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The design and analysis of a building with atrium and large cantilever

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ABSTRACT: Recently, the design of high rise buildings of a particular plan and shape has become possible in Japan as a result of development in computer analysis etc. This paper shall investigate the effects of lateral torsional coupling on the earthquake response, and the vertical motion of such type of buildings. For the analysis of the building, three vibration analysis models are introduced, and some practice design methods of the building which have complex shape are proposed based on the results of the analysis.

1 INTRODUCTION

The building dealt with herein is a 30 story office building for which a special plan has been selected in the course of design competition. The outlines of this building are as follows(Figure 1～4).

- Building name : Bunkyoku Civic Center (temporary name)
- Use : Ward office
- Location : Tokyo, Japan
- Number of basement floors : 4
- Number of stories above ground : 30
- Typical floor area : 1,400m²
- Total floor area : 44,000m²
- Standard floor height : 4,000mm
- Total height : 145,200mm
- Bay size : 6,300mm x 14,000mm
- Plan dimensions : 31,500mm x 44,100mm
- Basement structural type : steel reinforced concrete
- Structural type above ground : steel
- Total steel weight : 8600t
- Typical column size : 600 x 600mm (maximum thickness 60mm)
- Typical beam size : 900mm-depth, 350mm-wide H-section plate girder (maximum thickness 32mm)

This building has two special features in the planning. One is a large cantilever from the 25th～27th floors used as a meeting hall for the prefectural assembly and as a sightseeing place for the public.

The other is a large atrium through the floors used as a light court which gives a good condition for daylighting and ventilation of the office rooms, and plays an important part of the smoke exhausting system. The atrium is provided between the northwing and the southwing. The two wings are connected by cross beams etc. at certain levels.

2 STRUCTURAL SCHEME

In order to realize the two main design intentions (a large cantilever, a large atrium) in this high rise building, the following structural design methods are adopted.

1. Super-frame system: As there are almost no slabs and beams in the atrium to connect the wings, it is difficult to obtain proper lateral stiffness in the the Y-direction. Therefore, a super-frame with super-columns and super-beams is adopted in the 2,7-frames. Super-columns consist of two columns linked by V-braces, and super-beams are Vierendeel trusses in which floor beams are connected by posts to form trusses(Figure 3). In the X-direction, the ordinary moment-resisting frames are adopted.
2. Cross beams: There are cross beams at the 4th, 5th, 14th, 15th, and 22nd～28th floors to achieve sufficient floor diaphragms(Figure 1,2).
3. Suspension and opposite member balancing: The meeting hall is suspended from the top of the building and counterbalanced by the opposite member like a balancing toy (Figure 4).
4. Box girders: The perimeter members of the semicircular meeting hall are box shaped girders (h=1100mm, w=900mm) to resist a simultaneous action of the bending moment and torsional stress(Figure 2).
5. Weight balance: Special precaution is paid to secure a good weight balance between the northwing and the southwing of the building. For example, the heavy-duty-zone (in which live load is three times as much as that in the standard office) is assigned

