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Analysis of Diaphragms Stiffness in Precast Construction supported on Reinforced Concrete Walls

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Abstract This research presents results of the structural analytical behavior of diaphragms built with precast slabs supported on reinforced concrete walls. The current analysis approach that some designers propose for this type of construction, supposes the implementation of a rigid diaphragm model provided by the slab connections, stipulated in the construction regulations. However, given the loss of continuity between the slab panels in these prefabricated systems and the adoption of spaced connections, some differential displacements are released inside these diaphragms, which leads to a loss of their rigidity and changes in the behavior of the structural seismic resistance system. Two buildings were analyzed in this research, whose geometric characteristics were: length/width ratio of 1.5 and 2; with 5 stories high. The modeled structural system was reinforced concrete walls and the diaphragm consisted of precast slab panels hoisted in situ, joined by spaced connections. A chronological-spatial analysis was performed with acceleration records of scaled earthquakes; with the purpose of evaluating the behavior of the structure and checking the floor displacements throughout the building. With calculated displacement at various points of the diaphragm, flexibility indices and floor accelerations were determined. From these results, it was possible to establish whether the behavior of these diaphragms is rigid or flexible.

Keywords Precast Slabs, Reinforced Concrete Walls, Rigid Diaphragm, Flexible Diaphragm, Chronological Analysis, Flexibility Indices

1. Introduction

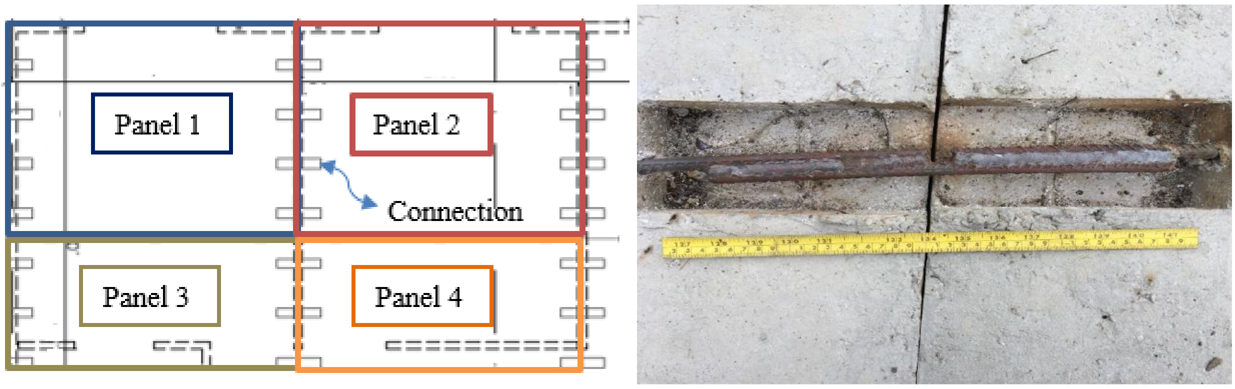
In recent decades, the construction of buildings has taken an important turn towards industrialization; the implementation of technology has optimized resources and execution times in construction practices, reducing costs [1]. There are many advantages that are achieved when industrialization is performed in modular construction with precast elements, like modular slabs supported on reinforced concrete walls [2] [3]. In this kind of construction, slab panels are built in a production facility and then erected on site on reinforced concrete walls.

The panels are distributed over the building's floor surface and then connected by welded joints in the reinforcing steel bars of the elements at established spacing (see Figure 1a.) [4]. A method to connect the panels is by welded splices on the reinforcing steel bars at a set spacing (see Figure 1b.). This type of connection has been studied through experimental research [5], and consists of placing bars of 12.7 mm (1/2 inch) in diameter, inside the prefabricated plates and later connecting them to the adjacent panels through an overlap with E7018 electrode welding. To achieve this, some openings must be made in the panels and the steel must be uncovered to develop the connection. Subsequently, concrete is poured over the connections (located at the wall axes) until a single diaphragm consisting of many panel elements connected to each other is obtained [3]. This diaphragm can support and transfer the gravitational and lateral loads as a continuous element, however, since its construction is not monolithic and the connections are spaced and not over the entire perimeter of each panel, there is some uncertainty concerning rigidity or flexibility consideration of the element and the way it distributes the inertial forces to the lateral supports of the structure.

The American Society of Civil Engineering (ASCE) [6] has developed a procedure to analytically corroborate the rigidity or flexibility of a floor diaphragm. This procedure is based on calculating (from the structural analysis) the displacements at some points of the diaphragm under seismic force, and then comparing the results of these displacements with those that would occur if the diaphragm were an undeformable solid. The ratio of the displacements under the seismic force of the diaphragm divided by the displacements expected for a non-deformable solid is known as the flexibility index; these indexes are defined under numerical limits that allow considering the rigid or flexible behavior of the structural diaphragm.

In many investigations, the behavior of the diaphragm's stiffness has been evaluated analytically in different buildings, some of them in a portal frame structural system, with flat slabs, lightened slabs and composite slabs with concrete and steel [7] [8], others in the structural system with or wood diaphragms [9], others in combined buildings [10] and others in the structural system with load-bearing walls in square buildings and prefabricated diaphragms, such as the one carried out by Armenta [5]. Even design provision of building codes such as NSR-10 (Colombian seismic resistant building code) has been studied for semi-rigid or flexible diaphragm behavior [11].

This research studied the stiffness behavior of prefabricated diaphragms supported on a structural system of load-bearing walls in buildings with rectangular floor plan dimensions. In this study, a structural analysis of two buildings with the mentioned characteristics was carried out; exposed to 5 earthquakes scaled for site conditions with Aa= 0.15, Av=0.10 and soil profile Type D; and the behavior of the floor diaphragms of these buildings (story 1 to 5) was evaluated at each instant of action of the earthquakes. Flexibility indexes were calculated for the whole-time interval of each story and the flexibility or rigidity of these elements was evaluated.



(a) (b)

Figure 1. Precast slab connections. a) Panels and connection distribution detail in floor plan b) Connection detail between panels

2. Diaphragm Background

2.1. Behavior

A diaphragm is a roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements [12], which transmits the horizontal inertial forces to the vertical elements of the seismic resistance system [8]. One of the relevant characteristics of these elements is the high mass concentration they have due to the accumulation of self-weight of elements.

This mass distributed in the diaphragms (M), when it is excited by ground accelerations (Ag), generates seismic forces (F= -M*Ag) that modify the static equilibrium of the building. To compensate for these seismic forces, the structure develops inertial forces in diaphragms, restoring forces of the lateral resistance system (walls or columns) and damping forces (Figure 2).

With the external and internal forces, an equilibrium equation is stated:

$$F_{ext} = \Sigma F_{int};$$

$$-M*Ag = M*A+ C*V + K*U \qquad (1)$$

A = Structure acceleration; V = Speed of the structure;
U = Structure displacement; K = Structural stiffness;
C = Critical damping constant

The following equations (2 to 4) describe the mechanisms of load transmission from the diaphragms to the elements composing the lateral load-bearing resistance system.

$$-M*Ag(0) = M*A(0) + C*V(0) + K*U(0) \qquad (2)$$

Dynamic equilibrium in time zero

Given U(0) = 0 and V(0) = 0

$$-M*Ag(0) = M\ \ddot{U}(0) \qquad (3)$$

$$F_{seismic} = F_{inertial}$$

$$-\ \ddot{U}g(0) = \ddot{U}(0) \qquad (4)$$

According to equation 4, the structure initially accelerates with a magnitude equal to the ground acceleration. Once the movements start, displacement and velocity values different from zero are obtained and the restoring and damping forces of the system are increased (Figure 3).

