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DYNAMIC INELASTIC ANALYSIS FOR TORSIONAL BEHAVIOR OF A SETBACK-TYPE BUILDING

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SUMMARY

In this paper, torsional inelastic behavior of a tentatively designed 11-story setback-type building is discussed based on the earthquake response analysis using three-dimensional frame model. First, the original test-designed building model is analyzed to investigate torsional behavior of a setback-type building subjected to a strong earthquake. Secondly, the modified models whose strength of each frame is adjusted according to its torsional response properties are analyzed to investigate rational treatment in seismic design for torsion.

INTRODUCTION

Recently, setback-type buildings are increasing because of the advantage of lighting and the uniqueness of shape. In setback-type buildings, some discrepancy exists between the center point of lateral shear force and that of stiffness at each story, which is eccentricity, and causes torsional vibration during earthquakes. As for the torsional inelastic vibration of eccentric buildings subjected to earthquakes, many researches have been already performed. But most of them treat buildings of which each floor plan is equal and the locations of center of gravity exist in a vertical line, and very few researches treat setback-type buildings. Investigation on the torsional inelastic behavior in setback-type buildings is needed to get useful information for the seismic design of this kind of buildings.

OUTLINE OF BUILDING

The building for earthquake response analyses is an 11-story reinforced concrete building which has been tentatively designed for the study of high-rise frame wall buildings conducted by the Japan Ministry of Construction. The plan of the standard floor (1F～5F) is shown in Fig.1, and the section of the longitudinal frames is shown in Fig.2. The longitudinal frames (Y0～Y2 Frames) are pure open frames, and the transverse frames are cantilever shear walls (X0～X10 Walls). The building has setbacks of one span per story above the 6th story in the longitudinal direction. The weight of the building, the sizes of columns, beams and shear walls, and the concrete strength are shown in Table 1. In the transverse direction, X6 to X9 Walls have the same sizes and reinforcement in columns and walls.

Since this building has setbacks in the longitudinal direction, its response in the transverse direction is torsional, and the strength of the building must be

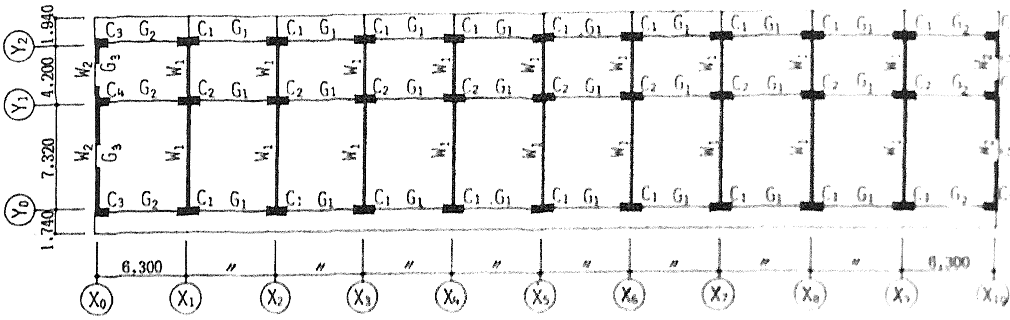


Fig.1 Standard Floor Plan (1F～5F)