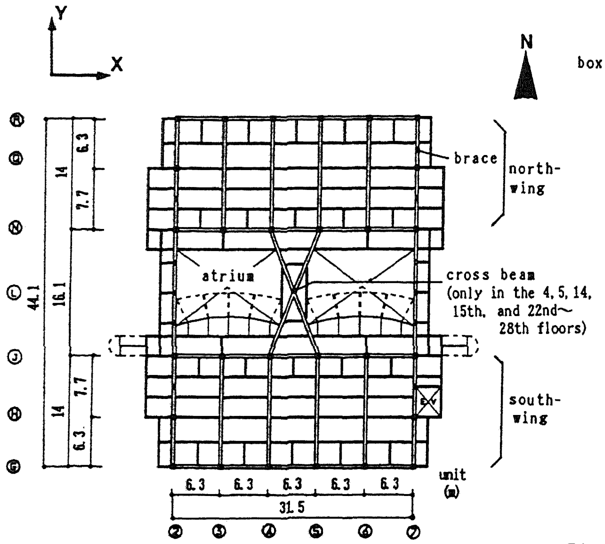


Figure 1. Standard Framing Plan

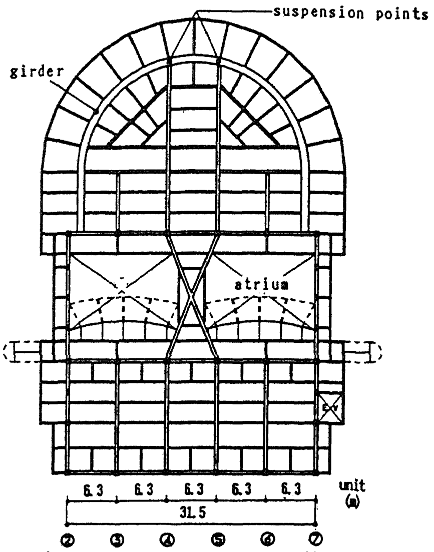


Figure 2. Framing Plan of the Meeting Hall

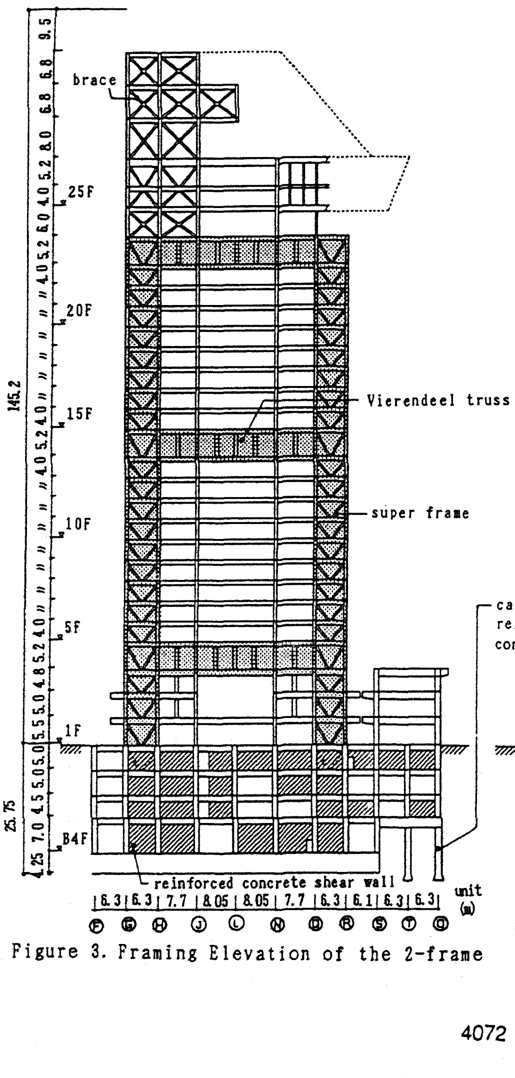


Figure 3. Framing Elevation of the 2-frame

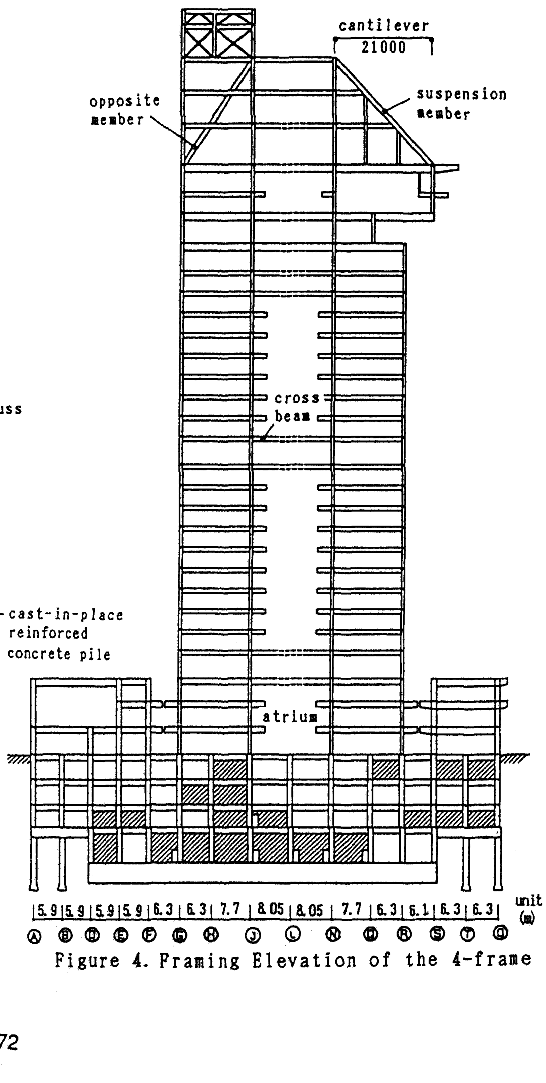


Figure 4. Framing Elevation of the 4-frame

to the northwing to eliminate weight unbalance due to different story numbers between the two wings.

3 EARTHQUAKE-RESPONSE ANALYSIS

In the earthquake-resistant design of this building, dynamic response analyses using three vibration models against the assumed design input ground motions have been carried out as shown below.

3.1 Vibration models

The following three vibration models are introduced.

1. The basic lumped mass model (B-model), which has a stiffness matrix with 1 degree of freedom (DOF) per floor. The natural periods of this model are shown in Table 1. In this model, the directions of input

