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Fragility curves for typical steel frames with semi-rigid saddle connections

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ABSTRACT

Steel structures with “saddle” type beam–column connections represent a popular construction practice in Iran. In the present paper, the seismic performance and the vulnerability of such structures are investigated. The seismic responses of three-story and five-story structures are modeled according to the nonlinear incremental dynamic analysis (IDA) procedure considering various frame configurations. Typical frames representing unbraced frames with masonry infill walls, braced frames with concentric bracings and braced frames with masonry infill walls were studied under the effect of 44 real ground motion records. Fragility curves were developed for these structures and the probability of exceedance at immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance limits was assessed for two seismic hazard scenarios (475 and 2475 years return period). For the 2% probability of exceedance in 50 years case, the probability of exceeding the CP performance limit is 90%, 74% and 56% for the 3-story structure consisting of unbraced frames with masonry infill, braced frames with concentric braces and braced frames with masonry infill, respectively. The corresponding values for the 5-story models are 94%, 69% and 62%. These alarming high values suggest that the seismic retrofit of such existing structures is of crucial importance.

1. Introduction

Due to its relatively simplistic method of construction and its lower construction costs, frames with “saddle” connections have been regarded as one of the main construction practices in Iran. In such frames, continuous parallel beams (primary beams) cross and encase the columns, and the semi-rigid saddle connections are formed by two welded angle sections on the joint positioned at the top and the bottom of each and every beam. A typical configuration of the connection is shown in Fig. 1. The beams in the direction perpendicular to the plane of these frames are essentially secondary floor beams typically with about one meter spacing to support the brick vault type floors. These beams are typically welded into (or on the top of) the primary beams with hinge connection. Fig. 1 shows some useful details according to typical construction practice. Fig. 2a depicts the typical beam into beam connection.

Common lateral load systems of these buildings are regarded as masonry infill walls, braces or a combination of both. In past major earthquakes of Iran (e.g. Manjil (Mw. 7.4, 1990) and Bam (Mw. 6.5, 2003), these steel structures exhibited unacceptable seismic performance and in many cases resulted in the complete structural collapse (Fig. 2 [1]).

Analyzing the 2006 census data for Iran [2], a total of 82% of the housing units are recognized as masonry or steel constructions nationwide. 47.3% of such units have been categorized as low seismic resistant constructions. For Tehran, low quality steel or masonry housing units accounts for about 50% of the total houses. The key point is that a large number of vulnerable steel structures are those with typical saddle connections. The seismic vulnerability assessment of such structures seems crucial in assessing urban risk throughout the country.

Different studies have been carried out for understanding the seismic behavior and the proper design method of saddle type structures. Amiri et al. [3] studied the seismic performance of such structures using nonlinear static procedure and suggested a method to obtain rigid saddle connections. Their study showed that the enhanced system is capable of providing the “life safety” performance with relatively high safety margin. Moghadam [4] has studied the performance of such structures in the 1990 Manjil earthquake (Mw. 7.4) and provided recommendations for their retrofit. Hosseini Hashemi and Hassanzadeh [5] studied a steel saddle frame building with infill panels that were damaged in the Bam earthquake (Mw. 6.5, 2003) using nonlinear dynamic analysis. They have utilized the recommendations of FEMA-356 [6] for the evaluation process. The results showed good correspondence of the overall behavior of the nonlinear model with the observed response. It was concluded that in such complex system, infill panels contribute in preventing the structure from collapse and most of the energy was absorbed and dissipated by them. Mazrouei et al. [7] proposed a method to retrofit saddle connections by adding top and bottom flanges

