TORSIONAL EFFECTS AND REGULARITY CONDITIONS IN RC BUILDINGS

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SUMMARY

Earthquake damages at the perimeter of buildings are often the result of excessive deformations caused by torsion during the earthquake. Large torsional responses can be expected if the buildings have large eccentricity and low torsional stiffness. Modern codes, including Eurocode 8, recognise the importance of the torsional stiffness on the seismic response and include such information in its static torsional provisions. Eurocode 8 states that the static torsional provisions can be applied to torsionally unbalanced buildings with regularity in elevation that satisfies the set of conditions presented in the Annex B of the Code. Present paper shows an overview on the EC8 torsional provisions including the code background and the comparison with other codes (UBC, etc.). On the other side the EC8 provisions could be improved during the revision process. In fact recent studies on the static torsional provisions of Eurocode 8 show that the static torsional provisions generally are satisfactory in order to limit additional demands when the building has a large torsional stiffness, but the provision seems to be deficient when applied to torsional flexible models. Moreover, the behaviour of buildings designed in High Class of ductility (“HD”) is more complex. This paper compares most of the results existing in the literature, suggests proposals of modification and underlines the importance of further studies in order to evaluate a condition of minimum torsional stiffness.

INTRODUCTION

The observations of buildings subjected to strong earthquakes showed that the excessive torsional response is one of the most important factors, which produces severe damage for the structures, even the collapse. The excessive torsional response is often due to the structural asymmetries codified, in most international rules, by a structural classification which characterises regular and irregular structures, in order to compulse the designer to use different methods for the structural analysis.

The general rules provided by modern codes should lead to the following hierarchy of analysis methods:

- planar static linear analysis sure in comparison with spatial static linear analysis;
- static linear analysis sure in comparison with dynamic linear analysis;
- linear analysis sure in comparison with non-linear analysis;

These goals should be obtained by means of simple code provisions so that in design offices they can be easily used.

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For irregular buildings, which not satisfy the criteria for regularity provided by codes, the structural analysis is based on a multi-modal dynamic analysis. For regular buildings a simplified procedure could be used, based on the application of equivalent static lateral forces in the mass center, with an additional planar eccentricity in order to be account an additional torsional effect at any story.

Therefore the structural regularity in plan is strictly related with the torsional behaviour. In fact the Uniform Building Code (1997) characterises the torsional regularity as the main plan regularity criterion. Its definition is based on the structural property to have, when diaphragms are not flexible, the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis less than 1.2 times the average of the story drifts of the two ends of the structure.

Also the Eurocode 8 (1994) provides as criterion for regularity in plan that the maximum displacement in the direction of the seismic forces, applied with the accidental eccentricity, is smaller than 20 percent of the average storey displacement. However the current code criteria regarding the regularity conditions and the torsional provisions are under discussion and different modifications are proposed. In fact the regularity conditions, effective for an elastic analysis and for a restricted class of buildings, appear not always appropriate to describe the torsional response, as recent researches proved.

2. CODE PROVISIONS FOR THE TORSIONAL EFFECTS

The methods proposed by current codes for the analysis of the structural effect due to the torsional actions are different. In the following the rules provided by UBC [1997] and Eurocode 8 [1994] are briefly analyzed.

Both UBC and EC8 codes account accidental torsional effects assuming that the mass is displaced from the calculated center of mass, in each direction, a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force considered L_i.

In the UBC, when the static lateral-force procedure is applied, the design eccentricity e_d is, for regular buildings, equal to:

$$e_d = e_o + 0.05\, L_i \tag{1}$$

defining with e_o as the eccentricity between the mass center MC and the stiffness center SC.

While for irregular buildings e_d is equal to:

$$e_d = e_o + 0.05 A_x\, L_i \tag{2}$$

where

$$A_x = \left[\frac{\delta_{max}}{1.2 \cdot \delta_{avg}} \right]^2 \tag{3}$$

while δ_{max} is the maximum displacement at level x and δ_{avg} is the average of the displacements at the extreme points of the structure at level x. The value of A_x must not exceed 3.0.

When the dynamic analysis is applied, in a spatial model the accidental effect can be taken into account using an appropriate modification of MC or adding the torsional static effects given by the only accidental eccentricity to the results of dynamic analysis.

In the Eurocode 8, when the simplified modal response spectrum analysis can be applied, in a planar model the accidental torsional effects are considered by amplifying the action effects in the individual load resisting elements, F_i, with a factor δ given by:

$$\delta = 1 + 0.6 \cdot \frac{x}{L_e} \tag{4}$$

where x is the distance of the element considered from the centre of the building measured perpendicularly to the direction of the seismic action under consideration and L_e is the distance between the two outermost lateral load resisting elements measured as previously.

