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PARAMETER IDENTIFICATION OF JACK-ARCH MASONRY FLOORS USING FORCED VIBRATION TEST OF 1/2-SCALE MODEL OF A 4-STORY STEEL STRUCTURE

M. GHAFORY-ASHTIANY
M. TIV
M. MAHMOUD-ABADI
M. TEHRANIZADEH

International Institute of Earthquake Engineering and Seismology (IIEES)
P.O. Box 19395/3919, Tehran, Islamic Republic of Iran

ABSTRACT

The rigidity of the so called jack-arch masonry floors has been studied by forced vibration test of a 1/2-scale model of a four-story steel structure. The model was examined under two conditions. 1) fully reinforced or rigid floors, and 2) with no reinforcement. A procedure to identify the relative translational stiffness coefficients of the entire model was employed. The ratios of these coefficients from the two conditions of the model indicate that a consistent reduction of stiffness of about 10 percent exists for these kind of masonry floors.

KEYWORDS

Floor rigidity; forced vibration; jack-arch masonry floors; stiffness identification; Iran

INTRODUCTION

Specific construction techniques have been in use for many years in Iran ever since the very first steel profiles were introduced to its building industry. Most of the existing steel structures in Iran have not been foreseen to resist lateral loads. They have two distinct differences with the so called 'standard' steel frame structures. The first and the more critical one of the two has to do with beam-to-column connections. In these connections, a pair of continuous beams cross several columns and connect to the sides of columns by means of angle sections (Fig.1-a,b). This type of construction saves not only on erection time and labor cost, but also the limitations on the availability and the cost of deep rolled sections in the country, makes the use of two parallel beams instead of one deeper beam the only alternative in most cases. Out-of-plane partial beam-to-column transfer of bending moments and early onset of failure in the angles are most likely the cause of failure under lateral forces in these connections.

The second distinction has to do with flooring systems. As it is shown in Fig. 1-a, simply connected steel joists are used to bridge the main beams. The space between these joists are filled with bricks and mortar as a bonding agent in a shape of an arch with approximately 2-5 cm from the head of the arch to its toe within a one meter span (Fig. 1-c). The 'one-way' action of these floors would result in larger stresses in the main beams when compared to 'two-way' floor systems where all beams in a bay between four columns share the same floor weight. This is another reason why two parallel beams are needed for taking up the floor loads.

More importantly, the rigidity of these so called jack-arch floors, for a well behaved diaphragm action, has also been subjected to questions under earthquake loads.

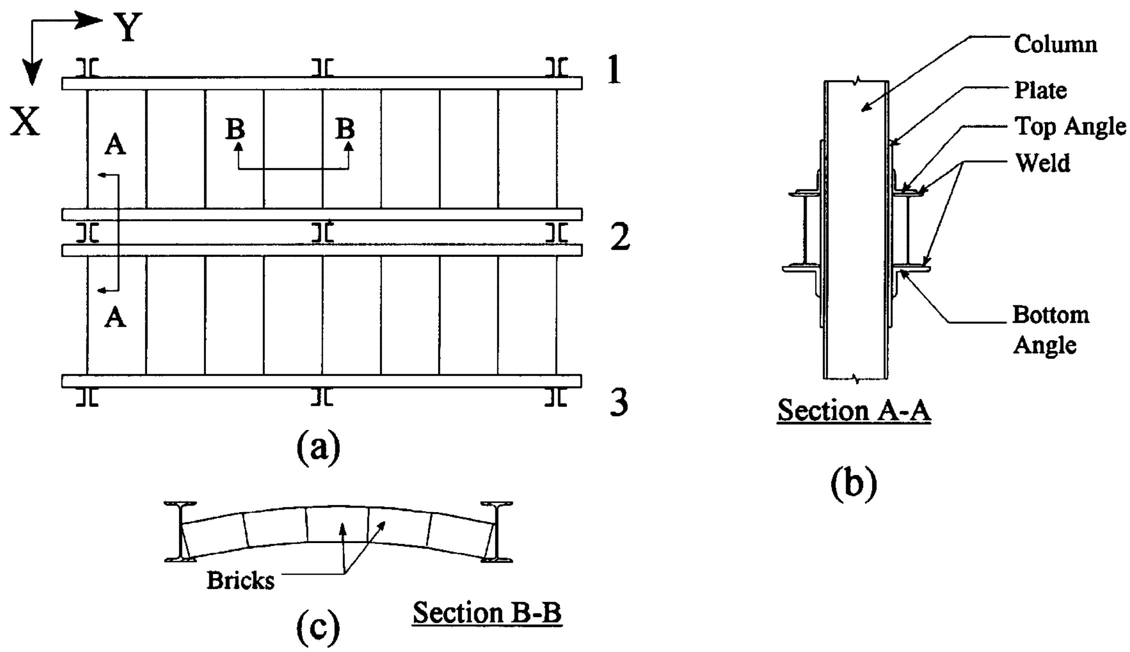

Fig. 1. (a) Floor plan, (b) beam-to-column connection, and (c) jack-arch floors of typical steel structures of Iran