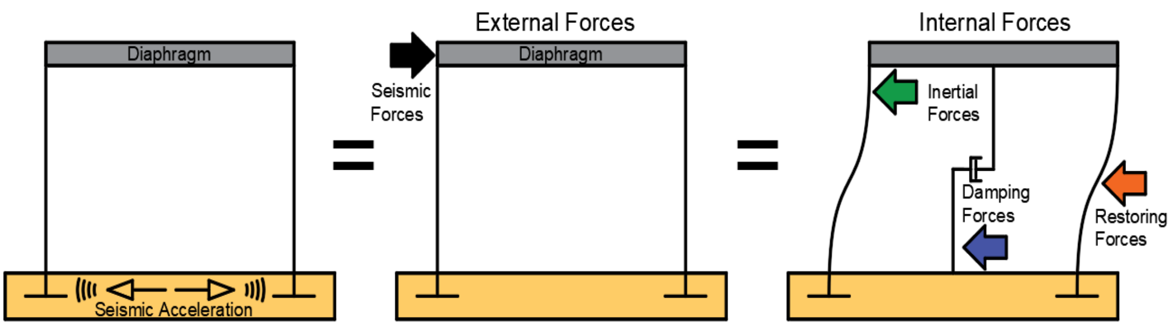


Figure 2. Seismic Acceleration Forces

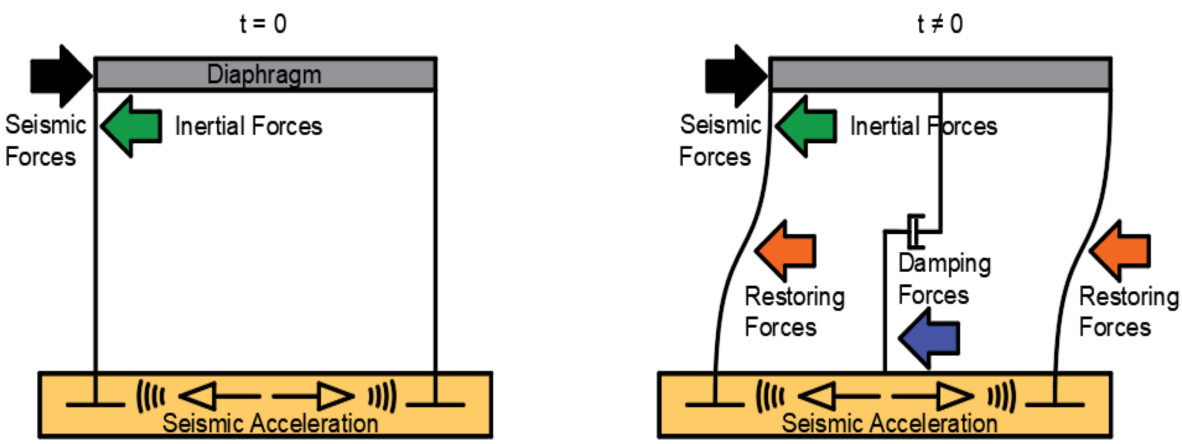


Figure 3. Load transfer

By dynamic equilibrium (equation 1); an increase of the restoring and damping forces, consequently, generates a decrease in the inertial force of the system, to maintain equilibrium. In this way, through the accelerations of the diaphragm, displacements are induced in the structure and a transfer of loads from inertial forces to forces in the structural resistance elements results.

From dynamic equilibrium in time zero

$$-M*Ag = M*A \uparrow$$

State in motion $t \neq 0$

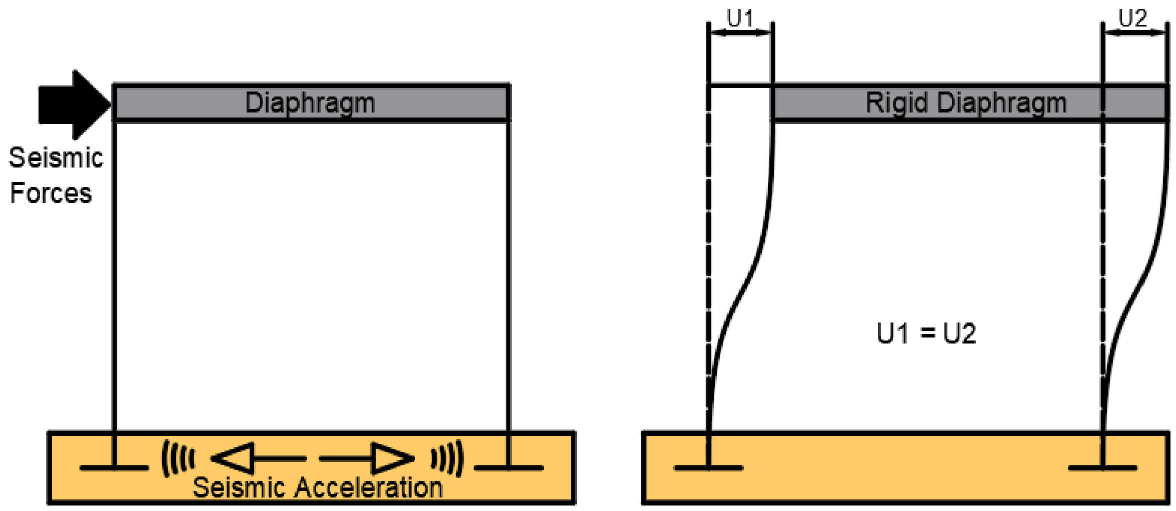
$$-M*Ag = M*A\downarrow + C*V \uparrow + K*U \uparrow$$

For the previous example, it was assumed that the seismic load at the time of the study was constant, this is not something that often occurs; however, it is useful to understand how the inertial forces resulting from the earthquake are transferred to forces in the lateral resistance elements. Certainly, the structure will be subjected to variable loads over time and will frequently change the direction of its movement; in the same way, the internal forces will also change to ensure equilibrium.

2.2. Classification

From the load transfer, diaphragms can be classified into rigid diaphragms or flexible diaphragms. Rigid diaphragms are structural elements that tend to behave in an almost completely rigid manner in relation to the displacements on their own surface; if this premise is valid, it is possible to determine the horizontal position of any point inside the diaphragm through 3 degrees of freedom, two orthogonal and one rotation with relation to the perpendicular axis [13]. Among the most important characteristics of the behavior of rigid diaphragms is the way in which they transfer the inertial forces to the structural lateral support system, providing and distributing loads to the elements by means of their stiffness [14].

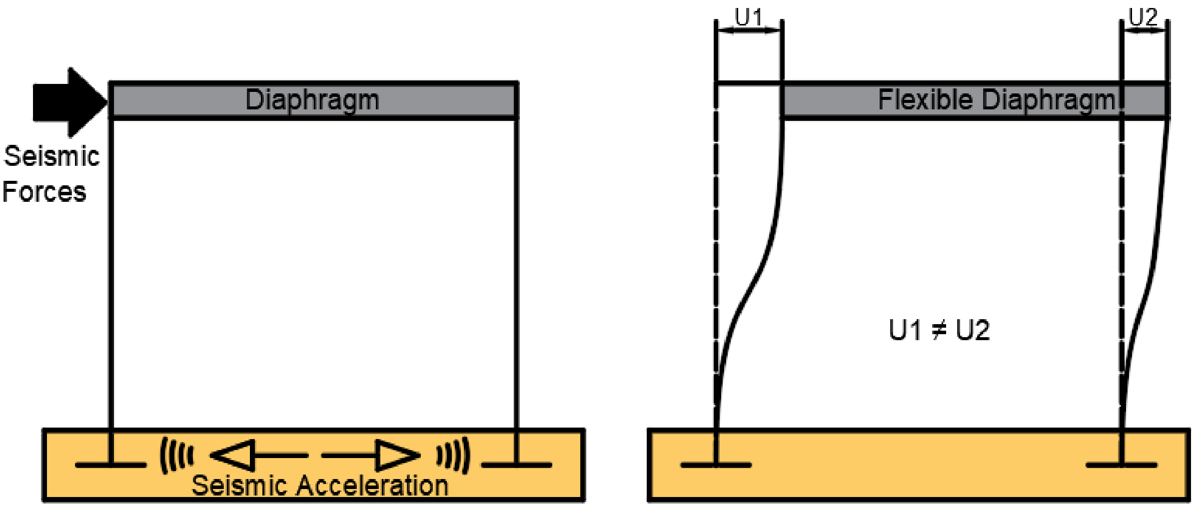
Figure 4 shows the behavior of a frame with a rigid diaphragm exposed to earthquake accelerations. Since the horizontal displacements at any point of the diaphragm are the same (since there are no considerable deformations). The proportion of restoring forces taken by each element depends exclusively on its stiffness [15]. The upper horizontal displacement of each column is the same.



