. Typical floor plan

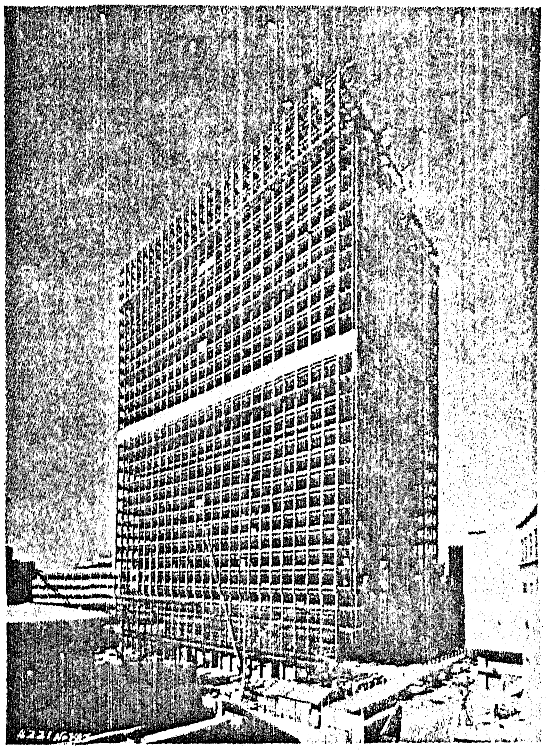

Fig.1-4. Steel skeleton

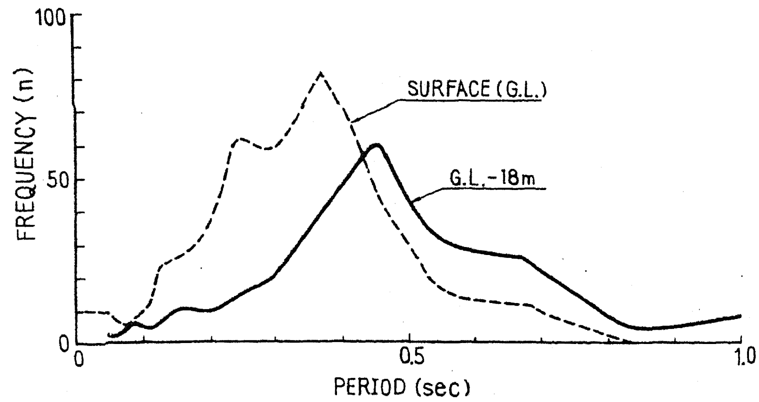

Fig.2-1. Period frequency curve of micro-tremors

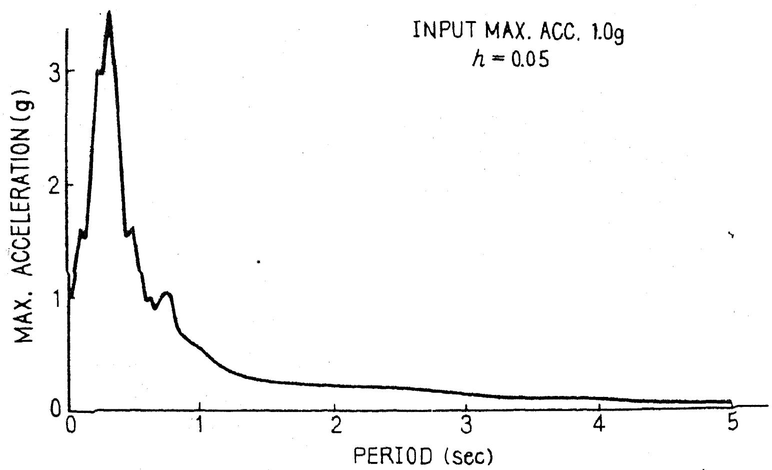

Fig.2-2. Response acceleration spectrum of earthquake wave recorded nearby the site

