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INFLUENCE OF GROUND MOTION INTENSITY ON THE INELASTIC TORSIONAL RESPONSE OF ASYMMETRIC BUILDINGS

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SUMMARY

The general trends of inelastic behaviour of plan-asymmetric structures have been studied, without any restrictions imposed by particular code requirements for torsion. Systems with structural elements in both orthogonal directions and bi-axial eccentricity were subjected to bi-directional excitation. Test examples include idealised mass-eccentric single-storey models, mass-eccentric five-storey steel frame buildings, regular in elevation, and a three-storey reinforced concrete frame building with mass, strength and stiffness eccentricities. The response in terms of displacements was determined by non-linear dynamic analyses. Sets of recorded ground motions normalised to different intensities (defined by peak ground acceleration or velocity) were used. The main findings limited to fairly regular buildings and subject to further investigations, are: The amplification of displacements determined by elastic analysis can be used as a rough estimate also in the inelastic range. Any favourable torsional effect on the stiff side, i.e. any reduction of displacements compared to the counterpart symmetric building, which may arise from elastic analysis, may disappear in the inelastic range.

KEY WORDS: torsion; asymmetric buildings; bi-axial eccentricity; seismic response; inelastic response; bi-directional ground motion

INTRODUCTION

The seismic response of asymmetric buildings in the inelastic range is very complex. Many more parameters influence the behaviour of such buildings compared to their elastic counterparts. In addition to the mass and stiffness of the structural system, the strength of the structure and its distribution in plan have an important influence on the response. Although extensive research has been performed world-wide in the field of inelastic torsional response, there is a lack of general conclusions. Unfortunately, until recently little attention has been paid to the most realistic but most complex case: multi-storey buildings with bi-axial eccentricity, subjected to bi-directional ground motion. Bi-directional ground motion has been used e.g. by De la Llera [1], Faella [2], Cruz [3], Rutenberg [4], De-la-Colina [5], and Stathopolous [6]. Non-linear dynamic analyses of asymmetric building structures subjected to bi-directional ground

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motion have also been performed by the authors (e.g. Fajfar [7]). An overview of recent research achievements was prepared by Rutenberg [8].

Research on torsion at the University of Ljubljana is aimed mainly at obtaining knowledge and data on the torsional response of structures in inelastic range, needed for the development of simplified non-linear assessment methods based on pushover analysis, like N2 method (Fajfar [9]). Thus, the objective of the study reported here is the exploration of the general trends of behaviour of plan-asymmetric structures with bi-axial eccentricity subjected to bi-directional ground motion, without any restrictions imposed by a particular code. The torsional code requirements were completely ignored. This does not necessarily mean that the results of the study are not relevant for code designed structures. Such structures are typically stiffer and stronger at the flexible edges than at the stiff edges. As a consequence, the stiffness and strength eccentricities are reduced and the torsional effect decreases, as demonstrated e.g. by Kilar [10]. The results obtained for structures not designed for torsion thus represent an upper bound for code designed structures as far as torsional influence is concerned. The response of code designed structures is expected to be similar to the response of structures investigated in this study if the eccentricity is comparable.

Extensive parametric studies have been performed. Test examples include idealised mass-eccentric single-storey models, mass-eccentric five-storey steel frame buildings, regular in elevation, and a three-storey reinforced concrete frame building with mass, strength and stiffness eccentricities. The response in terms of displacements was determined by non-linear dynamic analysis. Sets of recorded ground motions normalised to different intensities (defined by peak ground acceleration or velocity) were used. In this paper some of the main results of the studies are presented. Some additional information can be found in Peruš [11], Marušić [12] and SPEAR [13].

MATHEMATICAL MODELLING

Extensive parametric studies, required for the establishing of general trends, are feasible only if simple mathematical models are used. An important part of our study was performed with idealised single-storey models. For multi-storey buildings, a pseudo three-dimensional global mathematical model, usually applied in the case of linear analyses of multi-storey structures, was used. The model consists of planar frames connected together by means of rigid diaphragms. In this modelling approach, each column belonging to two frames (in two directions) is modelled independently in both directions, and is subjected to independent uni-axial bending in two directions, rather than to bi-axial bending. The compatibility of the axial deformations in columns belonging to the two frames is not considered. All beams and columns are modelled as planar elastic beam elements with two non-linear rotational springs at each of the two ends. The moment – rotation relationship for each spring is assumed to be elasto-plastic. The torsional stiffnesses and strengths of all elements are neglected. The influences of axial force - bending moment interaction in columns and of second order theory have not been taken into account in this study. Initial moments due to vertical loading are considered. Some of the simplifications made in the mathematical models may be significant in some cases. However, the focusing point of the study was the global torsional effect and we do not expect that the simplifications can have an important influence on the conclusions.

A large number of non-linear dynamic analyses have been performed. For single storey structures the program DRAIN-3DX was used. The hysteretic behaviour of all elements was bilinear, with 5% post-yield stiffness. In all cases 5 % mass proportional damping was assumed. The damping coefficients were determined based on the three uncoupled periods of the basic symmetric system. The analyses for multi-storey buildings were performed with the program CANNY. Elasto-plastic and Takeda's hysteretic rules with tri-linear backbone curve were employed for the steel and RC buildings, respectively. The damping matrix was proportional to the mass matrix and the instantaneous stiffness matrix. The target damping was 5% in the first two modes.

MATHEMATICAL MODELS AND GROUND MOTIONS

Single-storey structures

The basic symmetric model **M,** which consists of three identical lateral load-resisting elements in each direction, is shown in Figure 1. The geometric data are as follows: $L_x / L_y = 1.5$, radius of inertia $r = 0.347\ L_x$. The mass is uniformly distributed at the slab level. The total mass is denoted by M. The x-direction is stiffer and stronger than the y-direction. The global stiffness and strength in the y-direction amount to K and F_y respectively, whereas in the x-direction they amount to 1.778 K and 1.5 F_y. The stiffness k and strength f_y of each element are equal to one third of the global stiffness and strength in the relevant direction. The numerical values used in the parametric study are $K = 4575$ kN/m, $M = 18.5$ kNs^2/m, and $F_y = 0.256\ Mg$, leading to yield displacements of $u_y = 0.86$ cm and $u_y = 1.02$ cm in x- and y- direction, respectively, and to periods of vibration equal to $T_x = 0.3$ s, $T_y = 0.4$ s, and $T_z = 0.254$ s, where T_z corresponds to torsional vibration. These periods suggest that the structure is torsionally stiff. The eccentricity amounts to 15% of the corresponding plan dimension in both directions.