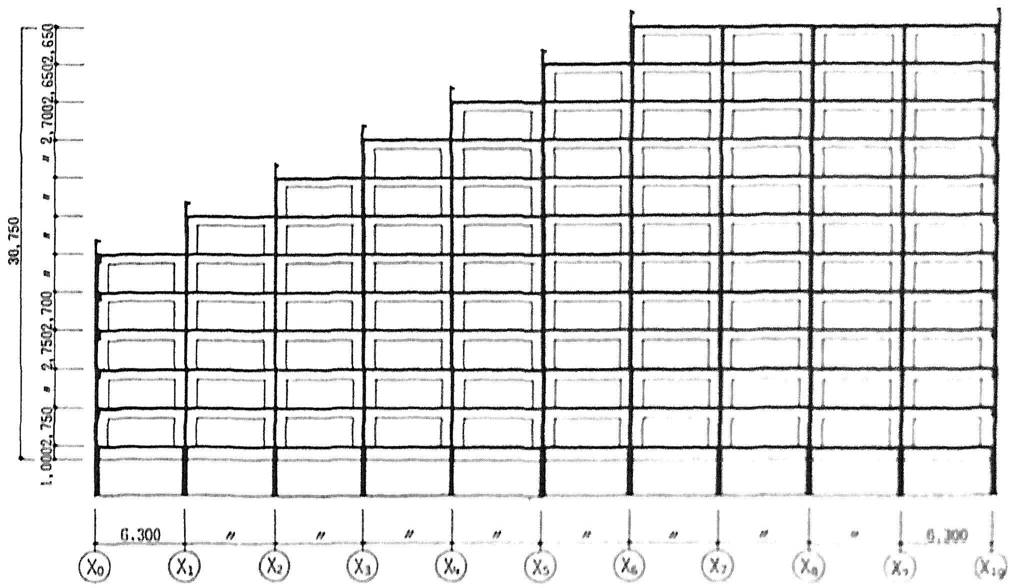


Fig.2 Section of Longitudinal Frames

Table 1 Data of the Test-designed Building

Floor	Mass (t)	Size of Columns (cm) External Frame (For Transverse Direction)	Size of Columns (cm) Internal Frame (For Transverse Direction)	Size of Beams in Y0 Y2 Frames (cm)	Thickness of Walls (cm)	Strength of Concrete (kg/cm²)
11	381	40 * 92.5	40 * 160	(11F,RF) 40 * 65	18.0	210
10	488					
9	592			(9F,10F) 40 * 70		
8	692					
7	818	50 * 92.5	50 * 160	(7F, 8F) 50 * 70		225
6	936				20.0	
5	1,096	60 * 92.5	60 * 160	(6F, 5F) 60 * 70		
4	1,127				22.5	240
3	1,127			(2F 4F) 60 * 75		
2	1,135				25.0	
1	1,144					

increased considering the effect of torsion following the current Japanese earthquake design code. In the structural design of this building, the strength of all walls and that of all stories are made 1.5 times as large as the wall strength determined by neglecting its torsional effect, based on the maximum eccentric rate of 0.56 at the 5th story.

MODELING OF BUILDING

Frame Model The building is modeled to three-dimensional frame composed of line members. Frame models of Y0 Frame in the longitudinal direction and X0, X3, X6, X10 Walls in the transverse direction are shown in Fig.6. In the figure, the distribution of frame damages calculated from the following earthquake response analysis is also shown. The floor is assumed as rigid body. The biaxial bending interaction and the axial deformation are neglected. Assumptions for the base condition are as follows. As for the columns in the longitudinal direction, the fixed support is adopted. As for the shear walls in the transverse direction, the rotational spring support according to the pile's stiffness is adopted because the influences of rocking are significant.

Member Model Structural members, i.e., columns, beams and shear walls are replaced by the same member model as shown in Fig.3. The inelastic bending and shear behaviors of each member are modeled by two rotational springs and one shear spring, respectively. The hysteresis models for bending spring and those for shear spring are assumed as the Modified-Takeda model and the origin-oriented model, respectively, as shown in Fig.4. The stiffness, strength and deformation of springs are evaluated from its dimension and its reinforcement of each member.

Fig.5 shows the natural periods, the modal shapes and the participation functions at the roof floor for the first and the second modes subjected to an excitation in the transverse direction.

RESPONSE ANALYSIS OF TEST-DESIGNED BUILDING MODEL

Outline of Analysis The input ground motion used in the response analysis is the acceleration record at Hachinohe Harbor in Aomori Prefecture during the Tokachi-oki Earthquake in 1968. In the analysis, the record is multiplied by 1.5 assuming a very strong earthquake, and the EW-component (max. 274 gal) and the NS-component (max. 338 gal) are used as inputs simultaneously for the longitudinal direction and the transverse direction, respectively. For numerical integration, Newmark's beta method with the beta value of 0.25 is used, and integration step is 0.001 sec. The damping is assumed to be proportional to initial stiffness, with its damping factor of 0.03 for the first mode.

