INELASTIC RESPONSE BEHAVIOR OF TORSION IN BUILDINGS SUBJECTED TO STRONG EARTHQUAKE

by

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SYNOPSIS

The effectiveness of maldistributed shear wall on the earthquake proof design were discussed through nonlinear response analyses, even though which causes the eccentricity of stiffness and induces the torsional vibration, from the viewpoint that the response displacement of the structure with maldistributed shear wall would decrease compared to the displacement of moment frame type structure without shear wall. The location of wall, ratio of sides, strength of frame are varied as parameters and the maximum peripheral displacement were mainly discussed to express the effectiveness of shear wall.

INTRODUCTION

Many damages of buildings with eccentric center of stiffness by the existence of maldistributed shear wall due to strong earthquake have been reported.

It is well known that for those buildings, there is a dynamic amplification of torque. The main concern of the building with torsion is that the eccentricity induces a rotational motion whose contribution to the displacement at the periphery causes an increasement displacement compared to the displacement corresponding to zero eccentricity.

The disadvantages of these maldistributed shear walls in the earthquake proof design have been very often indicated from the viewpoint of the increasement of displacement by this rotation mentioned above compared to the translational displacement of the center of mass.

This paper has discussed, on the contrary, through the inelastic response analyses, the effectiveness of the shear wall, when it locates in the reasonable position, even though which causes an eccentricity of stiffness, from the viewpoint that this shear wall will strengthen the stiffness and also strengthen the strength of whole structure so that the response displacement should be decreased compared to the response displacement of the moment resisting type structure without that shear wall.

Some researchers have also indicated the effectiveness of the maldistributed shear wall through the analyses for the damaged building by torsional vibration.

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These viewpoints imply that the aseismatic capacity of the structure with eccentric center of stiffness are able to be estimated from the aseismatic capacity of the moment frame type structure without shear wall by the function of eccentricity, ratio of sides etc.

If the relation between the response behaviors of moment frame type structure with zero eccentricity and the response behaviors of structure with shear wall in it were obtained, it would be available for the rough structural design of the structure with eccentric center of stiffness by shear wall to determine the reasonable location of wall or to determine the strength of frame to be possessed, and also it would be available for the approximate evaluation of structural safety of existing building which has maldistributed shear wall by the response displacement of the pure frame structure with zero eccentricity by neglecting the existence of the shear wall.

The response displacements of structures, consisting of moment resisting frames only, whose ratio of sides, stiffness and strength were varied by parameters, were compared to the response displacements of the structures which have maldistributed shear wall in the above-mentioned pure frame structures. The position of the shear wall in the plan of the structure and stiffness and strength of shear walls were varied. The maximum peripheral response displacements were mainly discussed.

EQUATION OF MOTION AND INPUT ACCELEROGRAMS

Structural models were assumed to have rigid floor diaphragm. The rigid diaphragm reduces the system to three degrees of freedom; two lateral displacements and a rotation about a vertical axis.

The dynamic equation of motion for the three degrees of non-linear system about the center of mass as shown in Figure-1, neglecting individual element torsional stiffness, is

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + \{F(U)\} = -[M]\{\ddot{U}_0\}$$

where

$$\{F(U_k)\} = \{F(U_{k-1})\} + [K_{k-1}]\{U_k - U_{k-1}\}$$

and $[K_k]$ is the tangent stiffness at time t_k,

The displacement vector $\{U\}$ is

$$\{U\} = \{U_X,\ U_Y\ \ U_\theta\}^T$$

U_X, U_Y : translational displacement of the center of mass
U_θ : rotation angle of the center of mass

The mass matrix then becomes

$$[M] = \begin{bmatrix} m & 0 & 0 \\ 0 & m & 0 \\ 0 & 0 & I_G \end{bmatrix}$$

m : mass
I_G : moment of inertia of mass

The displacement vector of each column can be obtained from the relation as,

$$\{u\}_{column} = \left\{ \sqrt{(U_X - U_\theta \cdot Y)^2 + (U_Y + U_\theta \cdot X)^2} \right\}$$

where X, Y distance from the center of mass to the column considered

The translational ground motions to X-direction were applied and the accelerograms of El-Centro NS 1940, Hachinohe NS, and EW (recorded at 1968 Tokachi-Oki Earthquake) were used in this analysis.

The maximum acceleration were modified to be 0.3g. The numerical integration method used is linear acceleration method and the time increment is 0.00125 sec. The maximum peripheral vector were discussed from the several points of view.

ANALYTICAL MODEL

In this report, moment resisting type structures with zero eccentricity were defined as uncoupled model, and structures with shear walls were defined as coupled model, respectively.