F1=K1*U1 and F2= K2*U2

U1= U2; force distribution by element stiffness

Figure 4. Behavior of rigid diaphragms



F1=K1*U1 and F2= K2*U2

U1= U2; force distribution by element stiffness and by individual displacement of the element.

Figure 5. Behavior of flexible diaphragms

In contrast, flexible diaphragms may have deformations in their own plane; consequently, it is possible that, at different points of the element, the displacements may not be the same, as is observed in Figure 5 where the upper horizontal displacement of each column is different. The NSR-10 [16] makes the following statement about a flexible diaphragm "it is assumed to be flexible when the maximum horizontal deflection inside the diaphragm, when subjected to seismic forces (Fs), is more than 2 times the average of its horizontal deflections".

Therefore, the ratio of restoring forces taken by each element does not depend exclusively on the stiffness of the element, but also on its individual displacement.

In practice, engineers prefer to design rigid and not flexible diaphragms, for the simple reason that it is more practical to determine the structural dynamic behavior. By establishing that the lateral support elements (columns or walls) move the same as the diaphragm; and by defining a maximum displacement of this diaphragm as the drift limit (established in regulations); we obtain the load proportion that each structural element takes, taking in consideration its lateral stiffness (K_{el}) (Equation 5).

$$F_{max} = K_{el}*(\text{Drift Limit}) \quad (5)$$

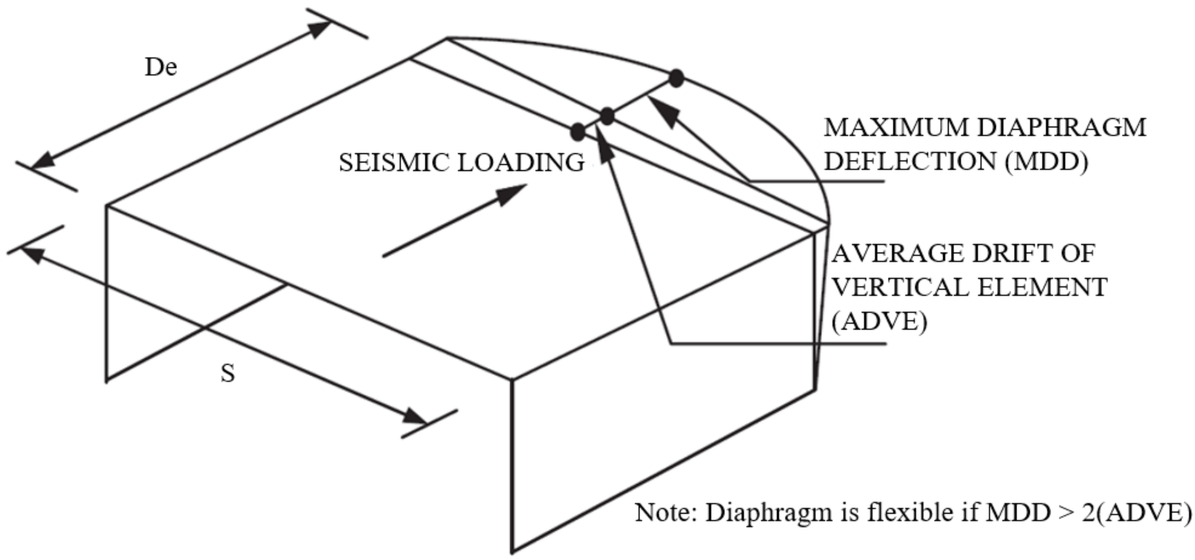
The restraint of deformations in the diaphragm plane (rigid diaphragm) limits the individual displacements that can occur in the structural elements and generate higher load concentrations to some of them. (Adapted from [14]). A continuous rigid diaphragm thus ensures all the lateral force-resisting components at the same level as a whole to resist lateral loads, which is difficult to adopt in larger structures as in high-rise modular buildings. Some studies suggest that the assumption of continuous rigid diaphragms is naturally not practical and there is a relatively flexible behavior which depends of the characteristics of the building itself [17] [18] [19].

3. Methodology

3.1. Diaphragm Evaluation

The effective properties of diaphragms of many precast concrete structures suggest that they may be flexible, this neglect the original assumption that the floor systems serve as rigid diaphragms between the vertical components of the lateral force-resisting system [15]. For that reason, the behavior of the diaphragm should be evaluated.

One way to evaluate whether the behavior of a diaphragm (rigid or flexible) is by the approach proposed by ASCE 7-10 (ASCE: American Society of Civil Engineering chapter 12.3) [6]. This methodology proposes to determine a flexibility index (α), obtained from the displacements generated in a moving diaphragm. The index is defined as the ratio of the in-plane diaphragm deformation and the absolute drift of the lateral system measured at the mid-height of the structure (Figure 6). The index values establish the degree of rigidity or flexibility that a diaphragm has.



(Adapted from ASCE 7-10 [5])

Figure 6. Flexibility index determination

This procedure is summarized in the following steps [20]:

a) Determine the maximum deformation of the diaphragm MDD
b) Determine the maximum average drift on each story ADVE
c) Calculate the flexibility index:

$$\alpha = MDD/ADVE \tag{6}$$

d) Consider a flexible diaphragm if

$$\alpha > 2 \tag{7}$$

e) Consider a rigid diaphragm if

$$0 < \alpha < 0{,}5 \tag{8}$$

This procedure was used to evaluate the behavior (rigid or flexible) of the diaphragm in prefabricated construction, proposed for the analysis in this research. For the buildings studied, the flexibility indices of each individual slab panel and the diaphragm were calculated.

3.2. Building Characteristics

Two buildings were studied in this investigation (Figure 7); each of 5 stories with a total height of 12.5 m (2.5 m each story). These buildings were subjected to accelerations of 5 seismic records scaled in the (Y-Y)-strong direction of the building. The structural system of these buildings consisted of concrete walls and prefabricated floor slabs (diaphragms). The diaphragm consisted of precast panels connected by spaced joints of 50 cm (Detail in Figure 7) and it was modeled taking into account the design considerations of chapter C.16 of the NSR-10 [16], regarding longitudinal and transversal ties to elements of the lateral load resistant system, the minimum number of ties in precast wall panels associated with a tensile load to resist, shear resistance in composite elements, among others. These provisions are based on the Precast/Prestressed Concrete Institute recommendations for the design of precast concrete load-bearing wall buildings [21]. These connections were made in the axes of the walls and perpendicular to them. In the other direction, the panels were connected by 9.5 mm (3/8 inch) diameter reinforcing bars, evenly distributed on the floor plan. The thickness of the walls and slabs was 10 cm.

Ratios of 1.5 and 2 were chosen to evaluate the behavior of the diaphragm with different rectangular geometries. The distribution of structural elements was kept the same in the two buildings, (e.g. Walls with maximum spacing between axes (Y-Y direction) of 3.5 m). The spacing was established in this way according to the maximum capacity (weight) that has to be considered in the design of the diaphragm. The spacing was established in order to take into consideration the maximum capacity (weight) of a crane to lift slab panels and place them on mezzanines or ceilings.

