Inelastic torsional behaviour of structures designed to EC8

A.M. Chandler\textsuperscript{a}, G.L. Hutchinson\textsuperscript{b}, X.N. Duan\textsuperscript{c}

\textsuperscript{a}Department of Civil and Environmental Engineering, University College London, Gower Street, London WC1E 6BT, UK
\textsuperscript{b}Department of Civil and Environmental Engineering, The University of Melbourne, Parkville, Victoria 3052, Australia
\textsuperscript{c}School of Civil and Structural Engineering, Nanyang Technological University, 2263 Singapore

ABSTRACT

Some typical examples of the elastic torsional response of buildings designed to Eurocode EC8 are presented, and the inelastic response of these structures to certain characteristic strong-motion earthquakes is then investigated. The non-linear hysteretic behaviour caused by yielding of the resisting elements in asymmetric buildings influences the torsionally coupled response and triggers behaviour different from that predicted by linear elastic theory. As current code analysis procedures for seismic design, including most major building codes around the world, are based largely on linear (or equivalent linear) elastic theory, it is important to assess the effect of such changes in response behaviour on the design efficiency. The results presented in this paper are used as a basis for comment on the question of extrapolating elastic torsional design procedures to the inelastic range, and to assist in identifying the limitations of this simplistic approach.

INTRODUCTION

It is widely accepted that the two main objectives of earthquake-resistant design are to ensure safety of life and to protect property. Recognising
these objectives, the present design philosophy adopted by codes requires that buildings in seismically-prone regions be designed to resist a major earthquake without collapse or failure (termed ultimate limit state design) and to resist moderate earthquakes without structural damage (serviceability or damageability limit state design). For asymmetric buildings (termed more generally torsionally unbalanced buildings), these objectives require firstly that in serviceability limit state design, element strengths are specified which are sufficiently high to ensure elastic behaviour when the building is subjected to relatively small ground motions, hence limiting deformations and consequent damage. Secondly, in satisfying ultimate limit state design criteria it is expected that no significant additional ductility demand should arise in the earthquake load-resisting elements (columns and/or walls), compared with corresponding symmetric or torsionally balanced systems responding only in translation.

To satisfy the above criteria, seismic building codes include torsional design provisions. It is evident when reviewing such provisions that current earthquake-resistant design for torsional effects, whether based on the equivalent static force procedure or the modal analysis procedure, relies largely on linear (or equivalent linear) elastic theory. In general, a non-linear inelastic analysis is not required by present code regulations, even for buildings with torsional irregularities, although it may be recommended in some special cases. This paper addresses the question of the influence that non-linear hysteretic behaviour due to yielding of the resisting structural elements has on the torsional effect in asymmetric buildings. Focus is placed on the ECS torsional provisions [1], as an example of the extrapolation of procedures based largely on elastic design criteria to the inelastic design of asymmetric buildings.

The base shear and torsional provisions of EC8 (which is presently in final draft form) have been adhered to. Also, localised strong-motion earthquake records having varying frequency contents have been used as the ground motion input for the inelastic studies. The main objectives are firstly to assess whether or not the EC8 code torsional provisions provide uniform protection to resisting elements situated on the flexible and stiff sides of stiffness-asymmetric structures. Secondly, a study is made to determine whether or not they provide consistent control over inelastic structural response (and hence damage) for symmetric and asymmetric buildings. Thirdly, limits on horizontal irregularity are assessed, in order to determine design criteria appropriate to the elastic and inelastic ranges of response; these limits have been compared with the existing code regularity requirement.
EQUIVALENT STATIC TORSIONAL DESIGN PROVISIONS

In the static force procedure, the design seismic shear force, \( V_b \) [Figure 1(a)] is obtained from the appropriate design spectrum, using the estimated fundamental lateral period, \( T_y \). Most codes specify a lateral shear force combined with a torsional moment at each storey level. The design moment is the product of the storey shear force and the appropriate design eccentricity, \( e_D \), as illustrated in Figure 1(a) for the case of an idealised single storey stiffness-asymmetric building with a rigid floor diaphragm, supported by three lateral load-resisting elements. Seismic codes specify two design cases based on the primary, \( e_{D1} \), and the secondary, \( e_{D2} \), design eccentricities, respectively. These design eccentricities are intended to account for the increased or the decreased strength demand in each resisting structural element, arising due to torsion, the more unfavourable loading effect for the element under consideration being adopted.