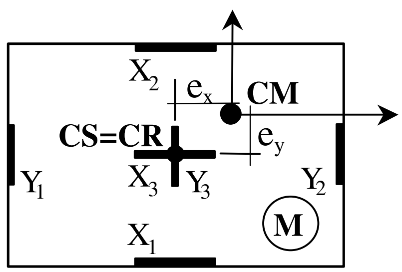

Figure 1: Plan of model M.

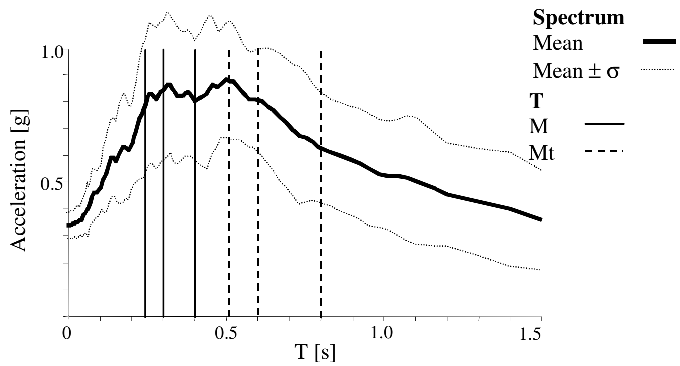

Figure 2: Mean and standard deviations of elastic spectra for 5% damping. Elastic periods for M and Mt models are indicated.

In addition to this model, a more flexible model **Mt**, whose initial uncoupled periods are equal to twice the initial uncoupled periods of the basic model, was also investigated. The increase in the periods was achieved by reducing the stiffness to one quarter of that of the basic model. In order to obtain a similar ductility demand as in the case of the basic model, strength was reduced by a factor of 1.5.

Two horizontal components of eight strong motion records (Sylmar and Newhall from Northridge 1994, Kobe J.M.A. from Kobe 1995, El Centro 1940, Petrovac, Ulcinj 1, Ulcinj 2, and Bar from Montenegro 1979) were used in the dynamic analyses. The records were normalised to the same peak ground acceleration in the horizontal plane a_g (the vectorial sum of both components). The ratio between the two components of each record was preserved. The mean spectrum and standard deviation of the set of 16 ground motion components are shown in Figure 2. Each pair of components was applied four times (all combinations of direction and senses were considered: in the second run the x- and y-directions were interchanged, and in the third and fourth runs the component oriented in the x-direction was multiplied by -1). So, altogether 32 time-histories were computed for each structural model.

Table 1. Peak ground acceleration, mean global ductility demand and periods based on secant stiffness to maximum displacement for M and Mt models.

Model→	Me	M – inelastic					Mte	Mt-inelastic				
a_g [g]	**0.1**	**0.2**	**0.3**	**0.4**	**0.8**	**1.0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.8**	**1.0**
$\mu_{mean,x}$	1.0	1.15	1.7	2.4	8.4	12.3	1.0	1.75	2.7	3.7	7.7	10.0
$\mu_{mean,y}$	1.0	1.7	2.9	4.6	12.0	16.0	1.0	1.8	2.9	3.9	8.0	10.6
T_x^* [s]	0.30	0.32	0.39	0.46	0.87	1.05	0.60	0.79	0.99	1.16	1.66	1.89
T_y^* [s]	0.40	0.52	0.68	0.86	1.38	1.60	0.80	1.08	1.35	1.57	2.26	2.61
T_z^* [s]	0.25	0.30	0.38	0.46	0.81	0.96	0.51	0.68	0.85	0.99	1.42	1.63

In order to investigate the influence of the ground motion intensity, which is related to the magnitude of the plastic deformations, the ground motion intensity was varied from $a_g = 0.1$ g to $a_g = 1.0$ g. Structures with elastic behaviour are denoted by **Me** and **Mte**. In Table 1, for different values of a_g the average ductility factors in both directions and the period determined based on the secant stiffness at maximum displacement of the symmetric system are listed. The initial elastic periods of the **M** model are on the "plateau" of the mean spectrum of ground motions, whereas all periods of the **Mt** model are in the descending range of the spectrum. Moreover, it can be seen from Table 1 that for the model **M** the ductility demand is much higher in the weak y-direction, whereas for the model **Mt** the ductility demands in both directions are approximately equal. This is due to the spectral shape in the relevant range.

The investigated systems are torsionally stiff and mass-eccentric. It has been shown (Peruš [11]) that the behaviour of a mass-eccentric system is very similar to the behaviour of a strength- and stiffness-eccentric system, in which strength and stiffness are linearly related. In real structures, strength and stiffness are strongly related.

Five-storey steel frame buildings

In the case of five-storey steel-frame buildings three different structural systems (S, F1, and F2) are employed. Schematic plans are shown in Figure 3, where moment-resistant frames are indicated by bold lines, and pin-connections by circles. The storey heights are 4.0 and 3.5 m for the first storey and for the other storeys, respectively. The symmetric structures S and F1 were designed according to pre-standards Eurocode 3 and 8 by Mazzolani and Piluso [14]. The design spectrum for stiff soil, normalised to a peak ground acceleration of 0.35 g, was used. A behaviour (reduction) factor of q = 6 was applied. A conservative estimate of the natural period was made, which resulted in a design base shear of about 10% of the total weight of the building. The detailed requirements of Eurocode 8, including the deformation limit, resulted in a large overstrength factor of about 2.7.

