# Impact of Joists Direction on the Diaphragm Behavior of Composite Floor Systems

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**ABSTRACT:** *The influence of the in-plane flexibility of floor systems on the seismic response of the structures may become significant, particularly when considerable floor slab cracking and yielding is expected. Since in recent years the composite floor systems (steel beams with upper concrete slab) have been used widely in the steel buildings, the investigation of the diaphragm behavior of composite floor systems subjected to lateral loads, is inevitable. As a part of a comprehensive study of the effective parameters in the diaphragm behavior and in-plane characteristics of the composite floor systems, two half-scale single story building models with the parameter of the direction of the joists were tested. The scale-model structures were steel buildings with composite floor system consisting of four square panels supported by side frames with X bracing. The structures were designed under the seismic load specified in the seismic code of Iran, but to ascertain the inelastic behavior of the diaphragm, the stiffness of the lateral load resisting system of the structures were increased by doubling the number of bracings, then the cyclic lateral load was applied to the structures up to failure. The paper reports on the results of the test program performed at the structural engineering laboratory of Building and Housing Research Center of Iran (BHRC). The results show that the composite diaphragms present good performance under lateral loads; however the diaphragms' in-plane characteristics such as stiffness, ultimate strength, stiffness degradation and the crack pattern of the floor slabs are affected significantly by the relative direction of the floor joists to the lateral load (perpendicular or parallel).*

**Keywords:** Composite floor system; Direction of the joists; Diaphragm; Seismic load; Cyclic loading; Experimental data; In-plane stiffness

## 1. Introduction

In buildings where the floor systems usually carry, the gravity loads to the vertical elements are required in order to transfer the lateral inertia forces to the lateral load resisting systems by a diaphragm action. The assumption of rigid floor diaphragm is often used to simplify seismic response analysis of many types of buildings. However, for some classes of structural systems, the effect of diaphragm flexibility cannot be disregarded, especially in the case of rectangular buildings with large aspect ratios where considerable inelastic floor slab behavior is expected [1]. Since the diaphragm behavior is one of the most important factors in the seismic response of the structures, researchers in many countries have conducted studies on this subject, but the studies do not have a long precedence and they have mostly been performed in the last three decades. The diaphragm behavior of different types of floor systems usually differs substantially and depends on the details of the floor system and in some cases the diaphragms behavior may be unknown, so experiments can be useful to better understand the

diaphragms behavior [2], as most of the studies on this subject have been conducted experimentally.

The studies on the low rise steel buildings with metal roof deck have shown that the period of the structure is influenced by the diaphragm in-plane flexibility and the forces in the resistant elements can be amplified due to dynamics of the flexible diaphragm. Also the shaking table results show that the diaphragm in-plane deformations are twice that of the values obtained from static analysis [3]. The investigations on the *PBFB* floors (pre-stressed beams and filler blocks) have shown that a 4*cm* thick slab for these floors assures the validity of "rigid diaphragm" hypothesis and the horizontal resistance capacity of the floors with cast-in-place concrete over the entire area were twice the floors without that and with transversal reinforcement. In addition, the failure modes of the specimens were very different, depending on material of filler blocks and of presence or absence of cast-in-place over concrete [4]. The seismic behavior of wood diaphragms in unreinforced masonry buildings has been studied through tests on three test specimens; also they were retested using different rehabilitation methods. The results indicate that *FEMA* 273 tended to overpredict the stiffness and significantly underpredict yield displacement and ultimate deformation levels, while *FEMA* 356 tended to underpredict stiffness and overpredict yield displacement [5]. An extended numerical parametric research was carried out to study the diaphragm behavior of *RC* floor systems (slabs and beams). The results showed that although there was no clear correlation between these two structural characteristics [7], the influence of aspect ratio on the criterion of the diaphragms rigidity ($\Delta_{diaphragm}/\Delta_{story}$) [6], was considerable.

In this project, the diaphragm action of the composite floor system in a typical steel structure under lateral loads with the influence of the gravity load is investigated. As part of a comprehensive investigation of the effects of the important parameters in the behavior of the composite diaphragms, the effect of the direction of the joists in the diaphragm behavior of the composite floor systems is studied through an experimental program. The aim of this research was to obtain experimental data on the effect of the direction of the floor joists on the in-plane characteristics of the diaphragm such as the diaphragm flexibility, stiffness, ultimate strength, yield point and crack pattern. The structures considered in this program were 3-*D* one-story typical steel buildings consisting of composite floor and *X* bracings, which are common in the country. The test specimens were half scale-models of the prototype buildings. The seismic design of the structures was performed considering the seismic code of Iran [8], and the composite floors were designed with *ASCE* code. The lateral load resisting system of the structures were strengthened by increasing the number of the braced frames to ascertain the nonlinear behavior of the diaphragm. In order to study the composite diaphragm characteristics in the nonlinear region, the cyclic lateral load with increasing amplitude was applied to the structures up to failure. This paper describes the test specimens and presents some of the main findings of the testing program.
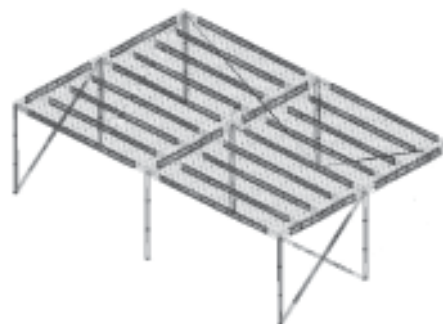
## 2. Test Structures and Experimental Procedure

