AN EARTHQUAKE-RESISTANT DESIGN OF HIGHRISE BUILDING WITH V-SHAPED FRAMING PLAN

By

Kiyoshi Muto[I], Masayuki Nagata[II], Eiji Fukuzawa[II]
Hiromichi Ueno[II], Minoru Hanajima[II]

SUMMARY

This paper presents an earthquake-resistant design of the upper 37 story tower-portion of the Akasaka Prince Hotel with a framing plan of notched V-shape, which is now under construction in Tokyo. The design of this hotel is based on feed-back dynamic design criteria.[1] In these criteria three classes of earthquake intensity, namely, "moderate earthquake-Class I", "severe earthquake - Class II" and "worst earthquake - Class III" are assumed, and earthquake response analyses are conducted in order to seek the optimum structural design. It results in obtaining the final designed framing sections, in which, for Class II earthquake, the stresses in all structural members are less than the allowable values, and for Class III earthquake, the structural members exceed the elastic limit slightly.

§1 INTRODUCTION

This hotel has the unique floor plan as shown in "Fig. 1". Hence, a new structural planning suitable to this novel archtectural planning is sought with much effort. Since the typical floor plan of this building is unusual and the frames which compose it are very slender, complex stresses and deformations by earthquake forces occur due to the three-dimensional (hereinafter called "3-D") effects, which are beyond the anticipation of the analyses of ordinary tall frame structures. Moreover, where the design earthquake forces act in the X-direction, torsional deformations are inevitable. Therefore a framing system and configurations of the slitted wall[2] that are most suitable to minimize the torsional deformation are determined. In this case, precise earthquake-resistant analyses are conducted by using the 3-D frame analysis computer program "FAPP IV"[3] which was previously developed by the authors.

In this paper, the computer program "FAPP IV", the stresses and deformations by static earthquake forces and the linear earthquake responses against the severe earthquake - Class II are described. The nonlinear responses against the worst earthquake - Class III[4] are not described due to lack of space.

§2 OUTLINE OF BUILDING STRUCTURE

The wide spread lower parts of this building below the 3rd floor, which are made of either reinforced concrete structure or steel composite reinforced concrete structure provided with thick bearing walls, are very rigid. Hence,

I. Member of Japan Academy, Director of Muto Institute of Structural Mechanics, Tokyo, Japan
II. Muto Institute of Structural Mechanics, Kajima Corporation, Tokyo Japan

assuming the 3rd floor as the base of the highrise tower, only the 37 story-tower-portion as shown in "Fig. 2" is viewed as the object of analyses.

The typical floor of the tower is as shown in "Fig. 1" and the individual story height is 3.2 meters. The framing system consists of the rectangular steel frames in the X and Y direction with a 4m x 4m grid plan as a unit. The slitted walls of reinforced concrete are configurated in the two wing parts and the center core part. The list of column and girder sections are as shown in "Table 1".

§3 SEISMIC DESIGN CRITERIA

The intensities of input design earthquakes are as same as in the case of the 20 story reinforced concrete building, reported in Reference 5. As the input design earthquake waves, the El Centro 1940 NS, Taft 1952 EW, Tokyo 1956 NS and Sendai 1962 NS are adopted. Then as the input maximum accelerations of "Class I", "Class II" and "Class III" at the base, 100 gal, 250 gal and 400 gal are assumed respectively. The seismic design criteria for the three classes are established as follows:

1) For the moderate earthquake - Class I, the vibration must be controlled so that human disturbance or disconfort is kept at minimum.

2) For the severe earthquake - Class II, the stresses in all structural members must be less than the allowable values and the story deflection angles must be less than 1/250.

3) For the worst earthquake - Class III, even if the structural members exceed the elastic limits, the story deflection angles must be less than 1/150.

§4 COMPUTER PROGRAM "FAPP IV"

The 3-D frame analysis computer program "FAPP IV" is used for the stress and deformation analyses by the assumed static earthquake forces, the natural vibration analyses and the linear earthquake response analyses against the earthquake of Class I and Class II. An outline of this program is as follows:

1) First, with precise consideration to the deformation of columns, girders, joint panels and slitted walls which compose the 3-D frames of the x and y direction as shown in "Fig. 3", the equilibrium equations between forces and displacements on each element in the local coordinate system are derived.