Results The distribution of calculated damages at Y0 Frame and X0, X3, X6, X10 Walls are shown in Fig.6, with the maximum values of story displacement. In the figure, the ductility factors for bending yielding of the members are also shown. First, as for the values of story displacement in transverse direction, it is found that the values at X0 and X10 Walls located at external side are twice and four times as large as those at X3 Wall, respectively. This result indicates that severe torsional vibration occurred in the building with its center of torsion near X3 Wall. Next, as for the distributions of shear cracking in transverse direction, it is found that cracks are severer in walls located farther from X3 Wall. In judging the above results, it is indicated that the distribution of

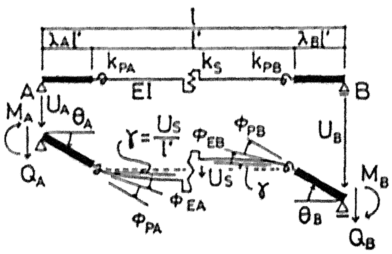
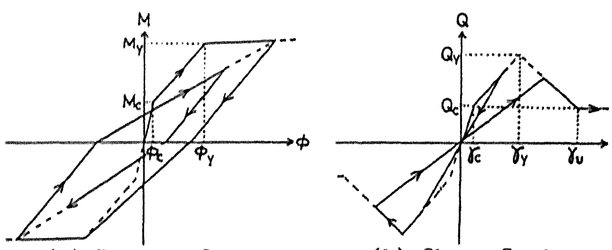


Fig.3 Member Element Model



(a) Bending Spring (b) Shear Spring

Fig.4 Hysteresis Models of Inelastic Springs

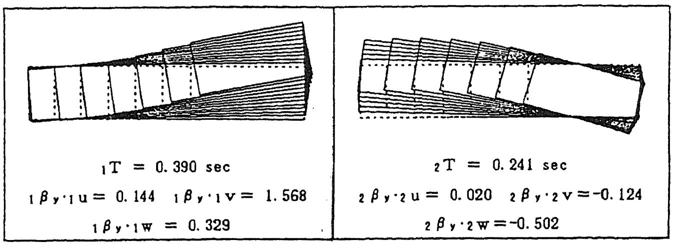
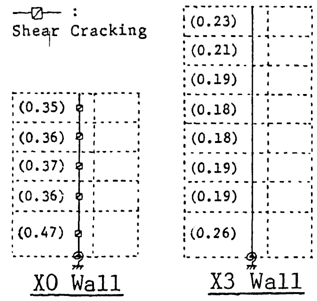


Fig.5 Natural Periods, Modal Shapes and Participation Functions at the Roof Floor



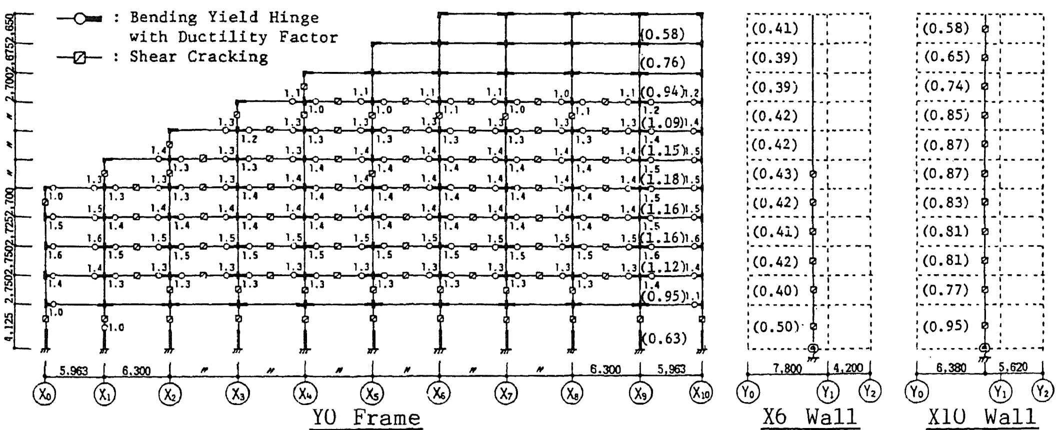


Fig.6 Frame Model and Distribution of Calculated Damages
* ( ) indicate the maximum values of story displacement (cm).

response values and damages in setback-type buildings are greatly influenced by torsion. Therefore, it is clear that the strength distribution of the building must be determined according to its torsional response properties.