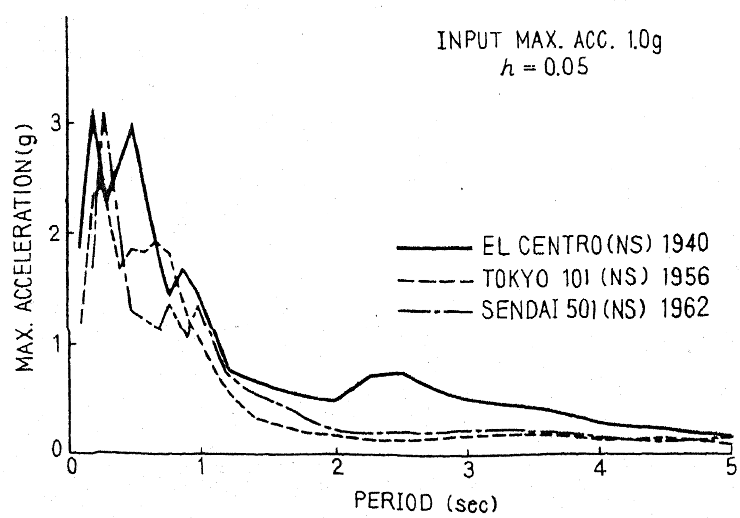

Fig.2-3. Response acceleration spectra of earthquake waves applied in dynamic analysis

