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Influence of bi-directional ground motions on the inelastic response of one-storey in-plan irregular systems

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Abstract

This paper examines the influence of bi-directional seismic excitations on the inelastic behaviour of in-plan irregular systems having one symmetry axis, schematised as one-storey models with resisting elements arranged along two orthogonal directions. Their strength is designed by means of the standard application of the modal analysis and by a procedure already proposed by the authors with reference to asymmetric models subjected to unidirectional ground motions. The stochastic nature of the seismic excitation is considered by analysing the structural inelastic response to 30 pairs of artificially generated accelerograms matching the elastic response spectrum proposed by Eurocode 8 for hard layer soil. The secondary horizontal seismic component is scaled to different values so as to examine the influence of its intensity on the ductility demand. The analyses show that the inelastic response is affected only in a minor way by the contemporary presence of the principal and secondary components of the seismic action, although the results are more scattered and significant increases of ductility demand in the elements along the asymmetric direction may sometimes arise. The proposed design procedure is almost always able to reduce the ductility demand of the resisting elements along the asymmetric direction to values comparable to those required by torsionally balanced systems. In most cases the adoption of Eurocode 8 provisions to combine the effects of the two seismic components allows the limitation of the orthogonal elements ductility demand.

Keywords: Asymmetric systems; Inelastic response; Bi-directional ground motions

1. Introduction

After moderate and severe seismic events, extensive damage and even collapse has been observed in buildings with in-plan irregular distributions of masses and resisting elements, owing to the torsional motion of the floors.

The elastic response of asymmetric systems has been thoroughly analysed in the past and the large number of papers on this subject has given an overview of the behaviour of systems with different dynamic properties (e.g. see [1]). Nowadays every structural engineer is able to correctly analyse even complex asymmetric structures by means of modal analysis. Nevertheless, all seismic codes still allow the use of static analysis and supply formulations of equivalent static eccentricities, which should provide a safe estimate of the elastic dynamic amplification; this approach is certainly simpler, but many researchers question the effectiveness of such formulations (e.g. see [2–4]).

The analysis of inelastic behaviour is much more complicated, because of the large number of design parameters which might influence the structural response (e.g. see [5]); a comprehensive list of the papers on this subject may be found in [6]. Furthermore, in contrast to the elastic case, the complexity of the input data required by computer programs which allow dynamic inelastic analyses, and the difficulty in interpreting their results, discourage common engineers from using them.

The basic aspects of inelastic response have already been highlighted. Firstly, the maximum displacements of structures well within the inelastic range are only a little dependent on the design criteria and on the strength consequently provided to the structural elements. Secondly, inelastic response is much less rotational than

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Nomenclature

a_{gx}, a_{gy} horizontal components of the ground acceleration
B dimension of the deck along the y direction
C_{M} mass centre
C_{R} centre of rigidity
$d_{0.50}$, $d_{0.95}$ 50% and 95% fractile of the normalised ductility demand of a resisting element (the subscripts x or y, if present, refer to the direction of the element)
e_{d} design eccentricity
e_r coefficient used for the evaluation of the proposed design eccentricity
e_{s} stiffness eccentricity, i.e. distance between C_{R} and C_{M}
E_{Ed} global effect due to the two horizontal components of the design seismic action
$E_{\mathrm{Ed}x}$, $E_{\mathrm{Ed}y}$ effect due to the application of the design seismic action along the x or y directions
k coefficient used for the evaluation of the proposed design eccentricity
K_x, K_y total lateral stiffness of the elements parallel to the x and y-axis
K_θ total torsional stiffness about C_{R}
L dimension of the deck along the x direction
m mass of the deck
q behaviour factor
r_k stiffness radius of gyration about $\mathrm{C_R}$: $r_k=\sqrt{\dfrac{K_\theta}{K_y}}$
r_{m} mass radius of gyration about C_{M}
T_x, T_y uncoupled translational period along the x and y directions: $T_x=2\pi\sqrt{\dfrac{m}{K_x}}$ $T_y=2\pi\sqrt{\dfrac{m}{K_y}}$
T_θ uncoupled torsional period: $T_\theta=2\pi\sqrt{\dfrac{mr_{\mathrm{m}}^2}{K_\theta}}$
γ_x share of torsional stiffness due to the elements parallel to the x-axis
ω_x, ω_y uncoupled translational frequency along the x and y directions: $\omega_x=\dfrac{2\pi}{T_x}=\sqrt{\dfrac{K_x}{m}}$ $\omega_y=\dfrac{2\pi}{T_y}=\sqrt{\dfrac{K_y}{m}}$
ω_θ uncoupled torsional frequency: $\omega_\theta=\dfrac{2\pi}{T_\theta}=\sqrt{\dfrac{K_\theta}{mr_{\mathrm{m}}^2}}$
Ω_θ uncoupled lateral–torsional frequency ratio: $\Omega_\theta=\dfrac{\omega_\theta}{\omega_y}=\dfrac{r_k}{r_{\mathrm{m}}}$

elastic response. This last point, in particular, justifies the satisfactory seismic performance of some structures designed in accordance with a simple provision of the Uniform Building Code, which requires that the strength of each element be not less than the value obtained by using a planar model (i.e. neglecting the in-plan rotation of the floors), although this limitation was introduced a long time ago for reasons which were not connected to the inelastic behaviour of asymmetric structures.

On the basis of the above mentioned issues and with the aim of complying with the dual level approach [7,8], in a previous paper [9] the authors proposed a design approach which consists of a double application of the multi-modal spatial analysis: firstly with the mass centre in its actual position, so as to obtain the elastic response; and secondly with the mass centre displaced toward the centre of rigidity of a quantity, called *design eccentricity*, so as to avoid excessive values of ductility demand. The use of modal analysis instead of static analysis reduces the numerical approximations. Indeed, the static analysis never matches acceptably the actual elastic response of asymmetric structures: the influence of higher modes of vibration is particularly remarkable in torsionally flexible structures, for which static and actual elastic responses are often in evident disagreement, and is not negligible in torsionally rigid structures in which the results of the static analysis need to be amplified on the flexible side in order to catch the dynamic effect of the structural eccentricity (e.g. see [10]). The effectiveness of the design procedure proposed by the authors and that of the analytical expression given for the design eccentricity was demonstrated by the statistical analysis of the results of a large number of step-by-step inelastic analyses, performed on one-storey schemes subjected to unidirectional ground motions.

The smaller rotation observed in structures well within the inelastic range is easily explained in schemes subjected to unidirectional ground motions: when the elements parallel to the direction of the seismic action undergo inelastic displacements, the orthogonal elements almost always remain elastic, even in strongly asymmetric schemes, thus granting a torsional over lateral stiffness ratio larger than that denoted when the behaviour remains elastic. In contrast, bi-directional ground motions might cause the contemporary yielding of elements along both directions. De la Llera and Chopra [11] observe that "buildings having substantial yielding in the orthogonal resisting planes…develop torsional mechanism in their inelastic response", while Paulay [12] points out the risk that "with all elements operating in the plastic domain, torsional restraint and consequently twist control is lost". Furthermore, while the response of a symmetric scheme to ground motions acting along the symmetric direction is always a pure translation, its response to bi-directional ground motions is contemporarily due to the translation along the symmetric direction and to the rotation of the deck caused by the orthogonal component which acts on asymmetric elements; because of that, the outer resisting elements in the symmetric direction yield at a different time, giving rise to dynamic eccentricity even in this direction. The applicability of conclusions obtained by the analysis of schemes undergoing unidirectional motions to the actual seismic events, during which two orthogonal components are always present, is thus questionable.

