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DUCTILITY CHARACTERISTICS OF BRACED STEEL FRAMES DESIGNED ACCORDING TO THE EGYPTIAN CODE

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ABSTRACT

In this paper, the ductility characteristics of low-rise braced steel buildings were evaluated using static pushover analysis. The low-rise braced steel buildings were represented by a 6-story braced steel frame which was designed according to the Egyptian code. Three different types of concentric brace configurations as well as one eccentric brace configuration were considered in this study. The concentric brace cases include regular-X, split-X and inverted-V configurations. The analysis was performed using the DRAIN-2DX computer program for non-linear analysis of building structures. The brace elements were modelled using a buckling element which is capable of representing the post-buckling behaviour of brace elements. The shear links where modelled using link elements capable of representing the shear and flexural behaviour of the shear links. The ductility characteristics of the braced frames were evaluated using a static push-over analysis. The distribution of global deformations of the frames as represented by the roof drift ratios and the story drift ratios were investigated. Local deformations in the frame elements as represented by axial displacements in the brace elements and the distortional angles of the shear link elements were evaluated. The results indicated that the eccentric bracing exhibits the best deformability among all the bracing cases. Concentric bracing cases designed according to a strength approach showed poor performance. It was found that some ductility requirements have to be considered in the design of concentric bracing in order to ensure satisfactory performance under the effect of lateral loading. The regular-X brace configuration exhibits the best performance among the concentric bracing cases.

Keywords

Braced Steel Frames; Buckling; Concentric Bracing; Ductility; Eccentric Bracing; Non-linear Analysis

1. INTRODUCTION

Braced steel frames have been proved to be cost-efficient lateral load resistance systems for multi-story steel buildings subjected to wind and earthquakes. They are capable of providing multi-story steel buildings with sufficient strength and stiffness in order to limit lateral deformations within acceptable limits (Jain and Goel [1] and Jain et al. [2]). The design of the bracing systems is a challenging task, because it involves a large number of possibilities for the arrangement of the bracing members. The selection of the bracing systems is usually undertaken by the designer based on a trial-and-error process and previous experience.

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Steel bracing systems used in constructing multi-story steel buildings are classified into two main types; concentric bracing and eccentric bracing. Concentrically braced frames (*CBFs*) are not expected to exhibit ductile performance in earthquakes as moment-resisting steel frames. This is reflected in the fact that response modification factors, *R*, for concentrically braced frames are generally less than those for moment resisting frames (UBC [3] and NBCC [4]). The reasons for the lower performance expected for *CBFs* are the possibility of poor hysteretic behaviour under lateral loads due to buckling of the bracing members which may lead to the development of a soft-story mechanism. The buckling behaviour of the brace members when subjected to lateral loading is characterized by pinching and softening of the brace hysteretic loops which are often considered less desirable than full hysteretic loops of ductile systems such as moment resisting frames [5].

Eccentrically braced steel frames (*EBFs*) are expected to exhibit ductile performance under the effect of earthquake loading [6]. The *EBFs* are structural systems in which eccentricities are deliberately introduced into the bracing configurations. The axial forces of the bracing members are transferred to the beams and columns through shear forces and bending moments developed in the eccentric links. Typically, the links are short and exhibit yielding in shear and therefore they are called shear links. Buckling of the bracing is prevented by designing the brace members to resist elastically the forces associated with the strengths of the links. The inelastic response of *EBFs* is dominated by the behaviour of their active links. A properly detailed link is capable of developing desirable full hysteretic loops like those of ductile moment resisting steel frames.

The evaluation of the seismic performance of newly designed structures in Egypt is required in order to determine the seismic level of protection afforded to these buildings by the new design provisions which have been introduced to the code as an as an implication of the 1992 Cairo earthquake. The ductility characteristics can be considered as one of the main factors that affect the seismic performance of building structures. The focus of this paper is on the evaluation of the ductility characteristics of low-rise braced steel frames designed according to the Egyptian code (Egyptian Code of Loads on Structures [7] and Egyptian Code of Design and Construction of Steel Structures [8]).