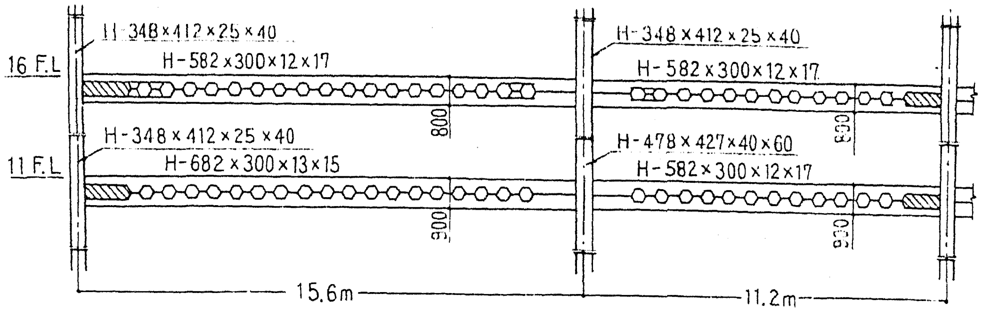

Fig.3-1. Frame detail

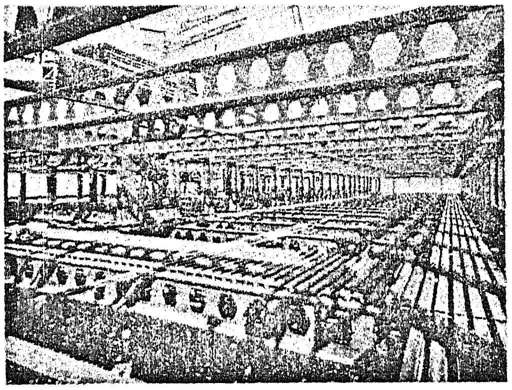

Fig.3-2. Castellated beam and deck plate

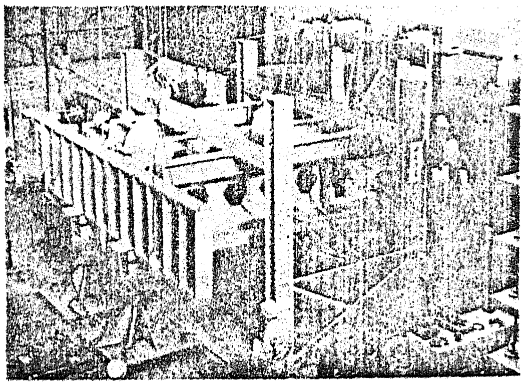

Fig.3-3. Test on column and beam connection

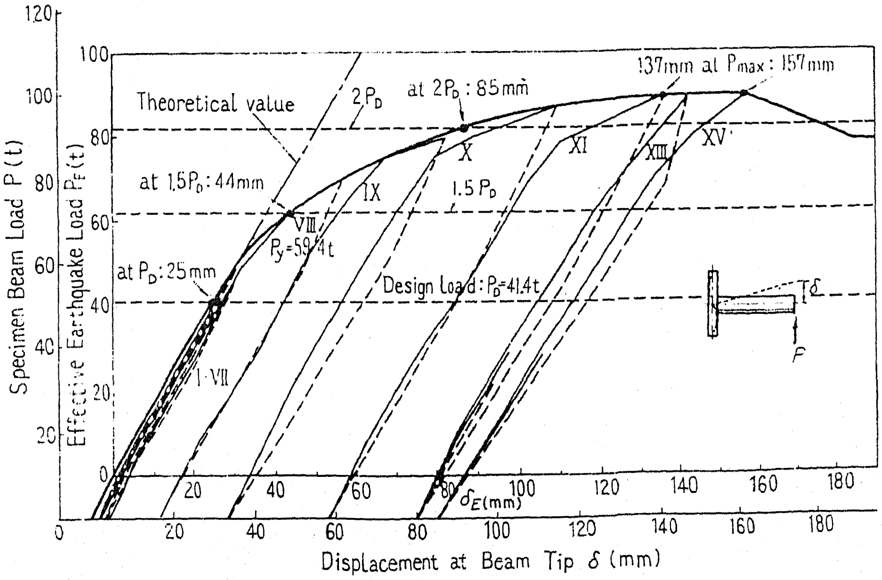

Fig.3-4. Load & deformation curve on column and beam connection

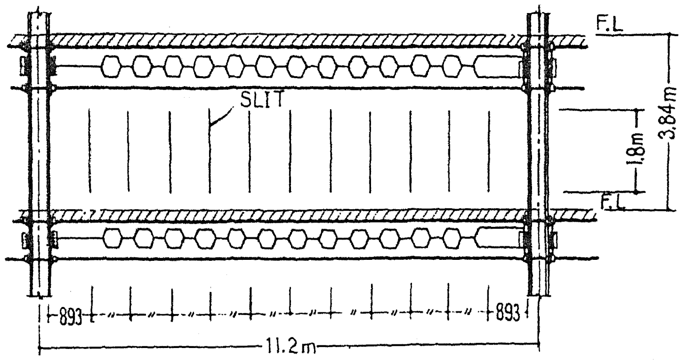