2) In the generalized coordinate system, the unknown displacements of ith floor are the rotations of the panel ${}_x\theta_{ijk}$, ${}_y\theta_{ijk}$, the shear deformation angles of the panel ${}_x\gamma_{ijk}$, ${}_y\gamma_{ijk}$, the vertical displacement w_{ijk} at the nodal point ijk, and the horizontal displacement U_i, V_i, the torsional rotation Φ_i at the gravity center which represent the ith floor displacement from assumption of a rigid floor slab as shown in "Fig. 4". Using these displacements in the generalized coordinate system, the aforementioned displacements in the local coordinate system are expressed. Then the stiffness of the ith story under separation as a unit segment by the matrix transformation calculation.

3) The stiffness of the entire 3-D structure system can be obtained by adding each segment story stiffness from the compatibility condition at each floor level. Then the stresses and deformations by the static earthquake forces can be calculated.

4) Besides the stiffness matrix introducing the mass matrix and the damping matrix, the natural vibration and the earthquake response analyses are executed.

§5 STATIC ANALYSES FOR EARTHQUAKE FORCES

The static behavior of this building by the design earthquake forces in the X-direction are examined. The base shear coefficient of the design earthquake forces is 0.06.

"Fig. 5" shows the horizontal floor displacements. It is noticeable that the torsional behavior can be observed clearly from the differences of displacements between the gravity center and the end frame. Further seeing the floor displacements at the roof level and the 20th level, it is clarified that each floor sways in the X-direction combined with torsional rotation.

"Fig. 6" shows the vertical displacements of column at the roof floor by the overturning moment. The columns of the frontal facade in the side where the earthquake forces act lift up due to the tension, whereas those of the back facade at the opposite side sink down due to the compression. It results in a eminent warp of the floor slab. The imaginary neutral axis by the bending deformation of the whole structure, flow in a V-shaped framing plan from end to end as shown in the dotted line, which is quite different from an ordinary rectangular frame structure.

The normal forces of column due to earthquake forces are large at both wing parts and the surrounding frames of slitted walls.

The bending moments of girder at the 20th floor due to earthquake forces are as shown in "Fig. 7". In the X-direction the bending moments of the adjacent girders to slitted walls are the severest at the center core part with the maximum moment value of 99tm (indicated by mark ◉). In the Y-direction the moment values of the girder are very small except the girder to which the differences of the vertical displacements of adjacent columns are large. The maximum value is 36tm (indicated by mark ○).

§6 NATURAL VIBRATION ANALYSES

The natural vibration periods of this building in the X-direction are shown in Table 2, and the normalized natural vibration modes, $\beta_x U$ are shown in Fig. 8.

§7 EARTHQUAKE RESPONSE ANALYSES

The design earthquakes are input to the base independently in the X or Y-direction which is the principal axis of the resistance of this building. Viscous damping is used assuming 2% damping factor for the 1st vibration mode. For the earthquake of Class II, the response results of the X-direction input for Taft 1952 EW, which is the severest among four adopted earthquake waves, are shown as follows:

The maximum relative horizontal displacements to the base as shown in "Fig. 9" are the largest at the end frame, where the maximum value at the roof floor is 34cm. The relative deformation angle of the roof floor to the base is 1/380, which is comparatively small. It is because the steel frames and the slitted walls coordinate to control the horizontal displacement together with the 3-D effect.

"Fig. 10" shows the response story deflection angles at the gravity centers and the end frames. The maximum values occur at the end frame of the 37 story. This value, 1.1/320, namely 1/290, is less than 1/250 of the design criteria.

§8 CONCLUSION

From the earthquake responses against Class II as described in the section 7 and the another nonlinear responses against Class III, it is assured that the final design of this structure is completely satisfactory to the design criteria.

It is clarified from this investigation that the structure with V-shaped framing plan is easily deformed torsionally by earthquake forces. These torsional deformations are minimized by the concentrated arrangement of the slitted walls at both wing parts.