### 2.1. Prototype Buildings

Figure (1) shows 10.8*m* x 7.2*m* x 3*m* prototype buildings considered in the study. The girders and floor joists are *I* shapes supported on box columns and lateral load resisting system of the structures is



| (a) The Floor Joists Parallel to the Lateral Load | (b) The Floor Joists Perpendicular to the Lateral Load |

**Figure 1.** The steel buildings prototype.

*X* bracings with box section. The overall geometry of the structures presented in Figure (1) is the same and the main difference is the direction of the floor joists. The spacing between the floor joists in the structures shown in Figures (1a) and (1b), was set to 108*cm* and 90*cm* respectively, and the thickness of the floor slab was 8*cm*.

The composite floors were designed due to *AISC* specifications and composite structures design handbook [9]. Also the seismic design of the structure was performed according to the seismic code of Iran. The specified seismic lateral load for the structure, *V*, is given by:

$$V = CW, \quad C = \frac{ABI}{R} \tag{1}$$

Where *C* is the seismic shear force coefficient, *A* is zonal acceleration, *B* is the seismic response factor, *I* is the importance factor, *R* is the force modification factor and *W* is the seismic weight of the structure.

For these structures in Tehran we have:

$A = 0.35$, $B = 2.5$, $R = 6$

So we have $C = 0.146$ and the total seismic load calculated for both of the structures, obtained from Eq. (1) is 54.1*kN*.

### 2.2. Scaling Requirements

Scaling the original structures is usually unavoidable in an experimental research, due to the geometrical, economical and technical limitations. However in a scaled test, the proportion of the quantities which are important in the test, must be determined. The relation of the basic dimensions between the prototype and model specimens is usually described by the scale factor ($S_L$) and Young's modular ratio ($S_E$). Table (1) presents the proportion of some of the quantities between the prototype and scaled models described as $S_L$ and $S_E$.

The models of the hypothetical prototype structures were constructed on a scale of 1:2, so we have $S_L = \frac{1}{2}$. The selection of the scale factor was dictated by the limitation of the laboratory and the relative situation of the structures frames to the strong floor. Also since in this testing program, the materials used to construct the scaled models were the same as the original buildings, therefore we have $S_E = 1$.

### 2.3. Test Specimens

The single-story half-scale building models used in

**Table 1.** Scale factors of the basic quantities.

| Scale Factor for $S_L = \frac{1}{2}$, $S_E = 1$ | Scale Factor | Dimension | Quantity | Type |
|---|---|---|---|---|
| 1 | 1 | - | Strain | Material Properties |
| 1 | $S_E$ | $FL^2$ | Stress | |
| 1 | $S_E$ | $FL^{-2}$ | Yong's Module | |
| 1 | 1 | - | Poison's Ratio | |
| 2 | $S_E / S_L$ | $FL^{-3}$ | Weight Density | |
| 0.5 | $S_L$ | $L$ | Linear Dimension | Geometry |
| 0.5 | $S_L$ | $L$ | Displacement | |
| 1 | 1 | - | Rotation | |
| 0.25 | $S_L^2$ | $L^2$ | Area | |
| 0.0625 | $S_L^4$ | $L^4$ | Moment of Inertia | |
| 0.125 | $S_L^3$ | $L^3$ | Section Modulus | |
| 0.25 | $S_E \times S_L^2$ | $F$ | Point Load | Loading |
| 0.5 | $S_E \times S_L$ | $FL^{-1}$ | Linear Load | |
| 1 | $S_E$ | $FL^{-2}$ | Pressure | |
| 0.125 | $S_E \times S_L^3$ | $FL$ | Moment | |
| 0.25 | $S_E \times S_L^2$ | $F$ | Shear Force | |