**EC8 torsional provisions**

The primary and secondary design eccentricity expressions of EC8 are based on the German seismic code DIN 4149, and their analytical basis has been given in [2]. The allowance for the amplifying effects of lateral-torsional coupling in the elastic range of response, on which the supplementary eccentricity \( e_t \) in equation (1a) below is based, arises directly from analytical studies using the linear elastic modal analysis procedure. The EC8 primary and secondary design eccentricities can be expressed as:

\[
\begin{align*}
  e_{D1} & = [1.0e_s + e_t] + 0.05b \\
  e_{D2} & = 1.0e_s - 0.05b
\end{align*}
\]  

where \( e_s \) is the static eccentricity between the centres of mass and stiffness, CM and CS, respectively (Figure 1).

The second term in equations (1a) and (1b), \( \gamma b (\gamma=0.05) \) is termed the accidental eccentricity component and accounts for all uncertainties potentially leading to increased torsional response. The amplification factor \( \delta_1 \), applied to the static eccentricity \( e_s \) in the case of the primary design eccentricity is:

\[
\delta_1 = 1.0 + e_t/e_s
\]

and the corresponding factor \( \delta_2 \) is 1.0 [equation 1(b)]. The proportion of the design force \( V_b \) assigned to structural elements on the flexible side of CS (elements 2 and 3 in Figure 1) is determined by applying equation (1a), and for element 1 on the stiffer side of CS the more unfavourable case for
Figure 1: Definition of the storey design eccentricity, $e_D$, a); asymmetric floor plan configuration, b).

all realistic structures except those with very low torsional stiffness is generated by equation (1b). It is noted that the latter equation leads to a reduction of design loading for element 1, compared with a reference system with design loadings (or yield strengths) allocated in direct proportion to the element stiffnesses.
The primary dynamic eccentricity amplification factor $\delta_1$, for buildings with a typical aspect ratio $a/b = 1/3$ has been summarised in Figure 2(a). The two curves separating for $e_s < 0.15b$ correspond, respectively, to

![Diagram showing the primary dynamic eccentricity amplification factor, $\delta_1$, for buildings with a typical aspect ratio $a/b = 1/3$. The two curves separating for $e_s < 0.15b$ correspond, respectively, to $\Omega = 0.7$ and $\Omega = 1.0$.](image)

![Diagram showing the primary dynamic eccentricity amplification factor, $\delta_1$, for buildings with a typical aspect ratio $a/b = 1/3$. The two curves separating for $e_s < 0.15b$ correspond, respectively, to $\Omega \leq 1.0$.](image)

Figure 2: EC8 primary dynamic eccentricity amplification factor, $\delta_1$. 
buildings with small and intermediate torsional stiffness \([\epsilon_\tau = 0]\) torsional to lateral frequency ratio \(\Omega = 0.7, 1.0\) and those with high torsional stiffness \(\Omega = 1.3\). For buildings with larger aspect ratio and \(\Omega \leq 1.0\), the code makes an increased allowance for eccentricity amplification effects for the primary design case [Figure 2(b)]. This is in accordance with the trends determined by elastic dynamic analyses [3], which show that the coupling effect between lateral and torsional response components reduces significantly with increasing values of both the key parameters, \(\Omega\) and \(\epsilon_\tau\), and with decreasing aspect ratio \(a/b\).

![Diagram](image-url)

**Figure 3**: Variation of primary design eccentricity with \(\epsilon_\tau\) for various seismic codes.

The primary design eccentricity \(\epsilon_{D1}\) [equation 1(a)], normalised to the building plan dimension, \(b\), has been plotted in Figure 3 for an example building with \(a/b = 1/3\). This relatively non-conservative case [Figure 2(b)] has been employed below in the evaluation of the inelastic performance of EC8 code-designed asymmetric buildings. Also shown in Figure 3 are the primary design eccentricity provisions of the seismic codes.
Soil Dynamics and Earthquake Engineering

[3] of New Zealand (NZS 4203), designated NZE, Canada (NBCC 1990, CAN), Mexico (1987, MEX), the U.S. code UBC 1988, and finally the draft Australian earthquake code AS 1170.4 (1992) [4], designated AUS. As concluded in several earlier studies, the linear provisions as specified by all codes except the Australian are less conservative than ECS for buildings with small or intermediate eccentricities. However, the Canadian and Mexican codes (with $d_1 = 1.5$ and $\gamma = 0.1$) are more conservative than EC8 for large static eccentricities $e_0$, greater than about 0.25$b$. In the draft Australian code, the primary design eccentricity expression is based on a second order function of $e_0$ (in the range $e_0 = 0$ to 0.33$b$), and a linear function ($d_1 = 1.4$) for large eccentricities. The AUS provision closely matches those of EC8 throughout the considered range of $e_0$, representative of most actual buildings.