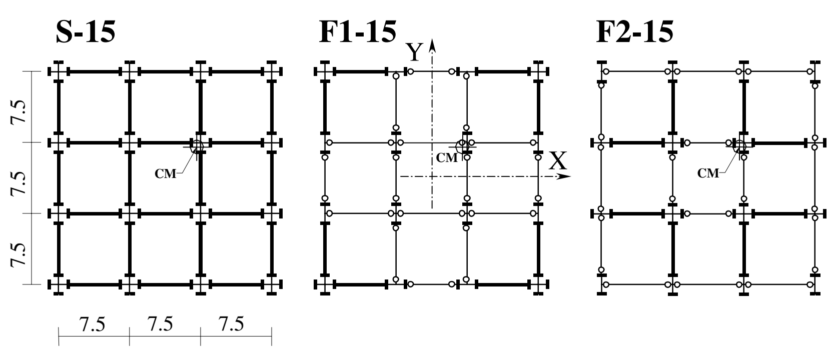

Figure 3. Schematic plans of the steel frame buildings with 15% eccentricity.

The main difference between the three systems lies in their torsional stiffness and strength. Buildings S and F1 are torsionally stiff (for both directions, X and Y). The first two modes are predominantly translational and the third mode is predominantly torsional. In structure S all the beam-to-column connections are moment-resistant. In structure F1 moment-resistant connections are only in the frames at the corner bays at the perimeter. The structure F2 consists of the same moment-resisting frames as those of structure F1, but they are located in the interior of the plan. In such a way, a torsionally flexible structure with a predominantly torsional first mode was created. Asymmetry was introduced by assuming different mass eccentricities, which amounted to 5, 10 and 15%, respectively, of the plan dimensions in both coordinate directions. The majority of results will be presented for 15% eccentricity. The structures with 15% eccentricity will be denoted as S-15, F1-15 and F2-15. The first three periods of vibration are listed for all buildings in Table 2. The second mode of vibration is purely translational in diagonal direction and therefore equal for the symmetric and asymmetric structure.

Table 2. The first three periods of vibration for symmetric and asymmetric variants of buildings S, F1 and F2.

	S				**F1**				**F2**			
e [%]	**0**	**5**	**10**	**15**	**0**	**5**	**10**	**15**	**0**	**5**	**10**	**15**
T_1 [s]	*1.25*	1.27	1.34	1.42	*1.26*	1.27	1.29	1.34	*2.13*	2.18	2.30	2.48
T_2 [s]	**1.25**	1.25	1.25	1.25	**1.26**	1.26	1.26	1.26	**1.26**	1.26	1.26	1.26
T_3 [s]	**0.96**	0.94	0.90	0.85	**0.73**	0.72	0.71	0.68	**1.26**	1.23	1.16	1.08

Dynamic analyses were performed using six different ground motion records (with 2 horizontal components each), which were obtained during the 1979 Montenegro earthquake (Bar, Ulcinj 1, Ulcinj 2), the 1994 Northridge earthquake (Newhall, Sylmar), and the 1995 Kobe earthquake (J.M.A.). For each record, the component with the higher peak ground velocity v_g was scaled to the same target value of v_g and applied in the Y-direction. Consequently, the Y-direction is effectively the "weak" direction, and the X-direction is the "strong" direction. Each accelerogram in the X-direction was scaled using the same factor as the corresponding accelerogram in the Y-direction. Excitation was applied simultaneously in both directions (X and Y). Elastic acceleration spectra for ground motions, normalised in the Y-direction to v_g = 40 cm/s, are shown in Figure 4.

In the study, different intensities of ground motion were simulated by scaling the ground motions to v_g = 10, 40, 70, 100, 160, and 250 cm/s. The structural response at v_g = 10 cm/s is elastic, and at v_g = 40 cm/s almost elastic. The magnitude of the deformation at v_g = 40 cm/s is, however, considerable (in the Y-direction more than 1% of the building height). Due to the very high strength of the structures, unrealistically high ground motion intensity is needed in order to produce substantial plastic deformations. The highest value used in the analyses amounts to v_g = 250 cm/s. The mean value of the top displacement (at the mass centre in the Y-direction) in the case of such ground motion amounts to about 6.5% and 7% of the height of the building S and F1, respectively, and the corresponding ductility factor amounts to about 6. Note that such extreme ground motion was used for the determination of trends. A more realistic analysis would require consideration of second order theory effects, and of possible strength deterioration. The global and local ductility factors for the asymmetric buildings are presented in Table 3. The ductilities for columns and beams correspond to maximum values throughout the structure.

Table 3. Mean values of maximum global and local ductilities for buildings S-15, F1-15 and F2-15 for different intensities of ground motion.

	S-15						F1-15						F2-15					
v_g [cm/s]	10	40	70	100	160	250	10	40	70	100	160	250	10	40	70	100	160	250
Global X	0.2	0.6	0.8	1.1	1.6	2.7	0.2	0.7	1.0	1.3	2.1	3.5	0.2	0.6	1.0	1.3	2.2	3.4
Global Y	0.3	0.9	1.4	1.9	3.4	5.8	0.3	1.0	1.6	2.1	3.7	6.2	0.3	1.0	1.7	2.6	4.9	8.7
Column	0.4	1.8	3.9	5.7	9.2	15.0	0.3	0.9	2.6	4.8	9.5	16.7	0.3	0.8	1.7	3.1	7.1	14.6
Beam	0.5	1.6	2.4	3.2	4.7	7.0	0.5	2.0	3.2	4.2	5.9	8.4	0.5	1.7	2.8	3.6	5.1	7.8