The structures are single story with two bays in transverse direction and two, three, four, five and twenty bays in longitudinal direction (whose ratio of sides are 1.0, 1.5, 2.0, 2.5, 20 respectively) as shown in Figure-2. The total stiffness and the total yield force in X direction are equal to those in Y direction in the uncoupled model.

Coupled model has a shear wall effective to X direction. It has the eccentricity alongside the Y-axis only as shown in Figure-1. The location of wall was varied as one of the parameters. The dimentions of column and girder or unit weight of structure were tabulated in Table-1.

The yield force of each column was calculated from the assumption that average cross sectional stress of column was 5 kg/cm^2, 7.5 kg/cm^2 and 10 kg/cm^2, when the yield hinges were formed at the top and the bottom of the column. The yield shear forces of each column are 8 tons, 12 tons and 16 tons respectively. This means that the yield shear coefficient K_y ($K_y = Q_y / W$, Q_y =yield force, W =total weight) of the moment frame type structure becomes about 0.3, 0.45, 0.6 respectively. The yield shear force of the shear force of the shear wall was calculated by assuming that the average cross sectional stress is 30 kg/cm^2.

The hysteresis model of column was assumed to be bi-linear for every direction where the stiffness after yield point was to be 10 percent of initial stiffness, and the yield function was assumed to be circle as shown in Figure-3. The hysteresis model for shear wall was elasto-plastic.

RESPONSE RESULT

The relation between the eccentric ratio in the vertical axis (E_y/L , E_y =eccentricity L =longitudinal length of structure) and the magnification factor of maximum displacement vector of peripheral column of coupled models to the maximum vector of moment frame type structure in the horizontal axis are shown in Figure-4.

In case that the ratio of sides of structure within about 1.5, eventhough the shear wall locates in the most outer frame, the maximum response is almost equal to the response of the moment frame type model taking off that shear wall from the coupled model. The ratio of sides becomes larger, the effect of torsional vibration becomes more clear, but when the eccentric ratio is smaller than one value, the magnification factor is also below 1.0.

This means that shear wall can reduce the response of structures compared to the response of moment frame type structure when this wall locates in reasonable position even though which causes an torsional vibration.

Figure-5 shows the same relationship as mentioned above to get the influence of the difference of the strength of frame. In this figure, ①, ②, ③ indicate the magnification factors whose yield shear coefficient of pure frame type structures is 0.3, 0.45, 0.6 respectively. There are very few difference by the difference of yield shear coefficient of frame.

CONCLUDING REMARKS

The brief summaries obtained from this research through inelastic response analyses for simple structural models are as follows:

1) Maldistributed shear wall which induces a torsional vibration by the eccentric center of stiffness will not be always infavorable in the earthquake proof design.

2) The maximum response displacement at periphery of structure whose ratio of sides are smaller than about 1.5, are almost equal to the response displacement of the moment frame type structure, even though the shear wall locates in the most outer frame.

3) The ratio of sides becomes larger, the response displacement at periphery gradually increases but when the eccentric ratio is smaller than one value, which depends on ratio of sides, the response displacement becomes smaller than that of moment frame type structure.

4) The response displacement of the structure which has maldistributed shear wall can be approximately obtained from the response displacement for the moment resisting type structure, by neglecting the existence of shear wall, by the function of ratio of sides and position of wall or eccentric distance.

5) It is possible to evaluate approximately the structural safety of existing building which has maldistributed shear wall by the response displacement to the pure frame structure of zero eccentricity by neglecting the existence of shear wall.

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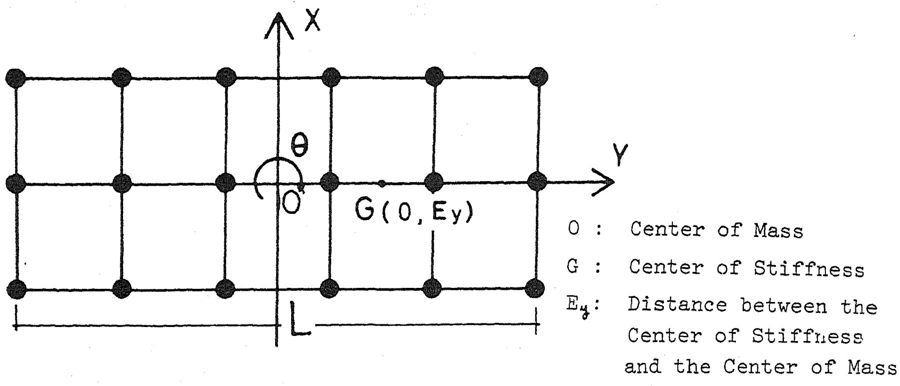