The seismic response of asymmetric systems to bi-directional ground motions has been examined in very few papers. Correnza et al. [13] discussed the reliability of the models used both for structure and seismic action, pointing out that, mainly in the short period range, "inclusion of the transverse load-resisting elements in the model definition affects significantly the performance of the edge elements"; nevertheless they concluded that, at least for mid to long-period systems, the response of realistic plan–eccentric structural systems (i.e. those with resisting elements in two orthogonal directions, subjected to simultaneous bi-directional ground motions) may be conservatively modelled in a simplified way with resisting elements along one direction and subjected to unidirectional ground motions. De Stefano et al. [14], analysing systems designed by means of static forces acting separately along two directions, found large increments of ductility demand (up to 50%) in the outer elements along the symmetric direction when the schemes were subjected to bi-directional ground motions.

This study aims to assess the influence of the secondary seismic component on the ductility demand of the elements along both directions, by analysing one-storey schemes covering a wide range of dynamic properties and subjected to a statistically significant set of bi-directional accelerograms. It will thus allow confirmation of the effectiveness of the design approach previously proposed by the authors to reduce the ductility demand in the elements along the asymmetric direction, and to discuss and verify a criterion of superposition of the effects of the two seismic components, which aims at avoiding excessive ductility demand in the elements along the symmetric direction.

Although the seismic response of multi-storey systems is much more complex than that of one-storey systems, the results obtained by analysing the latter yield basic information about the behaviour of the actual buildings, as confirmed by on-going studies [15,16].

2. Geometry and stiffness of the model

In order to investigate both the aspects previously mentioned, the analysed structures have been schematised by means of one-storey models with one symmetry axis (x axis). The deck, having infinite in-plan rigidity, is rectangular in shape (L=29.50 m; B=12.50 m) and is supported by resisting elements arranged along two orthogonal directions (Fig. 1); the elements have in-plan stiffness and strength only, and are characterised by an elastic–perfectly plastic behaviour. Many studies have discussed the influence of number and location of resisting elements, pointing out the opportunity of choosing a realistic model and the necessity of using more than two elements for each direction (e.g. see [5]). The selected model presents eight elements in the y-direction and three in the x-direction; this number, larger

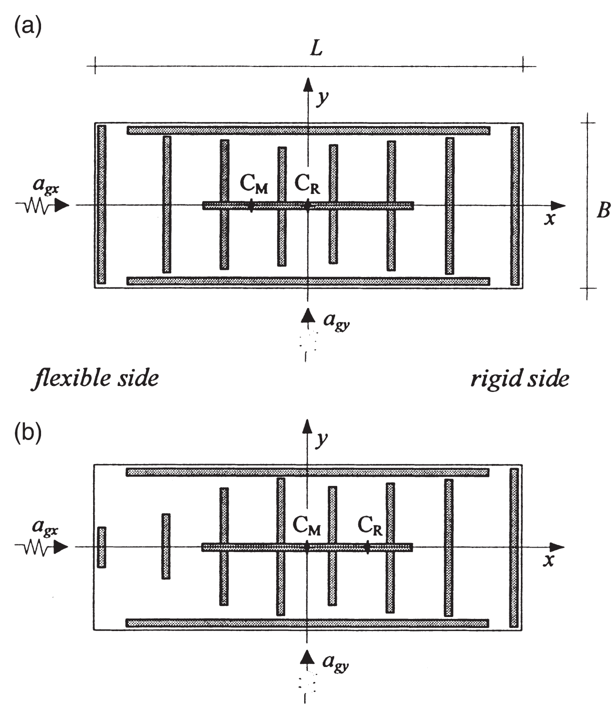

Fig. 1. Mass (a) and stiffness (b) eccentric systems.

than the minimum required, has been chosen for a better correspondence to the structure of actual buildings.

A numerical procedure [9] allows definition of the stiffness of each element so as to obtain structural systems with a given location of stiffness centre C_R, uncoupled torsional to lateral frequency ratio Ω_θ and global lateral and torsional stiffness K_x, K_y, K_θ. The location of the mass centre C_M and mass radius of gyration r_m (0.312 L) are assigned independently of the shape of the deck, supposing the total mass (m=1.00 t m$^{-2} \times B \times L$) might be non-uniformly distributed in plan. In this way it is possible to generate either stiffness eccentric systems (SES) or mass eccentric systems (MES) and to let the most important parameters vary within a wide range of values.

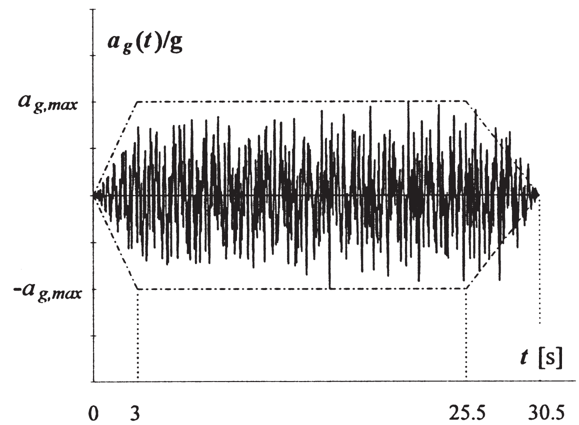

Fig. 2. Artificial accelerogram.

3. Ground motions

The possibility of using the horizontal components of historical records for the simulation of bi-directional ground motions has been discarded, because it would be inadequate for the requirements of a statistical analysis. In fact, while it is possible to select single components so as to obtain a given mean elastic spectrum, it is much more difficult to find a sufficient number of couples of components which satisfy assigned conditions.

The analysis of the relationship between the orthogonal components of a seismic recording, together with the definition of the instantaneous principal axes of motion, has been the object of many papers. For the sake of simplicity, in the present study seismic actions have been simulated by means of two uncorrelated sets of accelerograms, as proposed by Penzien and Watabe [17], artificially generated by the computer program SIMQKE [18,19].

Thirty couples of accelerograms $a_{gx}(t)$ and $a_{gy}(t)$, more than the minimum imposed by European seismic code EC8 [20], have been used in the analyses. Each component, defined by a stationary random process modulated by means of a trapezoidal intensity function, is characterised by a strong motion phase of 22.5 s, as advised in EC8 for accelerograms with a peak ground acceleration of 0.35 g, and by a total duration of 30.5 s (Fig. 2). The mean elastic response spectrum of each set of components approximates the one proposed by EC8 for hard layer soil (class A) and satisfies the requirements of the same code. Indeed no value of the mean spectrum is more than 10% below the corresponding value of the code elastic response spectrum; furthermore the average of the values in the constant acceleration range and that related to the null period are not smaller than the corresponding values of the code spectrum.

4. Resisting element strength

4.1. Standard design procedure

In an initial set of schemes the strength of the resisting elements has been defined by means of the standard application of modal analysis, which takes into account all three modes of vibration of the model. The contributions of the modes to the global behaviour have been superimposed by means of the complete quadratic combination rule [21]. The design spectrum has been obtained by the EC8 elastic response spectrum for hard layer soil (class A) with PGA=0.35 g, reduced by a constant value of the behaviour factor q.