Fig.3-5. Slit layout of shear wall

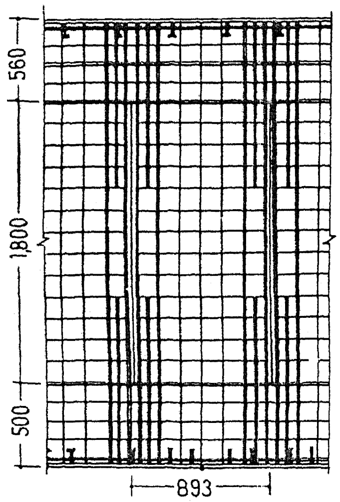

Fig.3-6. Detail of R/C slitted wall

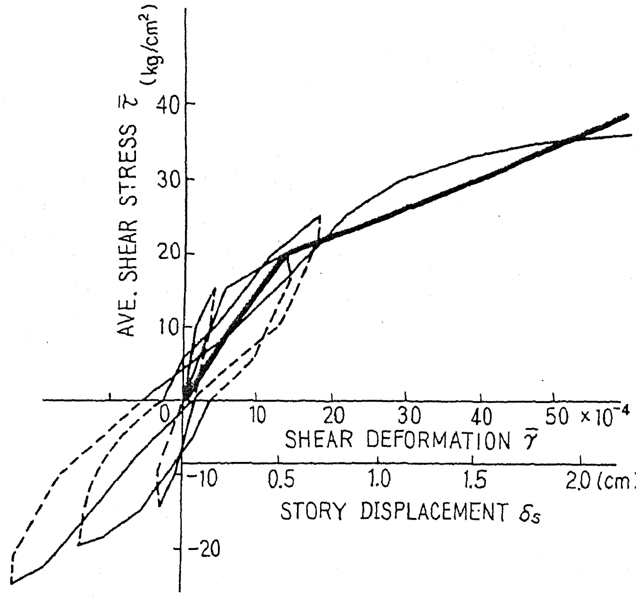

Fig.3-7. Shearing stress-strain curve of R/C slitted wall

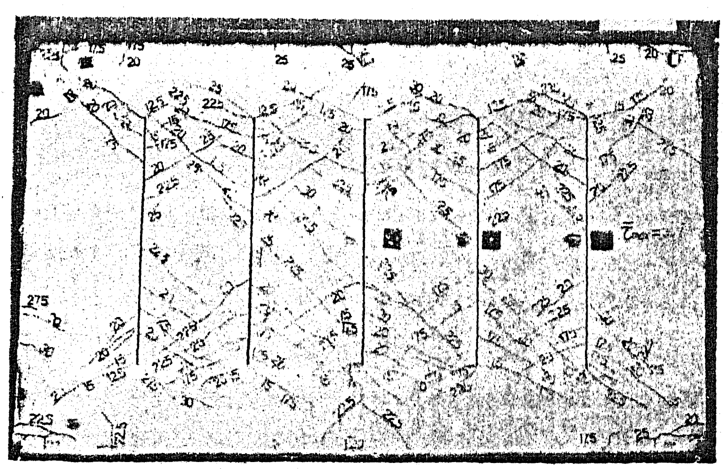

Fig.3-8. Crack pattern on test specimen of slitted wall

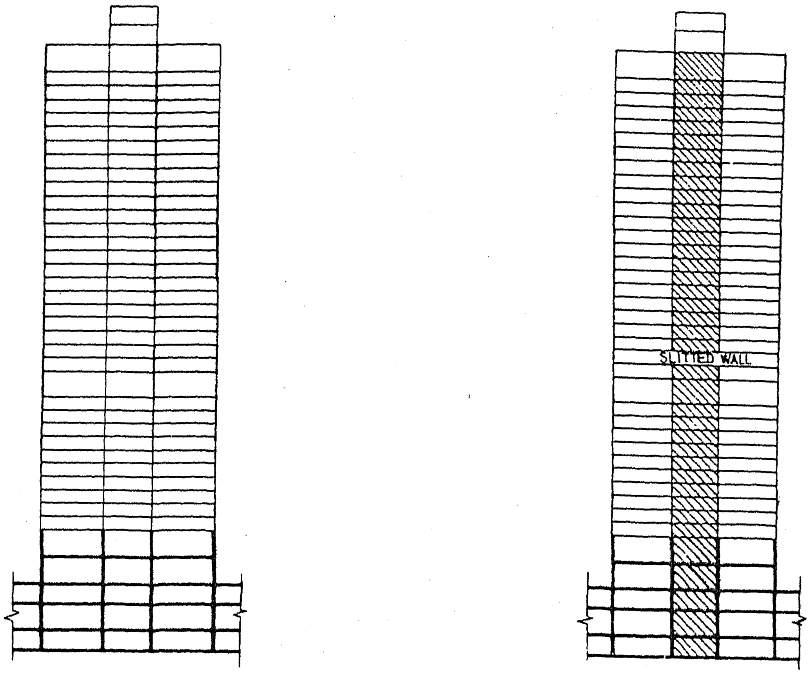

Fig.4-1. Frame view (transverse direction)