In this study, a 6-story braced steel frame has been designed according to the Egyptian code using both concentric and eccentric bracing. Various brace configurations including regular X-bracing, split-X bracing and inverted-V bracing were considered. A static push-over analysis was performed using the DRAIN-2DX computer program for non-linear dynamic analysis of building structures. The ductility characteristics of the frame have been evaluated in terms of the roof drifts, story drifts and the local deformations of the structural elements.

2. BUILDING DESIGN

The selected low-rise structure in this study is a 6-story office building. The floor plan of the building represents a three-bay by six-bay rectangular steel office building with bay width in both directions equals to 8 m. The story height for the building is 4.5 m for the ground floor and 3.6 m for the other floors. The lateral load resisting system of the building consists of perimeter braced steel frames. Only, the two perimeter braced frames in the short direction were designed and analyzed in the current study.

The structural steel is grade 52 in accordance with the Egyptian code [8]. The floor is considered to be made of reinforced concrete and is assumed to provide bracing against lateral buckling of the steel beams. The out-of-plane bracing of columns was assumed at both ends, so the effective length factors about major and minor axes were taken equal to 1.0.

The design dead loads include 2.5 Kpa weight of concrete slab, 1.5 Kpa covering load and 1.5 Kpa equivalent distributed load of the interior wall system. The design live, wind and earthquake loads were taken as those suggested by the Egyptian code [7]. The frame is designed for critical combinations of gravity, seismic and wind loadings based on the Egyptian Code [8].

Four design cases, shown in Fig. 1, are considered in this study. The first is a concentric regular-X bracing case (*CRX*), the second is a concentric split-X bracing case (*CSX*) and the third is a concentric inverted-V bracing case (*CIV*). The fourth design case is an inverted-V eccentric bracing case (*EIV*).

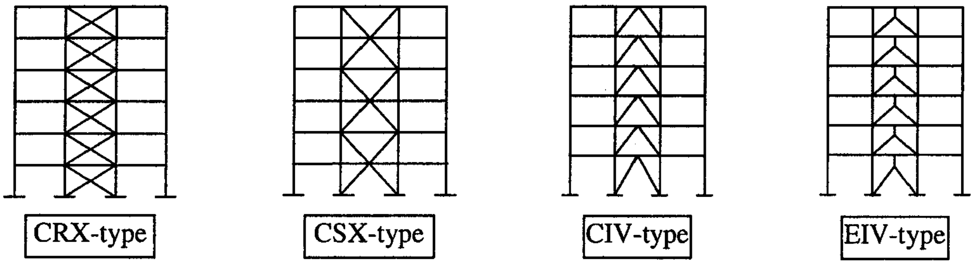

Fig. 1: Types of braced frames considered in the study.

For the regular-X bracing type, the buckling length of the brace members is taken equal to *0.5L* for in-plane buckling and *0.7L* for out-of-plane buckling [8]. For the split-X and inverted-V bracing cases, the buckling length of the brace members is considered equal to *1.0L* for in-plane and out-of-plane buckling [8].

In the eccentric bracing case, the steel brace members are designed as compression members with their axial strengths equal to 1.5 the axial forces associated with the strengths of the links [10]. The brace members are assumed to be pin-connected to the vertical link while the link itself is considered fixed to the beam. Equation 1, which was proposed by Popov and Malley [11], is used in calculating the link length to ensure that it yields primarily in shear.

$$e_{cri} = \frac{4.0\, b_f t_f}{t_w} \tag{1}$$

In equation 1, b_f and t_f are the width and thickness of the flange and t_w is the web thickness. The previous equation is for steel links having fixed ends with reverse curvature and equal end moments. Vertical shear links that have fixed connections with the beam and simple connections with the brace members are assumed to act as cantilevers; therefore e_{cri} in the previous equation is divided by two.The design information concerning the braced frames is presented in Table 1.

3. COMPUTER MODELS

A static push-over analysis was performed using the DRAIN-2DX computer program for non-linear dynamic analysis of building structures [9]. The beams and columns of the braced frames are modelled using the beam-column model of the DRAIN-2DX computer program.