A usual design approach consists of evaluating the effect of design forces acting separately in two orthogonal directions. In order to take into account their contemporary presence, many authors [22–26] and the Eurocode 8 itself suggest assigning the strength of each resisting element by combining the effects of the two seismic components by means of the square root of the summation of squares formula (SRSS), i.e. by the following expression

$$E_{Ed}=\sqrt{E_{Edx}^2+E_{Edy}^2} \quad (1)$$

where E_{Ed}, E_{Edx}, E_{Edy} are respectively the global effect due to the two horizontal components of the design seismic action and the effect due to the application of the design seismic action along the x and y directions.

This criterion has been adopted herein, obtaining some increase of the strength of the elements in the symmetric direction; the maximum variation for the outer elements has been about 20%, while the global strength increase has never exceeded 5% (Fig. 3). The strength of the elements in the asymmetric direction remained unchanged because the design seismic component along the x-axis does not cause displacements in the orthogonal direction.

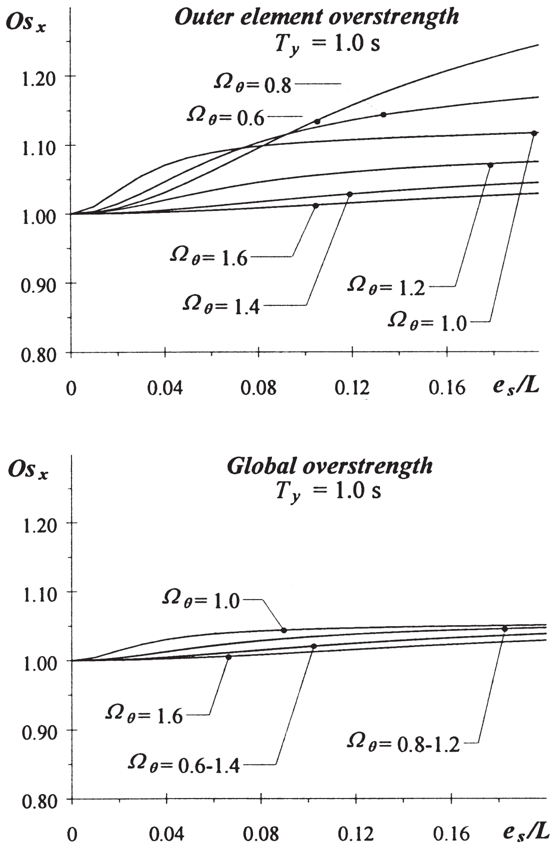

Fig. 3. Overstrength of the outer elements and mean overstrength of all elements acting along the *x*-direction, designed according to Eurocode 8 provisions as regards the combination of the effects of the two seismic components.

4.2. Proposed design procedure

A second set of schemes has been designed by the procedure proposed in [9]. It involves a double application of the multi-modal analysis: in the first one, the mass centre is assumed to be in its actual location; in the second one it is displaced from its nominal position toward the stiffness centre of a quantity e_d, called *design eccentricity*

$$e_d=k(e_s-e_r) \tag{2}$$

in which

$$k=\max\begin{cases}3.3\text{–}2.5\Omega_\theta+0.04q\\1\end{cases}$$
$$e_r=\max\begin{cases}0.1(0.5\Omega_\theta\text{–}0.4)L\\0.01L\end{cases} \tag{3}$$

The strength assigned to each resisting element is the maximum of those obtained by the two analyses. In this way it is possible to grant a safe response to both moderate and strong earthquakes. The first analysis, in fact, allows the matching of the elastic response of the structure subjected to forces, corresponding to the design spectrum (i.e. to the elastic one reduced by the behaviour factor), which at the same time represents the inertia forces induced by moderate intensity earthquakes. On the basis of this, it is therefore also possible to limit the interstorey drift to values which do not induce relevant damage to the non-structural elements. The second analysis increases the strength of the resisting elements wherever the elastic design underestimates the actual inelastic displacements, so as to reduce the maximum ductility demand; more precisely such analysis generally increases the strength of the elements on the flexible side of torsionally flexible structures and on the rigid side of torsionally rigid structures, while in structures characterised by values of the uncoupled lateral–torsional frequency ratio Ω_θ close to unity, it increases the strength of the elements at the centre of the scheme; the global strength increment depends above all on e_s, being about 10% when e_s=0.1 L and 15% when e_s=0.2 L [9].

Also in this case, the results of modal analyses performed with seismic excitation along the *x* and *y*-axes have been combined according to Eurocode 8 provisions.

5. Parametric analysis

The numerical analyses have involved both mass and stiffness eccentric systems, even if in other studies no remarkable differences have been noticed between the inelastic response of mass and stiffness eccentric systems subjected to unidirectional ground motions. In order to understand more thoroughly the behaviour of such schemes, the most important parameters e_s, Ω_θ, T, q and γ_x have been varied within a large range (Table 1). For each scheme, the element strength has been assigned twice, according to the standard modal analysis and to the proposed design procedure, so as to verify the effectiveness of the latter. The response of each asymmetric scheme has been compared to that of the corresponding torsionally balanced system, having the mass centre coincident with the stiffness centre.

The couples of artificially generated accelerograms have been scaled in different ways. One component, acting along the *x* or *y*-axis, has been considered as *principal* and scaled to a value of the peak ground acceleration PGA=0.35 g, representative of the intensity of strong ground motions. The other one, acting along the orthog-

Table 1
Minimum and maximum values of the main parameters used in the numerical analyses

Parameter	e_s	Ω_θ	T_x=T_y	q	γ_x
Minimum value	0	0.6	0.4 s	1	0.001
Maximum value	0.20 L	1.6	2.0 s	5	0.4

onal axis (y or x-axis respectively), has been called *secondary* and scaled to different values of the peak ground acceleration (0, 0.25, 0.50, 0.75 or 1 times the principal PGA) so as to closely examine the variation of the structural behaviour when the intensity of the secondary seismic component is increased.

For each geometrical scheme and combination of seismic components, the inelastic response of the system to the set of 30 accelerograms has been evaluated. Attention has been focused on the maximum value of the inelastic displacement of each resisting element and on the corresponding ductility demand. These quantities have been normalised by the values occurring, for the same accelerogram, in the corresponding torsionally balanced scheme. The maximum responses to the whole set of 30 accelerograms have been described by means of an equivalent gaussian probability density function having the same mean value and standard deviation of the set of the numerical results. In particular, the figures presented later and the discussion of the results are based on the *mean value*, i.e. the 50% fractile $d_{0.50}$, and the *characteristic value*, i.e. the 95% fractile $d_{0.95}$, of the normalised ductility demand.

6. Inelastic response of systems designed according to the standard procedure

6.1. Ductility demand of elements along the asymmetric direction

In order to analyse the influence of bi-directional ground motions on these elements, the seismic component along the y-axis has been assumed as principal and scaled to PGA=0.35 g, while the secondary one has been varied from 0 to 0.35 g.

The mean value of the normalised ductility demand of the resisting elements along the y-direction is shown in Figs. 4 and 5 as a function of their position. The global behaviour of the system is instead presented in Fig. 6, where both the mean and the characteristic values of the maximum normalised ductility demand among all the resisting elements are plotted versus the stiffness eccentricity. The same values are reported in Fig. 7 for different values of the uncoupled translational period T_y, so as to highlight some differences in the responses of low and high period schemes. The type of scheme (MES or SES), the behaviour factor q and the ratio γ_x of torsional stiffness due to the elements in the x-direction over the total torsional stiffness have little influence on the structural behaviour; for this reason the corresponding results are not shown here.