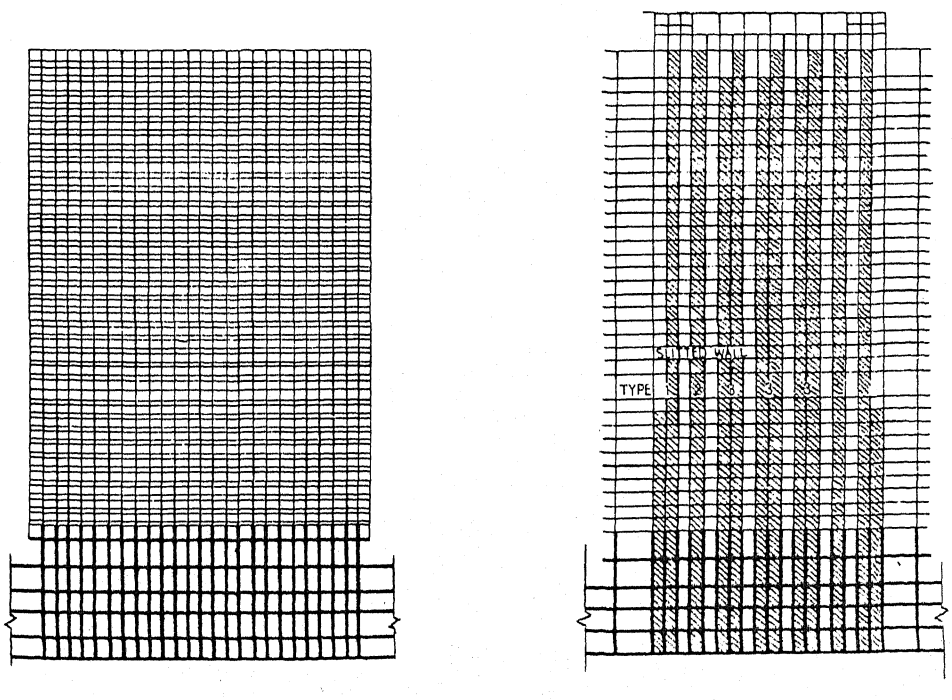

Fig.4-2. Frame view (longitudinal direction)