The Jain and Goel brace model [5] was chosen to model the inelastic buckling behaviour of steel braces. The model is able to mimic the behaviour of bracing members with effective slenderness ratio varying from 40 to 120. The main parameters that govern the hysteretic rules of the bracing model, shown in Fig. 2, are the yield load, P_y, the initial buckling load, P_c, the residual buckling load, P_r, and the effective length of the bracing member, *KL*. The initial buckling load, P_c, is calculated using the provisions of the Egyptian code [8]. The residual buckling load, P_r, is estimated using an equation proposed by Lee and Goel [12] for calculating the residual buckling load of tubular sections.

Steel links are subjected to high levels of shear forces and bending moments and therefore elastic and inelastic deformations of both the shear and flexural behaviour have to be taken into consideration. Ricles and Popov [13] modelled the link as an elastic beam element with nonlinear rotational and translational springs at each end. The flexural inelastic behaviour of the link was represented by the multilinear function shown in Fig. 3.a, while the inelastic shear behaviour of the link web was represented by the multilinear function shown in Fig. 3.b. The model of Ricles and Popov was implemented in the DRAIN-2DX computer program and was used in modelling shear links of the eccentrically braced frame. The hysteretic behaviour of the shear link element is shown in Fig. 3.c.

Table 1: Design information of braced frames.

Type	Story	Conventional Strength Design			Bracing	Modified Ductile Design		
		Beams	Columns			Beams	Columns	
			Exterior	Interior			Exterior	Interior
CRX	1	IPE 300	HEA 240	HEA 550	HSS 203*152*6.4	IPE 300	HEA 240	HEA 650
	2	IPE 300	HEA 160	HEA 280	HSS 178*178*6.4	IPE 300	HEA 160	HEA 320
	3	IPE 330	HEA 160	HEA 280	HSS 178*127*8.0	IPE 330	HEA 160	HEA 320
	4	IPE 330	HEA 140	HEA 220	HSS 178*127*6.4	IPE 330	HEA 140	HEA 240
	5	IPE 330	HEA 140	HEA 220	HSS 152*102*9.5	IPE 330	HEA 140	HEA 240
	6	IPE 330	HEA 140	HEA 220	HSS 152*102*4.8	IPE 330	HEA 140	HEA 240
CSX	1	IPE 300	HEA 200	HEA 340	HSS 178*178*9.5	IPE 300	HEA 200	HEA 450
	2	IPE 300	HEA 160	HEA 280	HSS 178*178*6.4	IPE 300	HEA 160	HEA 320
	3	IPE 300	HEA 160	HEA 280	HSS 152*152*9.5	IPE 300	HEA 160	HEA 320
	4	IPE 330	HEA 140	HEA 220	HSS 152*152*8.0	IPE 330	HEA 140	HEA 240
	5	IPE 330	HEA 140	HEA 220	HSS 152*152*4.8	IPE 330	HEA 140	HEA 240
	6	IPE 330	HEA 140	HEA 220	HSS 127*127*4.8	IPE 330	HEA 140	HEA 240
CIV	1	IPE 360	HEA 200	HEA 340	HSS 178*178*9.5	IPE 450	HEA 200	HEA 550
	2	IPE 360	HEA 160	HEA 260	HSS 178*178*6.4	IPE 450	HEA 160	HEA 320
	3	IPE 360	HEA 160	HEA 260	HSS 152*152*9.5	IPE 450	HEA 160	HEA 320
	4	IPE 360	HEA 140	HEA 200	HSS 152*152*8.0	IPE 400	HEA 140	HEA 240
	5	IPE 360	HEA 140	HEA 200	HSS 152*152*4.8	IPE 400	HEA 140	HEA 240
	6	IPE 330	HEA 140	HEA 200	HSS 127*127*4.8	IPE 360	HEA 140	HEA 240
EIV	1	-	-	-	HSS 152*152*4.8	IPE 450	HEA 220	HEA 450
	2	-	-	-	HSS 152*152*4.8	IPE 450	HEA 160	HEA 260
	3	-	-	-	HSS 152*152*13	IPE 400	HEA 160	HEA 260
	4	-	-	-	HSS 152*152*9.5	IPE 360	HEA 140	HEA 200
	5	-	-	-	HSS 152*152*6.4	IPE 330	HEA 140	HEA 200
	6	-	-	-	HSS 127*127*11	IPE 270	HEA 140	HEA 200