Figure-1 Coordinate of Equation of Motion

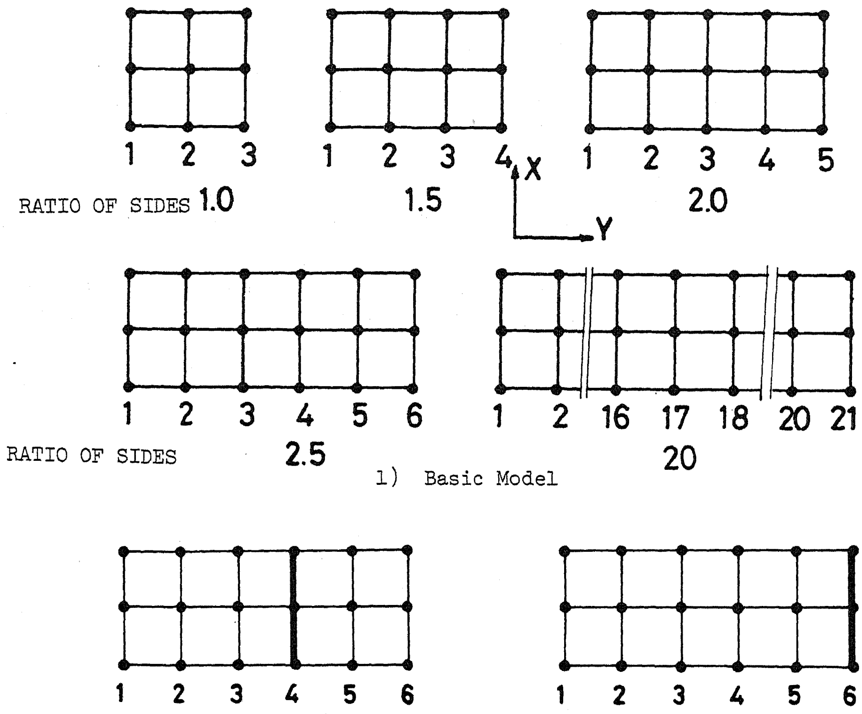
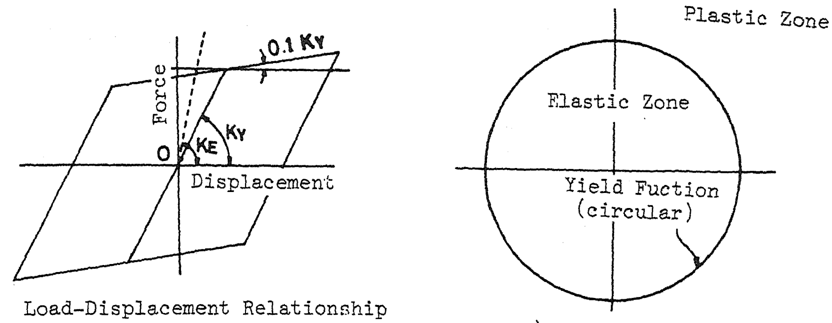


Figure-2 Analytical Structural Model



a) Load-Displacement Relationship

b) Yield Function

Figure-3 Hysteresis Model of Column

Table-1 Properties of Analitycal Model

Model	Eccent. Ratio	Natural Periods sec. 1 st	2 nd	3 rd	Model	Eccent. Ratio	Natural Period sec. 1 st	2 nd	3 rd
A-- 2	0.0	0.378	0.267	0.086	E-- 11	0.0	0.452	0.429	0.237
A-- 3	0.47	0.378	0.320	0.056	E-- 12	0.04	0.452	0.430	0.234
B-- 2.3	0.0	0.40]	0.302	0.104	E-- 13	0.07	0.452	0.431	0.227
B-- 3	0.16	0.401	0.315	0.094	E-- 14	0.11	0.452	0.434	0.216
B-- 4	0.47	0.401	0.359	0.060	E-- 15	0.15	0.452	0.436	0.204
C-- 3	0.0	0.414	0.328	0.119	E-- 16	0.18	0.452	0.439	0.190
C-- 4	0.23	0.414	0.354	0.095	E-- 17	0.22	0.452	0.441	0.177
C-- 5	0.46	0.414	0.382	0.066	E-- 18	0.25	0.452	0.442	0.164
D-- 3.4	0.0	0.423	0.347	0.132	E-- 19	0.29	0.452	0.444	0.153
D-- 4	0.09	0.423	0.353	0.126	E-- 20	0.33	0.452	0.445	0.142
D-- 5	0.27	0.423	0.379	0.097	E-- 21	0.36	0.452	0.446	0.133
D-- 6	0.45	0.423	0.397	0.072	A--pure		0.379	0.267	
					B--pure		0.401	0.302	
					C--pure		0.414	0.328	
					D--pure		0.423	0.347	
					E--pure		0.452	0.429	

note; D--4 : Model D, wall located in column line 4
B--2.3:Model B, wall located in between column line 2 and 3
column: 40cm x 40cm girder: 30cm x 60cm story height: 400 cm
each span: 600 cm equal unit weight: 1.5 ton