a) First Structure



b) Second Structure
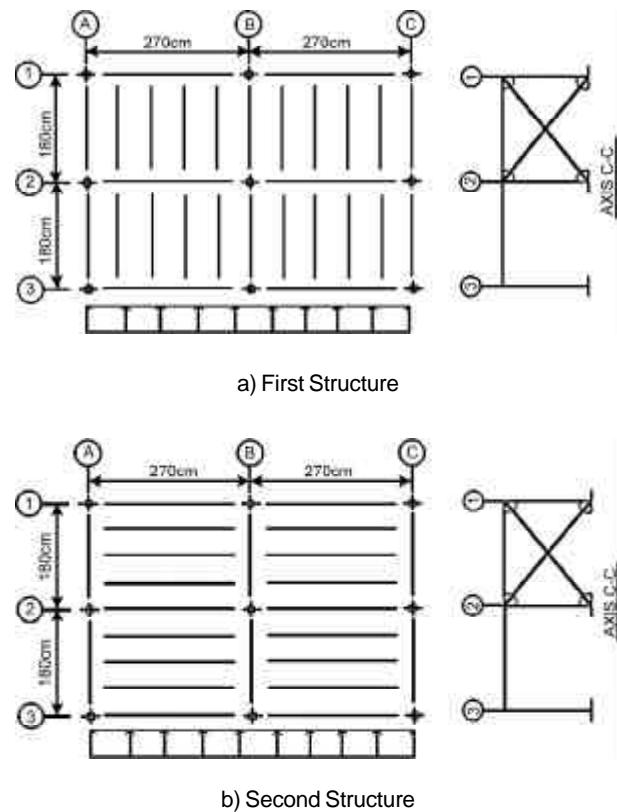
**Figure 2.** Building models.

the experimental study are shown in Figure (2). The dimensions were set to 5.4*m* x 3.6*m* x 1.5*m* and the thickness of the floor slab was 4*cm*. The lateral load resisting system were the side braced frames and all the connections of the beams and bracings to the columns, were hinged connections. The

columns and bracings sections were exactly half of the original ones, but the *I* shape beams were scaled considering the section modulus proportions. Also the shearkeys dimensions and spacing were set to half of the original ones.

The direction of floor joists is one of the most important parameters in the diaphragm behavior of composite floor systems, so it was considered as the varying parameter between the specimens. As shown in Figure (2), the direction of joists in the two test specimens were parallel and perpendicular to the direction of the lateral load and the spacing between the joists were 54*cm* and 45*cm* respectively.

One of the test specimens is shown in Figure (3). As shown in this figure, a strong steel foundation was considered for the structures. The foundation was made of plate girders and was locked to the strong floor. Due to the seismic design of the structures, each of the side frames consisted of one *X* bracing, but in the next parts of the testing program, the number of the bracings of the side frames was doubled in order to give priority to the diaphragms failure and to study the nonlinear behavior of the diaphragms.



**Figure 3.** One of the half scale model structures.

### 2.4. Testing Sequence

The testing program started 28 days after pouring the concrete of the floor slab. 15*cm* x 15*cm* x 15*cm* cubic prisms were made during construction of the specimens to determine the compressive strength of the slab concrete. The compressive strength of the concrete ($f'_c$) was obtained as 19.52 *MPa* and Young's modulus of the concrete were about 20.8*GPa*. Also yield stress and Young's modulus of the steel used in the column and bracing members were obtained as 383*MPa* and 205*GPa* respectively

from tensile tests.

Prior to the lateral loading test, the gravity load were applied to the structures, including dead and live loads and a load related to scaling and simulation requirements, see Table (1), row No. 5. The total gravity load as uniform pressure, for both of the structures is given by:

$$Q_{tot} = Q_{DL} + Q_{LL} + Q_\rho =$$
$$120 + 150 + 100 = 370 kg/m^2 = 3625 Pa \qquad (2)$$

The uniform gravity load was applied gradually using lead bars shown in Figure (4). The total weight of the required bars is:

$$W_{tot} = 370 \times 5.4 \times 3.6 = 7193 kg \qquad (3)$$



**Figure 4.** The gravity loading test.

### 2.5. Cyclic Quasi-static Testing

In this part of the test, shown in Figure (5), the stiffness of the lateral load resisting system of the structures were increased by doubling the number of the bracings of the side frames, in order to study the in-plane characteristics of the diaphragms such as ultimate strength, stiffness, deformation and crack pattern of the diaphragms under lateral cyclic load. The reason for increasing the lateral stiffness of the structure was to give priority to the failure of the diaphragm compared with the failure of the structure, and to ascertain the nonlinear behavior of the composite diaphragms.

The lateral load was applied at the roof level of the structures to simulate the inertia load caused by the seismic action. As shown in Figure (6a), the lateral load was applied by the two point loading method and the cyclic lateral load was applied by hydraulic actuators with a capacity of 50 tons, in each side, and the foundation was locked to the

strong floor as illustrated in Figure (6b). In order to study the crack pattern and distinguish yielded elements, the floor slabs and all of the steel elements, were covered by hydrated lime.