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(4) Muto, K., et al., 1978 "Nonlinear Earthquake Response Analysis of Highrise Building with V-Shaped Framing Plan" Proceeding of the 5th Japan Symposium on Earthquake Engineering, pp. 1273-1280 (in Japanese).

(5) Muto, K., et al., 1973 "Earthquake Resistant Design of a 20 Story Reinforced Concrete Building" Proceeding of the 5th World Conference on Earthquake Engineering, Vol. 2, pp. 1960-1969.

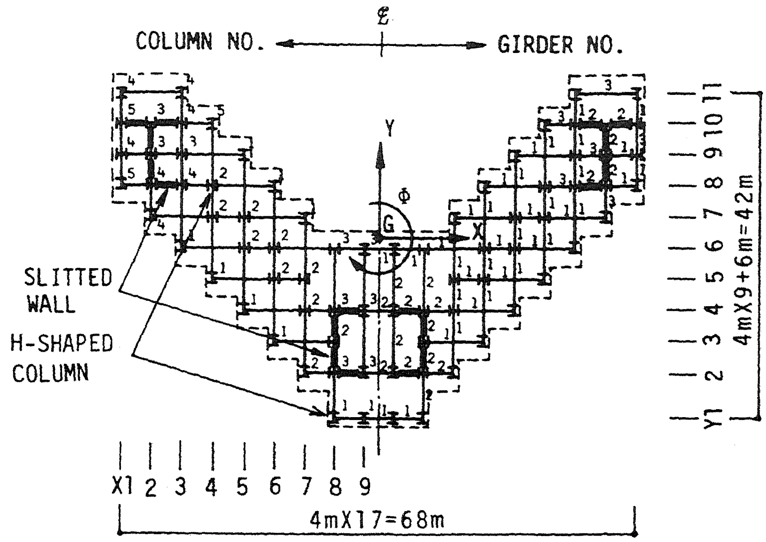


FIG.1 TYPICAL FLOOR FRAMING PLAN

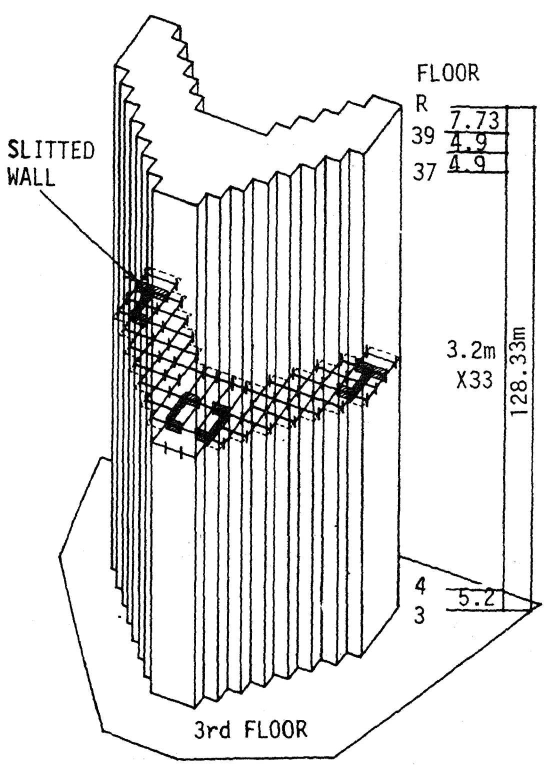


FIG.2 PERSPECTIVE DRAWING OF THE TOWER

TABEL 1 MEMBERS LIST

	NO.	STORY	MEMBER
COLUMN	1	39-14	H-418x412x25x30
		13- 3	H-428x412x25x35
	2	39-26	H-418x412x25x30
		25-14	H-428x412x25x35
		13- 3	H-438x412x25x40
	3	39-26	H-428x412x25x35
		25-14	H-438x417x30x40
		13- 3	H-458x417x30x50
	4	39-26	H-458x417x30x50
		25- 3	H-468x422x35x55
	5	39-26	H-468x422x35x55
		25- 3	H-478x422x35x60
GIRDER	1	R - 3	H-500x200x10x16
	2	R - 3	H-506x201x11x19
	3	R - 3	H-488x300x11x18
	1-3	3	H-600x200x11x17
WALL		39- 3	SLITTED WALL t=15cm