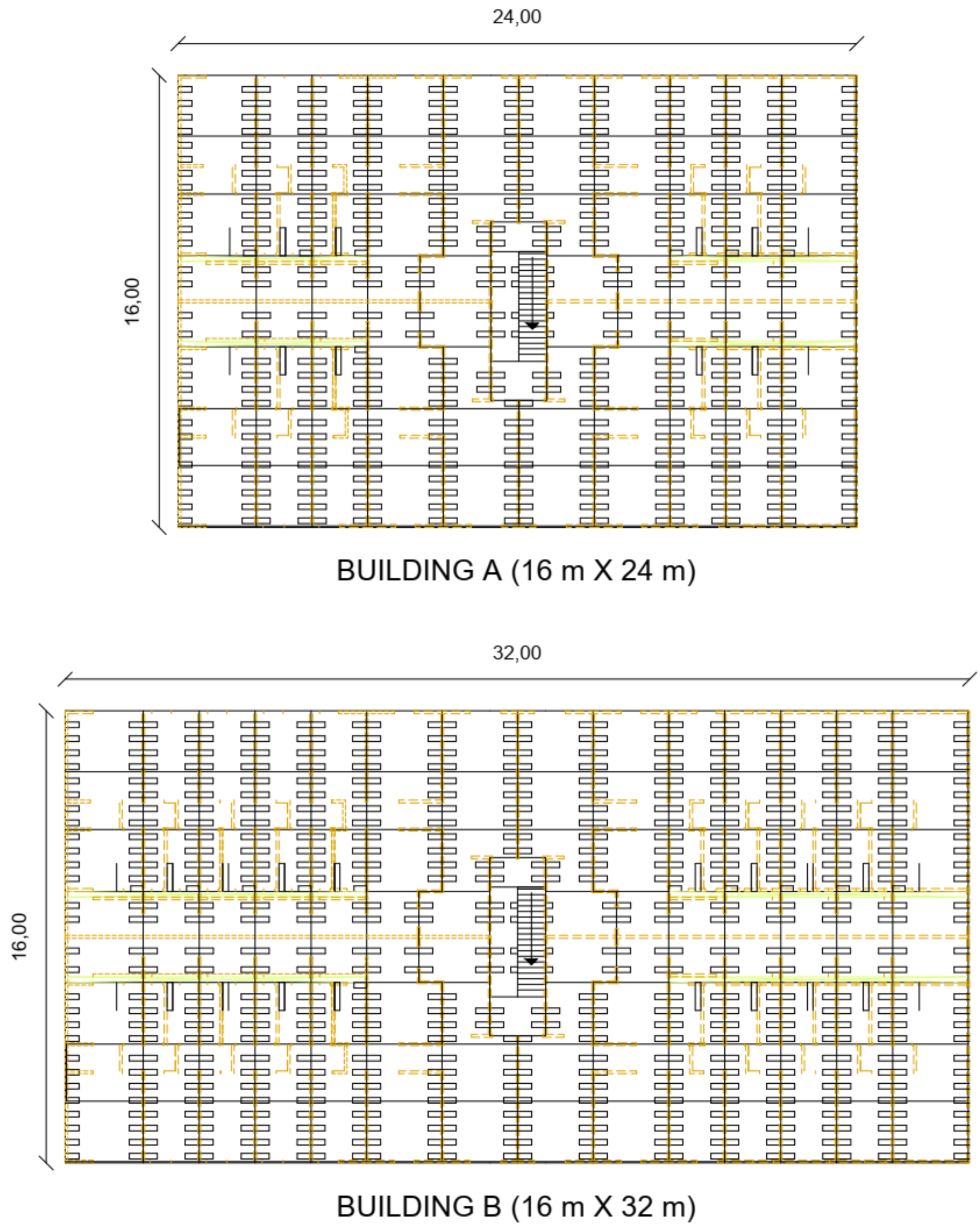


Figure 7. Building A and B plan, with precast panels connected by spaced joints. Smaller rectangles are reference to panel connections as is shown before in Figure 1

The concrete strength of all the structural elements chosen was f'c = 28 MPa, and the modulus of elasticity established in the analysis was obtained by the equation E=4700√(f'c) of chapter R19.2.2.1 of ACI 318S-14 [17]. The dead load addition to the self-weight of the structural elements considered in the analysis was 1.6 kN/m^2 (weight due to ceiling and floor finishes).

3.3. Seismic Parameters

For the seismic analysis of the buildings, the following data, detailed in Table 1, were considered:

Table 1. Seismic Parameters

Effective peak velocity (Av)	0.10
Effective peak acceleration (Aa)	0.15
Soil classification	D
Importance coefficient of the structure	1
Soil amplification coefficient (Fa)	1,5
Soil amplification coefficient (Fv)	2,4
Calculated peak hazard acceleration (g)	0,56

These data are typical values for a building constructed in an intermediate seismic hazard zone, in a type D soil profile. The design spectrum for these data is detailed in Figure 8.

3.4. Chronological Analysis

A chronological analysis was carried out, and the seismic record selected are presented in table 2.

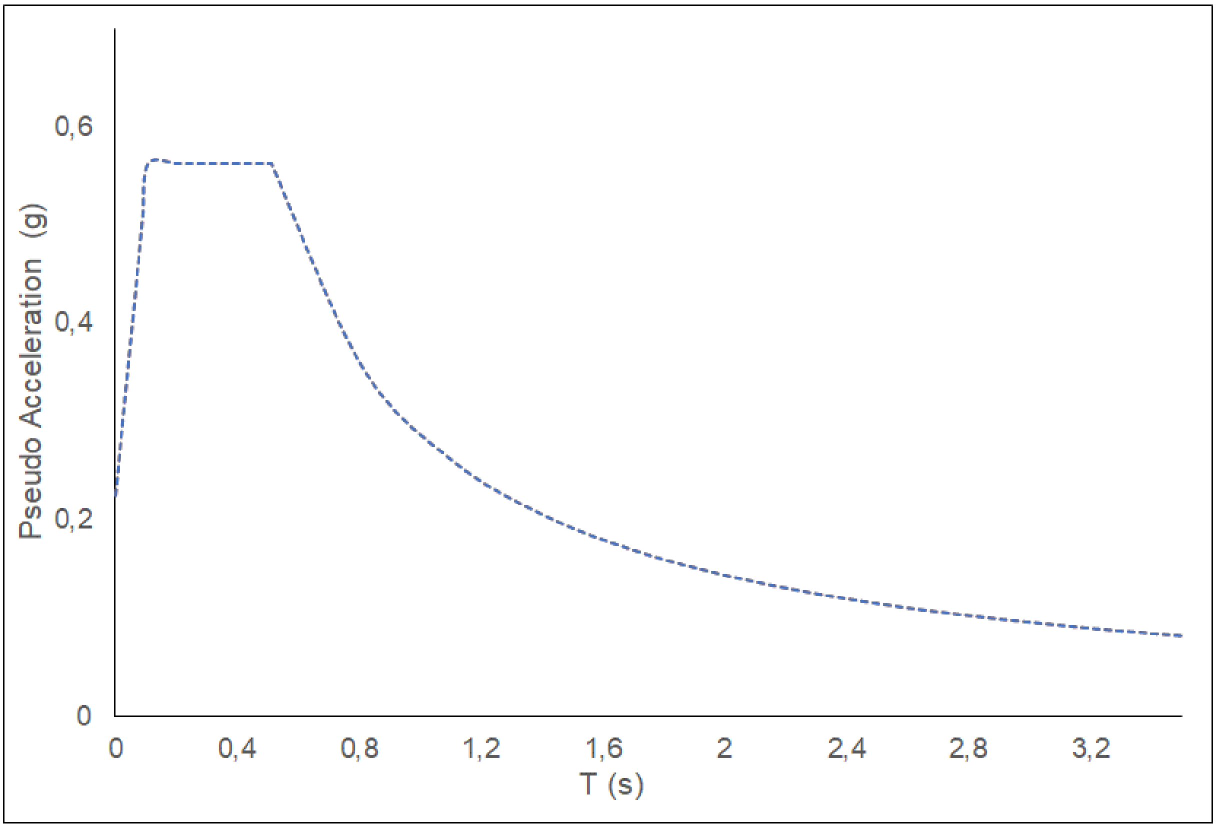


Figure 8. Elastic design spectrum

Table 2. Seismic Events

Seismic event	Location	Date	Magnitude	Depth
Oroville Airport	Oroville	08/08/1975	4,7 Mg	8,67 km
Sierra El Mayor -Cucapah	Baja California	04/04/2010	7,2 Mg	30,2 km
Landers	Burbank – Buena Vista	6/28 /1992	7,28 Mg	157,94 km
Cape Mendocino	Shelter Cove Airport	4/25 /1992	7,01 Mg	26,51 km
Kern County	Talft Lincoln School	7/21 /1952	7,36 Mg	38,42 km

The record of accelerations obtained in each seismological station is a representative record of the accelerations that occurred in the place of measurement; therefore, all records had to be scaled and adapted to the accelerations that could occur in the place of study of the buildings; as detailed in the design spectrum. The scale factor for each record was established according to chapter A.2.7 of NSR-10 [16]. The procedure details that the spectral ordinates of the scaled records, in the interval between 0.8T and 1.2T, should not be less than 80% of the accelerations obtained in the design spectrum. Additionally, it must be fulfilled that the average of these scaled ordinates, in the interval between 0.2T and 1.5T, must be greater than the ordinates of the same design spectrum. Table 3 and Figure 9 detail the scale factor determined for each seismic record.

