Torsional Behavior of Reinforced Concrete Multi-Story Building under Seismic Loading

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Abstract

Excessive torsional behavior of asymmetric building structures is observed to be the main cause of the poor seismic performance. Concepts of current design provisions for torsion are based on the assumption that the strength of the lateral load resisting elements can be adjusted without changing their stiffness. This paper investigates inelastic torsional effects of multi-story high-rise residential building in Korea on increase of strength demand and ductility of members using some methods published in literature. The methods analyze the reduction of strength and member ductility resulting from torsional mechanisms. This study shows that use of these concepts control inelastic torsion during preliminary seismic design of multi-story building of irregular plans.

1. Introduction

Torsional effects on asymmetric building require the increase of demanded strength, ductility, and stiffness. Most general consideration of torsional effects on building structures under earthquake loading is the consideration of eccentricity. In order to compute the design eccentricity, the center of mass shall be displaced from its calculated origin in each direction a distance of 5 percent of the building dimension (0.05b) perpendicular to the direction of acting force.

With respect to the New Zealand Building Code this principle can be rewritten in the following form:

$$e_d = \alpha e_s + \beta b$$
$$e_d = \alpha e_s - \beta b$$

where

$$\alpha = 1.0$$
$$\beta = 0.05b \text{ (AC=accidental eccentricity)}$$

The definition of these eccentricity modifiers has been extensively studied and is therefore not further addressed in this paper. As for preliminary design purposes, these modifiers may also be omitted and only the structural eccentricity may be used as it accounts for the coupled lateral-torsional effect.

Whenever there is torsional irregularity (section 1629.5.3 of the UBC 97), these effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor. The factor is defined as:

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$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}} \right]^2$$

where δ_{max}: maximum displacements at level I and δ_{avg} : average displacements at the extreme points at level i. This amplification factor will not be used in this paper.

There is a clear contradiction in the consideration of torsion effects during the design process because torsion effects are elastically considered, while most of the seismic code design methodology is based on the safety level EQ and, therefore, considerable inelastic behavior is expected.

This paper investigates the inelastic torsional effects of multi-story residential building on safety of current design practice in Korea.

2. Assumptions:

- walls are linked at each floor by an infinitely rigid diaphragm, which has no flexural stiffness and only translates/rotates in its plane
- walls are displaced by identical amounts at each floor and each wall shares an amount of the story force in proportion to its own stiffness
- the stiffness of the connection between floors and walls is neglected
- forces due to torsion of the members is calculated from the rotation of the walls

The bending moments and torsional forces can then be calculated from the shear forces on a simple cantilever system and by applying basic concepts of warping torsion. The rigidity/stiffness of the members can be computed using simple section calculations for uncracked elastic concrete. Stiffness degradation due to the cracking can be considered by applying formulas proposed by Paulay.

3. Typical Structural Planning of High-rise Residential Building in Korea

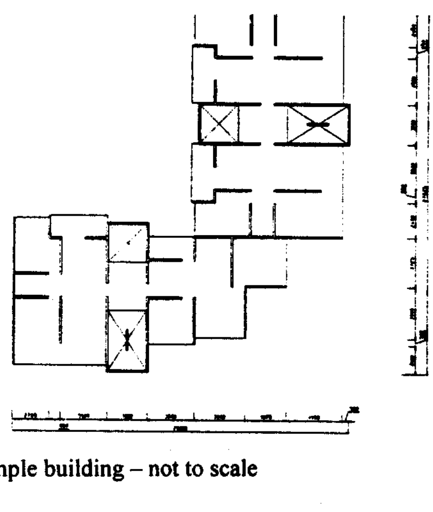

Fig. 1 Rough floor plan of sample building – not to scale

Table1. Dimensions of Walls

wall nr.	l_w [m]	h_w [m]	A_r	b_c/l_w	b_c [m]	Boundary element size
1/17	5.00	63.00	12.6	0.07	0.35	A_{wb} = 0.18 m^2 flange:b_1=20 cm b= 88 (100) cm

3/7/11/16	3.60	63.00	17.5	0.07	0.25	A_{wb} = 0.09 m^2 flange : b_1 =20 cm b= 45 (50 cm) piers : b_1=40cm b= 40 cm
6/14	5.70	63.00	11.1	0.07	0.40	A_{wb} = 0.23 m^2 pier: b_1=57 (60) cm b= 40 cm
8	4.00	63.00	14.3	0.07	0.28	A_{wb} = 0.11 m^2 flange: b_1=20 cm b= 56 (60) cm
9	5.00 3.80	63.00 63.00	12.6 16.6	0.07 0.07	0.35 0.35	A_{wb} = 0.18 m^2 flange:b_1=20 cm b= 88 (100) cm A_{wb} = 0.13 m^2 pier: b_1=40 b= 40 cm