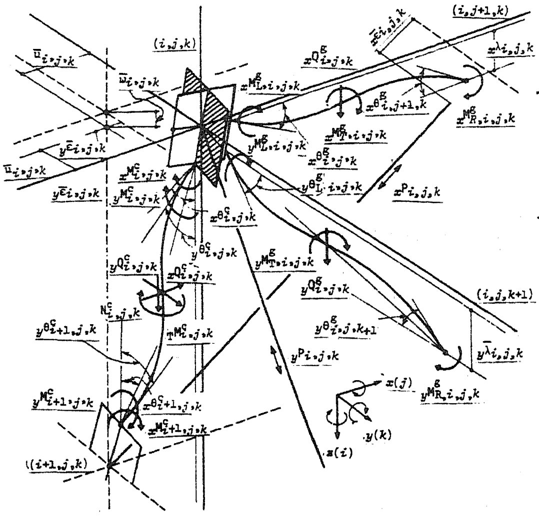
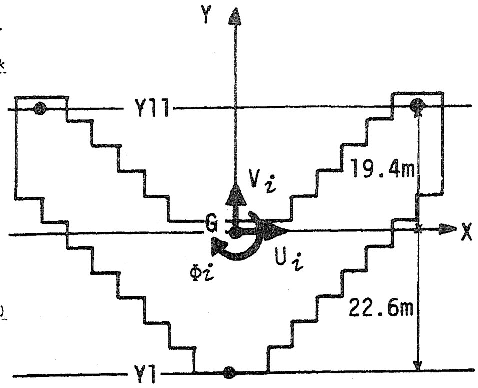


FIG.3 DISPLACEMENTS AND STRESSES OF THREE DIMENSIONAL FRAME

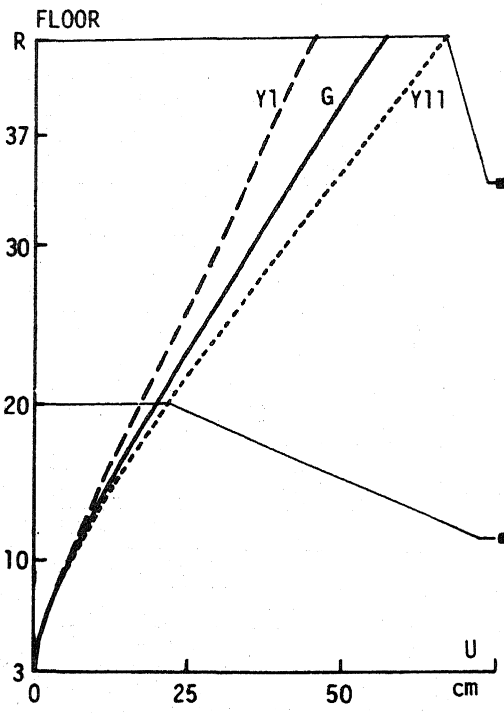


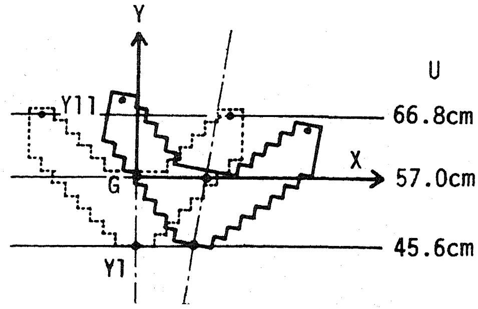
G:CENTER OF GRAVITY

Y1,Y11:END FRAME

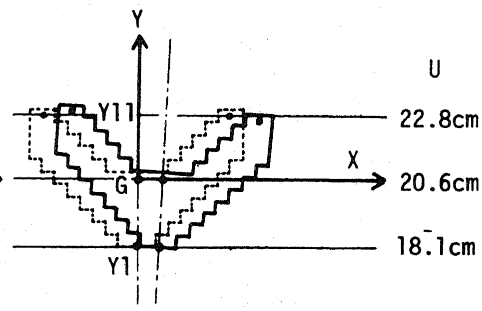
U_i, V_i, Φ_i:THREE DEGREE OF FREEDOM IN the i th FLOOR