Table 1 Natural Periods of B-model

	X			Y		
Order	1	2	3	1	2	3
Period(sec)	3.11	1.07	0.61	3.10	0.96	0.54

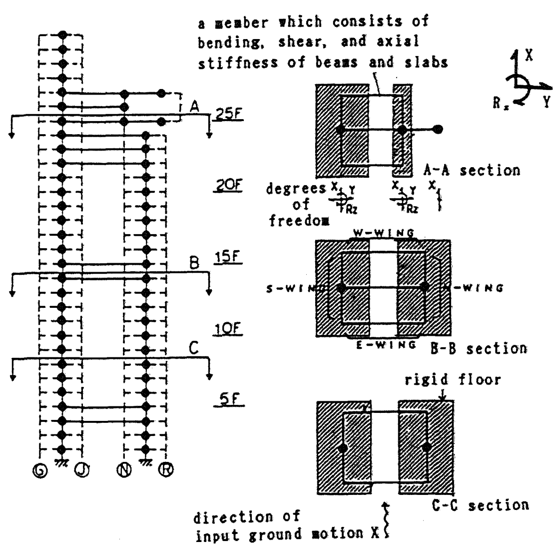


Figure 5. 3D-model Diagram

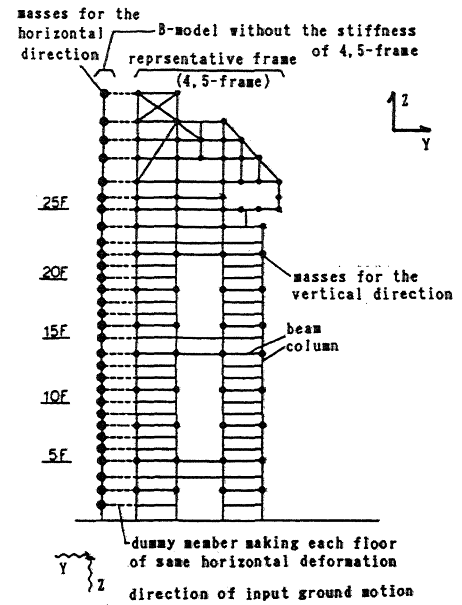


Figure 6. V-model Diagram

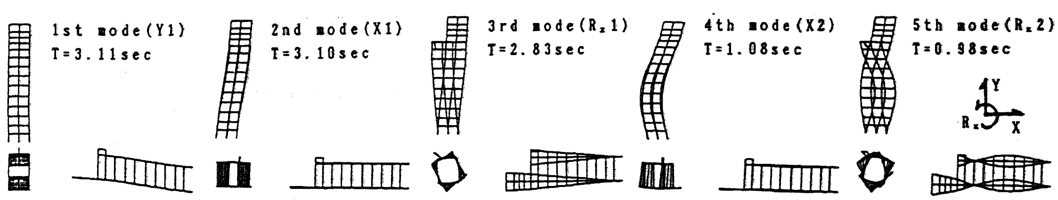


Figure 7. Natural Periods and Vibration modes(3D-model)

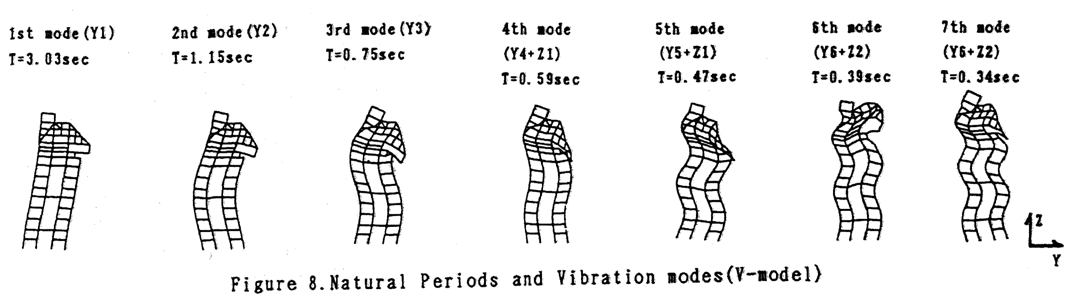


Figure 8. Natural Periods and Vibration modes(V-model)

ground motions are X and Y.

2. The 3-dimensional vibration model (3D-model), in which each floor is divided into two masses. Each mass has 3 DOF (X, Y, R_z) and is connected with members that have in-plane stiffness representing actual beams and slabs(Figure 5). This model was used to study the stress of the connecting members (ex. cross beams) between the southwing and the northwing, and the story shear force by the torsional effect. Figure 7 shows the natural periods and vibration modes. These natural peiods are very similar to those of the B-model. *In this model, full live load* for the southwing and no(0) live load for the northwing are taken to investigate effects of eccentric live load distribution. Here the direction of input ground motion is only X.

3. The vertical vibration model (V-model), which has a stiffness matrix with 1 DOF per floor for horizontal motion and 68 masses for vertical motion in a representative frame(Figure 6). This model was used to study the dynamic response (accerаration, displacement etc.) of the cantilever. Figure 8 shows the natural periods and vibration modes of this model in which there are many coupling modes of horizontal motion and vertical ones. In this model the directions of input ground motion are Y and Z.

3.2 Input ground motions

Table 2 shows the input ground motions used for the each model. Figure 9, 10 show the tripartite response spectra of them. Here the artificial wave was generated by the following process.

1. *Original acceleration response spectrum* (S_A') was prepared according to the design spectral coefficient (Rt) of Japanese Building Standard Law. In this case, the base shear coefficient is 0.5.

2. Original velocity response spectrum (S_V') is calculated by the formula;

$$S_V' = S_A' \cdot T/(2 \cdot \pi)$$

3. Required S_A and S_V for the analysis is obtained by reducing original ones using the damping factor h, that is;

$$S_A = S_A'/D_h \ , \ S_V = S_V'/D_h$$
$$D_h = 1.5/(1+10h) \ , \ h=0.02$$

4. Artificial wave is calculated by the inverse Fourier transform increasing or decreasing the amplitude spectrum. Phase spectrum of the artificial wave was adjusted using that of recorded ground motion.