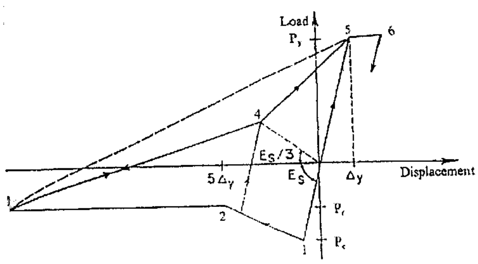

Fig. 2: The Jain and Goel brace model [5].

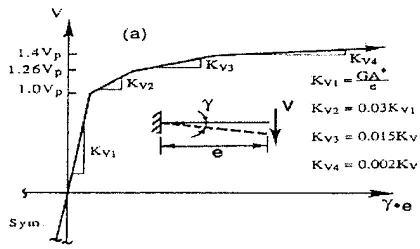

Fig. 3.a: The inelastic flexural behaviour of the link element [13].

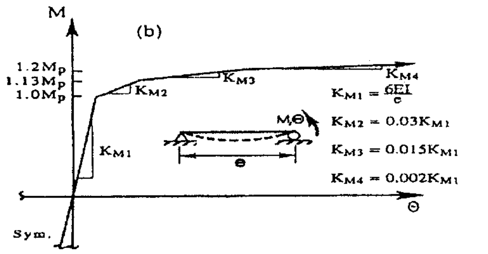

Fig. 3.b: The inelastic shear behaviour of the link element [13].

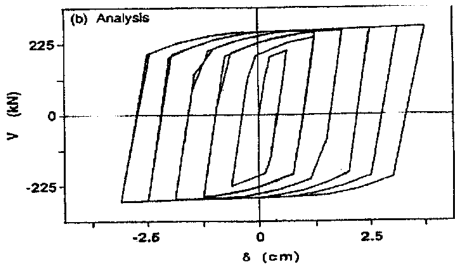

Fig. 3.c: The hysteretic behaviour of the shear link element [13].

4. PUSH-OVER ANALYSIS

Static pushover analysis can be considered as a viable tool to evaluate the deformability as well as the damage vulnerability of existing and newly designed building structures [14]. The objective of the pushover analysis is to obtain estimates of global and local deformations which the structure is likely to undergo when subjected to an earthquake loading. These estimates of deformations can be used in evaluating the integrity and the ductility of the structural system.

Static pushover analysis is carried out by applying a static lateral load having the distribution pattern specified in the Egyptian code [7]. A displacement controlled analysis is conducted until the structure reaches a pre-determined high level of lateral deformations. The result of the pushover provides estimates of the structure lateral strength and lateral stiffness. Also, it provides information on the load-displacement relationships of the roof level and the various stories of the structure. The distribution of the story displacements obtained from the pushover represents an important parameter in the evaluation of the overall ductility of the structure. In addition, local deformation and forces of the structure elements can be obtained which are important in determining the critical elements in the structure.

Static push-over analyses were conducted for the braced frames *CRX*, *CSX*, *CIV* and *EIV* using the lateral load distribution presented in the Egyptian code [7]. The results obtained are described in details in the following subsections. The global response parameters considered in the evaluation of the braced frames are; the base shear coefficient, the roof drift ratio, the story drift ratio and the maximum story drift ratio, *MSDR*. The local response parameters considered in the evaluation are the normalized axial displacement of the brace member and the normalized deformation angle of the shear link.

The base shear coefficient is defined as the total lateral load acting on the braced frame divided by the dead weight considered in the frame seismic design. The seismic resistance of the buildings in the short direction is provided only by the two perimeter braced frames. This means that the base shear coefficient of the braced frame is equal to the total lateral load divided by half the building dead weight.