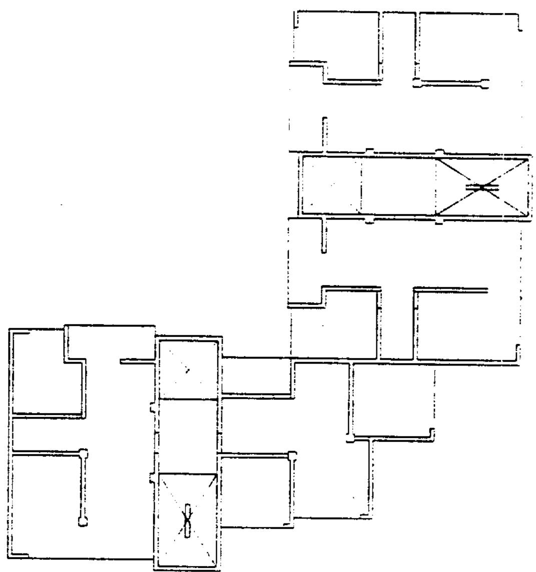

Fig. 2. Revised floor plan with flanges/piers

4. Strength Reduction due to Inelastic Torsion

New approaches to the analysis and design of torsionally weak buildings will be addressed. The paper by Bertero deals with a method that focuses on estimating the reduction in strength of members due to inelastic torsion. Its aim is to provide a simple way of determining "the suitability of structural layout, torsional redundancy, eccentricity, and relative strength among planes of strength with regard to inelastic torsion."

Member detailing and local seismic demand are not know during preliminary design phase and therefore a simplified global model of the 3D behavior will be adopted.

In the paper a few very simple buildings are analyzed using this method. Bertero also exhibits discrepancies between his method and the inelastic behavior obtained with computer numeric models of those buildings. Nevertheless, he comes to the conclusion that his method is an adequate procedure to obtain the reduction in strength for coaxially coeccentric elasic-perfectly plastic (CCEPP) buildings.

This concept will now be applied to the sample building. Some assumptions will have to be made to facilitate the calculations. In this case we may choose the building with or without coupling beams as no division of forces or deflections between coupled walls will become necessary as will be the case with the other approaches. The building may be considered CCEPP type.

1. Estimate the strengths

   At this point, we assume that the base yields correspond exactly with the forces obtained by hand analysis. This means that forces correspond with the stiffness distribution and no capacity design approach will be considered. Therefore we can use I_x and I_y instead of V^*_{xi} and V^*_{yi} for a first close estimate.

2. Center of strength
3. Estimate Accidental eccentricity
4. Total distance between centers
5. Earthquake attack in perpendicular direction
   α=0.3 will be used to account for earthquake forces in the perpendicular direction of 30% of the forces in the main direction.
6. Torsional lever arms

Wall No.	$x_i - x_R$ [m]	$y_i - y_R$ [m]	$I_x (x_i-x_R)$	$I_y (y_i-y_R)$
1'	13,71	7,44	571,64	85,65
2'	4,12	6,12	918,42	2224,37
3'	9,40	10,06	133,41	957,90
4'	8,92	17,11	102,69	713,52
Sum			1726,15	3981,44

With I_y = 511.84 m^4 and I_x = 290.24 m^4 , we find

$$\gamma = \frac{\sum I_y}{\sum I_x} = 1.76$$

Replacing V^*_{xi} and V^*_{yi} with I_x and I_y the formulas above may be rewritten as

$$j_x = \frac{\sum_{i=1}^{n} I_x |x_i - x_R|}{(1/2)\sum_{i=1}^{n} I_x} ; j_y = \frac{\sum_{i=1}^{n} I_y |y_i - y_R|}{(1/2)\sum_{i=1}^{n} I_y}$$

Without explicitly written calculations the torsional lever arm are

j_x = 11.89 m
j_y =15.56 m

7. Torsional Mechanism Indicator β

$$\beta = \frac{1}{2} \frac{j_x + \gamma j_y}{\frac{j_x}{2} + |x_R| + \alpha\left(\frac{j_y}{2} + |y_R|\right)} = 1.65$$

   Therefore no strength reduction is expected if all base shear capacities correspond with the stiffness distribution and no retrofit design of the floor layout will be necessary.