The cyclic lateral load with ascending amplitude was applied to the structure up to failure. The amplitudes of the lateral load for both of the structures is presented in Figure (7). Since the first
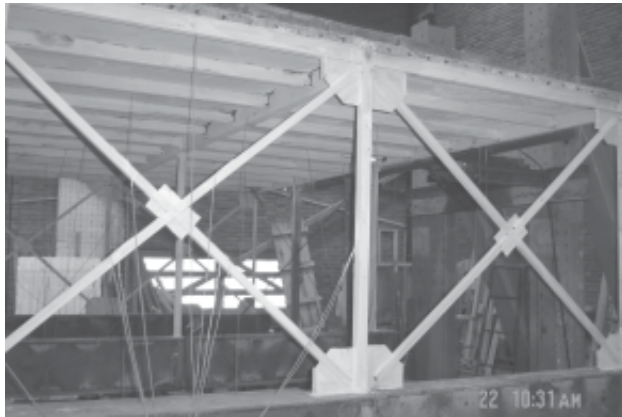


a) First Specimen
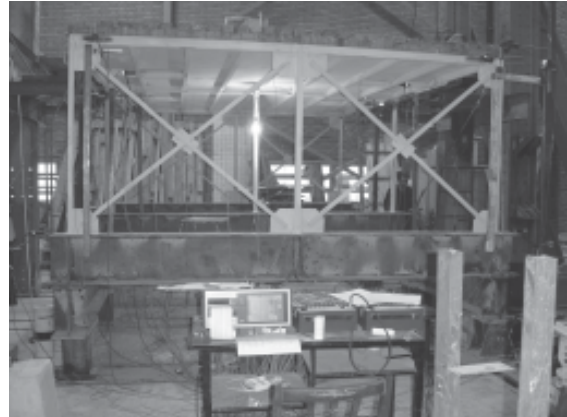
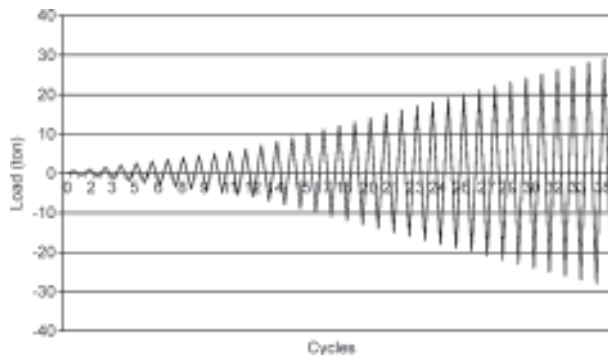b) Second Specimen

**Figure 5.** Increasing the lateral stiffness of the structures.
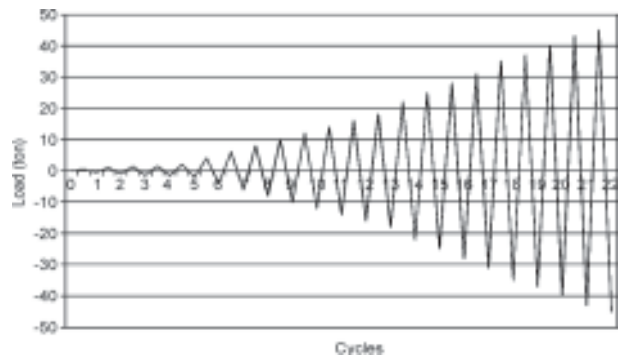


a) Two Point Loading Method

b) Locking the Foundation to the Strong Floor

**Figure 6.** Cyclic lateral loading setup.



a) First Specimen

b) Second Specimen

**Figure 7.** Cyclic lateral load amplitudes.

specimen was tested prior to the second one and the behavior of the diaphragm was rather unknown, in the first specimen the cyclic load was applied by closer steps compared to the second specimen.

## 3. Test Results

As mentioned in the previous sections, the net displacement of the diaphragm is the relative displacement of the mid frame to the side frames, which is given by:

$$\tag{4}$$

Where $\Delta_d$ is the diaphragm displacement and $\Delta_s$ is the displacement of the side frames or the story drift. The hysteretic response of the diaphragms were obtained by the hysteretic response of the braced side frames (*A-A* and *C-C*) and the mid frame (*B-B*), using Eq. (4). The hysteretic response of the diaphragms of the structures are presented in Figure (8).

As shown in Figure (8), there was no significant stiffness degradation, so the cyclic tests of the structures were based upon force-control method up to failure.

The envelope curves of hysteretic response of the diaphragms are presented in Figure (9a). Since usually there are some hidden eccentricities in experimental tests, in some cases the displacements of the inverse loading directions, differ slightly. So the mean values of displacements in two inverse loading directions (push and pull) were used in envelop curves of Figure (9b).