The roof drift ratio which is usually presented in percentage form is defined as the roof displacement divided by the building height. It represents an important parameter in determining the level of lateral deformation which the frame has experienced. The story drift ratio is defined as the story displacement divided by the story height. The story drift ratio is an important parameter in assessing the story damage due to lateral loading. The maximum story drift ratio, *MSDR*, is a key parameter in evaluating the global damage of the whole frame.

The brace normalized axial displacement is defined as the brace axial displacement divided by the brace yield axial displacement. The link normalized deformation angle is equal to the shear deformation angle divided by the yield shear deformation angle of the link element.

4.1 Roof Drift Response

The pushover analysis has been carried out for the three concentrically braced frames *CRX*, *CSX* and *CIV*. The relationships between the roof-drift ratios and the base-shear coefficients of the three frames are presented in Fig. 4. The analysis of these three frames has stopped at an early stage as shown in the figure. This is because the levels of axial forces in some of the frame columns have reached the buckling capacities of these columns. The analysis stopped at roof-drift ratios of 0.43%, 0.39% and 0.44% for the three frames *CRX*, *CSX* and *CIV*, respectively. Buckling of the frame columns at these early stages of deformations can be attributed to the fact that the brace hysteretic behaviour is unsymmetrical in tension and compression. Although, the frame design was based on the compression strengths of all the bracing members, some of braces under the effect of lateral loading will work in tension and their strengths can reach their tensile yield strengths. This causes a distribution of column axial forces differs substantially from those predicted using conventional design procedures. The design procedure presented in the Egyptian code is based on a strength approach with no requirements to avoid this type of undesirable behaviour.

The column cross sections of the concentrically braced frames *CRX*, *CSX* and *CIV* have been redesigned to account for the brace over-strengths. The added axial force at each joint connected with a brace member is equal to the difference between the strengths of the brace member in tension and in compression multiplied by *cosine* the angle between the diagonal brace and the column. Jain et al. [2] has suggested that the probability of all the braces to be overloaded simultaneously is low. They suggested that the loads in any column due to braces can be taken as the maximum loads induced at any level above the column considered, plus the square root of the sum of squares of all other brace-induced loads above that level. The modified column cross sections of the frames *CRX*, *CSX* and *CIV* are presented in Table 1. The responses of the frames after modification are obtained up to 1.0 % roof drift ratios and are shown in Fig. 5.

The ultimate base shear coefficients of the braced frames are 0.13, 0.11, 0.10 and 0.09 for the *CRX*, *CSX*, *CIV* and *EIV* frames, respectively. The ultimate base shear of the eccentric case was the lowest as compared to the concentric bracing cases. This may be attributed to the over strength of the braces working in tension which have been neglected in the brace design.

The three concentric bracing cases exhibit a softening in behavior after reaching the ultimate base shear levels which can be attributed to the buckling of the brace elements. On the other hand, the eccentric bracing case exhibits a stable behaviour during the total response.

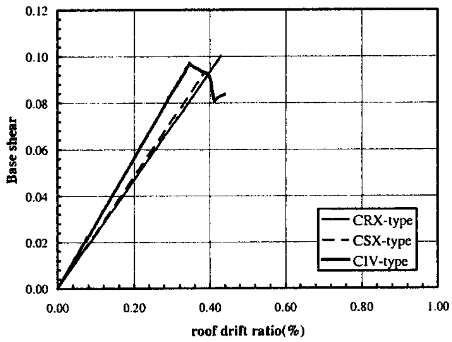

Fig. 4: Relationships between the roof drift ratios and the base shear coefficients of the frames *CRX*, *CSX* and *CIV* (Conventional strength design).

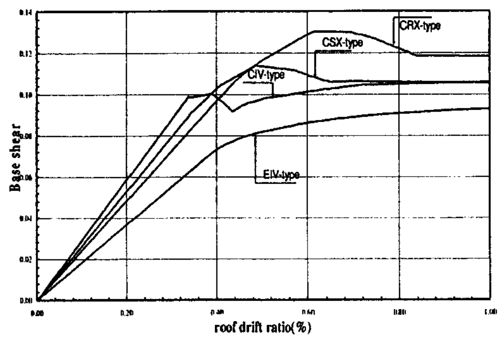

Fig. 5: Relationships between the roof drift ratios and the base shear coefficients of the designed frames (Modified ductile design).