In a 3D model, in addition to an eccentricity e_o between MC and SC and an accidental eccentricity of storey mass e_1 equal to 5 percent of planar dimension in the direction perpendicular to the direction of application of lateral force, an additional eccentricity e_2 is considered to account the dynamics effects as follows:

$$e_2 = \min\left\{\left[0.1(L+B)\sqrt{10\frac{e_o}{L}} \leq 0.1(L+B)\right];\left[\frac{1}{2 \cdot e_0}\left(l_s^2 - e_o^2 - r^2 + \sqrt{\left(l_s^2 + e_o^2 - r^2\right)^2 + 4e_o^2 r^2}\right)\right]\right\} \tag{5}$$

where $l_s^2 = (L^2 + B^2)/12$ is the square of radius of gyration, r^2 is the ratio of the storey torsional and lateral stiffness, L and B are the planar dimensions of the building and e_0 is the actual eccentricity between the stiffness center SC and the nominal mass center MC.

The additional eccentricity e_2 can be neglected if r, ratio of the storey torsional and lateral stiffness (square of "torsional radius"), is sufficient (i.e. $r^2 > 5 \cdot (l_s^2 + e_0^2)$).

When the multi-modal response spectrum analysis is applied in a spatial model, the dynamic effects due to eccentricity e_o are implicitly taken into account. Instead the accidental torsional effects may be determined as the envelope of the effects resulting from an analysis for static loading, consisting of torsional moments M_{1i} about the vertical axis of each storey i.

For buildings complying with the criteria for regularity in plan or with the regularity criteria given in clause A1 of Annex A the analysis can be performed using two planar models, one for each main direction.

3. PARAMETERS OF THE ELASTIC RESPONSE

For codes scopes, it is essential the use parameters of simple evaluation. Unfortunately the torsional response of the structures depends on many elastic and inelastic parameters.

A key elastic parameter is the ratio Ω of the uncoupled torsional frequency ω_θ to the uncoupled lateral frequency ω_h:

$$\Omega = \frac{\omega_\theta}{\omega_h} \tag{6}$$

If Ω is greater than 1 the response is mainly translational and the structure is defined as torsionally stiff; conversely, if Ω is lower than 1 the response is affected to a large degree by torsional behaviour and, then, the structure is defined as torsionally flexible.

Usually, torsionally stiff structures show simpler seismic behaviour than the torsionally flexible ones.

An alternative manner to express the ratio Ω is as follow:

$$\Omega = \frac{\omega_\theta}{\omega_h} = \frac{\rho_K}{\rho_M} \tag{7}$$

where ρ_K is the normalised inertia radius of the stiffness distribution (ρ_K =r/B) and ρ_M (ρ_M =l_s/B) is the normalised inertia radius of the mass distribution, computed with respect to the center of stiffness (CR) and to the center of masses (CM) respectively. Therefore, if the ratio Ω is known (through dynamic analysis), it is possible to evaluate ρ_K given the value of the mass inertia. This approach is generally used in the evaluation of ρ_K for multi-storey buildings.

In the practice, the mass inertia radius ρ_M is often obtained by assuming a mass uniform distribution and, then, in case of buildings with rectangular plan, ρ_M is given by:

$$\rho_M=[(L^2+B^2)/12]^{0.5}/B \tag{8}$$

where L and B are the two plan dimensions.

For a building with plan dimension ratio B/L equal to 2.5 the mass inertia radius is equal to:

$$\rho_M=[(L^2+B^2)/12]^{0.5}/B =0.31 \tag{9}$$

In this case, the value of the stiffness radius ρ_K corresponding to Ω=1 (i.e. the threshold value separating torsionally stiff from torsionally flexible structures) is obviously equal to:

$$\rho_K=0.31 \tag{10}$$

This result is of importance, considering that most studies have been referring to structures with rectangular plan and plan dimension ratio B/L equal about to 2.5.

The evaluation of ρ_M is very simple; in order to evaluate Ω and the torsional behaviour, the evaluation of ρ_K becomes essential.

An approximate manner to obtain ρ_K, which is definitely more suitable in the design practise, is to compute it as radius of the distribution of the cross section inertia of the vertical resisting elements. This procedure leads to "exact" results, i.e. compared with elastic dynamic analyses, in the case of framed buildings with very stiff beams and in the case of buildings with shear walls characterised by constant stiffness in elevation. In other cases the evaluation is more or less approximate.