FIG.4 THREE DIMENTIONAL COORDINATE SYSTEM





FLOOR DISPLACEMENT AT ROOF LEVEL



FLOOR DISPLACEMENT AT 20th LEVEL

FIG.5 HORIZONTAL FLOOR DISPLACEMENT U BY STATIC EARTHQUAKE FORCES ACTING IN X-DIRECTION

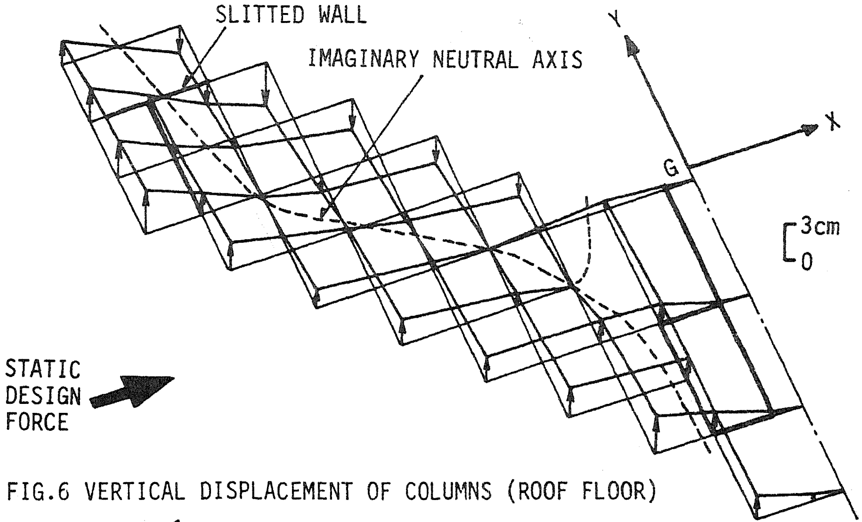


FIG.6 VERTICAL DISPLACEMENT OF COLUMNS (ROOF FLOOR)

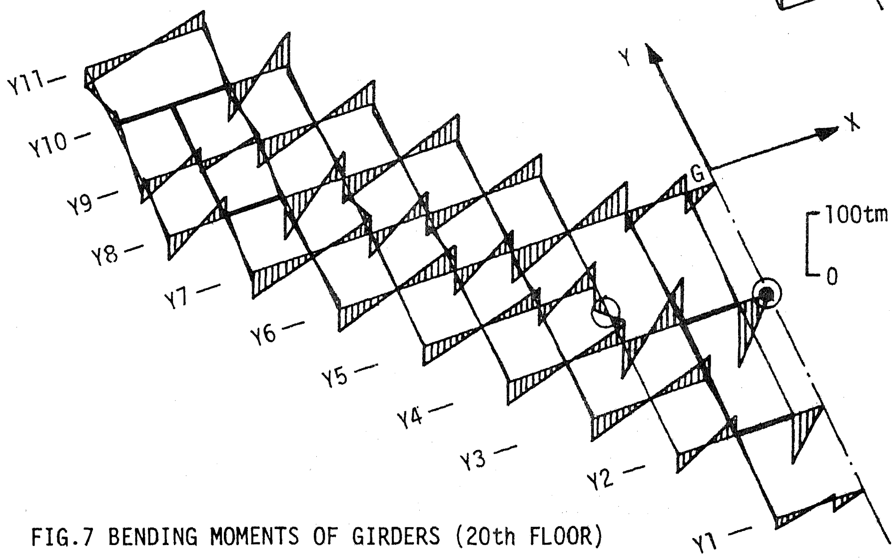


FIG.7 BENDING MOMENTS OF GIRDERS (20th FLOOR)

TABLE 2 NATURAL VIBRATION PERIODS IN THE X-DIRECTION

	1st	2nd	3rd	4th	5th	6th
NATURAL PERIOD (SEC.)	4.20	3.38	1.19	1.01	0.63	0.54
PREDOMINANT COMPONENTS	X- Φ	Φ -X	X	Φ	X	Φ