3.3 Results

1. The B-model and the 3D-model: In general, response story shear force of 3D-model (sum of two wings) is almost the same as that obtained in the B-model. Response story deflection in the 3D-model at 24th floor is

Table 2 Input ground motions

Model name	B	3D	V
El Centro NS	○	×	○*
El Centro UD	×	×	○*
TAFT EW	○	×	○**
TAFT UD	×	×	○**
Artificial wave	○	○	×

1)○:used, ×:unused
2)*,**:each of them is simultaneously input

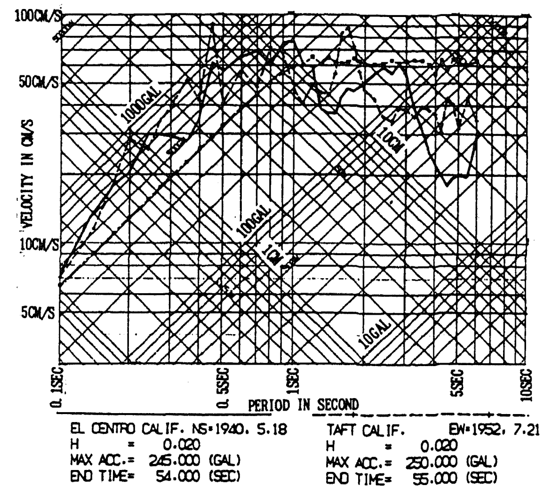


Figure 9. Tripartite Response Spectrum of Horizontal Ground Motion

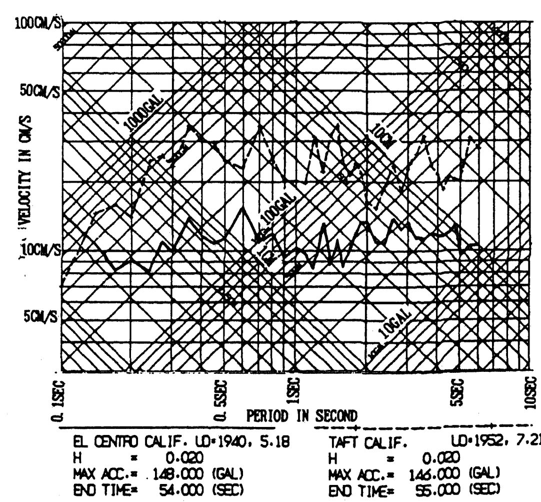


Figure 10. Tripartite Response Spectrum of Vertical Ground Motion