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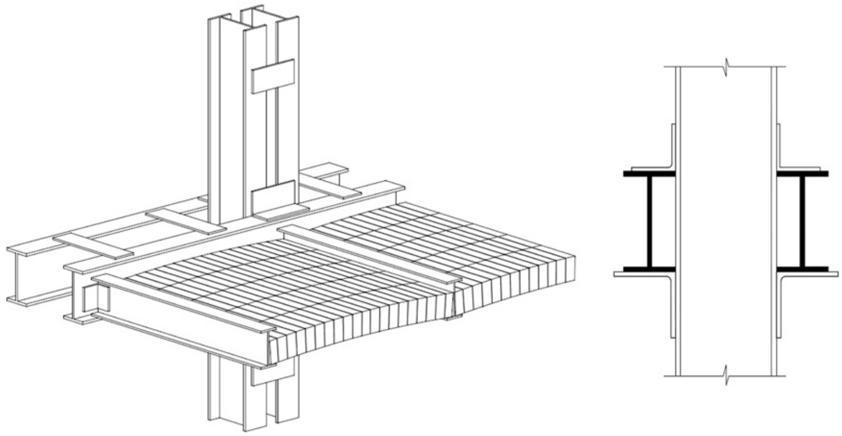

Fig. 1. Typical configuration of saddle connection.

and gusset plates. In their proposed method, a considerable increase of connection rigidity was achieved which resulted in better performance compared with the conventional practice. In 2011, Shakib et al. [8] investigated the seismic vulnerability and retrofitting of a 19-story steel building with semi-rigid connections located in the city of Tehran. The results showed that the building was strong enough to resist gravity loads but the strength was not adequate for lateral seismic loadings.

Most of the previous studies have been deterministic, rather than "probabilistic". Due to the random nature of the earthquake shaking, it would be beneficial to study the problem from probabilistic point of view. This paper is an attempt to assess the seismic fragility of such structures in a probabilistic framework. For this purpose, 3-story and 5-story building models with three types of lateral load resisting systems, unbraced frames with masonry infill walls, braced frames with concentric braces and braced frames with masonry infill walls (six prototypes in total) have been modeled and subjected to incremental dynamic analysis (IDA) utilizing 44 real ground motions based on the recommendations of ATC-63 (FEMA-P695) [9]. In creating fragility curves, the results of IDA procedure were combined with those of a PSHA study for a typical site located in Tehran, where the performance of each model was assessed at immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance limits.

2. Methodology

2.1. General structural behavior

Previous studies (e.g. [10–12]) have showed that although there is no high rotational rigidity at the connections (with initial rotational stiffness typically around 900 tonf·m/rad); but, due to the shear and torsional resistance for the connecting angle sections, they are generally categorized as "semi-rigid". Moment–rotation curves for saddle connections have been derived by experimental tests carried out by Karami and Moghadam [10], Mazrouei and Mostafaei [11], Moghadam and Aalaee [12] and Amiri and Aghakouchak [13]. Moghadam and Aalaee [12] provided the moment–rotation curves for six different conventional saddle connections using different beam sections and connection angle sections. They found that the length of the connecting angle sections played a major role in the strength of the connection. Also, based on Sadeghian and Moghadam [14] study, the connection generally cannot exhibit any ductile behavior due to the large stress concentration on the angle welds.

2.2. Typical building configuration

As the first step in this study, as-built drawings of some typical saddle steel frame buildings were collected in Tehran. The data provided the typical beam, column and brace sections, connection details as well as the thickness of the infill walls in addition to the overall geometry of the buildings. Based on the collected data, two sets of 3-bay (with typical span length of 5.0 m) frames with three and five stories (with typical story height of 3.2 m) were considered. Each set included three lateral load resisting systems supplied by masonry infill walls, braced frames with concentric braces and braced frames with masonry infill walls as depicted in Figs. 3 and 4. In Table 1, some structural specifications of the two model buildings are summarized. For the saddle connections, L-10 and L-12 angle sections (numbers indicate the leg size in centimeters) with a length of 20 cm were used at the top and

(a) (b)

Fig. 2. Brittle saddle connections failure — total collapse examples in Bam earthquake (Mw. 6.5, 2003) [1].