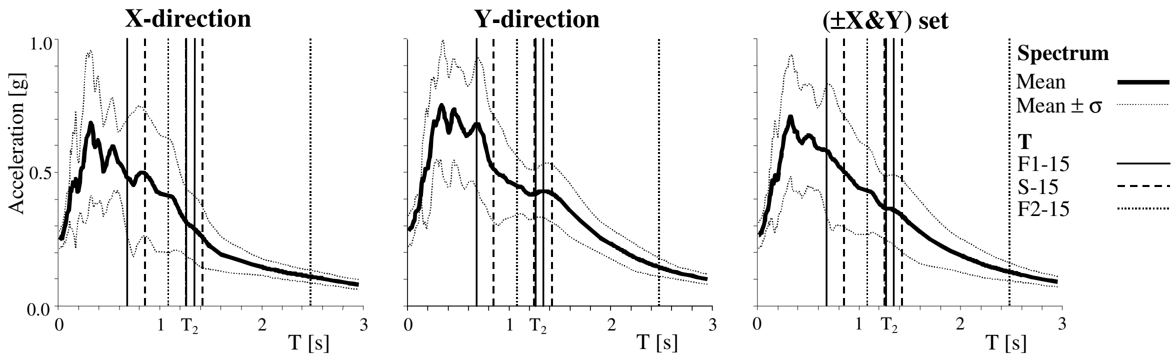

Figure 4. Elastic acceleration spectra for 5% damping (Y-dir. of the ground motion scaled to v_g = 40 cm/s); mean spectrum and standard deviation; the first three natural periods of vibration for structures F1-15, S-15 and F2-15 are indicated (lines for T_2 are overlapping).

Three-storey reinforced concrete (SPEAR) building

The analysed structure represents a typical older 3-storey building constructed in Mediterranean region (Figure 5). It was designed only for gravity loads, according to the concrete design code implemented in Greece between 1954 and 1995 with construction practice and materials used in Greece in early 1970's. Such a design was used for the test building in the SPEAR project [13]. In the mathematical model, the storey heights amount to 2.75 and 3.00 m for the first and upper stories, respectively. The eccentricities amount to about 10 % and 14 % in X- and Y-direction, respectively. The fundamental three periods of vibration of the building (considering some cracks due to gravity load), amount 0.59 s, 0.54 s, and 0.42 s. The first mode is predominantly in X-direction, the second predominantly in Y-direction, while the third mode is predominantly torsional.

In the parametric study bi-directional semi-artificial ground motion records were used. Horizontal components of seven recorded ground motions were fitted to EC8 design spectrum (Type 1C). The ground motions were scaled to different intensities defined by peak ground accelerations 0.05, 0.1, 0.2, 0.3, 0.5, 1.0, and 2.0 g in order to obtain results from practically elastic to different rates of inelastic response. The top displacements in the centre of mass and the global ductilities are very similar in both directions. When a_g = 0.3, 0.5 and 1.0 g, the mean top displacements in the centre of mass amount in the X-direction 1.2, 2.3, and 5.0% of the total height, and in the Y-direction 1.2, 2.5, and 5.3%, respectively. The global ductilities amount to about 2.8, 5.4 and 11.7 for X-direction, and 2.5, 5.2, and 11.2 for Y-direction, respectively. The ductilities were determined by assuming the yield point based on a bilinear idealisation of the pushover curve. The ground motion with a_g = 2.0 g is unrealistically high. It was applied in order to determine the trends.

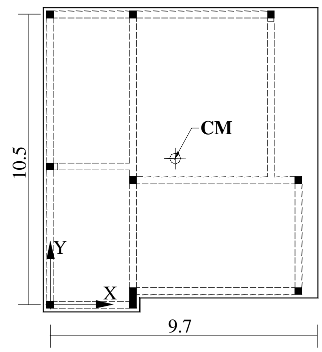

Figure 5. Schematic plan of the SPEAR structure.

RESULTS OF PARAMETRIC STUDIES

General

The results of the parametric studies indicate that the inelastic response of the investigated structures is in majority of cases qualitatively similar to the elastic response. Therefore it is important to understand the elastic structural behaviour under bi-directional ground motion. In many cases the inelastic response can be explained by phenomena observed in elastic range and considering the change of stiffness due to plastic deformations. In following subchapters, first some important effects, observed in elastic range, are presented. Then, the inelastic response is discussed. Results presented in sub-chapter Elastic response were obtained with modal analysis and idealised spectra. CQC combination rule was employed. Seismic loading was applied in each direction separately. The results for both directions were combined by the SRSS rule. All results presented in sub-chapter Inelastic response were obtained by time-history analysis and bi-directional ground motion.

Elastic response

Influence of the ratio between intensity of seismic excitation in two directions

Normalised displacements, which represent the torsional effects, presented in Figure 6, were obtained by applying reduced spectra in X-direction. The influence is small on the flexible side and substantial on the stiff side, especially in the direction of the smaller intensity, i.e. in X-direction. In X-direction, the normalised displacements on the stiff side increase with decreasing intensity of ground motion in X-direction, while in Y-direction they decrease. Consequently, an increase of the displacements at the stiff edge in the direction of the smaller seismic load can be expected. It is obvious that the rotation excited by ground motion in one direction influences the displacements in the orthogonal direction. The effect mentioned above is similar to the effect of different eccentricities in two orthogonal directions, where the increase of normalised displacements at the stiff edge in the less eccentric direction may occur.

Influence of the translational to torsional period ratio

The period ratios Ω_x and Ω_y, which are defined as the uncoupled translational period divided by the uncoupled torsional period in X- and Y- direction, respectively, have important influence on the torsional response. The smaller the period ratio, the larger is the influence of the predominantly torsional mode of vibration to the response in the direction considered (compared to the predominantly translational mode). It is usual to denote structures with period ratios larger than 1 as torsionally stiff and those with ratios smaller than 1 as torsionally flexible. Structures can be torsionally stiff in one direction and torsionally flexible in the other one.

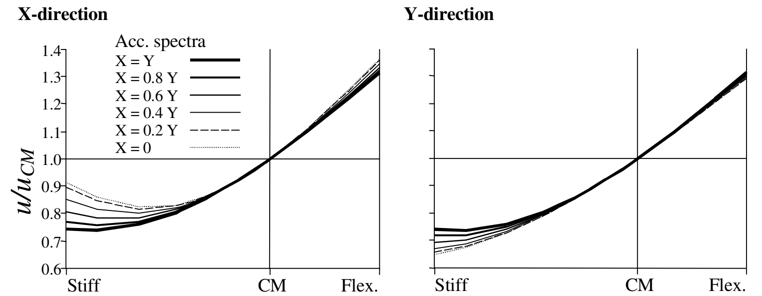

Figure 6. Normalised displacements in the horizontal plane at the top of the building S-15 (const. acceleration spectra).