As shown in Figure (9), for the first diaphragm the ultimate strength was 29 tons and the stiffness was constant until the lateral load was 9 tons, but after developing the cracks, it decreased about 50 percent of the initial stiffness. For the second diaphragm the ultimate strength was 47 tons and the force-displacement curve was linear in a large region and the diaphragm stiffness was approximately constant until the lateral load was about 29 tons, but, by developing the first cracks, the diaphragm stiffness decreased about 30 percent of the initial stiffness. Variation of the diaphragm stiffness (slope of load-displacement curve) is shown in Figure (10). The results show that in the second structure where the direction of the floor joists were perpendicular to the direction of the lateral load, the diaphragm has
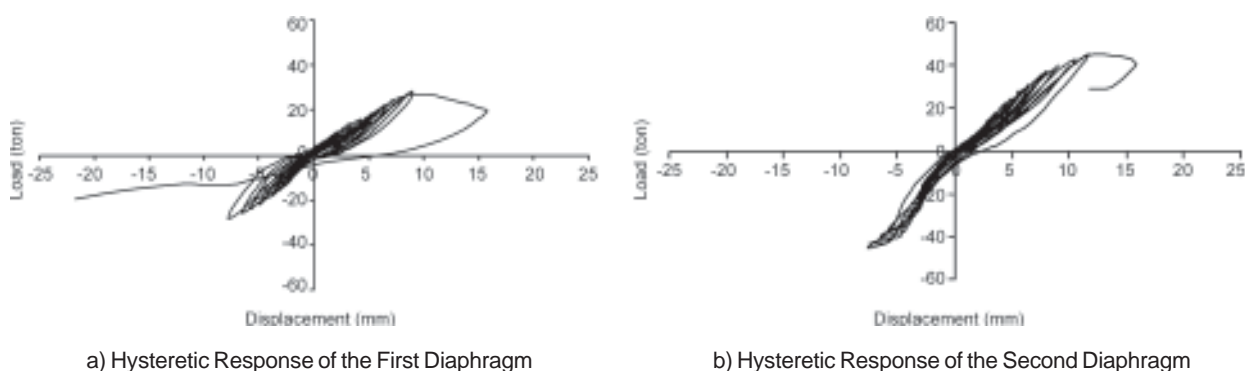
a) Hysteretic Response of the First Diaphragm     b) Hysteretic Response of the Second Diaphragm
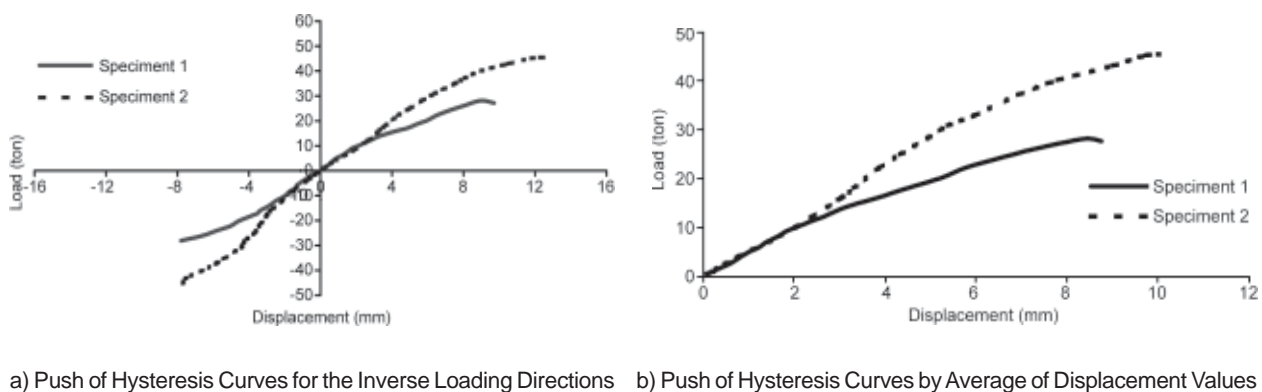
**Figure 8.** Hysteretic responses of the in-plane deformations of the diaphragm.

a) Push of Hysteresis Curves for the Inverse Loading Directions     b) Push of Hysteresis Curves by Average of Displacement Values

**Figure 9.** Envelop curves of the diaphragms in-plane deformations.

a) First Diaphragm                                    b) Second Diaphragm
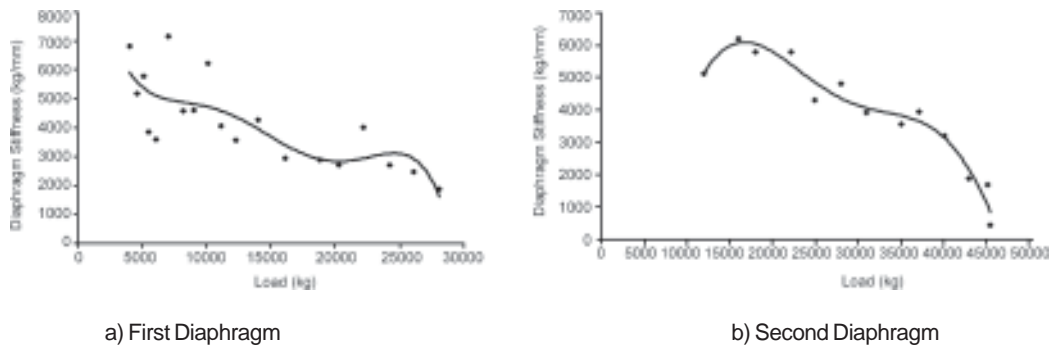
**Figure 10.** Variation of in-plane stiffness of the diaphragms.

higher in-plane stiffness and ultimate strength compared to the first structure and the direction of the floor joists was parallel to the direction of the lateral load. For example it was observed that the ultimate strength of the second diaphragm was higher than the first one by a factor of 1.63, and the major stiffness degradation of the diaphragms started when the lateral load was 32% and 85% of their ultimate load, respectively. These observations indicate that the second diaphragm generally shows a better performance under lateral loads compared to the second one [10].

