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Seismic rehabilitation of semi-rigid steel framed buildings—A case study

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ABSTRACT

This paper presents the seismic vulnerability and rehabilitation of a steel building with semi-rigid connections in Tehran. This 19-storey building with an asymmetric plan was constructed 30 years ago in three blocks. The qualitative vulnerability of the building was evaluated in the first step of the study, indicating its high seismic vulnerability. In the next step of the study, the quantitative vulnerability of the structure was investigated. The results show that the building was strong enough to resist gravity loads but the strength was not adequate for seismic loads. Finally, three seismic retrofitting methods consisting of concrete shear wall, steel shear wall, and steel bracing were proposed. The comparison of the three retrofitting alternatives in terms of architecture, implementation, and seismic performance showed the superiority of using the concrete shear walls over the two other alternatives.

1. Introduction

The use of steel buildings with a kind of semi-rigid connection called Khorjini has been very common in Iran in the past 50 years. In the construction of these steel buildings, a pair of continuous beams cross the two opposite sides of the columns and each beam is connected to the columns by top and bottom angles. The behaviour of steel buildings with this connection type in different earthquakes such as, Manjil, 1990, and Bam, 2003, revealed the failure of the connections before the failure of the beams and columns. In 1991, some tests on the physical model of Khorjini connection by Moghaddam and Karami [1] showed that this kind of connection is semi-rigid and it lacks the required ductile behaviour. In 1993, Tehranizadeh et al. [2] and also Barkhordari et al. [3] studied the degree of rigidity of this kind of connection. In 2000, Mirghaderi and Mazrouei [4] proposed a method to rehabilitate Khorjini connections by adding top and bottom flanges and triangle plates. In their suggested method, a considerable increase in the degree of rigidity of these connections was observed. The present paper investigates the seismic vulnerability and rehabilitation of a steel building with such semi-rigid connections located in the mega city of Tehran.

2. The building

The building investigated was designed and built between 1973 and 1978. As illustrated in Fig. 1, the building had 3 rather similar blocks. There was a distance of about 21 cm between the blocks. The blocks consisted of 19 stories: 3 underground stories, the ground floor and 15 stories on the ground. The area of each storey was approximately 1950 m^2. The three underground stories were used as car parking areas and the upper floors, being offices for roughly 2000 people, had an administrative usage. The typical storey height was 3.2 m. The southern view of the building is shown in Fig. 2.

3. Qualitative evaluation of the building vulnerability

In evaluating the qualitative vulnerability of the building, various construction parameters such as site slope, soil type, building height, opening dimensions, plan shape, load resisting system, floor diaphragm type and construction quality were considered. The results showed that the building may suffer substantial damage at VII intensity and it may collapse at VIII intensity in the MSK scale [5].

4. Quantitative evaluation of the building vulnerability

The following steps were taken in the quantitative vulnerability evaluation of the building: primarily, the structural performance level was selected and the data collection requirements were specified. Then the building configuration, material properties and site characteristics were identified and the seismic hazard was studied to obtain the necessary information. Finally, the building was modelled and analysed. These steps are described in more details below.

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Table 1
Steel material properties of beams and columns.

Section	Sample number	Elasticity modules (MPa)	F_y (MPa)	F_u (MPa)
Beams	1	20 768	200	370
	2	17 735	200	400
Columns	3	18 058	210	430
	4	23 290	210	410
Mean (expected material properties)		19 963	205	402
Lower bound material properties		17 361	199	377

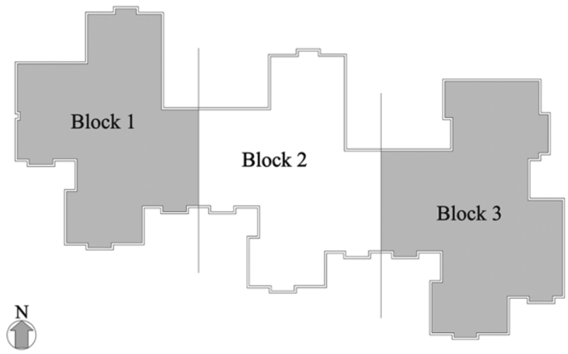

Fig. 1. Position of three blocks of the building.

Fig. 2. The southern view of the building.

4.1. Structural performance level and rehabilitation objective

The basic safety objective was selected based on the importance of the building function. According to FEMA356 [6], the basic safety objective is a rehabilitation objective that achieves the dual rehabilitation goals of life safety building performance level (3-C) for the BSE-1 earthquake hazard level and collapse prevention building performance level (5-E) for the BSE-2 earthquake hazard level.

4.2. Technical data collection requirements

Data on the as-built condition of the structure, the components, the site, and the adjacent buildings shall be collected in sufficient detail to perform the selected analysis procedure. Based on the selected rehabilitation objective for this building, it was decided that the usual data collection level of information about the building would be adequate, in which the knowledge factor, k, was considered to be one according to FEMA 356 [6].

4.3. Building configuration

As there were no structural design drawings available, the types and details of the structural elements, including columns, beams, connections and foundation were obtained by performing on-site investigations. For this purpose, similar elements from several points of the plan location and load type in each floor were studied and they were assumed to comprise five sets. Three on-site investigations were carried out on each set.

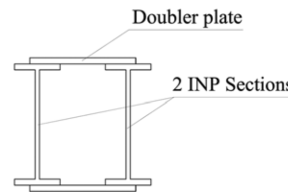

Fig. 3. Column section.

Table 2
Concrete material properties of foundation and diaphragm.

Sample number	Sample diameter (cm)	Sample height (cm)	f'_c (MPa)
1	18.7	9.4	35
2	15.3	9.4	28
3	17.3	9.4	25
4	18.3	9.4	25
5	18.3	9.4	32
Mean (expected material properties)			29
Lower bound material properties			24

These investigations indicated that the columns had a double INP section with doubler plates as shown in Fig. 3. The beams in the East–West direction had 2INP260 sections with doubler flange plates that were connected to the two opposite sides of the columns with top and bottom angles (Khorjini connection), as illustrated in Figs. 4 and 5. The beams in the North–South direction had CNP180 section (castellated beams) with pin connections.

Also, on-site investigations showed that strip footings with link beams were used in the foundation. The foundation thickness was 1.6 m. Fig. 6(a) shows the foundation plan and the column locations in block 3. The diaphragm system was a slab deck. However, there was no lateral force resisting system such as steel bracing or concrete shear wall provided in the building. Fig. 6(b) presents the structural plan view of block 3.

4.4. Properties of in-place material and components

Properties of in-place material and components were obtained according to FEMA356 [6] by conducting the usual tests. For this reason, low-stress regions such as flange tips at beam ends and external plate edges were sampled to minimize the effects of reduced area. The results of tests are presented in Tables 1–3.

4.5. Site characterization and geotechnical considerations

In order to identify the geotechnical characteristics of the site, four bore holes were drilled in the corners of the building site and the following results were obtained:

(a) In terms of geology, the building was located on continental deposits. These deposits contain compact cemented sand and gravel with silt.