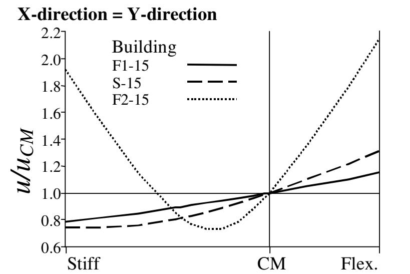

Figure 7. Normalised displacements in the horizontal plane at the top of the buildings F1-15, S-15, and F2-15.

Influence of the ratio between translational periods

Ratio between translational periods is related to the ratio between translational stiffnesses. In the more flexible direction the torsional effect on displacements is smaller than in the orthogonal (stiffer) direction. By decreasing only the translational stiffness in one direction (i.e. by increasing the uncoupled period of vibration for one direction, e.g. $T_y = 0.6$ s in Figure 8) while keeping the other two uncoupled periods unchanged, in general a decrease of the torsional effect on displacements in the considered direction is obtained. The situation is opposite in the case of decreasing uncoupled period of vibration (see Figure 8, $T_y = 0.3$ s). Note that this effect is always coupled with the effect related to the shape of response spectrum. In addition, by changing the stiffness of a structural element, the torsional stiffness (and the period ratios Ω_x and Ω_y) will probably change (depending on geometrical position of the element), too.

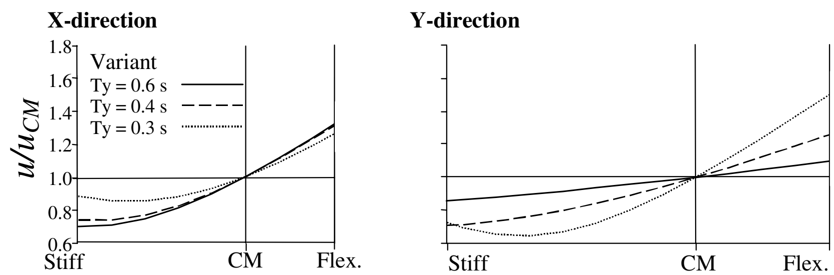

Figure 8. Normalised displacements u/u_{CM} in the horizontal plane at the top of the model M (variation of translational period/stiffness in y-direction, constant acceleration spectrum).

Influence of the response spectrum shape

The acceleration response spectrum shape may importantly influence the torsional response. A decreasing acceleration spectrum (i.e. proportional to $1/T$ or $1/T^2$) increases the influence of higher modes compared to the constant acceleration spectrum. Consequently, a decreasing acceleration spectrum, e.g. the medium-and-long-period part of the spectrum, increases the torsional effects in the case of torsionally stiff structures and reduces torsional effects in the case of torsionally flexible structures. Response obtained by using three idealised acceleration spectra is presented in Figure 9. For torsionally stiff structures, a very important influence of the spectral shape can be observed on the stiff side, while the influence on the flexible side is negligible.

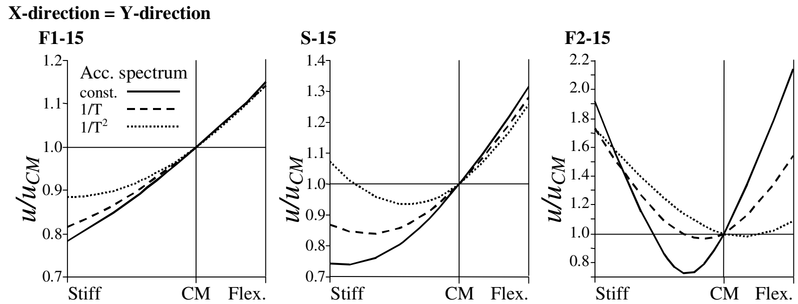

Figure 9. Normalised displacements in the horizontal plane at the top of the buildings. Acceleration response spectra: constant, proportional to 1/T and 1/T^2, respectively.

Inelastic response

During inelastic response plastic deformations occur and the parameters, important for torsional structural response, change. Due to the change in stiffness of individual structural members and of the whole structure, which is different in different directions, the periods and the period ratios change and the influences of the different vibration modes on the response may change accordingly. The eccentricities are also changing. Consequently, the effects observed in the case of elastic response and discussed in previous sub-chapter may be somewhat modified. In this sub-chapter the main results obtained by inelastic time-history analyses of the test structures will be presented and discussed. The test examples represent very different structural systems, which can be characterised as follows:

- Single-storey buildings: torsionally stiff; different stiffness and strength (larger in X-direction), and equal eccentricities and loading in two horizontal directions; initial periods of models **M** and **Mt** in the flat and decreasing part of the acceleration spectrum, respectively; elasto-plastic hysteretic rules.
- Multi-storey steel buildings: S and F1 torsionally stiff, F2 torsionally flexible; equal structural characteristics in both horizontal directions: larger loading in Y-direction and, additionally, equal loading in both horizontal directions; initial periods in the decreasing part of the acceleration spectrum; elasto-plastic hysteretic rules.
- Multi-storey RC building: torsionally stiff; different stiffness, strength, eccentricity (all values are larger in Y direction) and equal loading in two horizontal directions; initial periods in the flat part of the acceleration spectrum; Takeda's hysteretic rules.

For each structure, results will be presented in terms of displacements normalised by the displacement in the mass centre (u/u_{CM}) for different ground motion intensities. One type of figures presents mean of maximum values of normalised displacements at the edges as a function of ground motion intensity and the other type presents the mean envelope of normalised top (roof) displacements in the horizontal plane.