4.2 Story-Drift Response

The story-drift ratio is an important performance parameter for evaluating the deformability of the braced steel frames. At a specific roof drift ratio, the high *MSDR* indicates local concentration of deformations and damage in one story of the frame while the low *MSDR* indicates a uniform distribution of deformation and damage among all the stories of the frame. In the four bracing cases studied, *MSDRs* corresponding to 1.0 % roof-drift ratio were found equal to 3.4%, 3.9%, 4.2% and 2.1% for the *CRX*, *CSX*, *CIV* and *EIV* frames, respectively. These results indicate that the eccentric bracing type is superior in its deformability compared with the concentric bracing cases. The results also indicate that the regular-X bracing type has the best deformability among all the concentric bracing cases.

The *MSDRs* occurred in the 2nd story in all types of concentric bracing cases. The distribution of story drift ratios along the frame stories corresponding to 0.5%, 1%, and 2% of *MSDRs* for the concentric bracing types are shown in Fig. 6. The figure shows that the story drift ratios tend to distribute uniformly among the frame stories in the early stages of the analysis (elastic stage). In the late stages of the analysis (inelastic stage) the story drifts tend to concentrate in the 2nd story of the frame leading to the development of a soft story mechanism. This may be attributed to the brace buckling behaviour which is characterized by strength softening.

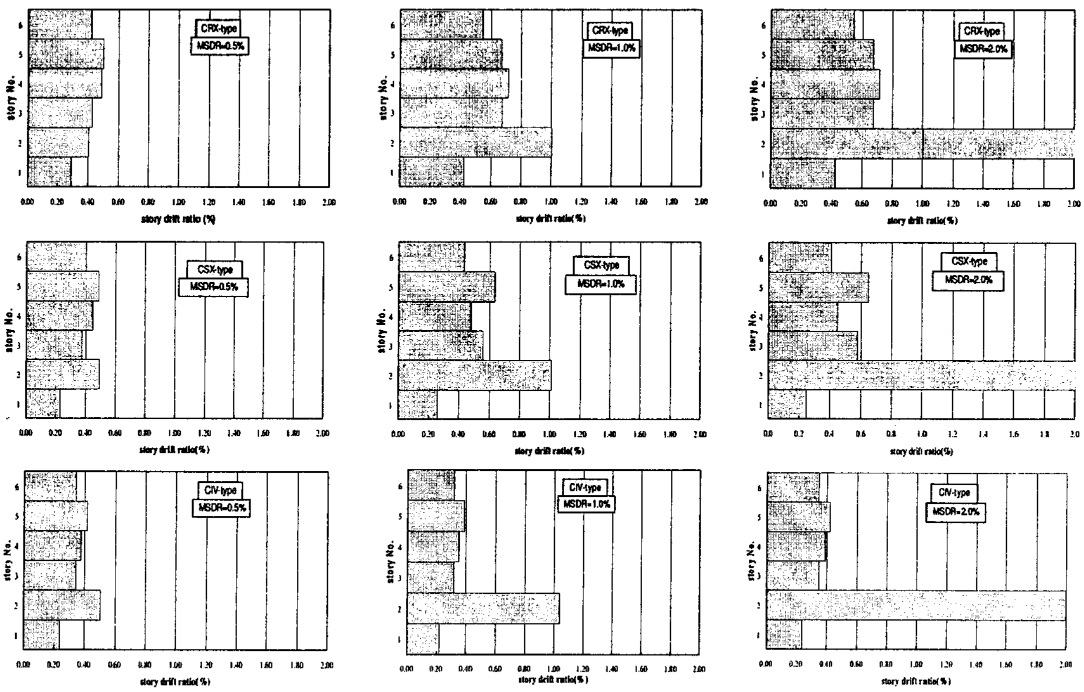

Fig. 6: The distribution of story drift ratios of the concentric bracing cases corresponding to *MSDRs* of 0.5%, 1% and 2%.