Following the June 21, 1990 Manjil earthquake of Iran and collapse of many of the typical steel structures (Maheri, 1990,) a series of research has been focused on these types of structures. These studies range from mathematical modeling and analysis to the experimental studies of typical connections (Karami and Moghadam, 1987; Kouhian and Moghadam, 1995; Moghadam and Kouhian, 1995), jack-arch flooring systems (Maheri, 1995), and model structures (Aghakouchak and Memari, 1994). A 1/2- scale, typical 4-story model having the above two characteristics was built on a 3x3 meter strong floor at IIEES to better evaluate the dynamic behavior of these types of structures and to propose the proper formulation for seismic design as well as the strengthening method for existing structures. The design and construction of this model was based on the common practice of Iranian engineers and steel workers the details of which are given by Tiv *et al.* (1995). A pair of sinusoidal force vibration exciters is used to excite the model at its top floor in a frequency range of 1-20 Hz. with increments of 0.01 Hz. An 8-channel data acquisition system measures the response in the form of acceleration time histories.

OBJECTIVE

The rigidity of a floor is related to the rigidity of the frame supporting that floor. In other words, a certain type of floor might act quite rigidly in a flexible framing system, but the same floor could behave semi-rigidly in a relatively stiffer frame. In this paper the rigidity of the jack-arch masonry floor systems has been examined under harmonic excitation of the entire model. Comparisons were made between the fully rigid condition of the floor and the condition where floors were allowed to show their semi-rigidity by inducing relative motion between the middle frame, 2, and the two side frames, 1 and 3 (Fig.1-a). Considering one translational degree of freedom per floor in the Y-direction (Fig. 1-a), a technique to identify the stiffness coefficients was adopted. The ratio of the corresponding stiffness coefficients of the two conditions give us a measure of the rigidity of these flooring systems.

TESTING PROGRAMS

Three test series A, B, and C were conducted. In each of the series the acceleration response of the model ranged from 0.06g to 0.35g depending on the frequency of the rotating exciters with higher accelerations associated with higher modes of vibration. Measurements were taken around resonance frequencies in order to find the peak in the displacement-frequency spectrum. Damping coefficients were computed using the half-power method to obtain the undamped natural frequencies. Mode shapes were obtained using the amplitude and the phase difference of different channels.

For each of the these tests the model was tailored differently to meet our needs. The description of the model as well as the parameters that were identified are given bellow for each of these test series.

Test A: The model had no vertical bracings along the Y-direction and the only lateral resisting mechanism was the semi-rigid beam-to-column connections. All floors were horizontally X-braced so that all frames (1, 2, and 3) had identical motion in terms of amplitude and phase (Fig. 2-a). For a unit mass at floor number 4, the stiffness coefficients K_{ij} (i, j = 1,...,4) together with other floor masses were identified.

Test B: The same model in test A was used with the exception of adding vertical cable bracings in the plane of frames 1 and 3, as shown in Fig. 2-b. For a unit mass at floor number 4 and the other floor masses computed in test A, the stiffness coefficients were identified. Again, because of the floor bracings, all frames showed identical motion.

Test C: All of the floor bracings of test B were cut. Frames 1 and 3 were still cable braced under exact conditions of test B. Relative motion in terms of only amplitude was observed between frame 2, and frames 1 and 3. The stiffness coefficients were again identified.

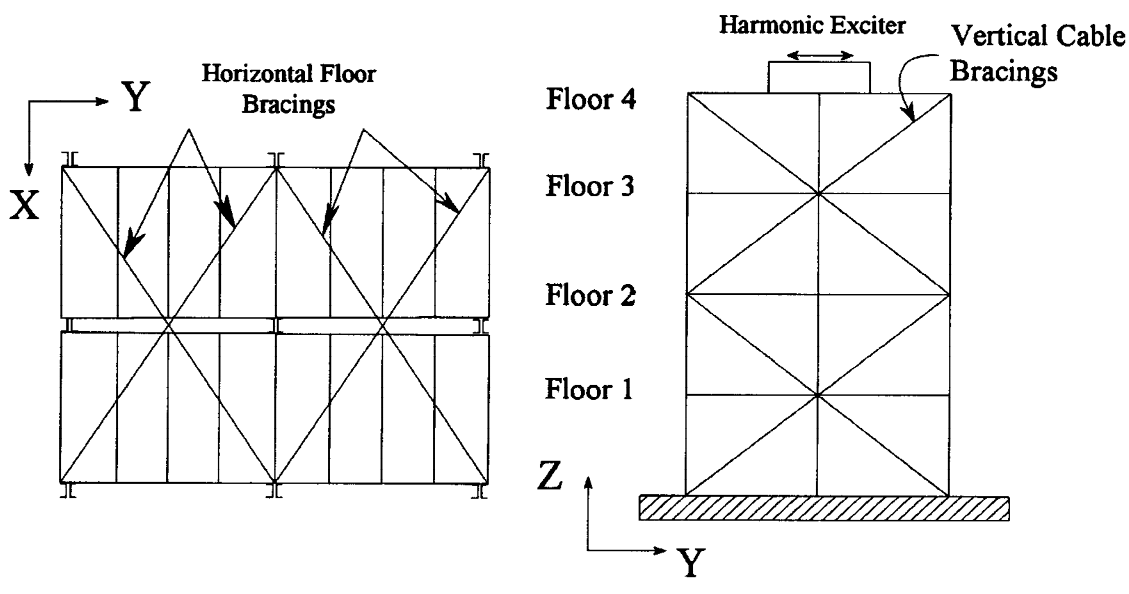

Fig. 2. (a) Horizontal floor bracings, and (b) vertical X-bracings of frames 1 and 3