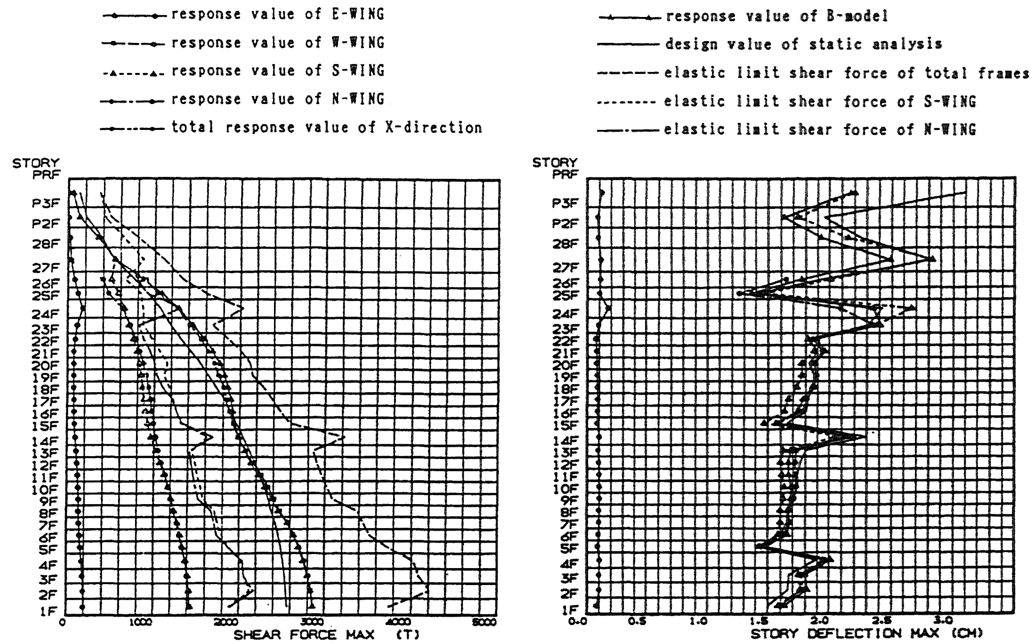


Figure 11. The Maximum Responses of 3D-model (Artificial wave)

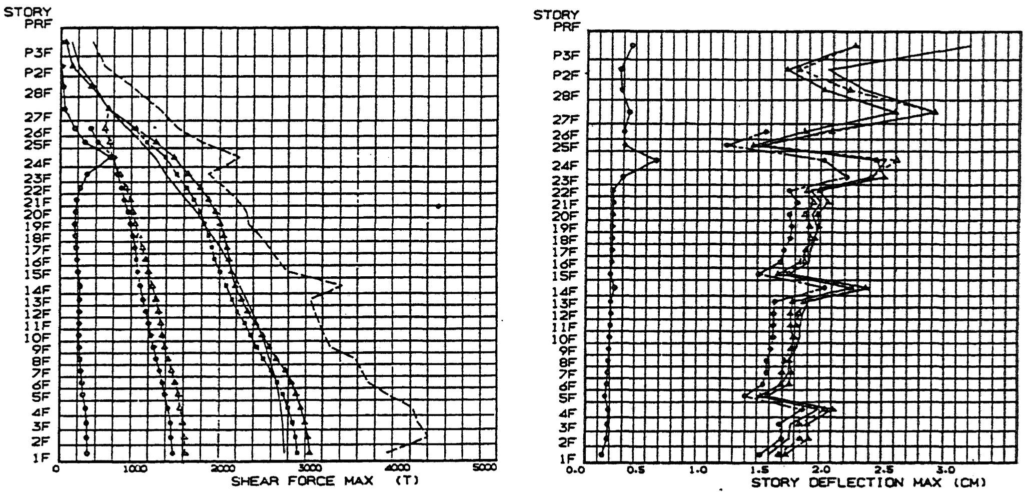


Figure 12. The Maximum Responses of 3D-model
(Artificial wave with weight eccentricity)

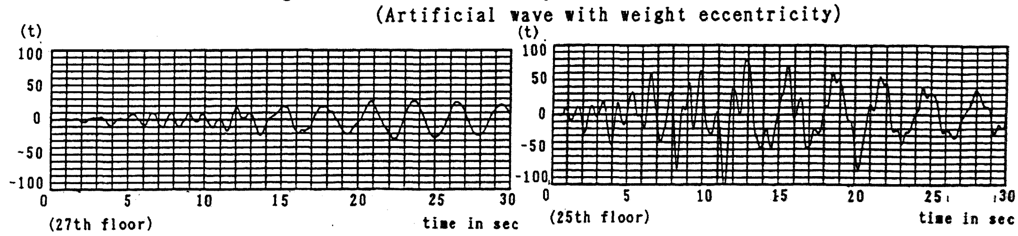
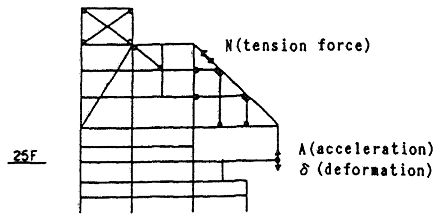


Figure 13. Timehistory of the Shear Force Transferred through the Cross Beams
(Artificial wave)

Table 3 Maximum resonse value

ground motion		A(G*)	δ (cm)	N(t)
El centro	NS	0.526	5.98	173.
	UD	0.152	0.85	52.
	NS+UD	0.542	6.13	178.
TAFT	EW	0.365	4.39	118.
	UD	0.423	1.73	153.
	EW+UD	0.488	4.42	165.

