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Seismic performance evaluation of Egyptian code-designed steel moment resisting frames

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Abstract Despite the fact that steel is a ductile material, the significant damage, during earthquake events, highlighted the need to thoroughly investigate the seismic performance of steel structures. Seismic design procedures have been developed to enable structures to achieve specific acceptable level of damage under dynamic loads in accordance with particular levels of ductility. The ductility of steel moment resisting frames is developed through flexural yielding of beams, shear yielding of column panel zones, and flexural yielding of columns. Meanwhile, the frame must develop the required ductility without failure in the beam-to-column connection. The observations on panel zone behavior revealed that it can afford high ductility; however, localized deformations at corners of panel zone may increase the likelihood of fracture in vicinity of beam flange welds. On the other hand, the observations on flexural yielding behavior of columns reported potential soft story collapse. Consequently, counting on ductility due to shear yielding of panel zone and flexural yielding of columns is not recommended. Hence, the focus of this study has been directed toward flexural yielding of frame beams. The effect of beam profile slenderness (according to the Egyptian code design limits) has been examined against ductility, over-strength and redundancy with numerical assessment for the anticipated seismic force reduction factor. The reduction factor has been evaluated using both the N2-method and the time history analysis method. Accordingly, a guideline has been established for the Egyptian code provisions to enable professionals to assign the steel moment resisting frame between adequate-ductility, intermediate-ductility and limited-ductility.

Introduction

The Egyptian code provisions have been evaluated for the seismic design of steel moment-resisting frame buildings through parametric and comparative studies [1–7]. These studies have been performed with a variety of analysis methods and numerical models in order to assess the seismic performance of the steel moment-resisting frames, at either structure-scale or beam-to-column connection-scale [1–7]. The analysis

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procedures have been evaluated for their ability to predict the deformation demands in terms of inter-story drifts, potential failure mechanisms and story shear demands [5–7]. The seismic force reduction factor (R) has been one of the most important components in the development of seismic design provisions. This factor has been utilized in current design codes to reduce the linear elastic design response spectra to the design forces at the strength level. In other words, it represents the ratio of the strength required to maintain the structure elastic to the inelastic design strength [8]. The reduction factor (R) has been used since the 1950s and its values that were given by the American codes [8] had not been greatly modified. However, in the mid-1980s, it was proposed to divide the reduction factor into three components ($R = R_\mu$, R_S, R_R).

The ductility reduction factor (R_μ) reflects the ability of a structure to dissipate hysteretic energy through plastic deformations. It is the reduction in strength demand due to nonlinear hysteretic behavior. It is considered the main component of the reduction factor. The ductility (μ) represents the ability of the structure to undergo large plastic deformations without significant loss of strength, representing the ratio between the ultimate deformation at an assumed collapse point and the yield deformation. On the other hand, the hysteretic energy is the energy dissipated by inelastic cyclic deformations. It is given by the area within the load–deformation curve. Newmark and Hall [9] presented formulas to relate the ductility reduction factor to the ductility of the system. These formulas have been the most well-known and widely used analytical method in seismic design of ductile structural systems. The proposed functional form is dependent on the natural period of the structure. In the long period range, elastic and ductile systems with the same initial stiffness have the same maximum displacement; accordingly, the equal displacement approach can be applied. In the short period range, elastic and ductile systems have the same seismic force; accordingly, the energy approach can be applied. The proposed formulas [9] are as follows: $R_\mu = 1$, for $T < 0.03$ s; $R_\mu = (2\mu - 1)^{0.5}$, for $0.12\text{ s} < T < 0.5$ s; and $R_\mu = \mu$, for $T > 1$ s.