Table 3. Scale factor seismic logs

Building	Oroville Airport	El Mayor -Cucapah	Landers	Cape Mendocino	Kern County
A	3.5	1.5	7.9	0.2	2.4
B	3.9	2.0	7.7	0.19	2.6

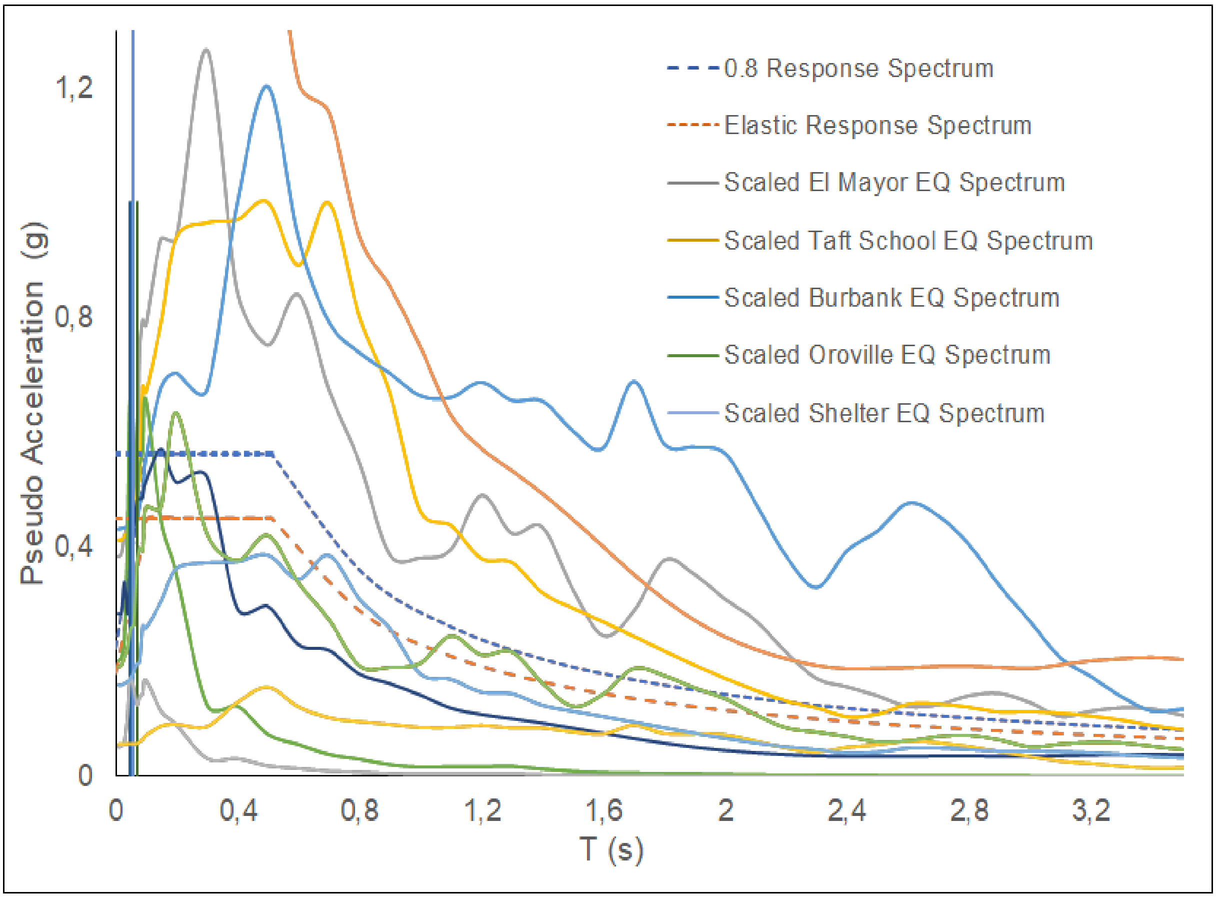


Figure 9. Design response spectrums and scaled response spectrums

3.5. Computational Analytical Model

A linear-three-dimensional analytical model (Figures 10 and 11) was developed in software SAP2000 [23], with geometric characteristics as established before for buildings A and B (Figure 7). The walls and diaphragm were modeled by 10 cm thick shell elements. The diaphragm consisted of slab panels connected by 10 cm wide by 50 cm long elements (space where the bars are welded and the concrete is poured); the 50 cm length is distributed in 25 cm towards each panel (Figure 11). The axes of the walls were located at distances of less than 3 m, in total 15 wall axes for building A (16 m x 24 m) and 17 for building B (16m x 32m) and. The largest spacing between walls was 2,7 m according to the project plans.

The supports of the structure were assumed to be embedded and the meshing of diaphragms was limited to elements with an average dimension of 40 cm.

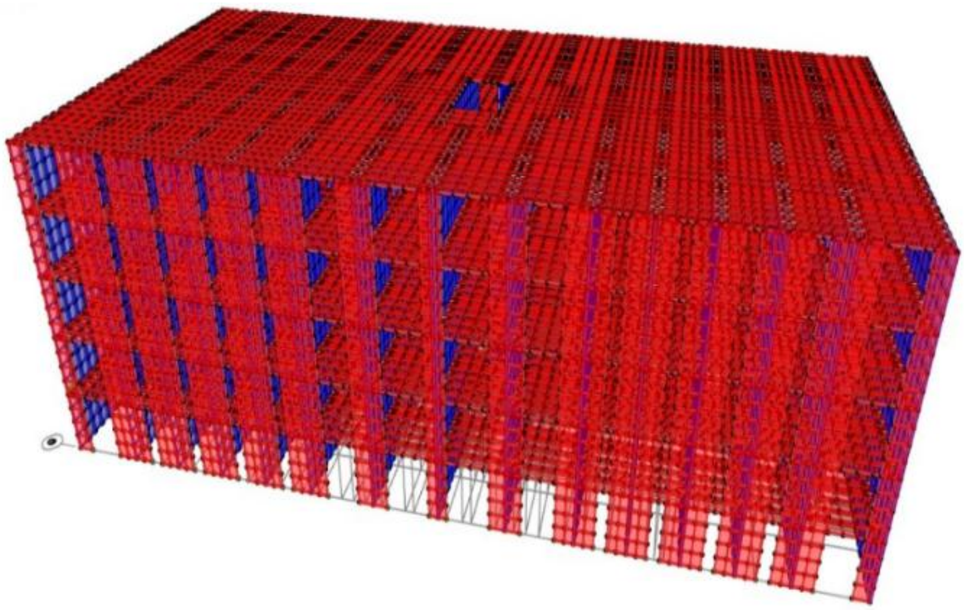


Figure 10. 3D computer model of Building B

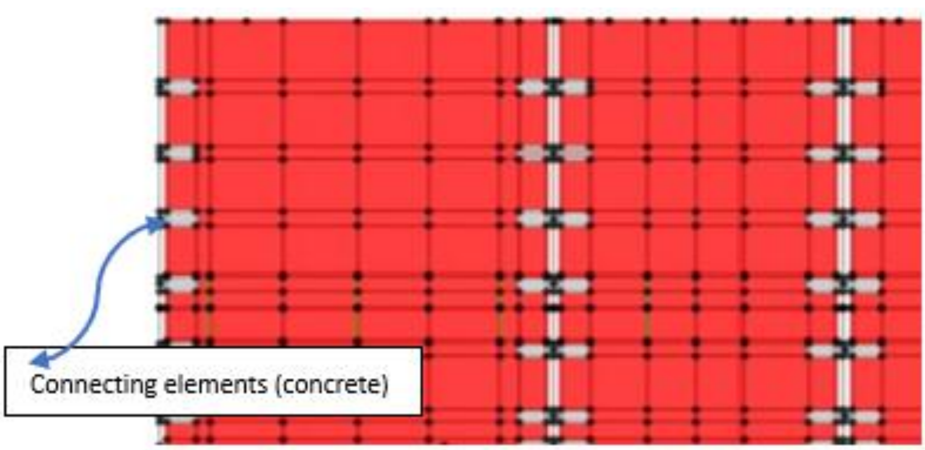


Figure 11. Detail of connecting elements between slab panels (Connections to wall shafts)

3.6. Determination of Flexibility Indices

The flexibility indexes were determined with the displacements obtained in the measurement points of each panel: one in each lateral wall (left and right) and one in the center, as detailed in Figure 12; indexes were also measured for the complete diaphragm, with the displacements of the edge walls and the displacements of the center of the building (Figure 13). With these values, Equations 7 and 8 were used to determine whether the behavior of the diaphragm corresponded to that of a rigid or flexible element.