Certain irregularities in these calculations may exists due to a very rough coupling of the walls as presented here.
The method presented above may nevertheless be considered as appropriate for preliminary design of high-rise buildings with irregular floor plans. Although not presented above, the following formulas can be used to find appropriate stiffness distributions of walls if the torsional coupling indicator β < 1.2. Computer analysis may still be necessary for design purposes to verify that the floor plan layout can be considered less prone to plastic failure in wall members or even resistant earthquake.

5. Estimation of System and Element Ductility Demand

The concept presented in this paper focus on estimating the ductility demand of wall members in buildings subjected to earthquake loads. Torsionally restrained and torsionally unrestrained mechanisms are presented and guidelines are given on how to allocate strength to elements to prevent a collapse in structures.

Torsionally unrestrained or limited torsionally-restrained buildings have no elements in the transverse direction of the earthquake attack and therefore can not resist the induced torque other than by yielding in the elements in the plane of the attack. Since the sample building presented in this paper has a large number of elements in the transverse direction for attacks in x- or y-direction to withstand torque, we can consider it torsionally-restrained.

A very useful tool could be a simple computer program based on the concepts presented above that only needs the rough floor plan of the building and then calculates the ductility demand, degree of translation-torsion coupling and indicates critical members which could lead to a collapse of the building. Sound engineering expertise can of course replace those tools but a simple computer print out would at least be useful to cut down the time on conducting the retrofit design in general.

To estimate the variable displacement demands Δy_i of each element for the torsionally restrained mechanism, we first have to predict the angle of twist of the entire structure. Paulay suggests that the residual torsional stiffness may be only considered provided that the floor diaphragm is indefinitely rigid.

$$k_i = \frac{V_{ni}}{\Delta_{yi}}$$

where

$$\Delta_{yi} = \frac{\phi_{yi} h_{wi}^2}{\eta}$$

h_{wi} : height of wall/member
ϕ_{yi} : yield curvature
η : coefficient for type of loading (e.g. inverted triangle, point load)
V_{ni} : nominal shear strength of member

The yield curvature may be taken from

$$l_w \phi_{yi} = 2\varepsilon_y$$

where

l_w : length of wall in floor plan
ε_y : yield strain of steel used, equal to 0.002 for Grade 60 steel

In order to facilitate the computations, we may simplify the sample building with the new properties as follows:

Note that some walls have been deliberately omitted as a result from previous rigidity/force distributions. The rigidity of those walls can be considered minor and they have been neglected therefore.
)
Table 2. Properties of Walls

Wall No.	1	2	3	4	5	6	7	Σ
x_i [m]	-14.00	-5.50	-5.50	5.50	5.50	5.50	5.50	
y_i [m]	-5.50	-5.50	-5.50	0.00	7.50	7.50	16.00	
x_{ri} [m]	-9.00	-0.50	-0.50	10.50	10.50	10.50	10.50	
y_{ri} [m]	-8.50	-8.50	-8.50	-3.00	4.50	4.50	13.00	
I_x [m^4]	24.00	20.00	0.00	0.00	0.00	20.00	0.00	64.00
I_y [m^4]	0.00	0.00	20.00	20.00	20.00	0.00	24.00	84.00
l_w [m]	11.00	11.00	3.00	11.00	11.00	3.00	11.00	
h_w [m]	63.00	63.00	63.00	63.00	63.00	63.00	63.00	
ϕ_{yi} [1/m]	0.00036	0.00036	0.0013	0.00036	0.00036	0.0013	0.00036	
Δ_{yi} [m]	0.48	0.48	1.72	0.48	0.48	1.72	0.48	
V_x [kN]	0	0	2000	3000	2000	0	3000	10000

V_y [kN]	3000	2000	0	0	0	2000	0	7000
k_i [kN/m]	6250	4167	1163	6250	4167	1163	6250	

$\eta = 3$ for an equivalent single load at h=0.7H , see [P1]
V_x/V_y were taken from a separate EXCEL spreadsheet incl. on CD-ROM in the Appendix, and rounded up to the next full amount to account for 10-20% strength surplus over the calculated forces. The angle of twist for both principal directions of earthquake attacks are calculated as

$$\theta_{tu} = \frac{e_{vy}\sum V_{ni}}{\sum x_{ri}^2 k_{yi}}$$

$$= 3 \times 8000/[(-9)^2 \times 6250 + (-0.5)^2 \times 4167 \quad + 10.5^2 \times 1163)]$$

$$= 0.037 \text{ [1/m]}$$

$$\theta_{tu} = \frac{e_{vx}\sum V_{ni}}{\sum y_{ri}^2 k_{xi}}$$

$$= 5 \times 7000/[(-3)^2 \times 6250 + 4.5^2 \times 4167 + 13^2 \times 6250)]$$

$$= -0.029 \text{ [1/m]}$$

With the angle of twist determined, we may now compute the lateral displacement of all members on top of each wall and consequently find the maximum ductility capacity.