**Special features of EC8**

EC8 is the only major earthquake-resistant design code which explicitly considers the plan aspect ratio, $a/b$ (Figure 1) and the torsional to lateral stiffness ratio in its torsional provisions. In the case of the single storey building considered herein, the latter ratio is related directly to $\Omega$, as follows [5]:

$$\Omega = \frac{\rho}{r}$$

In equation (3), $r$ is the mass radius of gyration of the floor slab about $CM$, and $\rho^2$ is the structure's torsional to lateral stiffness ratio. That is, $\rho^2 = K_{th}/K_y$, where $K_{th}$ is torsional stiffness of the storey, calculated about $CS$, and $K_y$ is the total storey stiffness in the lateral $y$-direction. The parameter $\rho$ is termed by EC8 the 'resilience radius'. For a building with a plan aspect ratio $a/b = 1/3$, equation (3) can be expressed as:

$$\Omega = 3.29(\rho/b)$$

Building models with the ratio $\rho/b$ around 0.3 (and hence $\Omega \approx 1$) are representative of structures with intermediate torsional stiffness [6], and correspondingly those with $\rho/b < 0.25$ and $\rho/b > 0.4$ represent torsionally flexible structures and torsionally stiff structures, respectively.

**PERFORMANCE OF ELASTICALLY RESPONDING STRUCTURES**

For small or moderate earthquakes, the aim of design for torsional effects is to ensure that elements on the flexible side of $CS$ have sufficient strength to accommodate the additional deformation arising from rotational floor response, whilst remaining in the elastic range and thereby minimising the
resultant damage (affecting mainly the non-structural components). The peak deformation of element 3 at the flexible-edge of the structure under consideration has been calculated by dynamic modal analysis, using the standard Newmark-Hall median elastic design response spectrum [7] for 5% damping to represent the earthquake input. The peak deformation of element 3 obtained from dynamic analysis for structures with a wide range of normalised static eccentricities, e,/b (0 to 0.5), has then been used to determine the primary dynamic eccentricity (\(e_d = \delta e\)) applicable to each case. The dynamic eccentricity has been derived for fundamental lateral periods, \(T_y\), up to 5.0 seconds, this value representing multistorey buildings of about 40-50 storeys in height.

Figure 4 shows the mean dynamic eccentricity obtained by modal analysis (to which the accidental eccentricity component, 0.05b, should be added to determine the primary design eccentricity \(e_{d1}\)). Results are shown for buildings with a range of plan aspect ratios, and correspond to the worst-case amplification of the torsional response component, occurring for \(\alpha = 1.0\) [3]. The results have been compared in Figure 4(a) with the design provisions of EC8, for the case \(\alpha \leq 1.0\) (see Figure 3). The EC8 torsional provisions follow the trends of the elastic dynamic analyses very closely, and with a reasonable degree of conservatism, over the full range of realistic eccentricities. This is to be expected, since the provisions are based explicitly on results of elastic dynamic analyses [2]. Since torsional coupling effects tend to reduce for \(\alpha < 1.0\), whereas the EC8 primary design eccentricity remains constant, the code also provides conservative estimates of the response of torsionally flexible asymmetric buildings.

All the other code provisions considered, with the notable exception of the draft AUS code, are non-conservative for systems with small to moderate eccentricities [Figure 4(b)]. It is well known that such cases give rise to the most severe modal coupling, when the system response remains in the elastic range. It is to be expected, therefore, that for such systems designed for example to the UBC or NZE codes, the flexible-edge element will undergo a degree of inelastic deformation even when subjected to ground motions with small or moderate intensity. This may give rise to unacceptable damage in non-structural elements, which are expected to remain intact under low levels of seismic loading.

**RESTRICTIONS ON STATIC PROCEDURE IN EC8**

EC8 limits the application of the static torsional design procedure by specifying certain regularity conditions. If any one of these conditions is not satisfied, the static design procedure is not permitted and generally a multi-modal response spectrum analysis is recommended. The code
requires that the plan configuration of buildings should be compact and regular and that regularity is maintained in the vertical configuration such that the stiffness and mass properties are approximately uniformly distributed over the building's height. Therefore, the storey centres of stiffness, CS (or, more accurately, the shear centres) should lie

Figure 4: Comparison of the primary dynamic eccentricity based on code static torsional provisions with the corresponding results of dynamic modal analysis, $\Omega = 1.0$. 
approximately on a vertical line and the centres of mass, CM, of the individual floor slabs should also lie on a vertical line.