Fig. 7 shows the distribution of story drift ratios of the eccentric bracing type corresponding to MSDR of 0.5%, 1.0%, and 2.0%, respectively. The distribution tends to be better at all stages of the analysis than the distribution of the concentric bracing cases shown in Fig. 6. However, the maximum story drift was shifted from the 2nd story in case of concentric bracing types to the 4th story in the eccentric bracing case.

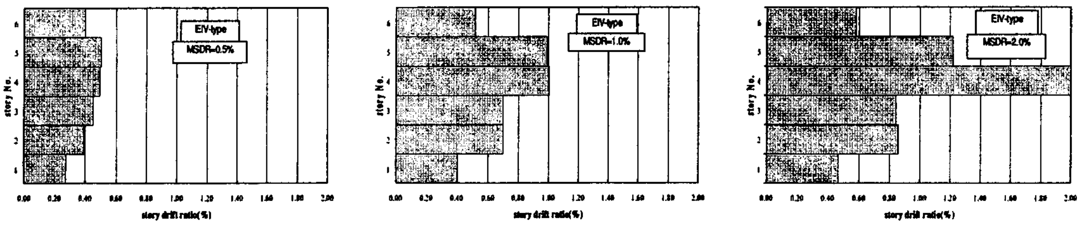

Fig. 7: The distribution of story drift ratios of the eccentric bracing case corresponding to *MSDRs* of 0.5%, 1% and 2%.

A plastic mechanism parameter (*PMP*) is introduced to evaluate the deformability (the distribution of story drift ratios) of the braced frames. This parameter can be calculated empirically by the following equation:

$$PMP = 1 - \frac{\sum_{i=1}^{i=n} \left| S_i - R \right|}{\left(\sum_{i=1}^{i=n} S_i \right) + (n-2)R} \tag{2}$$

where, S is the story drift ratio (in percentage), R is the roof drift ratio (in percentage), n is the number of stories and i represents the story number.

The case when *PMP* equals 1.0 represents a desirable state, where the story drifts are equal in all the stories. In this case, the *MSDR* and the frame damage are minimal. The case of *PMP* equals zero indicates an undesirable situation, where all the deformation is concentrated in only one story. In this case, *MSDR* and the frame damage are maxima.

Fig. 8 shows the relationship between the *PMP* and the roof drift ratio of all the designed cases. The value of the parameter *PMP* tends to deteriorate with the increase of inelastic deformations. The deterioration in the values of the parameter *PMP* is more obvious in the concentric bracing cases. This may be attributed to the brace buckling behaviour which is characterized by strength softening. The values of *PMP* at 1.0% roof drift ratio for the *CRX*, *CSX* and *CIV* frames were 0.55, 0.46 and 0.39, respectively. By considering the value of the parameter *PMP* at 1.0% roof drift ratio as an indicator of the deformability of the concentrically braced frames, the case *CRX* can be considered the best in deformability, while the case *CIV* has the worst deformability.

The deterioration of the value of the parameter *PMP* with the increase in the inelastic deformation is minimal in the eccentric bracing case *EIV*. This may be attributed to the stable inelastic responses of the shear links under the effect of lateral loading. The value of *PMP* for the eccentric bracing case was 0.75 at 1.0% roof drift ratio. This means that the case *EIV* can be ranked as the first in deformability among all the designed cases.

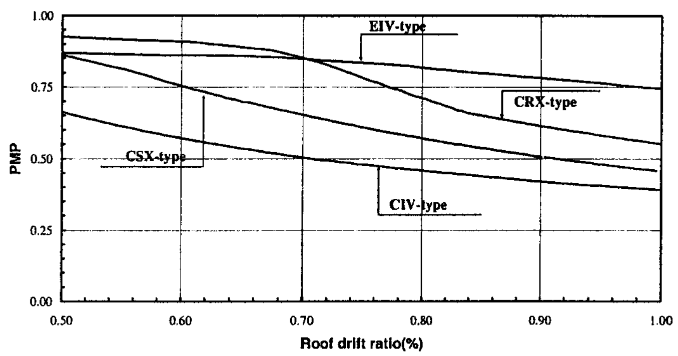

Fig. 8: The relationship between the *PMP* and the roof drift ratio of the designed frames.