Another fundamental parameter of the torsional response in the elastic range of behaviour is the well known stiffness (or static) eccentricity e, i.e. the distance between center of mass and center of stiffness.

The evaluation of the stiffness centre is another complex problem for multi-storey buildings, where a clear definition is not possible; again, the definition of the center by means of the distribution of the cross section inertia of the vertical resisting elements is exact in the cases just defined for ρ_K.

In recent years many studies have considered another important parameter, i.e. the ratio Δ of the lateral displacement at the stiff side δ_{min} to that at the flexible side δ_{max}, as a suitable measure of structure sensitivity to torsional behaviour. Namely, it can be seen (Tso and Wong,1995) that the above-mentioned ratio Δ is related to both the stiffness eccentricity and to the stiffness radius trough:

$$\Delta = 1-\left(\frac{e/B}{\rho_K^2}\right)\left\{1+\left(\frac{e/B}{\rho_K^2}\right)\cdot(0.5+\eta)\right\}^{-1} \tag{11}$$

where ηB is the distance between the stiffness center and the geometric center.

From the previous formula it is possible to obtain a further estimate of ρ_K. The advantage of this evaluation is that is based on simple evaluation for the practitioners.

Therefore two different approaches can be introduced in the Codes about the influence of stiffness distribution on the torsional response: rules based on the concept of torsional radius r or rules related to the displacements distribution, as the deviation of maximum story displacement respect to the average one.

The first approach gives the benefit to be based on meaningful mechanic parameters, but its disadvantage is to have a not unequivocal application to multi-story structures.

The second approach is simple and clearly useful for multi-storey buildings, referring to interstory drift.

The current version of EC8 is not clear in the choice of the approach; in fact it uses the second approach in the Part 1-2 and the first approach in Annex A.

4. A CRITICAL ANALYSIS OF INELASTIC RESPONSE

The inelastic torsional behaviour of the structures has been deeply investigated in the last years. Many researchers have developed different non-linear analyses of asymmetric buildings using different procedures. However these numerical results have shown that the non-linear response, in addition to the parameters of the elastic response, is also dependent on the strength distribution, and therefore on the adopted seismic code, on the code over-strength, and to a large extent on the input ground motion.

It should be also notice that non linear response is generally dependent on the assumed hysteretic behaviour; however, examination of this last aspect is beyond the scope of this paper.

In the following, the results obtained from several studies have been grouped according to the model considered in the analyses and critically analysed. Results for one-story models are reported in Figure 1 and those for multi-story building models in Figure 2. For the sake of simplicity only the non-dimensional stiffness eccentricity (e/B) and the non-dimensional stiffness radius (ρ_K) have been considered to characterise each analysed model thus neglecting the effect of other geometrical and mechanical parameters.

The meaning of the markers in Figures 1 and 2 is the following:

- white markers represent cases of good behaviour, i.e. the assumed damage parameter (ductility demand, interstorey drift, etc.) obtained for the asymmetric system is comparable to that of its TBS (torsionally balanced system), that is characterised by the coincidence between center of stiffness CR and center of mass CM;

- green markers represent cases of satisfactory behaviour, i.e. response of the asymmetric structures is characterised by a slight increase in damage with respect to their the TBS (as an example, increase of 10-20% in ductility demands);

- red markers represent cases of poor inelastic response of the asymmetric structures (as an example, ductility demands larger than 50% compared to their TBS).

When discussing the whole variability of the results, drawn in fig 1, regarding the one-story buildings, it is necessary to observe that:

- the obtained results are dependent on the code provisions adopted for strength design (as an example results obtained by De Stefano et al., 1996 and those by Tso and Zhu, 1992). Namely, according to Tso and Zhu (1992), systems having the same elastic parameters performed poorly when designed according to the New Zealand Code (NZS 4203, 1984) or to the Uniform Building Code (UBC, 1988), whereas they performed well when strength distribution was defined based on torsional provisions of the Canadian Code (National Building Code of Canada, 1990). De Stefano et al. (1996) have shown that the inelastic response of asymmetric systems designed according to UBC 1994 torsional provisions is definitely better than that of EC8 (1994) designed systems. For this reason, in case of ρ_K equal about 0.3 results obtained by De Stefano et al. are represented with a yellow marker since the corresponding marker (white and red) would overlap on the graph of Figure 1;

- results from non linear analysis are affected by the input ground motion; therefore, in most studies average values of response parameters obtained from dynamic analyses under several earthquake records have been considered;

- non-dimensional stiffness eccentricity larger than 0.3 are uncommon and refer to building structures very irregular for which static analysis should be definitely avoided as design tool;

- stiff side of buildings is usually the critical one, since provisions of many codes under-design it and then large ductility demands activate. On the contrary, flexible side is usually well protected by code provisions, though increase in lateral displacements arise during excitations. This last behaviour may lead to some concern regarding damage in the non-structural elements;

- it appears suitable designing flexible side strengths based on current code torsional provisions, while elements placed at the stiff side should be designed as if they located in a TBS counterpart;

- as general remark, the level of inelasticity strongly affects the torsional response in the system and the elastic response can be strongly modified in the non-linear field.