Furthermore, EC8 requires that appropriate horizontal regularity be maintained such that at any given storey the eccentricity $e_i$ should not exceed 15% of the resilience radius, $\rho$, defined above. Application of this condition restricts the static design procedure to asymmetric buildings having small eccentricity, even for buildings having high torsional stiffness. For example, even for a building having $\Omega = 1.3$, the condition $e_i \leq 0.15\rho$ (where $\rho$ is expressed in equations (3) and (4) in terms of $\Omega$) implies $e_i < 0.06b$ in any given storey. The results given in Figure 4 show clearly that this condition is highly conservative for elastic seismic design. The EC8 allowance for torsional effects in this range of response is found to be adequate even for buildings with very large eccentricity; hence the regularity condition is regarded as superfluous under these circumstances. It is noted that these restrictions are similar in principle to the torsional stability provisions of the New Zealand seismic design code NZS 4203 (1992) [8].

In the following section, an important extension to previous research has been provided by evaluating the inelastic torsional response of EC8-designed asymmetric structures subjected to severe ground motions. In view of the above comments, for the purposes of this study the horizontal regularity condition has not been directly adhered to. Hence, limits on the values of the key parameters $e_i$ and $\Omega$ have not been considered. Such a procedure enables the horizontal regularity condition of EC8 to be re-evaluated for buildings designed to ultimate limit state criteria.

**INELASTIC RESPONSE TO SEVERE GROUND MOTIONS**

**Linear analysis design spectra**

The design base shear (or nominal lateral design strength), $V_b$, on which the calculation of element design strengths by the static procedure is based, is specified by EC8 as:

$$V_b = \alpha S\beta(T_y)W$$  \hspace{1cm} (5)

where $\alpha$ is defined as the ratio of the site peak ground acceleration to the acceleration of gravity, $g$. In this study, the parameter $\alpha$ is taken to be 0.3, representing severe earthquakes. Also in equation (5), $S$ is the soil parameter, being 1.0 for soil profiles A (rock or extended layers of very stiff soil) and B (stiff soil), or 0.8 for soil profile C (soft soil); $W$ is the
design spectra of EC8 taking $a = 0.3$ and $b = 4$, $q = 0.5$.

European strong motion earthquake records with the corresponding
comparison of the 5% damping elastic response spectra of selected

Figure 5: Normalised elastic and inelastic design spectra of EC8.

(a)
weight of the building; \( \beta(T_y) \) is the normalised design spectrum specifying the variation of the design base shear with respect to the uncoupled lateral period \( T_y \). The design spectra \( \beta(T_y) \) of EC8 based on the recommended parameters for the above three soil types are shown in Figure 5(a), where \( q \) is termed the structural behaviour factor. This factor, which is similar in concept to the force reduction factor employed by other seismic codes, takes into account the structure’s energy dissipation capacity through inelastic ductile behaviour. Except at very short periods \( T_y < T_1 \) [Figure 5(a)], the design base shear is obtained simply by dividing the elastic strength demand by \( q \). In the analyses which follow, the structural behaviour factor \( q \) is taken to be 4, a value commonly employed for ductile steel and reinforced concrete structural systems.

**Strong-motion earthquake records**

Three records selected from European strong-motion earthquake events have been utilised as ground motion input for the assessment of the EC8 torsional provisions for inelastic (ultimate limit state) seismic structural design. These records have distinctly different frequency contents and have high (Patras), intermediate (Thessaloniki) and low (Romania) ratios of peak ground acceleration to velocity, respectively. The 5% damped elastic acceleration response spectra of these selected records, normalised to a common peak ground acceleration of 0.3g, have been plotted in Figure 5(b). Also shown in this figure are the corresponding design spectra stipulated by EC8, with the above values for \( q \) and \( \alpha \).

**Inelastic torsional effects**

The peak inelastic displacement ductility demand, \( \mu_* \), arising in the edge elements 1 and 3 in asymmetric buildings having small, intermediate and large eccentricities, \( e_1 = 0.1b, 0.2b \) and \( 0.3b \), respectively, have been computed. The results have then been normalised to the peak element ductility demand, \( \mu_* \), of the corresponding reference symmetric systems having \( e_1 = 0 \) and uniformly distributed stiffness and strength (see Figure 6). The three records lead to ductility demands that differ greatly in magnitude; this results from the relationship between the elastic strength demand represented by the response spectra plotted in Figure 5(b), and the corresponding inelastic design spectra. Since the aim of the present study is to identify response trends rather than to evaluate specific numerical results or carry out any statistical analysis, such differences in the ductility demands arising from the different ground motions are of no direct concern, and are largely eliminated by the above normalisation procedure.