The determination of the flexibility indexes was carried out throughout the duration of the earthquake; in other words, displacement measurements were taken for each instant of the seismographic record and indexes were calculated throughout the course of time (Detail in Table 4).

Table 4 shows calculations of flexibility indexes of panel 1 of Building A, when time is equal to 0,18 seconds, as an example. In the research exercise, these data were analyzed up to the duration of the earthquake recording (Oroville).

Table 4. Flexibility index calculations in an instant of time for panel 1

Flexibility indices calculations - Panel 1 (Shelter EQ)							
Time (s)	**Q**	**Right wall displacement (mm)**	**Drift average (mm)**	**Maximum center measurement (mm)**	**Maximum center deformation (mm)**	**Flexibility index (mm)**	**Diaphragm behaviour**
0,18	0,0012	-0,0009645	-0,001	-0,00103	-0,0000372	0,034903	Rigid

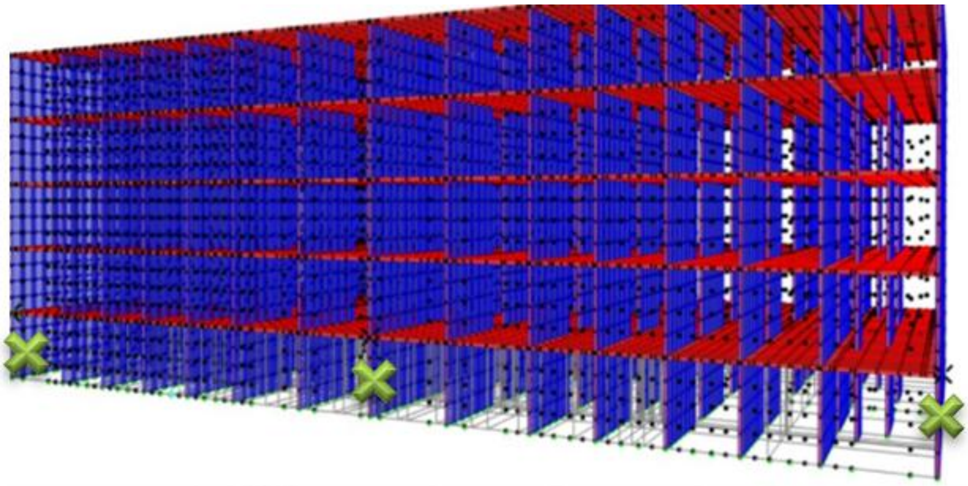


Figure 12. Displacement measurement points for calculation of panel flexibility indices 2 – Floor 1

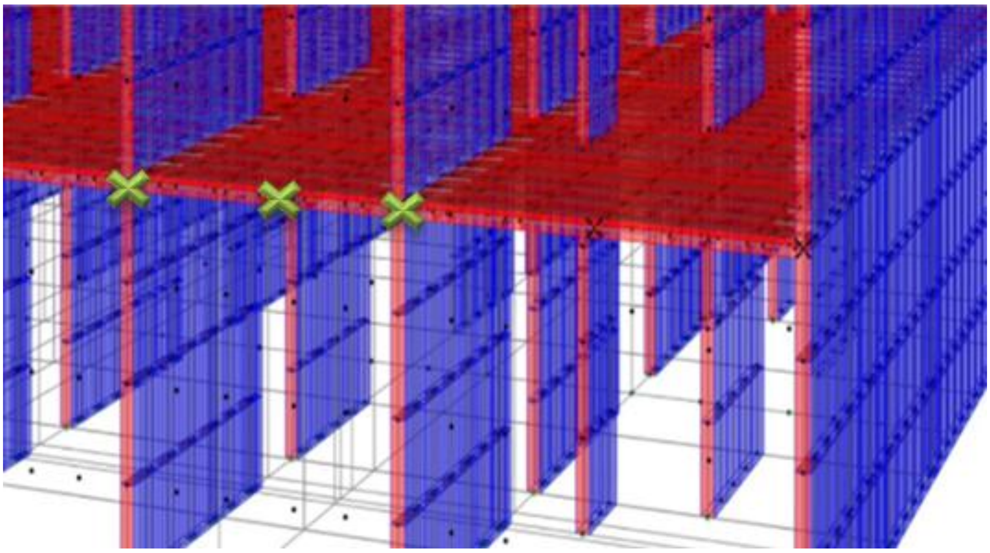
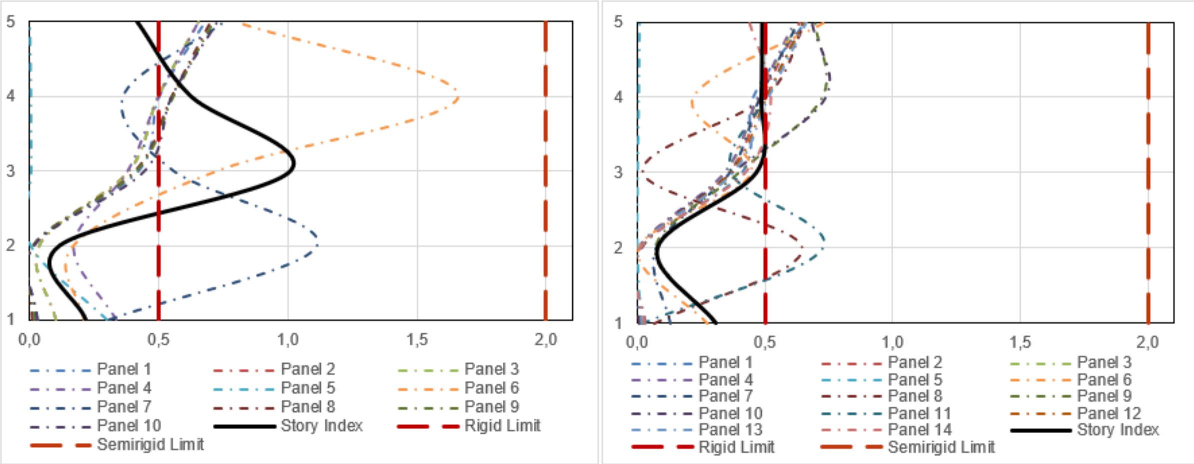


Figure 13. Displacement measurement points for calculation of flexibility indexes in the full diaphragm. – floor 1, building B.

4. Results

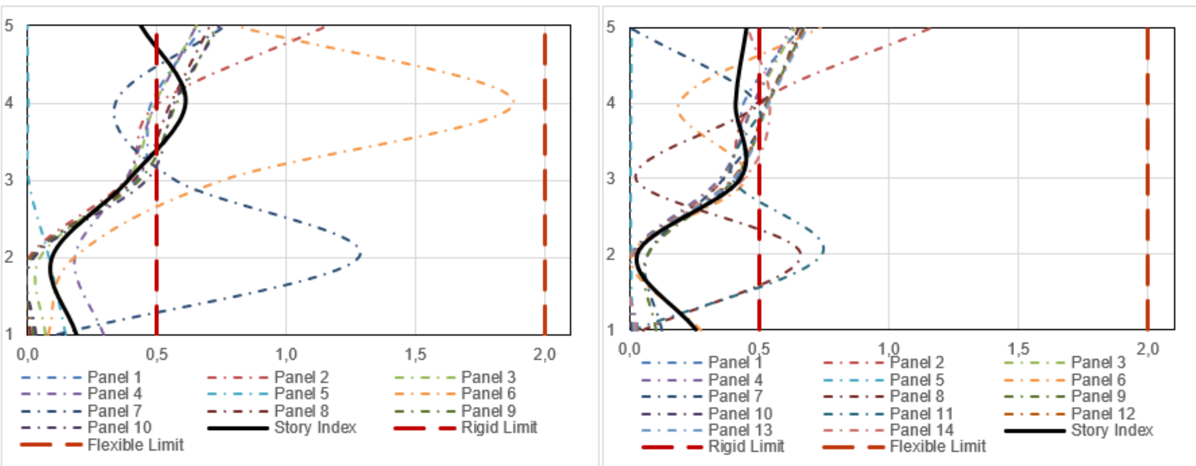
In the present investigation, the behavior of all the panels of Building A and B, on each of the floors, was determined for the five scaled seismic records; the overall behavior of the total diaphragm of the building was also verified.