The over-strength factor (R_S) accounts for the fact that the maximum lateral strength of a structure generally exceeds its design strength. The sources of structural over-strength are as follows: redistribution of internal forces in the inelastic range, difference between actual material strength and the strength used to calculate the capacity in design, strain hardening, effect of using oversized members, detailing, effect of considering multiple loading combinations, effect of minimum requirements on member sections to meet the stability and serviceability limits in design, effect of structural elements that are not considered in the prediction of lateral load capacity and the contribution of nonstructural elements [8]. Accordingly, the over-strength factor plays an important role in the survival of buildings during severe earthquakes. Uang [10] expressed the various sources contributing to the over-strength factor in the formula: ($\Omega = \Omega_o\ F_1\ F_2 \ldots F_n$), where Ω is the actual over-strength factor, Ω_o is the over-strength factor using the nominal material properties, and $F_1, F_2, \ldots F_n$ are factors that account for different sources contributing to the over-strength. Uang reported a value of 2 to account for over-strength based on the nominal material properties. In addition, a value of 1.05 was reported by Elingwood et al. [11] to account for the difference between actual and nominal yield strength; meanwhile, 10% increase was considered to account for the strain rate effect. Accordingly, the value of the over-strength factor can be taken as 2.3, where this value is considered conservative as it accounts only for three sources of the over-strength. This theoretical work was accompanied by experimental work using the shaking table testing of multistory reinforced concrete and steel structures, where over-strength of 2–3 has been reported for 4- to 12-story special steel moment resisting frames located in high-seismic region. Miranda and Bertero [12] showed that strength reduction factors specified by American codes [13] are much larger than those due to nonlinear hysteretic behavior for system undergoing displacement ductility ratios equal to four. Hence, Miranda and Bertero [12] inferred that structures designed according to these seismic provisions must possess a sufficient level of over-strength in order to avoid excessive inelastic deformations.

The redundancy factor (R_R) is related to the configuration of the structure. The main concept behind it is to allow the seismic load to be distributed over a number of load-resisting components, which in turn shall decrease the possibility of failure of all components at the same time [8]. Uang [10] reported that the allowable stress factor is used to account for the differences between allowable stress and ultimate design philosophies, the ratio between the first significant yield level and the design force level. Hence, this factor equal to 1.0 when the ultimate design is used. On the other hand, it has been reported that the redundancy factor ranges between 1.4 and 1.5 [10,14] for the allowable stress design. According to AISC-1989 ASD specifications [15], the value for the redundancy factor can be estimated to be 1.58, where the average allowable stress of 60% of the nominal yield stress has been increased by twenty percent, as permitted by the specifications, and a shape factor of 1.14 is assumed for the wide-flange sections according to the formula: [$R_R = 1.14/(0.6 \times 6/5) = 1.58$].

Steel had been considered an excellent material with respect to ductility, where it was recommended for seismic areas [16,17]. For steel moment resisting frames, the main source of ductility is the rotation capacity of plastic hinges [18,19]. Beams and columns with moment resisting connections resist lateral forces by flexure and shear in beams and columns. The ductility is then developed through: (1) flexural yielding of beams; (2) shear yielding of column panel zones, and (3) flexural yielding of columns. Accordingly, there are three possible locations for plastic hinges formation at: (1) beam ends; (2) panel zone; and (3) column ends, where the required ductility should be developed without failure in the beam-to-column connection (bolts and welds) [20]. The observations on panel zone behavior revealed that it can afford high ductility [21]. However, three concerns have been reported [21] as follows: (1) the localized deformations "kinking" at corners of panel zone may increase the likelihood of fracture in the vicinity of the beam flange groove welds; (2) the building code provisions have varied greatly on panel zone design; and (3) further research is needed to better define the acceptable level of panel zone yielding. Consequently, counting on panel zone ductility is not recommended; moreover, the current AISC seismic provisions [22] permit limited yielding at panel zone. On the other hand, the observations on flexural yielding behavior of columns reported potential soft story collapse [23]. Consequently, flexural yielding of columns should be avoided.

Accordingly, the focus of this study has been directed toward plastic hinge formation at beam ends. Modern design codes and recommendations for ductile moment resisting steel