Torsionally flexible systems (Ω_θ<1) are influenced to a negilible extent by the secondary seismic component. The mean normalised ductility demand of the resisting elements does not show remarkable variations with the

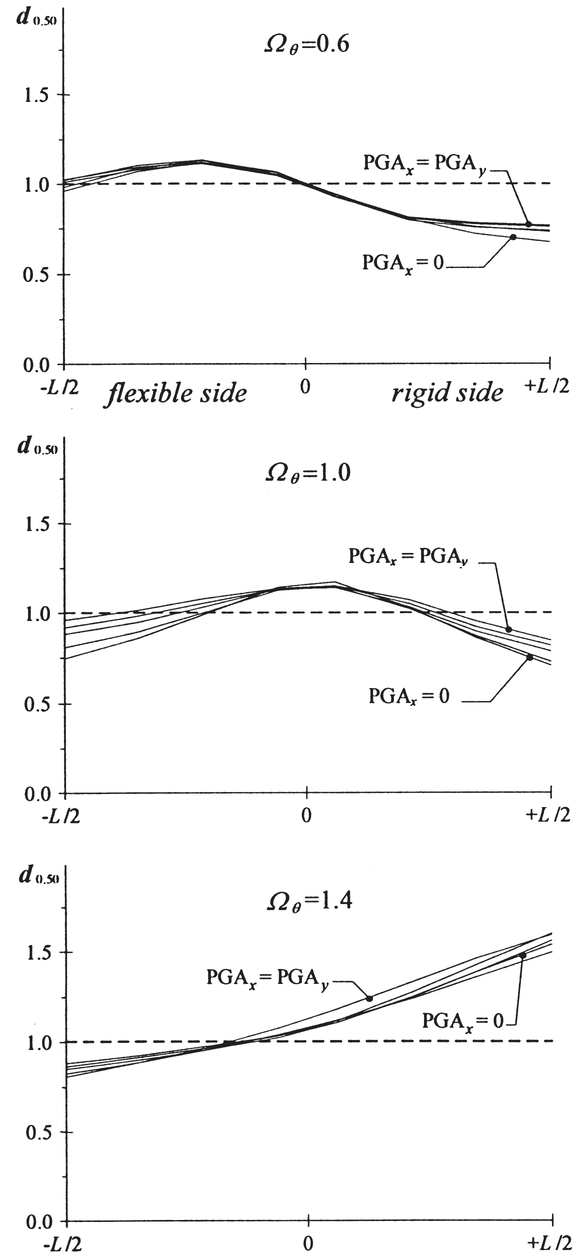

Fig. 4. Distribution along the deck of the mean normalised ductility demand $d_{y0.50}$ of the elements in the y-direction, for different values of the intensity of the secondary seismic component (PGA$_x$=0, 0.25, 0.50, 0.75, 1.00 PGA$_y$). Asymmetric systems designed by means of standard modal analysis; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5, e_s=0.10 L.

growth of the intensity of such a component, particularly for very low values of the uncoupled lateral–torsional frequency ratio. Some increase may be noted for the elements on the stiff side when Ω_θ is close to unity; nevertheless the maximum value of the ductility demand, which is always achieved in the central elements, is also in this case affected in a minor way. Only for some torsionally flexible schemes (Ω_θ=0.6, T_y≤1 s) with low values of stiffness eccentricity (e_s<0.10 L) the mean and, mainly, the characteristic value of the maximum

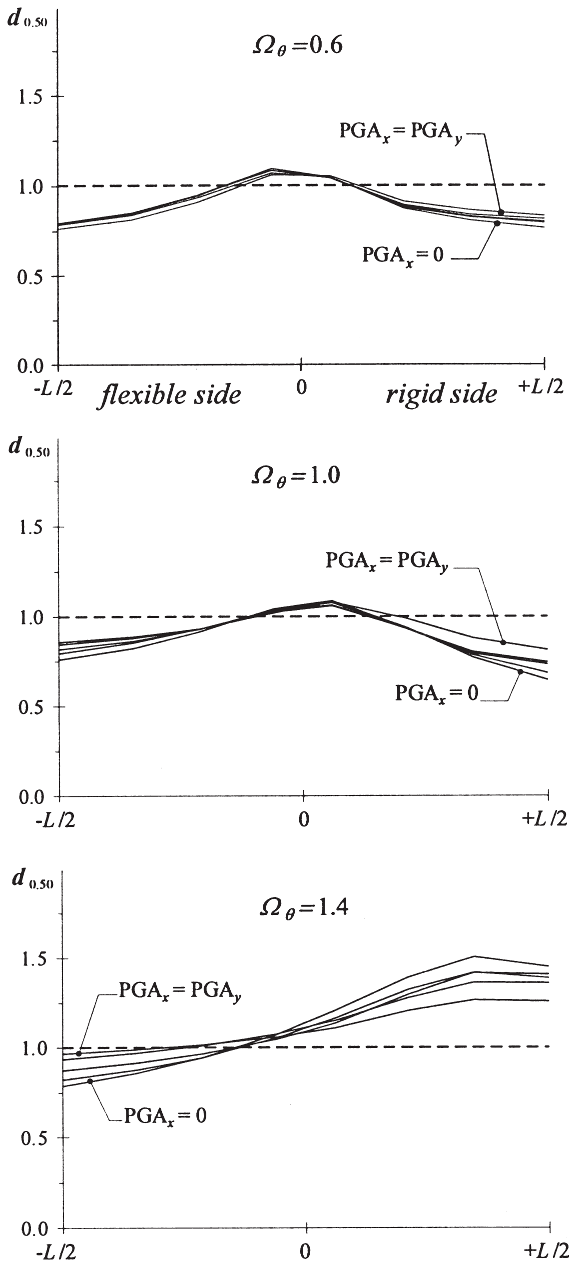

Fig. 5. Distribution along the deck of the mean normalised ductility demand $d_{y0.50}$ of the elements in the y-direction, for different values of the intensity of the secondary seismic component (PGA_x=0, 0.25, 0.50, 0.75, 1.00 PGA_y). Asymmetric systems designed by means of standard modal analysis; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5, e_s =0.20 L.