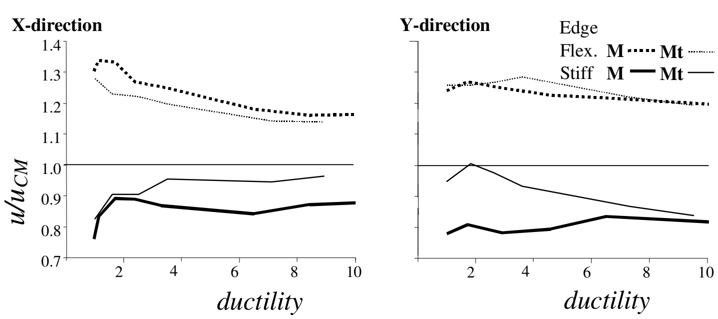

Figure 10. Influence of the global ductility demand on the normalised displacements at edges for single-storey models M in Mt.

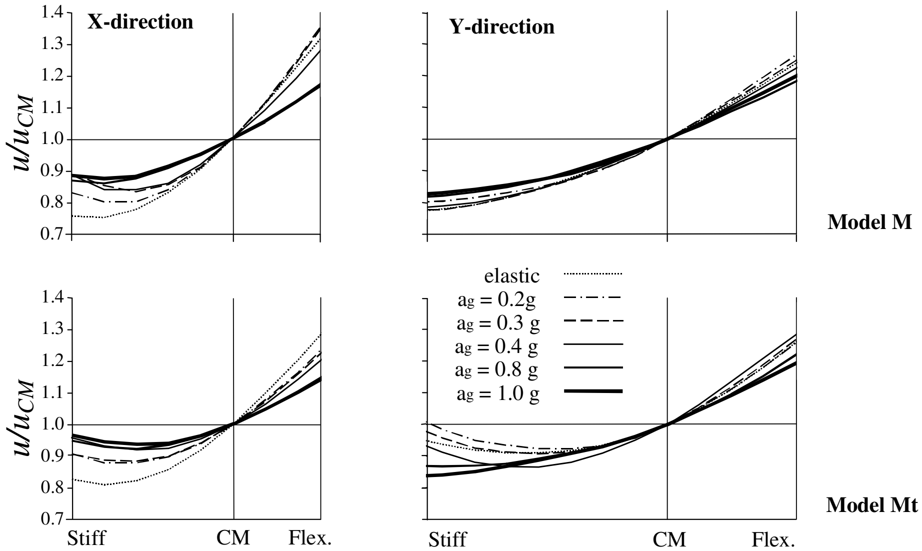

Figure 11. Envelopes of the normalised displacements for single-storey models M and Mt in the horizontal plane.

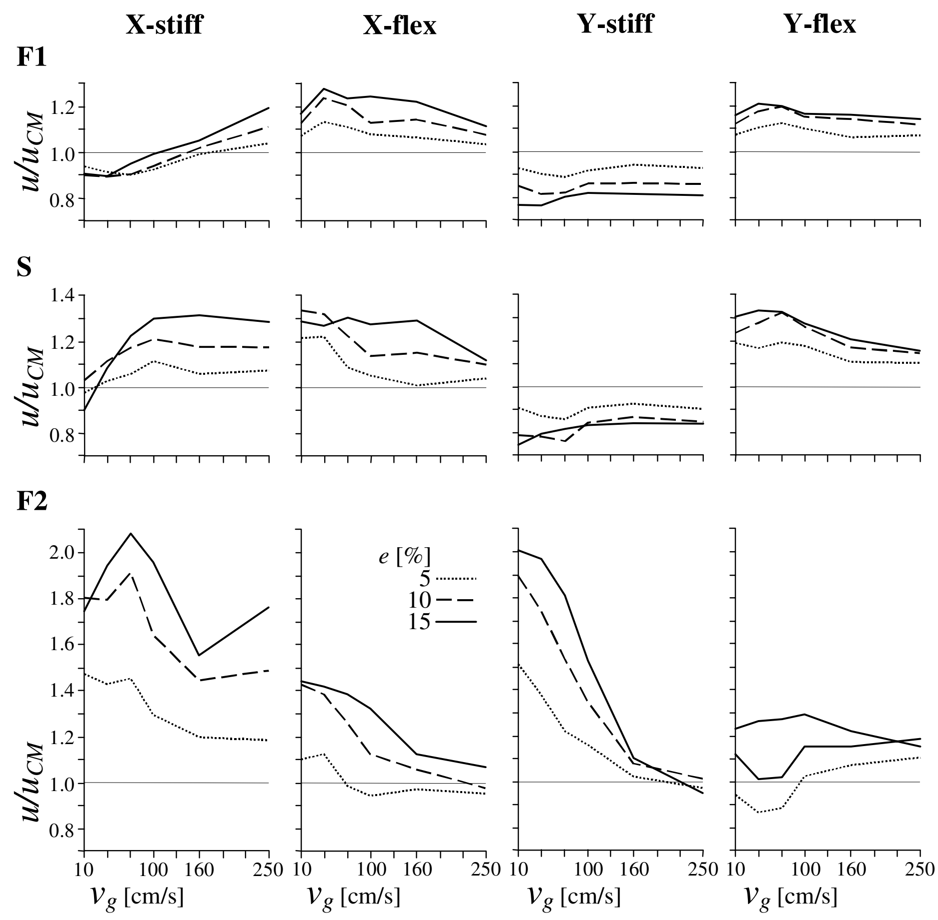

Figure 12. Influence of the ground motion intensity on the normalised displacements at edges for multi-storey steel buildings with different eccentricities

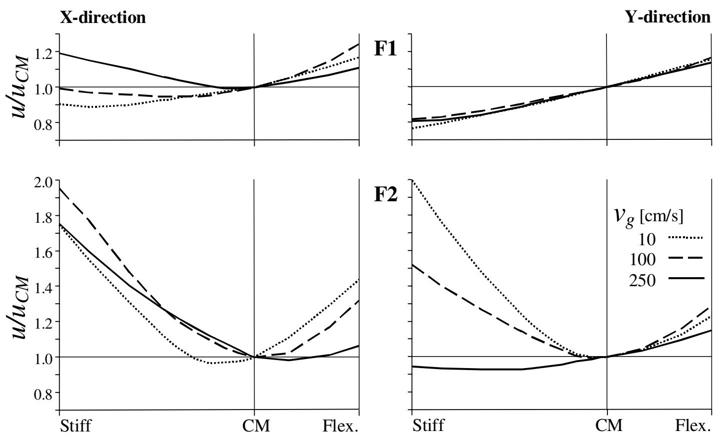

Figure 13. Envelopes of the normalised displacements at the top of F1-15 and F2-15 in the horizontal plane. (Results for S-15 are presented in Figure 15.)