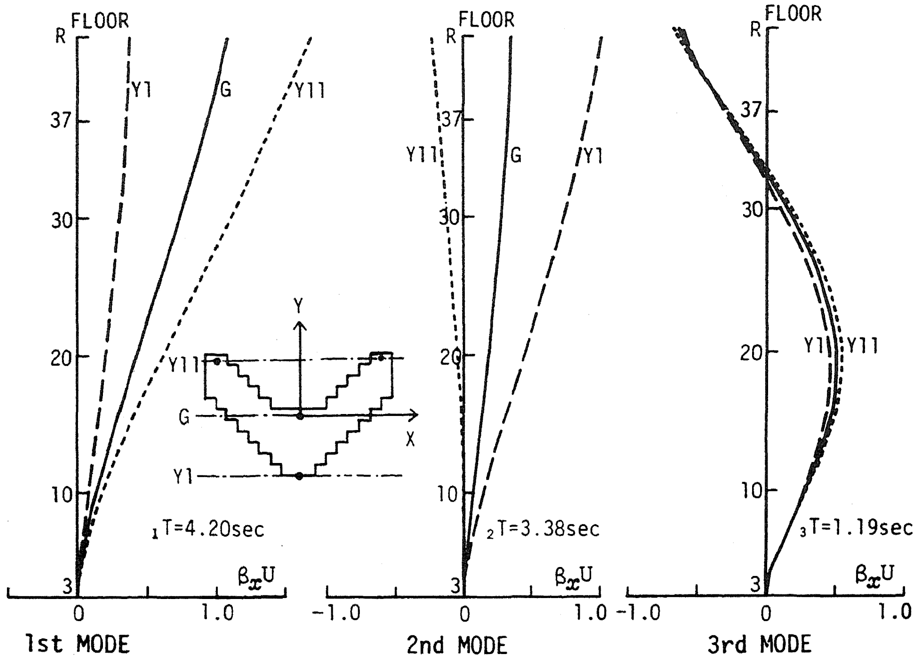


FIG.8 NATURAL VIBRATION MODE SHAPES

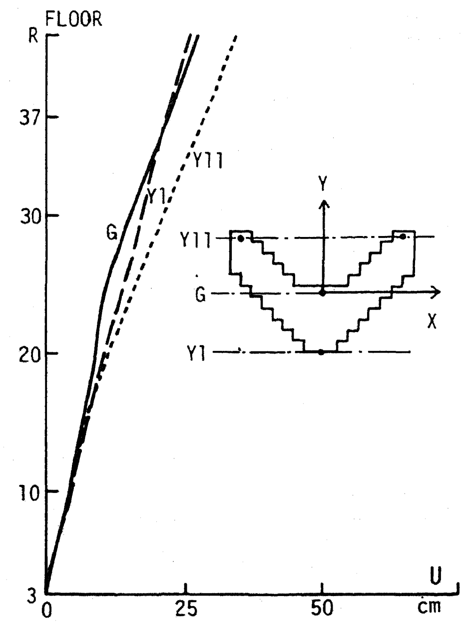


FIG.9 MAX. RESPONSE HORIZONTAL DISPLACEMENTS BY TAFT EARTHQUAKE

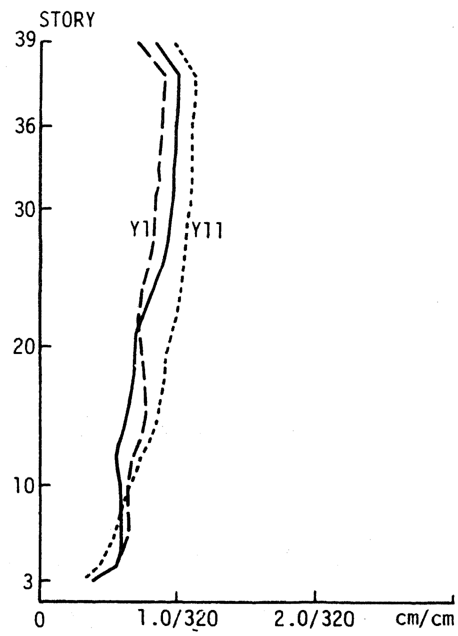


FIG.10 MAX. RESPONSE STORY DEFLECTION ANGLE BY TAFT EARTHQUAKE