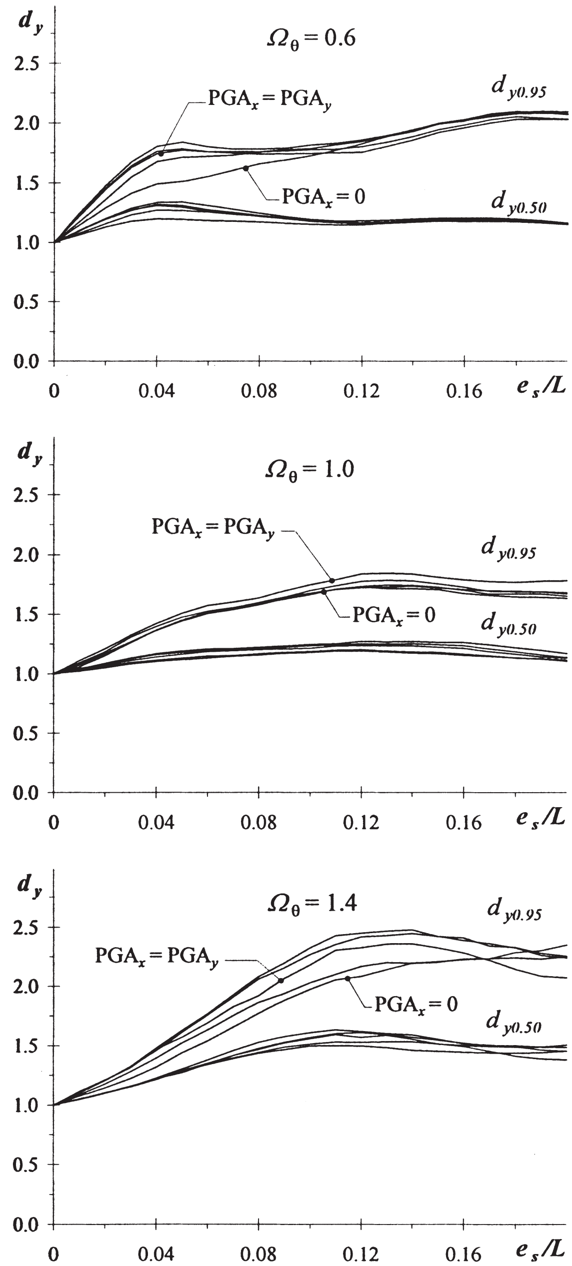

Fig. 6. Mean and characteristic values $d_{y0.50}$ and $d_{y0.95}$ of the maximum normalised ductility demands of the elements in the y-direction, for different values of the intensity of the secondary seismic component (PGA_x=0, 0.25, 0.50, 0.75, 1.00 PGA_y). Asymmetric systems designed by means of standard modal analysis; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5.

ductility demand increase in a significant way with the intensity of the secondary seismic component.

Torsionally rigid systems (Ω_θ>1) highlight a greater dependence on the value of the intensity of the secondary seismic component. For unidirectional seismic excitation, the inelastic response always presents a deck rotation smaller than the one shown in the elastic range (used in the design phase); for this reason the ductility demand of the element at the rigid edge is greater than that required in the corresponding balanced system. When the secondary component is taken into account, the inelastic behaviour of the system is sometimes even less rotational in the case of small stiffness eccentricity, while for greater eccentricity the inelastic response is a little more similar to the elastic one. In the first case (Fig. 4), a slight increase of the ductility demand may be noted on the rigid side. In the second case (Fig. 5) the demand increases on the flexible side, but its normal-

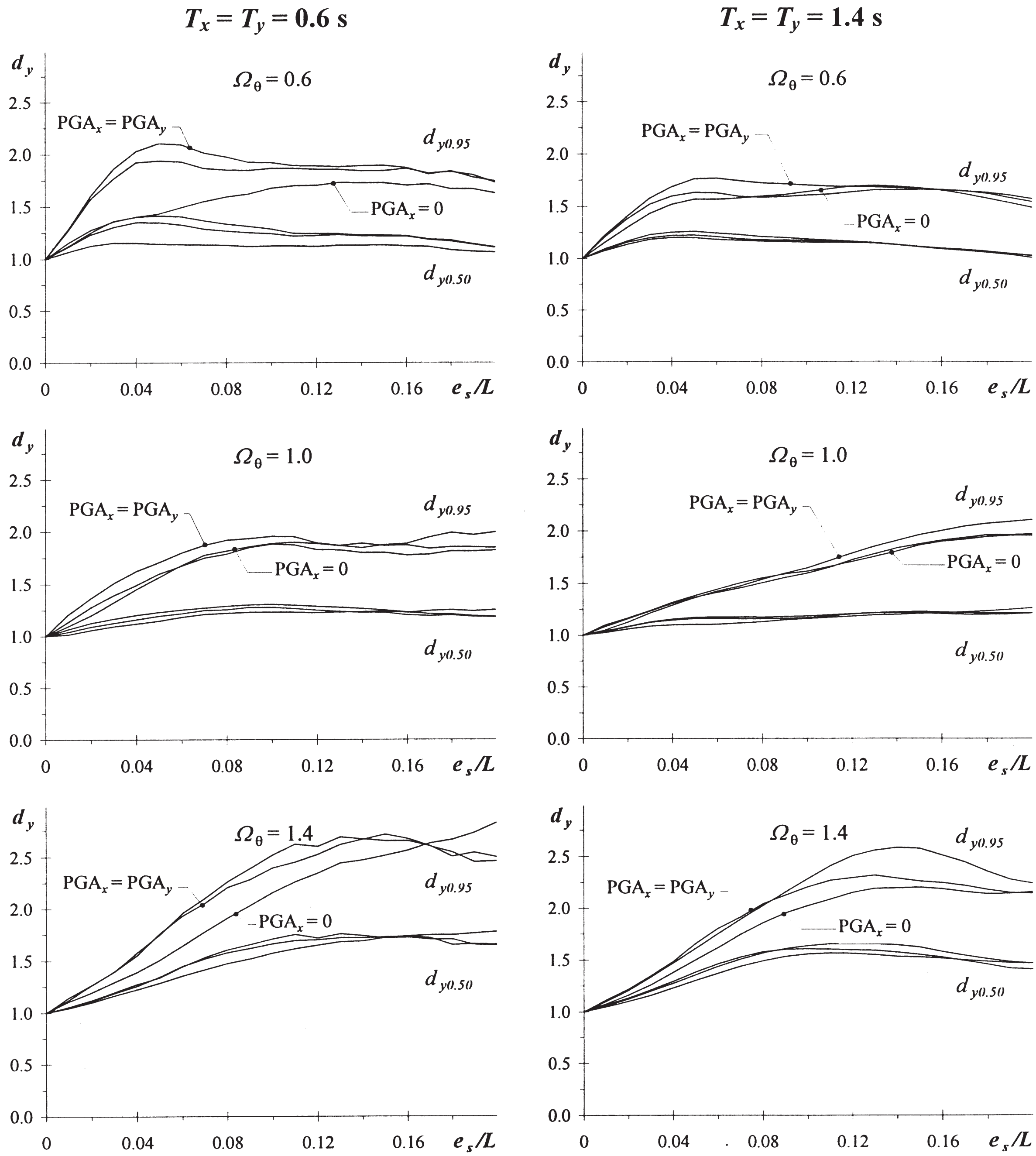

Fig. 7. Mean and characteristic values $d_{y0.50}$ and $d_{y0.95}$ of the maximum normalised ductility demands of the elements in the y-direction, for different values of the intensity of the secondary seismic component (PGA$_x$=0, 0.50, 1.00 PGA$_y$). Asymmetric systems designed by means of standard modal analysis; design parameters: MES, Ω_θ=variable, T_x=T_y=variable, γ_x=0.2, q=5.

ised value never exceeds unity; contemporarily, the ductility demand decreases at the rigid edge of the deck, where in unidirectional conditions of seismic excitation such structures sustain the greatest damage. In any case, the difference is negligible in terms of mean normalised ductility demand (Fig. 6), while it is quite remarkable in terms of characteristic value, indicating that these phenomena are related only to a few seismic events.