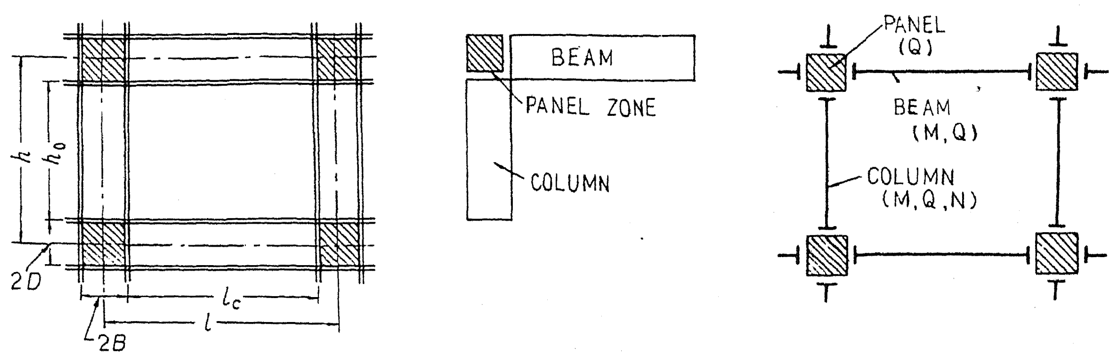

Fig.4-3. Framing components

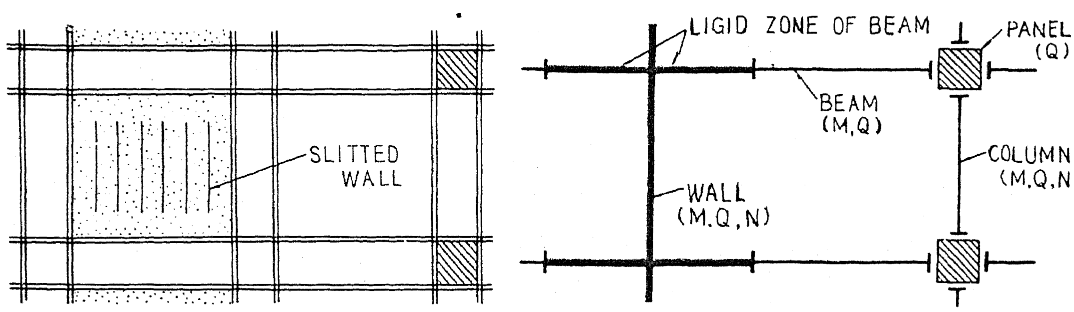

Fig.4-4. Analysis model with wall

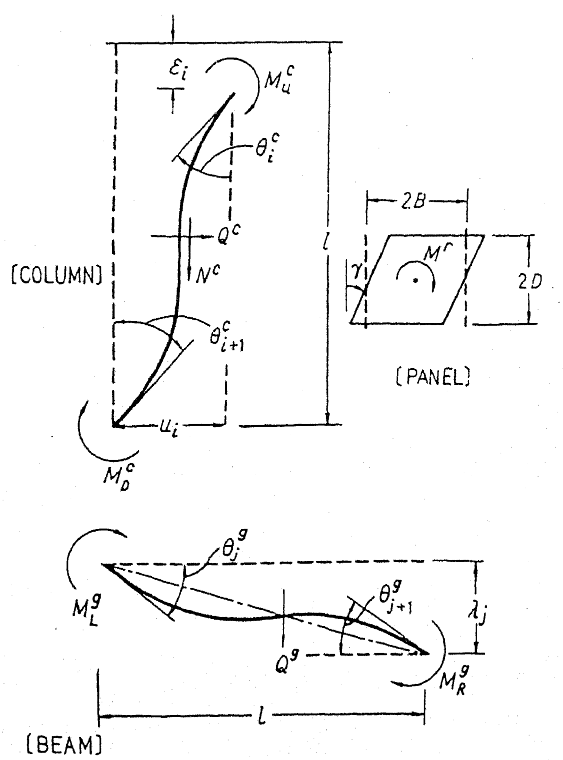

Fig.4-5. Description of force & deformation of structural element

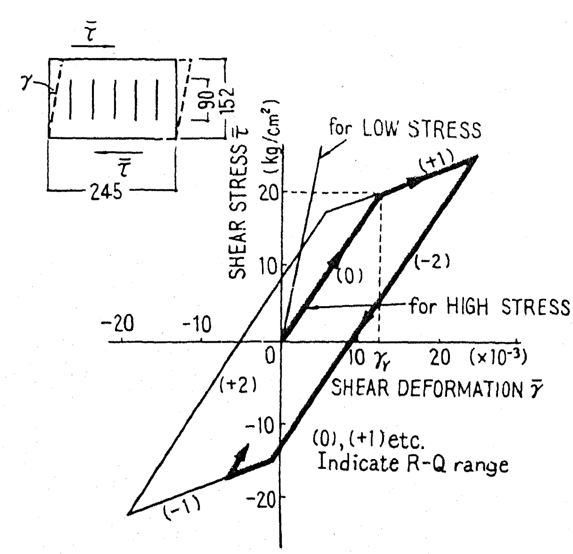

Fig.4-6. Shearing stress-strain relationship of R/C slitted wall used for dynamic analysis