### 3.1. Floor Crack Pattern

The crack patterns of the diaphragms during the test are presented in Figure (11). The numbers specified in the figure beside the cracks, show the lateral load in tons when the cracks appeared. For the first specimen there were no marked cracks, until the lateral load reached 4 tons, and the first cracks appeared when the lateral load was 4.5 tons.

For the next levels of lateral loading, most of the cracks developed parallel to the direction of the joists, but in the high levels of lateral loading a few cracks appeared near the braced frames which were inclined at about 45° to the joists. Finally when the lateral load was 29 tons, due to wide developing cracks and decreasing diaphragm stiffness, the slab concrete crushed and subsequently the connection of beam to column in the intersection of axes 1-1 and *C-C* was ruptured and the structure failed while the diaphragm displacement was 8.4*mm*. However the columns and bracings remained elastic until failure.

For the second specimen there were no marked cracks until the lateral load was 35 tons and the first cracks appeared when the lateral load was 37 tons. For the next levels lateral loading, all the cracks were developed near the braced frames and were inclined at about 45° to the joists. The ultimate load was reached just before the concrete adjacent to the shearkeys on the joints were crushed when
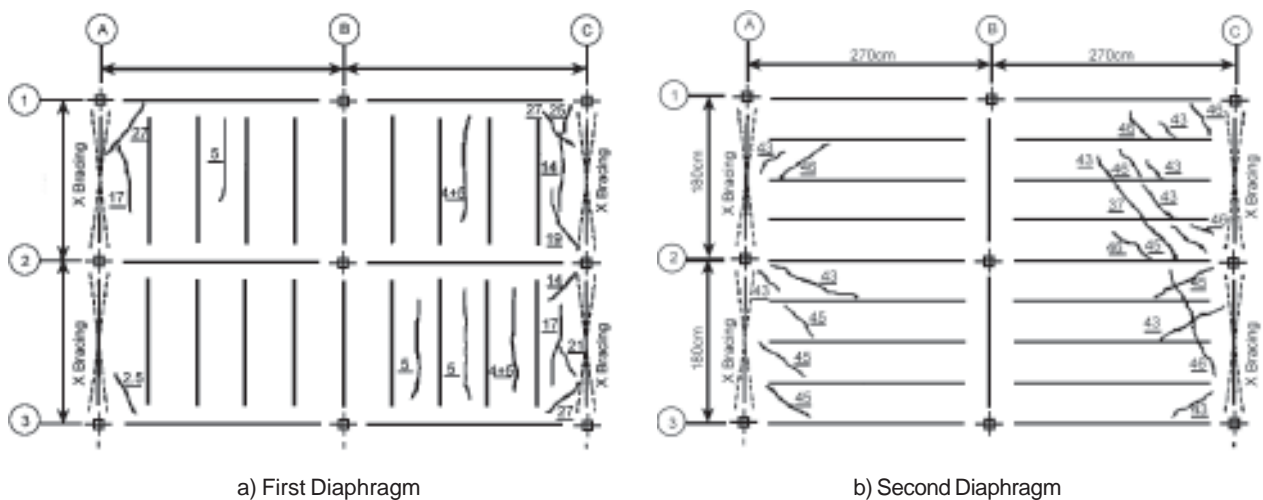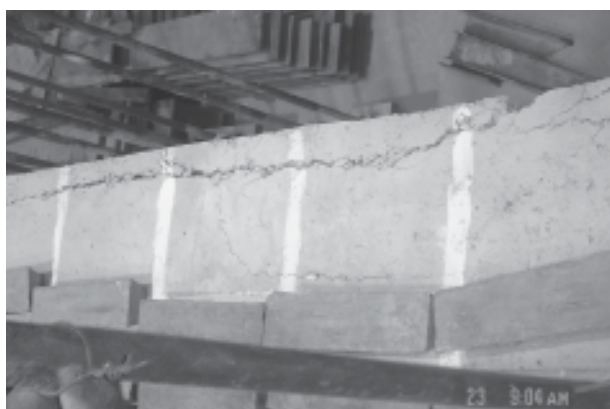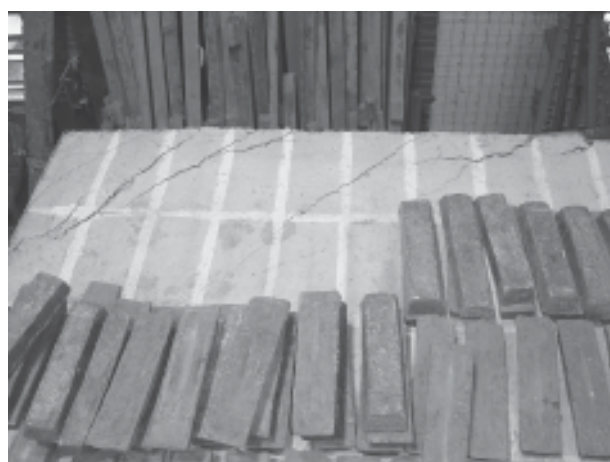