6.2. Ductility demand of elements along the symmetric direction

The influence of bi-directional ground motions on these elements has been analysed by assuming the seismic component along the x-axis as principal and varying the component along the y-axis. The main results are shown in Fig. 8, where the mean and the characteristic

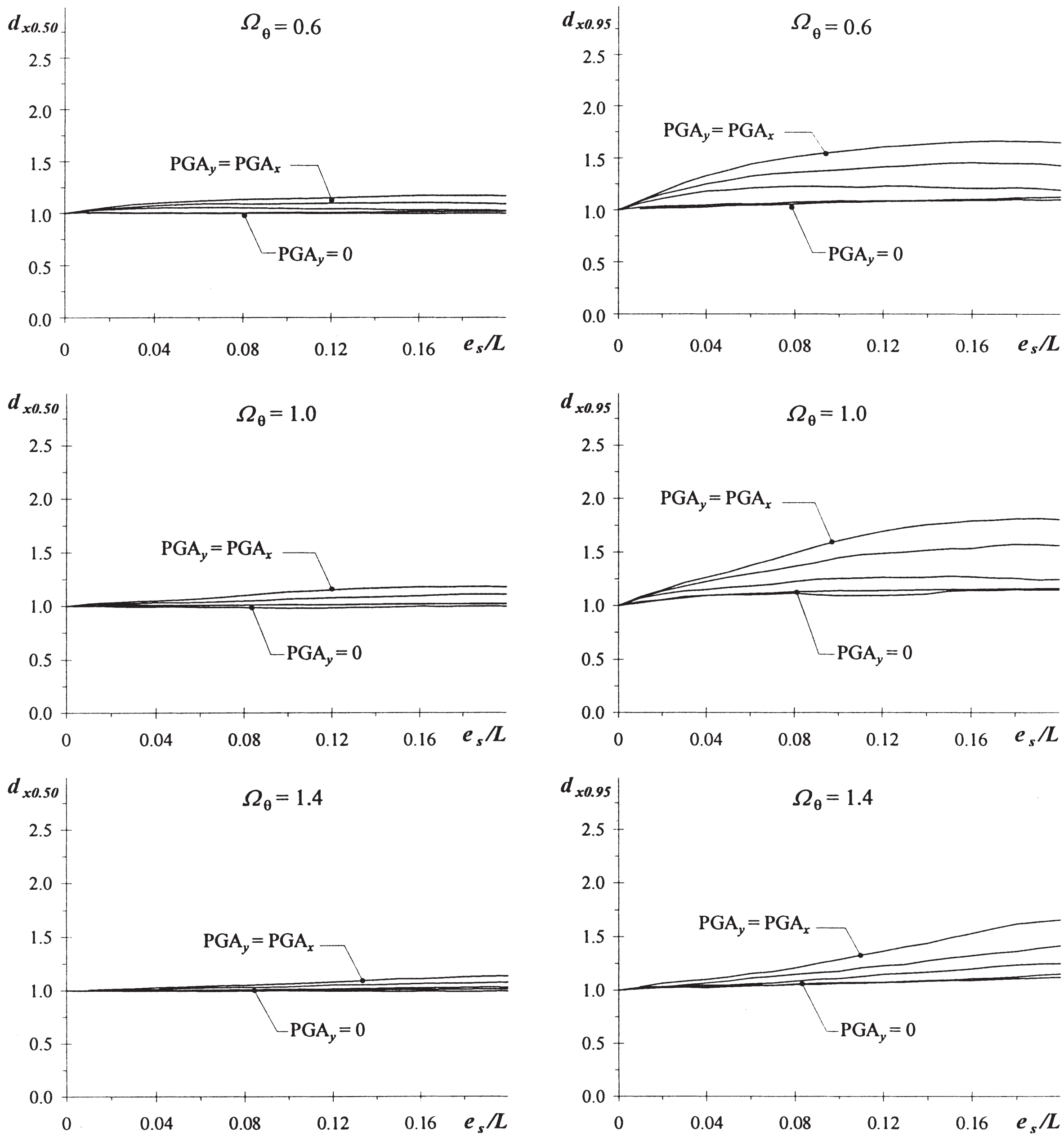

Fig. 8. Mean and characteristic values $d_{x0.50}$ and $d_{x0.95}$ of the maximum normalised ductility demands of the elements in the x-direction, for different values of the intensity of the secondary seismic component (PGA$_y$=0, 0.25, 0.50, 0.75, 1.00 PGA$_x$). Asymmetric systems designed by means of standard modal analysis; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5.

values of the maximum normalised ductility demand among all the resisting elements are plotted versus the stiffness eccentricity.

Independently of the value of the uncoupled lateral–torsional frequency ratio, the normalised ductility demand increases with the intensity of the secondary seismic component and with the stiffness eccentricity. The difference is not particularly remarkable in the mean (about 15% for PGA$_y$=PGA$_x$ and e_s=0.20 L) but the results of the analyses are more scattered. Until the secondary component is significantly smaller than the principal one (PGA$_y$≤0.5 PGA$_x$) the characteristic value of the normalised ductility demand does not increase too much (about 25% when e_s=0.20 L), but it becomes much larger (up to 80%) when PGA$_y$ is equal to PGA$_x$.

Previous studies [14] have shown large increases (25–50%) of the mean ductility demand in the elements along the symmetric direction, designed without any particular provision to account for the contemporary presence of two orthogonal seismic components, but subjected to bi-directional ground motions. The results referred to here demonstrate that the provisions of Eurocode 8 are quite effective in reducing the increase of ductility demand in the mean, although they cannot avoid a significantly worse response to very few seismic excitations.

7. Inelastic response of systems designed according to the proposed procedure

7.1. Ductility demand of elements along the asymmetric direction

The influence of the value of the secondary seismic component (along the *x*-axis) on the mean normalised ductility demand of the resisting elements along the *y*-direction is shown in Figs. 9 and 10, as a function of their position. Independently of the stiffness eccentricity, the presence of this component increases the ductility demand at the rigid edge of the torsionally flexible schemes and at the flexible edge of the torsionally rigid ones. The maximum normalised ductility demand among all the resisting elements remains unchanged when $\Omega_\theta<1$, being once again achieved in the central elements, while some increase may be noted when $\Omega_\theta\geq1$.

This trend is confirmed by Figs. 11 and 12, where the maximum normalised ductility demand among all the resisting elements is plotted versus the stiffness eccentricity. Its mean value is affected only in a minor way by the secondary seismic component, except for the case $\Omega_\theta\cong1$. In contrast, the characteristic value of the maximum normalised ductility demand among all the resisting elements is always increased by the presence of the secondary seismic component, thus confirming the greater scattering of the results already highlighted in the previous section; the difference is however small both for torsionally flexible and stiff schemes (excluding the case Ω_θ=0.6, T_y=0.6 s), but it is quite significant when the uncoupled lateral–torsional frequency ratio Ω_θ approaches unity.

The effectiveness of the design procedure in reducing the ductility demand of these elements is confirmed by the fact that, in spite of the value of the secondary seismic component, the mean normalised ductility demand nearly always remains smaller than unity for all the elements. Only for systems with an uncoupled lateral–torsional frequency ratio close to unity do the mean and characteristic values of the normalised ductility demand increase with the intensity of the secondary seismic component, and the beneficial effect of the design eccentricity gradually comes down. Nevertheless, in this case the proposed design procedure also appears to be quite effective for secondary components scaled up to a half of the peak ground acceleration of the principal component (PGA_x=0.5 PGA_y).