The following conclusions can be obtained:

1. the old provision subscribed by the Eurocode 8 (1989) to limit the applicability of static analysis, i.e. the ratio of the stiffness eccentricity to the stiffness radius must be less than 0.15, appears too conservative. In fact, both from Figure 1 and from Figure 2 it can be seen that many cases of good and satisfactory behaviour are found on the right side of the red straight line representing the relation $\bar{e}/\rho_K$=0.15 (with $\bar{e}$ equal to e/B). Namely, according to the above-mentioned EC8 provision, for systems having e/B>0.1 the static analysis can be applied only if the non-dimensional stiffness radius is very large, exceeding the value of 0.5 which is an upper bound value for systems having lateral-load resisting elements aligned along one direction only (this threshold value is reported with dashed horizontal lines in Figures 1 and 2). Values of ρ_K larger than 0.5 can be obtained for systems having resisting elements acting along two orthogonal directions;

2. considering both the limitations: ρ_K>0.3 and e/B<0.3, all the analysed buildings show an acceptable torsional behaviour.

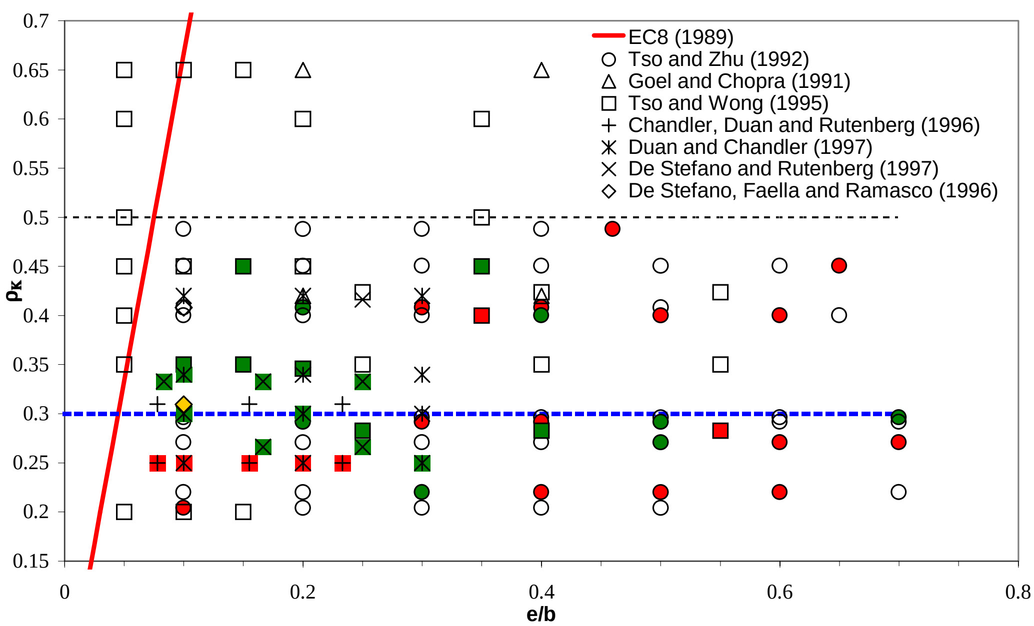

Figure 1: Results for one-story models

In figure 2 the results on multi-story building are shown. There are few analyses that have usually considered displacement parameters, such as top story drifts and interstory drifts, as damage indicators. Among the selected studies only one [Duan and Chandler, 1993] refers to ductility demands, demonstrating that - in particular in case of large eccentricity building - response is significantly dependent on the adopted seismic code. Namely, poor inelastic performances has been found for building designed according to the EC8 (1989) and to the New Zealand Code (1992), whereas good behaviour characterised the response of UBC (1988) buildings. These cases, characterised by the same elastic parameters, are represented with yellow square markers since the corresponding markers (white and red) would overlap on the graph of Figure 2.