a) First Diaphragm                                    b) Second Diaphragm

**Figure 11.** The crack patterns of the diaphragms during the test.

the lateral load was 47 tons. During the next cycle due to strong degradation of the diaphragm and increasing crack widths, the concrete near the braced frame along axis *C-C* crushed and finally due to failure of the shearkeys, the diaphragm failed when the lateral load was 47 tons, with a diaphragm displacement of 12*mm*. Also under the ultimate load, one of the compressive bracings buckled, and the failure of the structure and the diaphragm occurred simultaneously as shown in Figure (13a).

The crack pattern of the diaphragm after overall failure is shown in Figure (12). As shown in this figure, in the first diaphragm the cracks are parallel to the direction of the lateral load and the flexural performance is dominant, however in the second one the cracks mostly incline at about 45° to the direction of the lateral load where the diaphragm performs as a deep beam and the shear performance is dominant [10].



a) First Diaphragm



b) Second Diaphragm

**Figure 12.** Crack patterns of the diaphragms after failure.



a) Buckling of the Compressive Bracing of the Second Specimen



b) Shearkeys Action after Failure of the Slab Concrete

**Figure 13.** Test observations.

### 3.2. Deformed Shape of the Diaphragms

The experimentally obtained in-plane deflected shape of the diaphragms for the ascending amplitudes of the lateral load is presented in Figure (14). The lateral load values related to each curve, are the peak values of the graphs presented in Figure (7).

The curves indicate that the responses are rather smooth and the deflection curves are symmetric. The parabolic shape observed suggests that in addition to the shear deformations of the diaphragm, the flexural deformations are considerable. However the second diaphragm exhibits very high flexural stiffness.

The reason of high performance of the second diaphragm under lateral load is illustrated in Figure (15). Performance of diaphragms is controlled by a combination of shear and flexural actions. If we consider any section of the equivalent beams of the two diaphragms, in the first diaphragm which
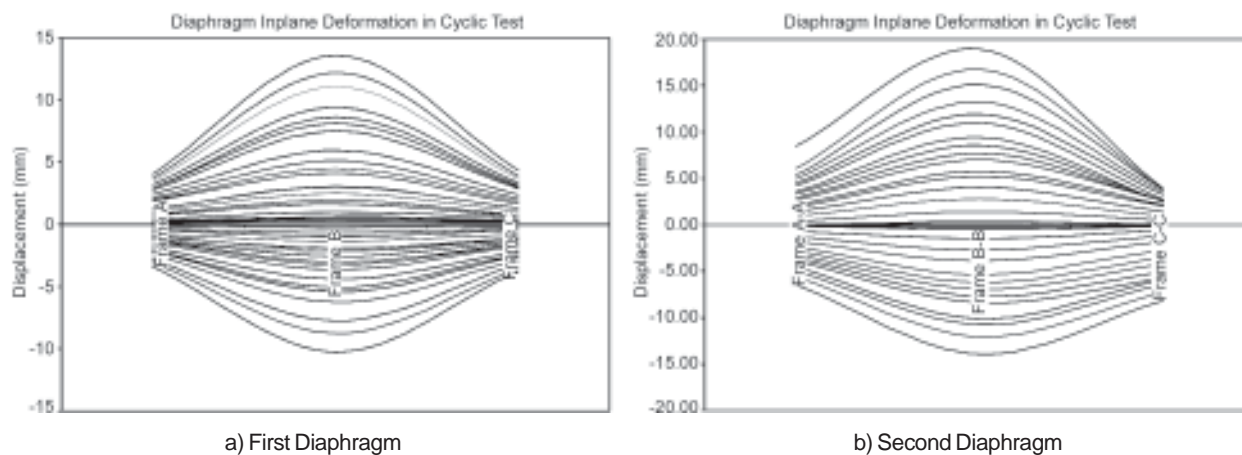
a) First Diaphragm

b) Second Diaphragm

**Figure 14.** Diaphragms in-plane deformations in the cyclic test.



a) Joists Parallel to the Lateral Load
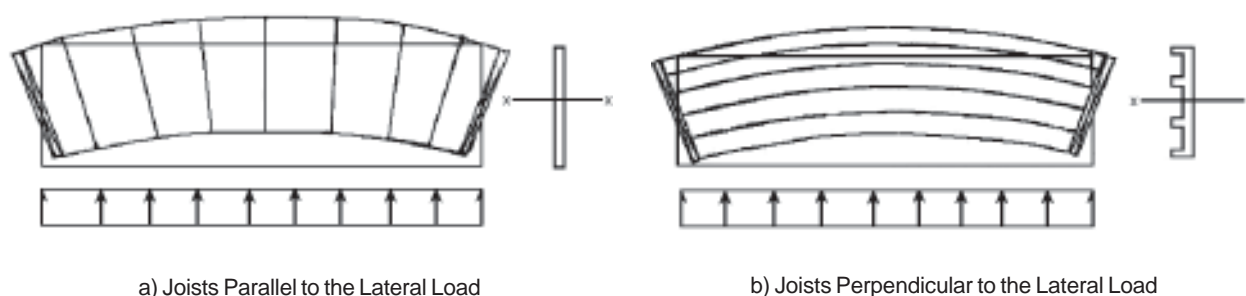
b) Joists Perpendicular to the Lateral Load