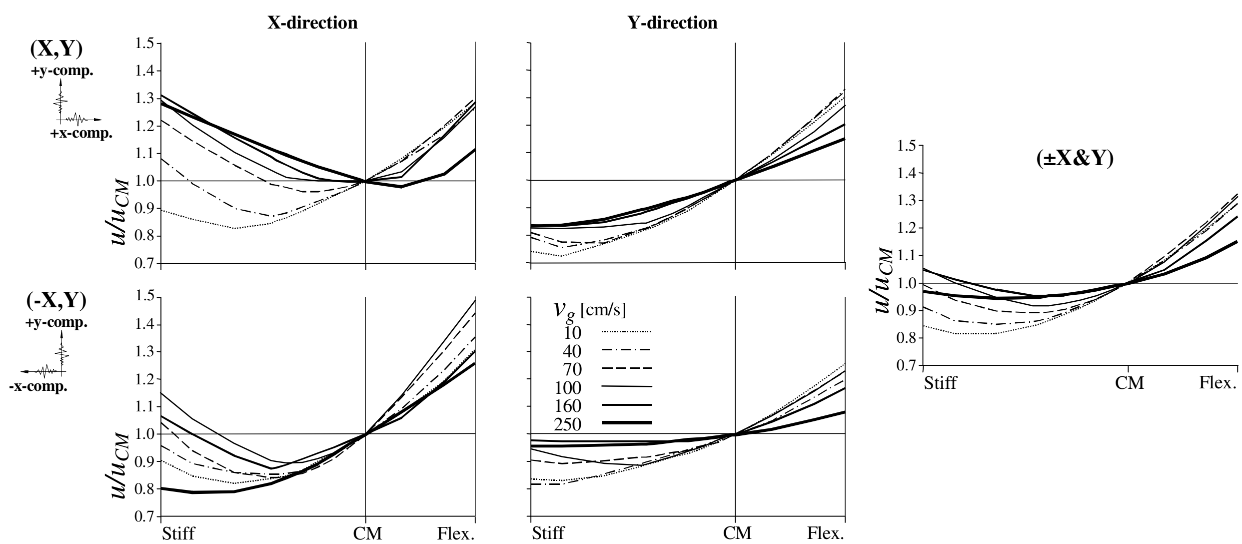

Figure 14. Envelopes of the normalised displacements at the top of S-15 in the horizontal plane. Influence of the ground motion set.

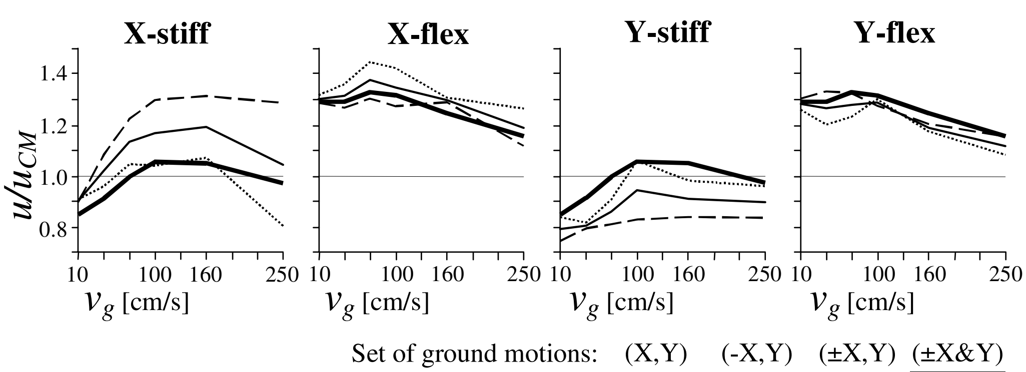

Figure 15. Influence of the ground motion intensity on the normalised displacements at the edges of S-15 for four sets of ground motions.

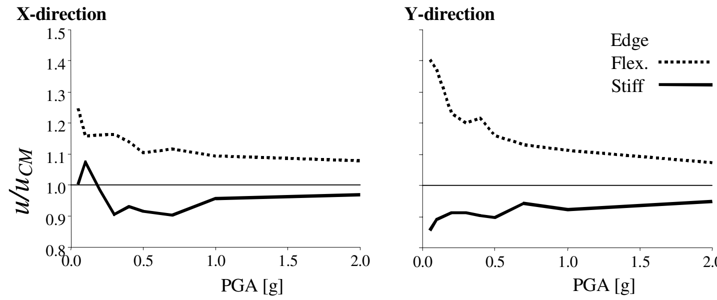

Figure 16. Influence of the ground motion intensity on the normalised top displacements at the edges of the multi-storey RC (SPEAR) building.

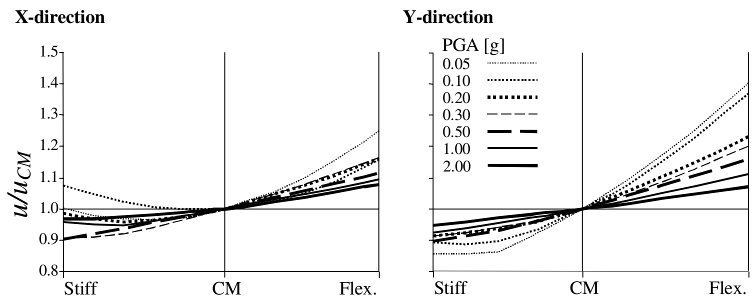

Figure 17. Envelopes of the normalised displacements at the top of the multi-storey RC (SPEAR) building in the horizontal plane.