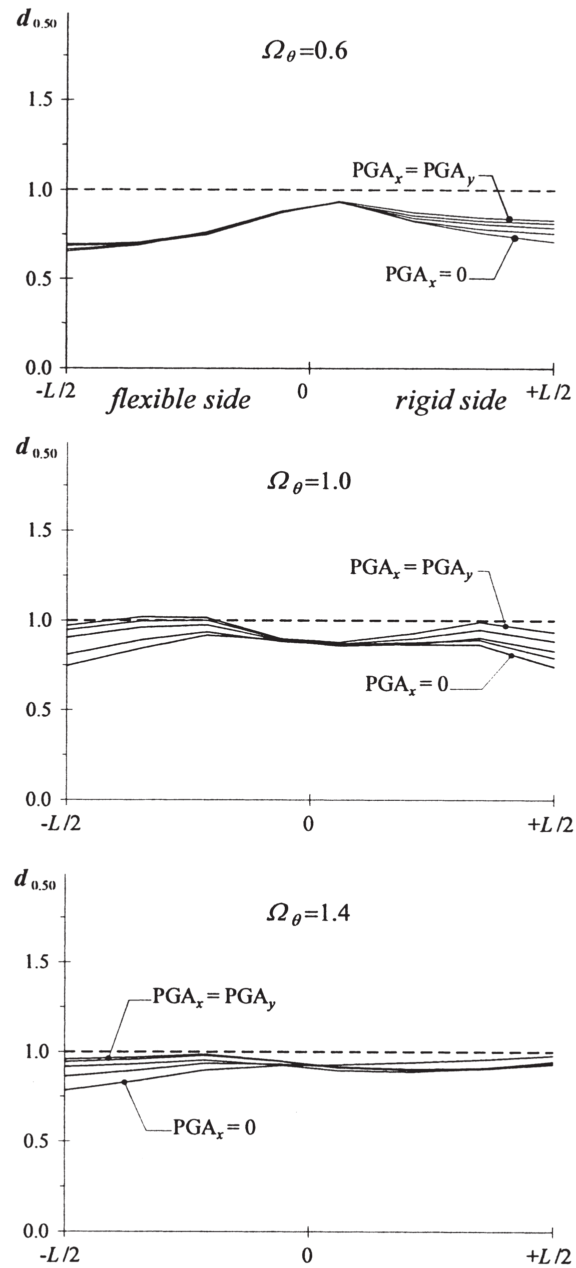

Fig. 9. Distribution along the deck of the mean normalised ductility demand $d_{y0.50}$ of the elements in the *y*-direction, for different values of the intensity of the secondary seismic component (PGA_x=0, 0.25, 0.50, 0.75, 1.00 PGA_y). Asymmetric systems designed according to the proposed procedure; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5, e_s=0.10 *L*.

7.2. Ductility demand of elements along the symmetric direction

The increase of strength of the elements along the asymmetric direction, due to the proposed design procedure, does not change the behaviour and the maximum displacements of the system in a significant way. The ductility demand of the elements along the *x*-axis is therefore always affected in a negligible way.

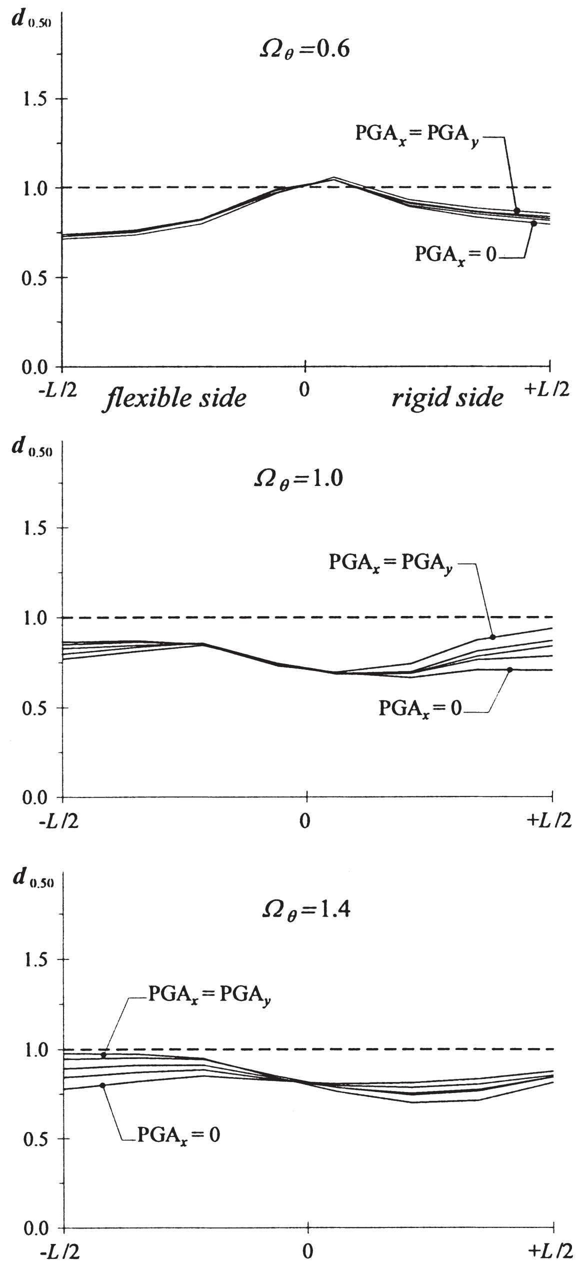

Fig. 10. Distribution along the deck of the mean normalised ductility demand $d_{y0.50}$ of the elements in the y-direction, for different values of the intensity of the secondary seismic component (PGA$_x$=0, 0.25, 0.50, 0.75, 1.00 PGA$_y$). Asymmetric systems designed according to the proposed procedure; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5, e_s=0.20 L.