4.3 Brace Response

The local deformability of the brace members was evaluated using the axial displacement ratio (*ADR*). The relationships between the *ADR* and the *MSDR* for the three concentric bracing cases are presented in Fig. 9. In tension, the levels of *ADR* were the highest for the *CRX* type and the lowest for the *CIV* type. While in compression, the levels of *ADR* were the lowest for the *CRX* type and the highest for the *CIV* type. This shows that the local ductility demands are minimal in case of the *CRX* type (*ADR*= 4.3 at 2% *MSDR*) and are maxima in case of the *CIV* type (*ADR*= 10.0 at 2% *MSDR*).

4.4 Shear Link Response

The local deformability of the shear link was evaluated using the link deformation angle, γ. Kasai and Popov [15] have stated that the ultimate deformation angles for the same link section and stiffener configuration are not sensitive to the loading routine. The ultimate link deformation angle is defined as the deformation angle before the occurrence of considerable strength deterioration due to severe flange and web buckling of the link. Michael and Popov [16] have found that the ultimate link deformation angle for well stiffened shear links may approach 0.1 rad.

The relationships between the γ and the *MSDR* for the eccentric bracing case are presented in Fig. 10. The *MSDR* corresponding to a deformation angle of 0.1 is equal to 1.98 %. This indicates that the deformability of eccentrically braced frames is controlled by the ultimate level of the link deformation angle. These results indicate that the ultimate level of the story drift ratio is 1.98 % for the eccentrically braced frame considered in the current study.

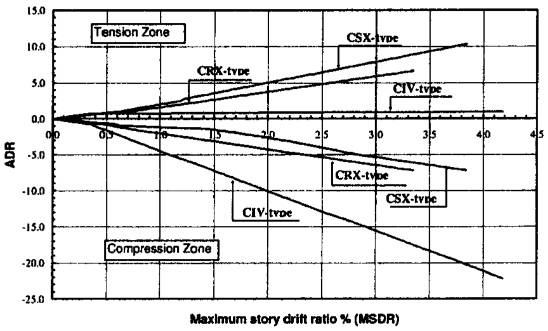

Fig. 9: The relationships between the *ADR* and the *MSDR* for concentric bracing cases.

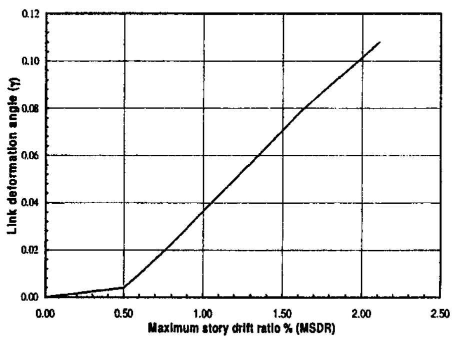

Fig. 10: The relationship between γ and the *MSDR* for eccentric bracing case.

5. CONCLUSIONS

From the results obtained, it can be concluded that:

1. Conventional design procedures based only on strength approach such as the one presented in the Egyptian code may provide concentrically braced frames with undesirable inelastic performance under the effect of lateral loading. Ductility requirements have to be considered in the design of concentrically braced frames to prevent nonductile modes of failure such as buckling of the frame columns due to brace over strength.
2. A parameter *PMP* was introduced to evaluate the global deformability of both the existing as well as the newly designed braced frames. The nearer is the *PMP* to 1.0, the better is the global deformability of the frame. The level of *PMP* at 1% roof drift ratio can be considered as an indicator of the frame deformability. The eccentric bracing case has the best deformability (*PMP*=0.75) among all the designed cases because of the stable behaviour of the shear links. The concentric regular-X bracing has the best deformability

among the concentric bracing types with *PMP*=0.55, while the concentric inverted-V bracing has the worst deformability with *PMP*=0.39.

3. For the concentrically braced frames, the local ductility demands of the brace members are minimal in case of the regular X-bracing type and are maxima for the inverted-V bracing type. For the eccentrically braced frames, the deformability is controlled by the ultimate level of the link deformation angle. In the current design case, the ultimate level of the story drift ratio is 1.98 %.

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