RESPONSE ANALYSES OF MODIFIED-STRENGTH MODELS

Modified Models In this section, according to the results of the analysis of the test-designed building model, several modified models which have the strength distribution different from the original test-designed building in the transverse direction are considered based on its elastic torsional response. As pointed out before, the total strength of the original test-designed building model in the transverse direction is increased because of the existence of setback. In the following analyses, modified-strength models are determined by modifying the bending and shear yielding strength of each wall in the transverse direction, using the modification factor m which is determined according to its torsional response properties. Let Q and Q' be the elastic shear force at each frame without consideration of torsion and that with consideration of torsion, respectively. Then, the m-value at each frame is given from the following equation as

$$m = Q' / Q \tag{1}$$

In this analysis, m-value at each wall is calculated from the base shear force at each wall, and it is assumed that values in each floor are same in each wall. In the modified-strength models, the bending and shear yielding strength of each member is determined by multiplying that in the original test-designed building model, by modification factor m.

According to the method mentioned above, two types of models are considered. One is the statically modified-model based on the result of the elastic static response analysis. The other is the dynamically modified-model based on the result of the elastic earthquake response analysis performed by the same way as the inelastic analysis mentioned above. In the following, they are called Model-S1, Model-D1, respectively. The total strength of these two models are nearly equal to the original test-designed building model. Besides these two models, another two models whose m-value is reduced by 20% are considered. In the following, for the static modified model, it is called Model-S2, and for the dynamic modified model, it is called Model-D2. Fig.7 shows the distributions of the modification factor at each wall for four models.

Results The maximum values of story displacement and the distribution of calculated damages at X1, X6 and X9 Walls are shown in Fig.9 through Fig.12. Those in the original test-designed building model are shown in Fig.8. In these figures, the ductility factors for bending at the base of each wall are also shown. First, as for the bending yielding at the base of walls, they are seen at X6 Wall in Model-S2 and at X9 Wall in Model-D2, in both of which total strength have been reduced. But maximum value of ductility factor is only 1.4. Next, as for the distribution of ductility factors for bending at the base of walls, the difference between maximum value and minimum value is about 0.2, 0.5, 0.3 and 0.3 in Model-S1, Model-S2, Model-D1 and Model-D2, respectively. Compared to the difference of about 0.5 in the original test-designed building model, it is recognized that these values are less in these strength-modified model. These results indicate that the determination of strength at each frame according to its torsional response properties is effective to make its damage level uniform. Moreover, judging from the distribution of ductility factors for bending, it seems that the dynamically modified models are a little better than the statically modified models.

Finally, to investigate the amount of torsion, the torsional response ratio R in X10 Wall calculated from the following equation at the 1st, 5th and 11th stories are shown in Table 2 with that in the original test-designed building model.

$$R \doteq |\theta|\,\mathrm{max} * \ell \,/\, |\delta|\,\mathrm{max} \qquad (2)$$

where θ is the rotational displacement, ℓ is the distance between X10 Wall and the center of gravity, and δ is the lateral displacement at X10 Wall. In the table, the values for all models are almost equal. This result indicates that the torsional displacement is not increased by the strength reduction.

CONCLUSIONS

The following remarks can be deduced from these analyses.

1) In a setback-type building, when consideration to torsional response behavior is lacking in the seismic design, the torsional influence appears significantly in the response and the damage distribution.
2) For controlling the the damage level in each frame uniform, the strength of each frame must be determined according to its torsional response properties. The distribution of elastic response shear distribution can be used to determine adequate strength distribution.
3) Under the strength distribution taking account of torsional response properties, requirement for the total strength can be reduced about 20% as compared with the current design value.

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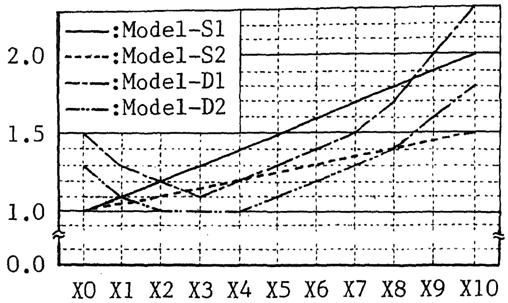

Fig.7 Distributions of Modification Factor

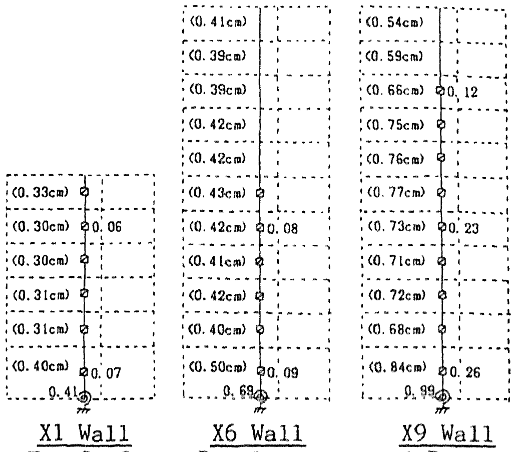

Fig.8 Story Displacement and Damage Distribution (Test-designed Building Model)