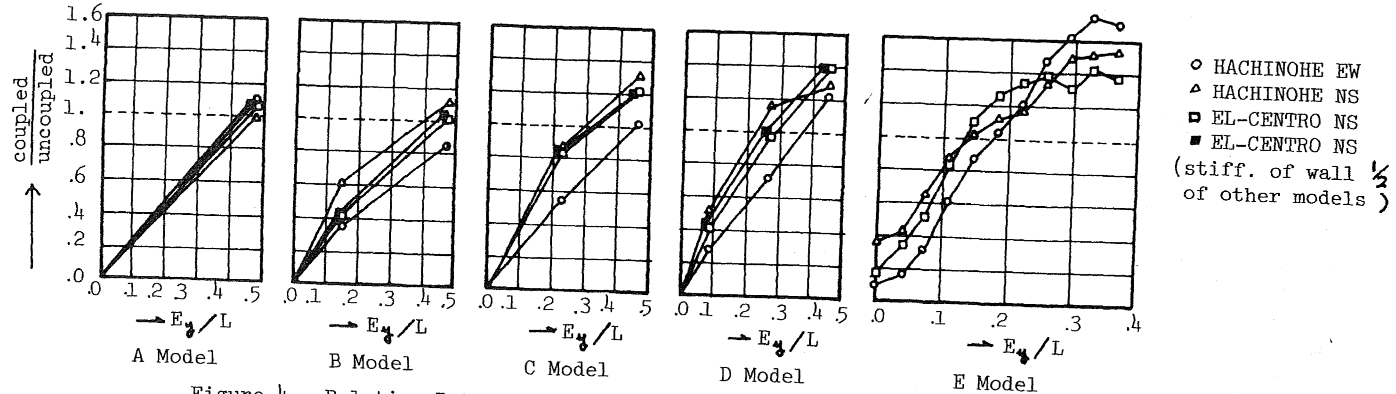


Figure-4 Relation Between the Eccentric Ratio and Magnification factor of Displacement to Displacement of the Uncoupled Model (for K =0.3 Models)

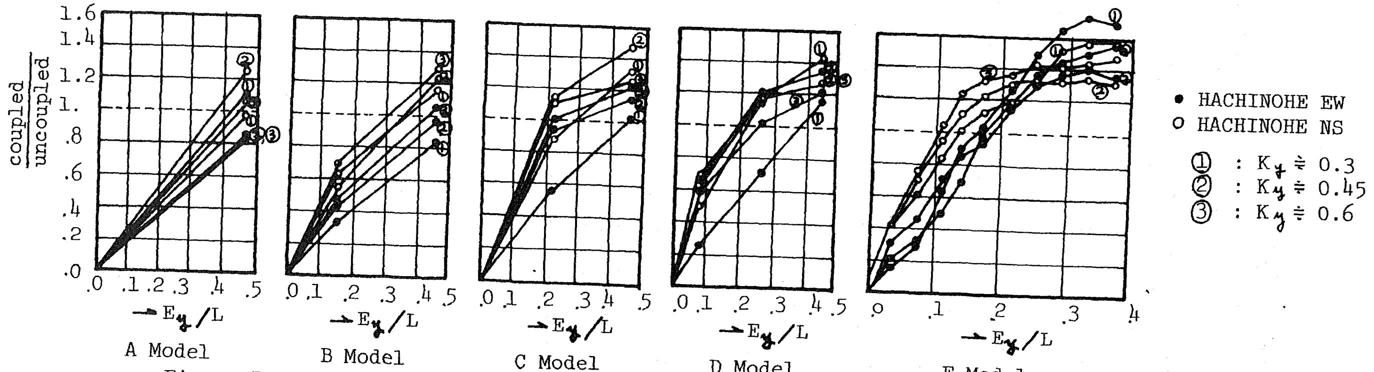


Figure-5 Relation Between the Eccentric Ratio and Magnification Factor of Displacement to Displacement of the Uncoupled Model (comparison by the difference of yield shear coefficient)