Results from De Stefano et al. (1995) refer to square plan buildings (while in the other cases rectangular plan buildings have been considered); for such structures the threshold value Ω=1 corresponds to ρ_K=0.4. The selected response parameters are interstory drifts and top displacements that are seen to be strongly dependent on the input ground motion.

Harasimowicz and Goel (1998) have studied 9-story shear wall buildings with stiffness eccentricity varying from one floor to the other, indicated with a series of markers aligned on two horizontal lines, covering the entire range of floor stiffness eccentricity. It has been found that static analysis leads to underestimate lateral forces on shear walls located at the stiff edge of torsionally flexible buildings.

Tso and Moghadam (1998) have analysed four 7-story buildings with constant e/B=0.1 and different plan location of resisting elements, leading to four values of ρ_K . The comparison is carried out with reference to the interstory drifts obtained from static analysis (force distribution given by EC8) and those obtained from non linear dynamic analysis (values obtained averaging results from analyses under 10 accelerograms). The analyses, conducted by using the computer code " CANNY", have shown that static analysis correlates the results from dynamic analysis in a suitable manner only for torsionally stiff systems (ρ_K>0.3).

It is very important for the application to note that, also in the case of multi-storey buildings, the limitations ρ_K>0.3 and e/B<0.3 are sufficient to provide an acceptable behaviour.

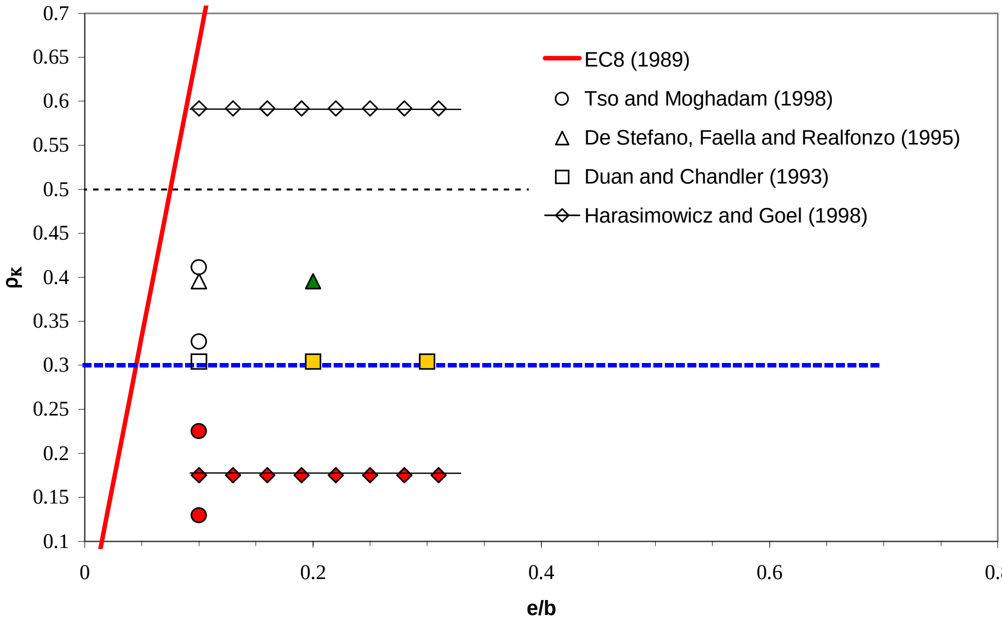

Figure 2: Results for multi-story building models

5. CONCLUSIONS AND RESEARCH NEEDS

The discussion of results from the selected studies suggests the following remarks both for one-storey and multi-storey buildings:

- for values of e/B>0.1 the old clauses of EC8 allows the application of the static analysis only for structures characterised by very large values of the stiffness radius and therefore they are believed to be over-conservative;

- systems having ρ_K greater than 0.3 and e/B less than 0.3 usually show good or satisfactory inelastic torsional response.

Further studies are necessary to evaluate the general effectiveness of these conclusions. In fact the majority of the researchers have analysed buildings with rectangular plan and ratio between the plan dimensions close to 2.5 and others researches are needed to validate these results varying the plan shape. Furthermore the studies on the torsional response of multi-storey buildings are very few and more analyses are necessary.

Another matter of discussion is the influence of both resistance and ductility demand distribution that, in particular for high ductility structures, can strongly affect the torsional response. This problem was recently studied by the TG8 of EAEE and in New Zealand [Paulay, 1997] but further researches are needed. However, the TG8 studies have provided some recommendations that could be included in seismic codes.

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