The envelopes of normalised displacements in the horizontal plane generally flatten with increasing plastic deformations, indicating that the influence of torsional modes of vibrations and of the ground motion in the orthogonal direction, i.e. the torsional effects, generally decrease in inelastic range. The flattening is more pronounced in the "weak" direction. "Weak" is the direction which experiences larger plastic deformations. It is characterised by smaller stiffness and strength or by larger intensity of ground motion in the relevant period range or by a combination of these characteristics. The magnitude of eccentricity exhibits also an influence.

On the flexible side of torsionally stiff buildings a consistent trend can be observed. In all cases normalised displacements slightly to moderately decrease with increasing intensity of ground motion, i.e. with increasing plastic deformations. A small exception may occur in the case of low global ductilities of less than 2. Namely, in the case of small to moderate plastic deformation the eccentricity may substantially increase due to yielding on the flexible sides, whereas the elements on the stiff sides remain elastic. As a consequence, the maximum normalised torsional rotations may be larger than in completely elastic structures or in structures in which all elements yield (Tso [15]).

The stiff side of torsionally stiff buildings is less predictable. On the contrary to the flexible side, the response on the stiff side generally strongly depends on the effect of several modes of vibration and on the influence of the ground motion in the transversal direction. These influences depend on the structural and ground motion characteristics in both directions. In elastic range are the normalised displacements at stiff edges in all cases smaller than 1 or very near to 1. Plastic deformations may substantially change the torsional effects at the stiff edges. This observation is consistent with the results presented in the previous sub-chapter for elastic systems, which indicate that the influence of the change of ground motion and structural parameters is much larger at the stiff edge than at the flexible edge. In the majority of cases, the normalised displacements increase with increasing plastic deformations. The reduction of displacements on the stiff side due to torsion decreases with increasing plastic deformations. In the case of buildings F1 and S in X-direction (Figures 13 and 14) even an amplification of displacement occurs at the stiff edge, which is typical for torsionally flexible structures. In these buildings, X-direction is the "strong" direction, characterised by smaller plastic deformations due to smaller ground motion intensity. In some cases, some individual results deviate from the prevalent trend, especially results for the highest intensity in the case of steel buildings ($v_s = 250$ cm/s). This deviations may be a consequence of a relatively small number of ground motions used in statistical studies. Only for the **Mt** model in Y-direction, the trend is opposite to the prevalent trend (Figure 11). The reason for such behaviour is a combination of the effects of the response spectrum shape (note the increase of periods from model **M** to the model **Mt**, Table 1), of the contribution of ground motion in transversal direction, and of the flattening effect in inelastic range.

Only one example of a torsionally flexible building (F2) was investigated. The results obtained for this building suggest that torsional effects decrease with increasing plastic deformations substantially. In the case of very large plastic deformations, the building behaves in the "weak" Y-direction as a torsionally stiff building.

The results presented elsewhere (Peruš [11], Marušić [12]) indicate that the dispersion of results in inelastic range is larger than in elastic range, and at the stiff edge is larger than at the flexible edge. An illustration is presented in Figures 14 and 15, where results for different sets of ground motions are shown. The first set (X,Y) is the basic set of ground motions. The second set (−X,Y) consists of the same six ground motions. However, the X-, i.e. the weaker horizontal component, is multiplied by –1. The third set (±X,Y) represents the mean of the results obtained for the first and the second set. The fourth set (±X&Y) corresponds to all four combinations of directions and senses of two components of each record. For the fourth set, the results for X- and Y-direction are the same. The influence of the intensity of ground motion on the normalised displacements is qualitatively similar in all four cases. However, quantitatively there is a substantial difference at the stiff edges. Note the large difference which results from the change of the sense of one ground motion component. This difference suggests that the seismic response of two adjoining identical asymmetric buildings, located symmetrically with respect to one axis, may be substantially different. A similar effect has also been noticed by Rutenberg [4].

CONCLUSIONS

A large number of structural and ground motion parameters influence the inelastic torsional response of building structures. Although extensive parametric studies have been performed (only some of them are presented in this paper), it was possible to explore only a very limited number of cases. Nevertheless, several combinations of input parameters have been investigated and some consistent trends have been observed.

Based on the results of these studies it can be concluded that, in general, inelastic torsional response is qualitatively similar to elastic torsional response and that, quantitatively, the torsional effects generally decrease with increasing plastic deformations. A decrease of torsional effect is manifested mainly in a decrease of amplification of displacements due to torsion on the flexible side. An exception to this rule may occur if structures are subjected to small plastic deformations, characterised by a ductility of less than 2. In such a case the amplification may be slightly higher than in elastic structures.

For the stiff side, it is difficult to make general conclusions. The response on the stiff side generally strongly depends on the effect of several modes of vibration and on the influence of the ground motion in the transversal direction. These influences depend on the structural and ground motion characteristics in both directions. Reduction of displacements due to torsion, typical for elastic torsionally stiff structures, usually decreases with increasing plastic deformations. In some cases even a transition from de-amplification to amplification may occur. On the other hand, the amplification typical for elastic torsionally flexible structures usually decreases with increasing plastic deformations.

As an additional effect of large plastic deformations, a flattening of the displacement envelopes in the horizontal plane usually occurs, indicating that torsional effects in inelastic range are generally smaller than in elastic range. The flattening is more pronounced in the “weak” direction.

The dispersion of results has been only briefly discussed in this paper. However, based on results presented elsewhere, the dispersion is generally larger in the inelastic range than in the elastic one.

Based on the results of the studies reported in this paper and elsewhere the following conclusions relevant for the development of simplified analysis methods and code procedures can be drawn:

- The amplification of displacements determined by elastic analysis can be used as a rough estimate also in the inelastic range.
- Any favourable torsional effect on the stiff side, i.e. any reduction of displacements compared to the counterpart symmetric building, which may arise from elastic analysis, will probably decrease or may even disappear in the inelastic range.

The above conclusions are limited to fairly regular buildings and subject to further investigations.

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