* 'G' means the acceleration of gravity



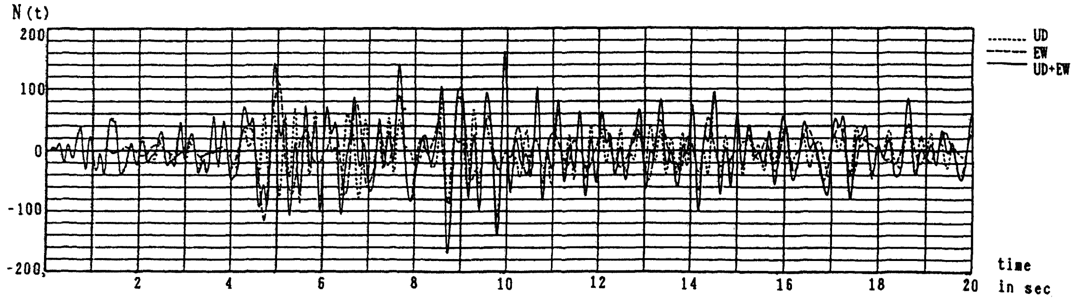


Figure 14. Timehistory of Suspension Member's Tensile Force (TAFT)

approximately 1.1 times that in the B-model (Figure 11).

It was noted that response story shear increases remarkably in the direction perpendicular to the ground motion due to the floor weight eccentricity ; however, the increase in the direction parallel to the ground motion was negligible(Figure 12).

Figure 13 shows the timehistory of the response shear force in plane transferred by the cross beams provided at the 27th and the 25th floor. Its maximum value is 110t at the 27th floor, and the shear force transfer is not so large in the lower floors(15F, 14F, 5F, 4F). For reference the weight of the meeting hall is 632t at the 27th floor level and 485t at the 25th.

2.The V-model: The V-model analysis shows that the maximum response acceleration in the vertical direction at the free end of the cantilever is 0.542G. The maximum response displacement in the same direction is 6.13cm, that is the angle of inclination between the free end and the fixed end is 0.33 $\times 10^{-3}$rad. The maximum response tensile force of the suspension member (in which the permanent tensile force is 370t) is 178t (Table 3).

Figure 14 shows the timehistory of the tensile force of the suspension member for TAFT of which vertical response value is relatively large.

4 CONCLUSIONS

Interpretation of the presented results leads to the following conclusions and design recommendations for similar type buildings.

1. Considering the torsional response practically (if a simple model such as B-model is used), it is sufficient to add 5~10% response in the direction of ground motion. Moreover 15%(generally)~25%(in the 23rd~25th stories) of the response shear force in the direction parallel to the ground motion should be considered to act simultaneously in the direction perpendicular to that due to the weight eccentricity.

2. Cross beams are effective to increase the plane stiffness and decrease the torsional effect of the whole building, but according to the result of the analyses large shear transfer and bending moment have to be considered in the floor diaphragms. Therefore, in the design of this building, steel panels under concrete slabs are adopted where they occur (23rd~27th floors), so that no additional stress in the general beams is generated.

3. In order to secure structural safety, it is preferable to design the cantilever members to remain within the elastic range even under the action of severe earthquakes. For this reason it is recommended to limit the stresses due to the dead & live loads of the members in such a large cantilever to below 40~50% of its allowable value(yield stress), considering the dynamic effects of vibration in the vertical direction.

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