The results are presented in Figures 14 to 18, which details the values of the indices when the maximum displacements occur in the walls of a panel; that is, the values of indices were taken when the maximum displacement occurred in the left lateral wall and the right lateral wall, and then the greater of the two was plotted. The same procedure was performed for the entire diaphragm. At the end of the results, some tables of the behavior of the panels and the complete diaphragm when displacements other than the maximum displacements occur in walls previously described are detailed.



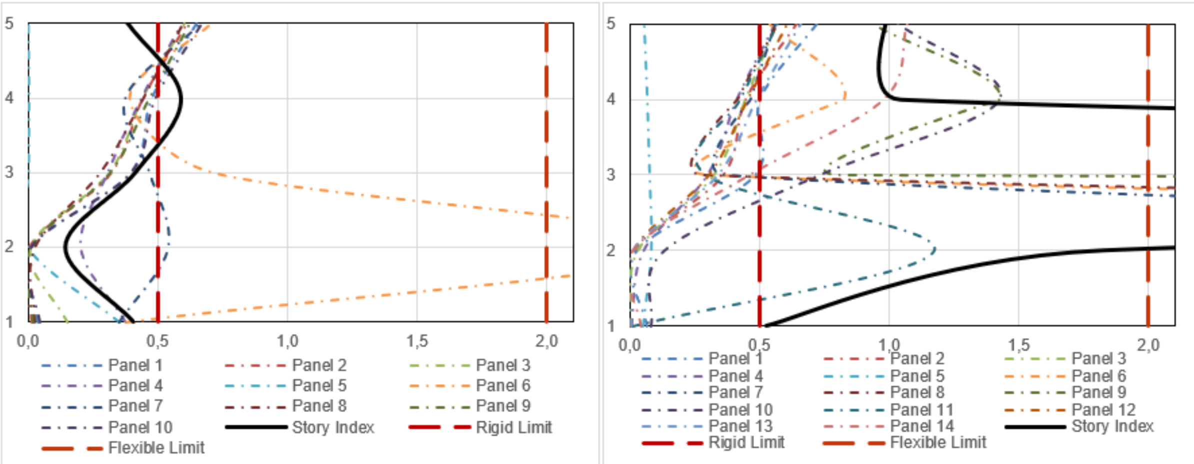
a) b)

Figure 14. Flexibility index for maximum panels displacement and story diaphragm (story index), Kern County Earthquake. X axis (Flexibility index) Y axis (Story level). a) Flexibility Index Building A b) Flexibility Index Building B



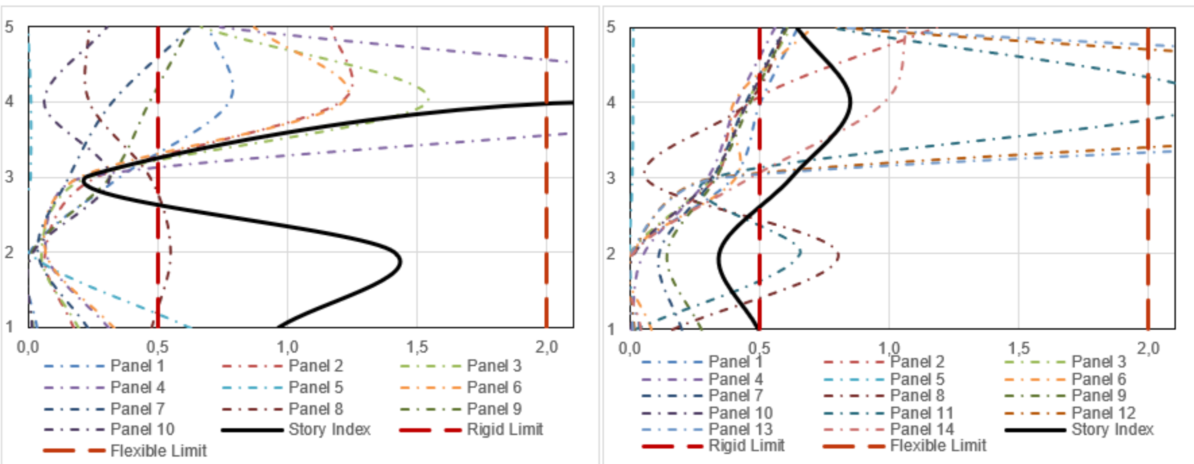
a) b)

Figure 15. Flexibility index for maximum panels displacement and story diaphragm (story index), Landers Earthquake. X axis (Flexibility index) Y axis (Story level). a) Flexibility Index Building A b) Flexibility Index Building B



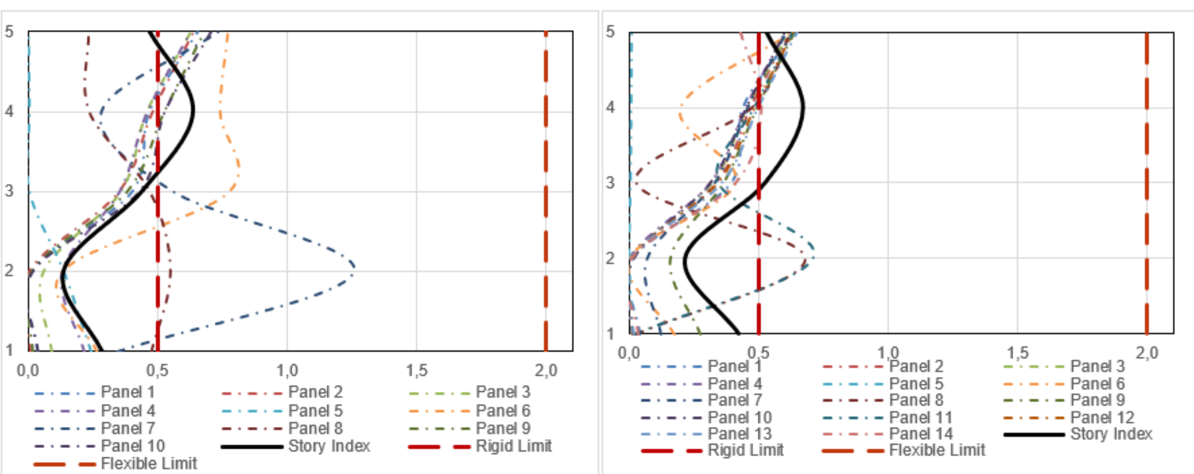
a) b)

Figure 16. Flexibility index for maximum panels displacement and story diaphragm (story index), Oroville Airport Earthquake. X axis (Flexibility index) Y axis (Story level). a) Flexibility Index Building A b) Flexibility Index Building B



a) b)

Figure 17. Flexibility index for maximum panels displacement and story diaphragm (story index), Cape Mendocino Earthquake. X axis (Flexibility index) Y axis (Story level). a) Flexibility Index Building A b) Flexibility Index Building B



a) b)

Figure 18. Flexibility index for maximum panels displacement and story diaphragm (story index), El Mayor Earthquake. X axis (Flexibility index) Y axis (Story level). a) Flexibility Index Building A b) Flexibility Index Building B

In the entire duration interval of each earthquake, numerical values of flexibility indices greater than 2 were found, associated with a flexible behavior of panels and complete diaphragms on each floor. The values presented in Tables 5 to 9 were obtained for wall displacements different from the maximum displacements.