**Figure 15.** Effect of the direction of the joists in strengthening the beam section.

the floor joists are parallel to the lateral load, the contribution of the floor joists in strengthening the equivalent beam section is negligible as shown in Figure (15a). But in the second diaphragm in which the floor joists are perpendicular to the lateral load, the floor joists strengthen the equivalent beam significantly, by increasing the moment of inertia and area of the beam section as shown in Figure (15b). This leads to high capacity of resistance in both shear and flexural actions [10].

## 4. Conclusions

In designing and retrofitting the structures, it is highly important to consider the stiffness and ultimate strength of the diaphragms. Strengthening the resisting elements without considering the strength capacity of the diaphragm will be hazardous, therefore it is necessary to know the in-plane characteristics of the diaphragms [2]. In this study the lateral in-plane characteristics of the composite diaphragms in typical steel buildings with composite floor system (which are common in many countries), were studied by experimental tests on two half scale one-story steel building models. The direction of

floor joists was the varying parameter between the models. The direction of the joists in the first model was parallel to the direction of lateral load, but in the second model it was perpendicular to the direction of lateral load. The seismic lateral load was simulated by quasi-static reverse cyclic loading. The results show that the second diaphragm has higher lateral in-plane stiffness and there was no significant stiffness degradation until 85% of the ultimate load, but the first diaphragm has lower stiffness and the stiffness degradation started at a load of 32% of the ultimate load. Also the ultimate strength of the second diaphragm was considerable and was about 1.62 times the ultimate strength of the first diaphragm. Thus it seems that one of the most important parameters in the diaphragm behavior of the composite floor systems is the direction of the joists relative to the lateral load. If two individual diaphragms are designed under gravity load with the same conditions, the diaphragm with joists perpendicular to the lateral load, exhibits a better performance under lateral loads. In both of the tested structures, since the buildings were single story with a low aspect ratio of 1.5, the diaphragms under the code

seismic load acted as a rigid diaphragm. Generally it is recommended that in a building with low plan aspect ratio, the composite floor systems are so constructed that the direction of the joists in the vicinity of braced frames or shear walls, is perpendicular to the direction which the main lateral load resisting elements act, or the joists are set in a staggered manner all over the plan. If a building has a high plan aspect ratio, directing the joists in the long direction would lead to better performance of the composite diaphragm; however in some cases directing the joists to be perpendicular to the lateral load is effored with some penalties. Because if the joists are in the long direction of the diaphragm, they are less efficient under gravity load and it would become more costly.

## Acknowledgments

## References

1. Panahshahi, N., Reinhorn, A.M., and Kunnath, S.K. (1994). "Earthquake Simulation Study of a One -Sixth Scale-Model RC Building with Flexible Floor Diaphragms", *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, IL.

2. Naeim, F. (1989). "The Seismic Design Hand Book", Van Nostrand Reinhold, New York, 210-237.

3. Tremblay, R., Berair, T., and Filiatrault, A. (2000). "Experimental Behavior of Low-Rise Steel Buildings with Flexible Roof Diaphragms", *12WCEE*.

4. Damian, A., Kovacs, L.A., Cornel, T.B., Gosa, O., Maniu, H., Dico, C.S., and Tokes, A. (2000). "Diaphragm Behavior of the Floor with Pre-Stressed Beam and Filler Blocks", *12WCEE*.

5. Peralta, D.F., Bracci, J.M., and Hueste, M.B.D. (2004). "Seismic Behavior of Wood Diaphragms in Pre-1950 Un-Reinforced Masonry Buildings", *J. Struct. Eng.*, **130**(10),1487-1496.

6. Euro Code 8 (1988).

7. Syrmakezis, C. and Chronopoulos, M. (1990). "A Parametric Analysis of R.C. Diaphragms", Earthquake Resistant Construction and Design, Balkema, Rotterdam.

8. BHRC (2005). "Iranian Code of Practice for Seismic Resistant Design of Buildings", Standard No. 2800, Tehran, Iran.

9. Viest, I.M., Colaco, J.P., and Leon, R.T. (1997). "Composite Construction Design for Buildings", Mc Graw-Hill.

10. Sarkissian, L. (2005). "Study of the Behavior of the Composite Floor Systems under Lateral Load", M.Sc. Thesis, Tehran University.