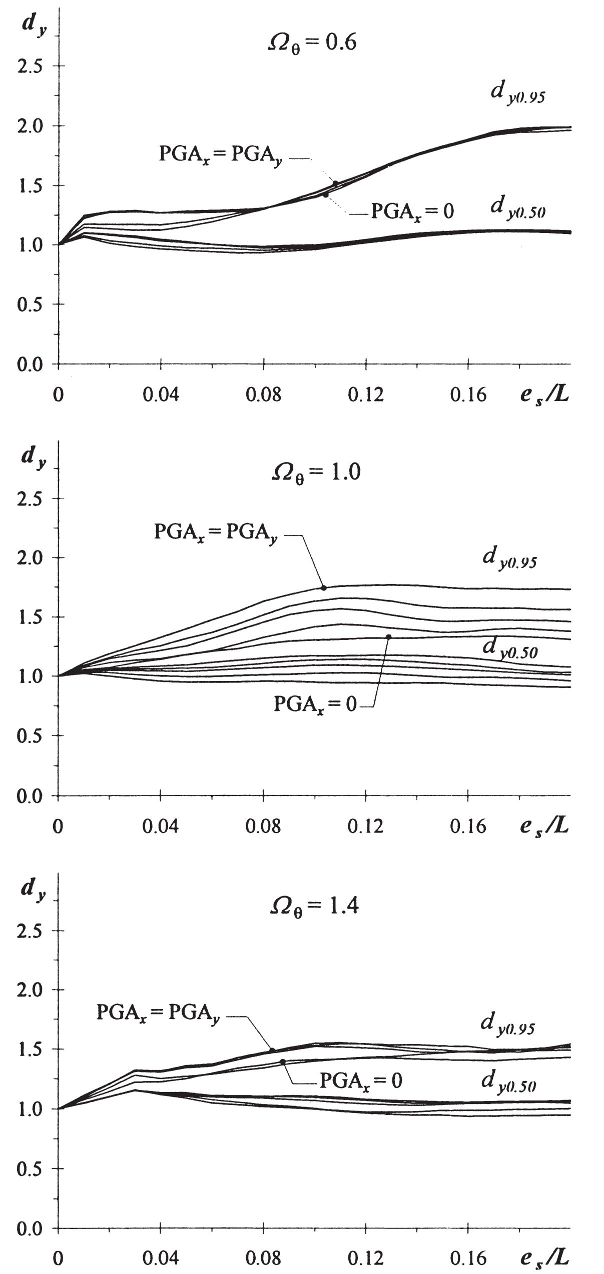

Fig. 11. Mean and characteristic values $d_{y0.50}$ and $d_{y0.95}$ of the maximum normalised ductility demands of the elements in the y-direction, for different values of the intensity of the secondary seismic component (PGA$_x$=0, 0.25, 0.50, 0.75, 1.00 PGA$_y$). Asymmetric systems designed according to the proposed procedure; design parameters: MES, Ω_θ=variable, T_x=T_y=1 s, γ_x=0.2, q=5.

8. Conclusions

The analyses show that the inelastic response of asymmetric systems to ground motions is affected only in a minor way by the contemporary presence of the two horizontal components of seismic action. In most cases the rotation of the deck is smaller than that predicted by an elastic analysis. This peculiarity has been explained, in the case of unidirectional seismic motion, by the fact that the orthogonal elements almost always remain elastic while the elements parallel to the seismic action undergo inelastic displacements. The persistence of this characteristic also in the case of bi-directional ground motions may be related to the lack of correlation between the two components, which might cause plastification of orthogonal elements at different times, and allows the maintenance of some torsional rigidity for most of the time history. The unavoidable variations

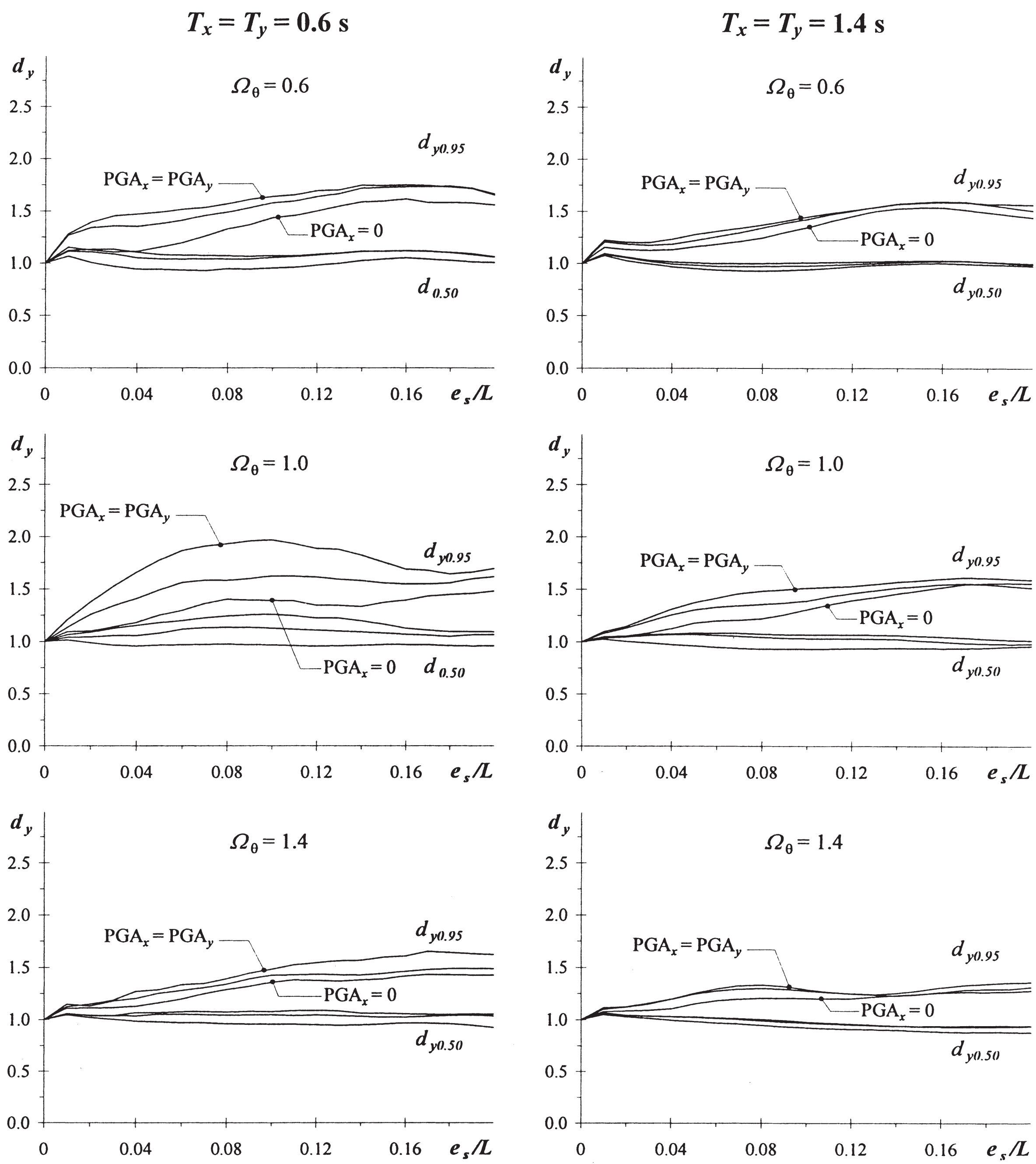

Fig. 12. Mean and characteristic values $d_{y0.50}$ and $d_{y0.95}$ of the maximum normalised ductility demands of the elements in the *y*-direction, for different values of the intensity of the secondary seismic component (PGA_x=0, 0.50, 1.00 PGA_y). Asymmetric systems designed according to the proposed procedure; design parameters: MES, Ω_θ=variable, T_x=T_y=variable, γ_x=0.2, q=5.

induced by the secondary component are negligible in the mean, although the results are much more scattered and the 95% fractile of ductility demand often increases (in some cases in a significant way) with the secondary component.

The proposed design procedure (consisting of a double application of the modal analysis) is able to reduce the ductility demand of the elements along the asymmetric direction to values which are comparable to those required by torsionally balanced systems, in most cases even when the secondary component has a peak ground acceleration equal to the principal one. Only for systems with an uncoupled lateral–torsional frequency ratio close to unity does its effectiveness appear to be limited to cases in which the secondary component is scaled to half the value of the peak ground acceleration of the principal component.

The mean ductility demand of the elements along the symmetric direction (designed according to the Eurocode 8 provisions as regards the combination of the effects of the two seismic components) is only slightly affected by the secondary component; just for a few seismic motions, the scattering of the results makes a large increase of the ductility demand possible. The overstrength given to the outer transverse elements in any case appears able to improve behaviour along this direction sufficiently.

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