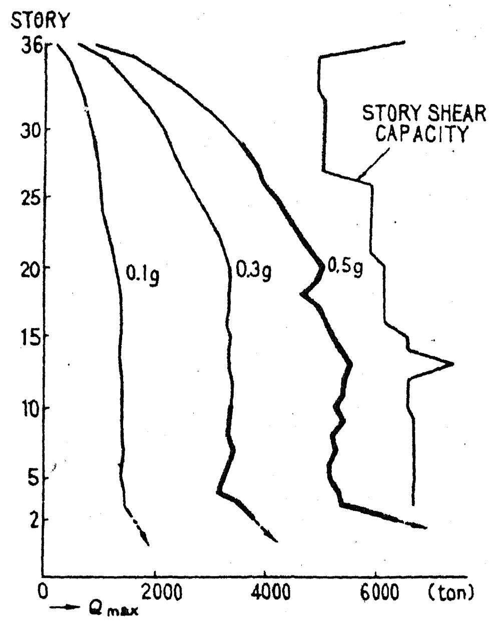

Fig.6-1. Response max. story shear force

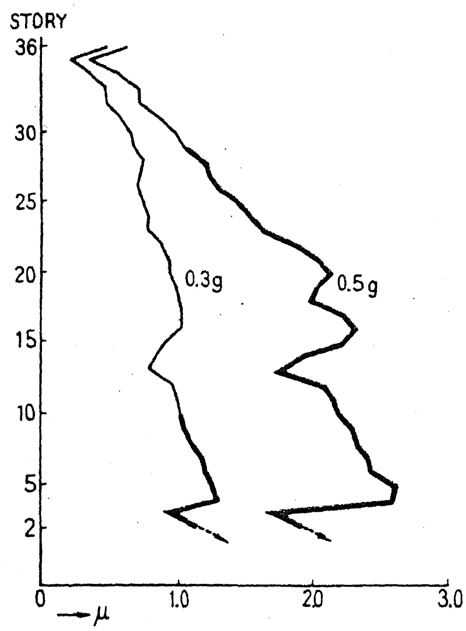

Fig.6-2. Response max. ductility factor (slitted wall)

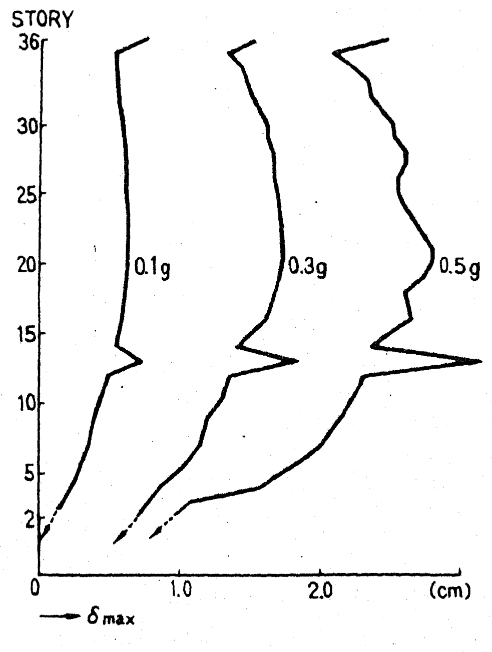

Fig.6-3. Response max. story drift

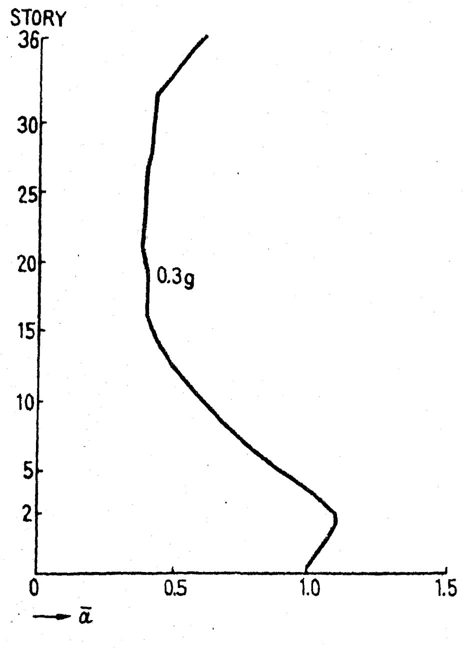

Fig.6-4. Response max. acceleration distribution