It is apparent that the EC8 static torsional provisions result in significant additional ductility demand (ratios \( \mu_*/\mu_* > 1 \)) in the stiff-edge element 1. This is attributed to the reduction of element 1 strength (and consequently its
Normalised displacement ductility $\mu \mu_i$.

Figure 6:

- Normalised peak
- Uncoupled lateral period $T^*$ (sec)
- Normalised displacement ductility $\mu \mu_i$

yield deformation) compared with the corresponding reference systems, resulting from application of the secondary design eccentricity $e_{d2}$. Such reductions lead to a substantial underestimation of strength demand for the stiff-edge resisting element. The results presented in Figure 6 indicate that this underestimation occurs across the full period range ($T_y$) considered, namely up to 2.0 seconds, and for all three selected earthquake records with widely varying spectral characteristics.

Also, the additional ductility demand in element 1 increases significantly with the magnitude of the eccentricity $e_*$. For $e_* = 0.3b$, for example, the calculated additional ductility demand is generally greater than 2.0 and may reach values as high as 5.0, for medium-period systems subjected to the Thessaloniki and Romania earthquake records [Figures 6(b),(c)]. Similarly high levels of additional ductility demand occur in the short-period range for buildings subjected to the Patras earthquake record [Figure 6(a)]. However, for buildings with small eccentricity, $e_* = 0.1b$, the additional ductility demand in the stiff-edge element is controlled reasonably satisfactorily by the EC8 torsional provisions, averaging about 1.2 to 1.3.

A recent paper [9] has investigated the influence of EC8 detailing provisions on element design strengths, and concluded that the minimum reinforcement requirements lead to design overstrengths, particularly in stiffer elements. Such conservative restrictions will clearly influence the inelastic performance of these elements, and their significance is currently being investigated in more detail.

In contrast, the results in Figure 6 show clearly that the EC8 primary design eccentricity provision, which determines the design strength of element 3, gives conservative control of the flexible element ductility demand in the inelastic range of response, over the full range of eccentricities and periods considered. In fact, this provision may be regarded as somewhat over-conservative for buildings with intermediate or large $e_*$. For example, medium-period or long-period systems subjected to the Thessaloniki or Romania earthquake records may have peak ductility demands as low as 20-30% of that computed for the corresponding symmetric systems. In some cases, this element may only just reach the threshold of inelastic behaviour ($\mu_y = 1$). This is due to the allowance for significantly increased strength in this element, accounting mainly for lateral-torsional coupling effects in the elastic range of response. Nevertheless, since the yield displacement of this element has been increased in accordance with its increased strength, the peak deformation of elements at the flexible edge of the building may still be very high compared with symmetric, elastic systems (as discussed above), due to a combination of the torsional effect and inelastic response. The deformation of the flexible element may therefore become the primary criterion for design.
DISCUSSION AND CONCLUSIONS

The results shown in Figure 6 illustrate the difficulty arising in asymmetric structures when attempting to specify element strength distributions which satisfy both elastic and inelastic design criteria with reasonable (and consistent) levels of conservatism. This problem has still to be satisfactorily resolved, with regard to the performance of both the flexible-edge element (which is the critical element with regard to elastic deformation and corresponding strength demand) and the stiff-edge element (which is the critical element with regard to additional inelastic ductility demand). These issues have been addressed in more detail in Reference [5], and are the subject of on-going research studies.

It is concluded that despite the increase in the total lateral yielding strength of asymmetric buildings compared with the design base shear applicable to the corresponding symmetric systems (by approximately 20% for buildings with \(e_i\) in the range 0.1b to 0.3b, see [5]), the EC8 torsional provisions lead to an element strength distribution for systems with intermediate or large stiffness eccentricity (\(e_i \geq 0.2b\)) which does not achieve the design aim of consistent protection given to symmetric and asymmetric buildings against excessive inelastic response and consequent structural damage. This implies that some restriction on eccentricity should be imposed when using the static torsional provisions for ultimate limit state design. However, on the basis of the presented results, the existing EC8 regularity provision may still be too severe even when inelastic response is considered.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the support received from the British Council for the Ph.D research studies of Dr. X.N. Duan, carried out at University College London between 1988 and 1992, under the Technical Co-operation Training Scheme between the U.K. and the People's Republic of China.

REFERENCES