This will be presented only for the far most remote wall from the center of rigidity CR, that is wall #7 for an earthquake attack in y-direction:

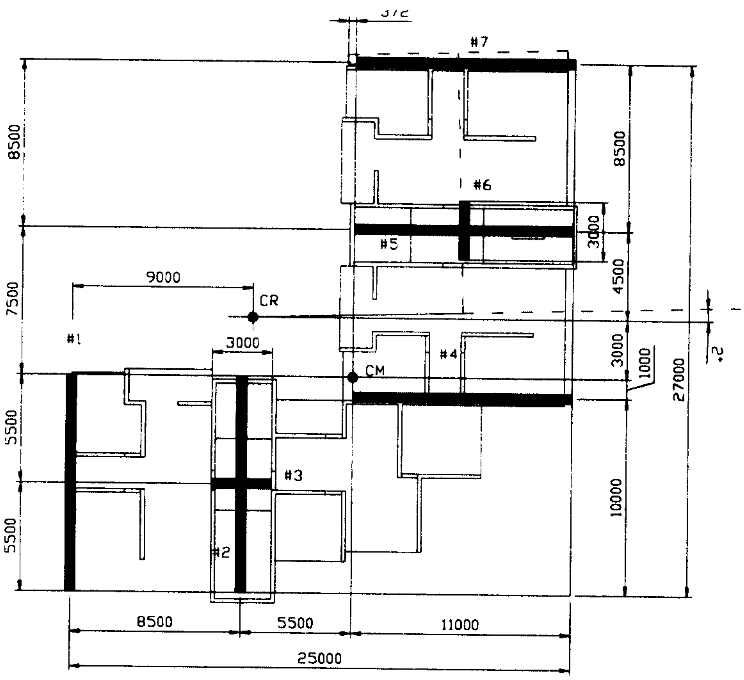

Fig. 3 Displacement of wall #7

The ultimate displacement capacity of wall #7 is $\Delta_u = \mu\Delta_{y,u}$. With μ=5.0 and Δ_y =0.48 from Tab. 2 we find

$$\Delta_u = \mu \Delta_y = 5 \times 0.48 = 2.4 \text{ m}$$

From Fig. 3 we take Δ_y = 0.37 m. Therefore the element ductility demand is

$$\mu = \Delta_u/\Delta_y = 2.4/0.37 = 6.5$$

Hence, wall #7 or wall #17 as indicated in the original floor plan of the sample building should be designed for μ =6.5 which means that special detailing provisions should be considered.

To undermine this the center of rigidity CR should be deliberately positioned closer to the center of mass CM as proposed for earthquake resistant buildings in general.

In his paper [P1], Paulay also suggests several ways to quantify the implications of skew seismic attacks. This will not be followed any further here, but should be generally investigated as well. This is due to the fact that a torsionally restrained building may become a torsionally unrestrained as all members yield and the ultimate ductility demand of the members will be exceeded.

6. Evaluation

The approaches above to design torsionally weak structures show that with rather simple calculations the displacement behavior of the structure or the need for a retrofit based on strength reduction in critical members can be estimated.
No full elastic or inelastic dynamic analysis of the building was necessary and it is the author's belief that the methods presented above provide a good means to further analyze the structure in the preliminary design phase and indicate critical members in the floor plan. Torsionally weak structures can by means of retrofit in terms of redistribution of member strength be made torsionally restrained or even earthquake safe.
In addition numeric computer analysis can be also cross-checked by the means above.

The only problem that may arise with such complex type of buildings as established in this paper is that in order to find a close prediction of the behavior of the building, the member strength in all directions must be rather adequately estimated or even determined prior to the application. In the case of the sample building this is a far more difficult task as all walls have member strength in both principal directions.

Therefore the computations presented above should be re-performed with different variations of member strength in order to determine the influence of overstrength. This can of course be neglected when designing only for the prevailing member forces thus risking plastic hinges in those members if the earthquake attack is in the transverse direction of the one for which those members have been detailed.

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