Table 2 Torsional Response Ratio in X10 Wall (Unit:%)

Story	Test-designed Build. Model	Model S1	Model S2	Model D1	Model D2
11	16.3	16.1	17.1	15.0	14.8
5	63.9	62.6	64.2	62.0	63.3
1	61.6	65.5	65.2	60.0	60.3

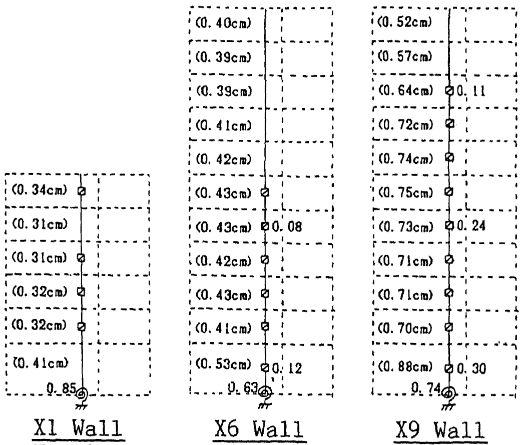

Fig.9 Story Displacement and Damage Distribution (Model-S1)

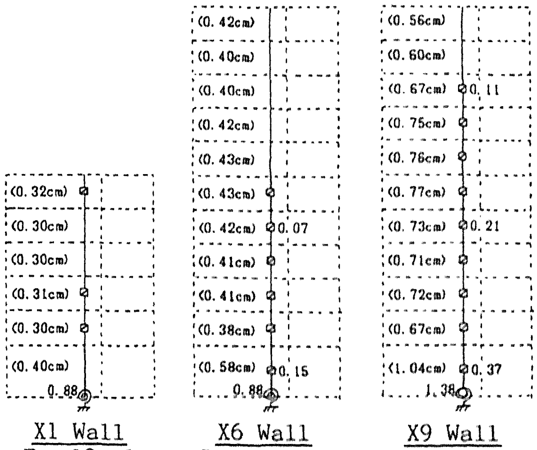

Fig.10 Story Displacement and Damage Distribution (Model-S2)

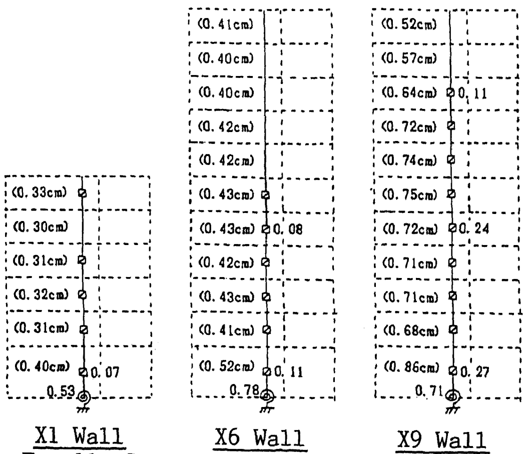

Fig.11 Story Displacement and Damage Distribution (Model-D1)

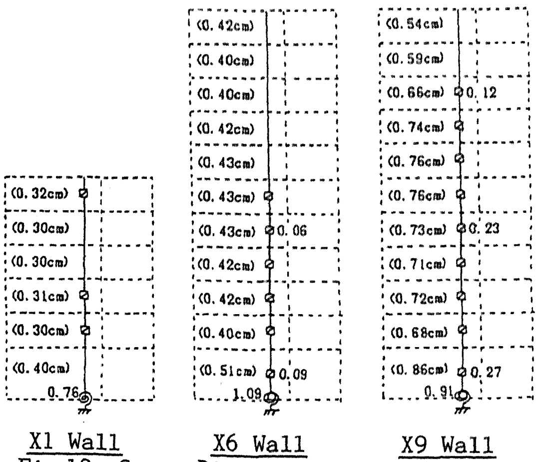

Fig.12 Story Displacement and Damage Distribution (Model-D2)