MASS AND STIFFNESS IDENTIFICATION

Since only the ratios of stiffness coefficients were required, a simple procedure was adopted to identify the relative mass and stiffness parameters. Consider the associated eigenvalue problem of a multi-degree-of-freedom system in free vibration as given by eq. (1).

$$[\mathbf{K}-\omega_i^2\mathbf{M}]\Phi_i = 0 \quad (1)$$

where **K** and **M** are the stiffness and the mass matrices of the system, respectively. For a given natural circular frequency, ω_i, and its corresponding mode shape vector, Φ_i, there are infinite sets of **K**'s and **M**'s that satisfy the above relationship. However, having measured the natural frequencies and the mode shapes, one may obtain a set of stiffness coefficients and floor masses if only one or more of these stiffness coefficients and/or floor masses are known. Having done that, it is possible to reach at a set of equations with certain number of unknowns by simply rearranging the coefficients in eq. (1). For the case of lumped mass systems, and one degree of freedom per floor, the number of unknowns, p, is given by :

$$p = (n+1)n/2 + n - l \quad (2)$$

where n is the number of lumped masses, and l is the number of known, estimated, or assumed coefficients. Thus, for each natural frequency, ω_i, and its corresponding mode shape, n equations with p unknowns may be written where in most cases p>n. Writing the same set of equations for other natural frequencies and corresponding mode shapes, the number of equations could be as high as n×n. However, in some cases natural frequencies of the structure fall beyond the operational frequency range of the exciters and measurements are not possible. Never-the-less, the minimum number of equations must be equal to p. For cases where the number of equations exceeds the number of unknowns, p, a multiple linear-regression could be used for better estimation of the parameters by pre-multiplying both sides of the system of equations by the transpose of the coefficient matrix.

RESULTS

Applying the identification process mentioned here to the results of each of the test series A, B, and C, the stiffness coefficients given in Table-1 were obtained. As mentioned previously, for a unit mass at floor number 4, the stiffness coefficients and the remaining floor masses were identified from test series A. This test was done because the model is a simple moment frame and it is easier to interpret the results to the actual response of typical moment frames.

Table 1. Stiffness coefficients of the three test series A, B, and C for a unit mass at floor number 4 (M_4=1.0)

Stiffness Coefficients	A	B	C	C/B
K_{11}	2629.7	2642.0	2385.9	**0.90**
$K_{12}=K_{21}$	-1503.0	-1482.0	-1316.7	0.89
$K_{13}=K_{31}$	329.8	324.2	237.3	0.73
$K_{14}=K_{41}$	-47.0	-209.9	-149.3	0.71
K_{22}	2346.7	3161.3	2801.8	**0.89**
$K_{23}=K_{32}$	-1483.4	-1589.3	-1362.2	0.86
$K_{24}=K_{42}$	329.7	176.1	124.3	0.71
K_{33}	2182.8	2378.9	2124.4	**0.89**
$K_{34}=K_{43}$	-1001.9	-1270.5	-1138.6	0.90
K_{44}	692.6	1466.8	1323.2	**0.90**
Floor Masses: M_3=1.64, M_2=1.63, M_1=1.60				

Figure 3 shows the direction of each of the stiffness coefficients schematically which complies with the actual response of typical moment frames with joint rotations.

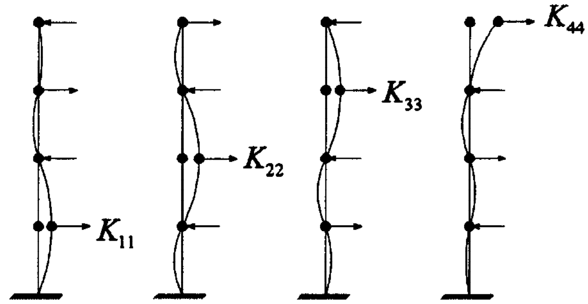

Fig. 3. Direction of each of the stiffness coefficients and the corresponding deformations of the entire model.

DISCUSSION

The ratios of the stiffness coefficients from the two test series B and C indicate a consistent reduction of stiffness of about 10 percent due to semi-rigidity of these masonry floors when compared to the fully rigid case. Noting the range of acceleration response (0.06g to 0.35g) of this model (or the prototype as well), it can be implied that as long as the integrity of these masonry floors is maintained, their rigidity is acceptable for the distribution of lateral forces.

There are some sudden changes in the values given for test series B and C when compared with the test series A. It should be kept in mind that these abnormalities are due to the connection scheme of horizontal X-bracing cables in test series B and C, where the cables brace floors 2, and 4 and skip floors 1, and 3 (see Fig. 2-b).

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