Table 5. Flexible behaviour in another instant of time - Landers Earthquake

Flexible behavior in another instant of time - Landers Earthquake					
Location	Time Interval (s)	Building A		Building B	
		Panel or Story (S) α > 2	Some α values	Panel or Story (S) α > 2	Some α values
Story 1	(1- 54)	1, 2, 3, 4, 5, 6, 7, 8, 10	6.5, 10.4, 3.4, 6.3	1, 2, 3, 4, 5, 6, 7, 8, 9, 11, 12, 14	5.2, 2.7, 4.7, 7.6, 16.2
Story 2		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	2.8, 3.7, 2.1, 8.3	1, 3, 4, 5, 6, 7, 8, 9, 10, 11, 13, 14	6.6, 9.9, 2.3, 5.8, 10.2
Story 3		1, 2, 3, 4, 6, 7, 8, 9, 10	14.2, 18.8, 6.0, 3.4,	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13, 14	9.8, 5.0, 3.5, 8.6, 15.1
Story 4		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	4.8,7.0, 6.3, 2.6, 4.2	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13, 14	5.4, 12.7,7.3 4.4, 10.8
Story 5		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	3.2, 6.2, 3.5, 6.8	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13, 14	7.2, 5.4, 8.8, 3.7, 2.7
Diaphragm		(S) 1, 2, 3, 4, 5	4.3, 6.3, 10.5, 5.9	(S) 1, 2, 3, 4, 5	7.5, 6.4, 5.7, 8.1,7.3

Table 6. Flexible behaviour in another instant of time -Kern County Earthquake

Flexible behaviour in another instant of time - Kern County Earthquake					
Location	Time Interval (s)	Building A		Building B	
		Panel or Story (S) α > 2	Some α values	Panel or Story (S) α > 2	Some α values
Story 1	(1- 54)	1, 2, 3, 4, 5, 6, 7, 8, 10	9.6, 2.0, 5.8, 4.0	1, 2, 3, 5, 6, 7, 8, 9, 10, 12, 13, 14	4.5, 8.3, 7.2, 3.1, 4.5
Story 2		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	10.6, 6.0, 2.3, 3.0, 5.1	1, 3, 4, 6, 7, 8, 9, 10, 11, 13, 14	6.2, 9.6, 4.5, 2.5, 4.7
Story 3		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	7.0, 10.6, 3.7, 4.6	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13	8.0, 4.3, 6.3, 8.8, 2.8
Story 4		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	15.8, 12.0, 13.4, 6,2	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13	6.5, 3.0, 7.7, 4.5, 5.7
Story 5		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	4.8, 13.0, 3.8, 4.8, 3.4	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13	5.7, 3.7, 2.6, 8.4, 3.1
Diaphragm		(S) 1, 2, 3, 4, 5	11,0, 7.8, 15.4, 10.0	(S) 1, 2, 3, 4, 5	5.3, 4.7, 6.2, 7.0, 7.4

Tabla 7. Flexible behavior in another instant of time -Cape Mendocino Earthquake

Flexible behaviour in another instant of time - Cape Mendocino Earthquake					
Location	Time Interval (s)	Building A		Building B	
		Panel or Story (S) α > 2	Some α values	Panel or Story (S) α > 2	Some α values
Story 1	(1- 30)	1, 2, 3, 4, 5, 6, 7, 8, 9, 10	10.7, 2.3, 4.0, 7.1, 2.0	1, 4, 5, 6, 7, 8, 9, 10, 13	3.5, 4.4, 8.7, 2.4, 6.6
Story 2		1, 2, 3, 4, 5, 6, 7, 8, 9	5.8, 3.8, 12.1, 2.0, 5.1	3, 4, 5, 6, 7, 8, 9, 11, 13	3.3, 4.8, 7.9, 2.4, 6.4
Story 3		1, 2, 3, 4, 6, 7, 8, 9, 10	14.3, 4.7, 2.1, 3.9, 2.0	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	3.0, 4.7, 5.8, 4.5, 9.3
Story 4		1, 2, 3, 4, 6, 7, 8, 9, 10	5.1, 2,0, 14.6, 5.1, 7.8	1, 2, 3, 4, 6, 7, 8, 9,10, 11, 12, 13	5.8, 6.3, 8.3, 2.4, 9.2
Story 5		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	7.2, 3.0, 6.6, 2.7	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13	3.9, 2.4, 4.8, 3.3, 8.0
Diaphragm		(S) 1, 2, 3, 4, 5	7.5, 2.7, 3.3, 3.7, 4.7	(S) 1, 2, 3, 4, 5	5.6, 7.9, 4.3, 11.5, 7.8

Tabla 8. Flexible behavior in another instant of time -El Mayor Earthquake

Flexible behavior in another instant of time - El Mayor Earthquake					
Location	Time Interval (s)	Building A		Building B	
		Panel or Story (S) α > 2	Some α values	Panel or Story (S) α > 2	Some α values
Story 1	(1-250)	1, 2, 3, 4, 5, 6, 7, 8, 9, 10	10.7, 2.3, 4.0, 7.1, 2.0	1, 4, 5, 6, 7, 8, 9, 10, 13	3.5, 4.4, 8.7, 2.4, 6.6
Story 2		1, 2, 3, 4, 5, 6, 7, 8, 9	5.8, 3.8, 12.1, 2.0, 5.1	3, 4, 5, 6, 7, 8, 9, 11, 13	3.3, 4.8, 7.9, 2.4, 6.4
Story 3		1, 2, 3, 4, 6, 7, 8, 9, 10	14.3, 4.7, 2.1, 3.9, 2.0	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	3.0, 4.7, 5.8, 4.5, 9.3
Story 4		1, 2, 3, 4, 6, 7, 8, 9, 10	5.1, 2,0, 14.6, 5.1, 7.8	1, 2, 3, 4, 6, 7, 8, 9,10, 11, 12, 13	5.8, 6.3, 8.3, 2.4, 9.2
Story 5		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	7.2, 3.0, 6.6, 2.7	1, 2, 3, 4, 5, 6, 7, 8, 9,10, 11, 12, 13	3.9, 2.4, 4.8, 3.3, 8.0
Diaphragm		(S) 1, 2, 3, 4, 5	7.5, 2.7, 3.3, 3.7, 4.7	(S) 1, 2, 3, 4, 5	5.6, 7.9, 4.3, 11.5, 7.8

Table 9. Flexible behaviour in another instant of time - Oroville Earthquake

Flexible behaviour in another instant of time - Oroville Earthquake					
Location	Time Interval (s)	Building A		Building B	
		Panel or Story (S) α > 2	Some α values	Panel or Story (S) α > 2	Some α values
Story 1	(1- 250)	1, 2, 3, 4, 5, 6, 7, 8, 9, 10	2.2, 3.4, 3.7, 2.1	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	4.8, 2.17, 6.7, 7.5
Story 2		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	2.0, 6.2, 7.3, 3.5	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	10.13, 2.4, 3.6, 5.8
Story 3		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	4.8, 10.5, 8.0, 2.6	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	2.6, 3.1, 7.2, 2.2, 2.1
Story 4		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	7.9, 4.5, 3.3, 6.1	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	2.0, 2.8, 5.9, 13.6
Story 5		1, 2, 3, 4, 5, 6, 7, 8, 9, 10	11.1, 2.17, 2.0, 2.3	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	2.5, 2.0, 3.9, 2.9
Diaphragm		(S) 1, 2, 3, 4, 5	7.3, 2.5, 4.8, 7.5	(S) 1, 2, 3, 4, 5	2.0, 12.5, 3.8, 2.4, 4.4, 8.0

5. Discussion

The model proposal developed in this research was based on a modal dynamic historical analysis, which is a notable difference with respect to other investigations. The flexibility indexes found expose values lower than 0.5, higher than 2.0 and intermediate values; this means that in the term of measurement of displacements due to earthquake effects in the studied structures (with a ratio of sides in plan 1.5 and 2) a rigid, semi-rigid and flexible behavior of the prefabricated diaphragm system was evidenced.

In previous research, such as that of Armenta [5], developed on the same prefabricated mezzanine system with slab panels supported on concrete walls, the diaphragms detailed a rigid behavior for buildings with square geometry (ratio of sides in plant 1).

However, research such as that of Tena, Chinchilla, and Juarez [7], have shown that certain mezzanine systems such as lightweight, composite (Steel deck) and waffle type flat slab, can behave as rigid, semi-rigid and flexible diaphragms depending on the ratio of sides of the building as is demonstrated in this investigation. This same appreciation is evidenced in the research developed by D'Arenzo, Casagrande and Seim [9], which details variable behaviors in terms of stiffness of the floor slab systems with plywood diaphragms supported on walls; for the same floor slab system but in hybrid buildings with concrete walls, Avila, Dechent and Opazo [10], corroborated that the diaphragm behaves to a greater extent as semi-rigid (66%) and flexible (33%), for some of the cases studied.

According to the evidence presented in some research [17] [19], it is possible to affirm that there are many variables that can influence the behavior in terms of stiffness of the diaphragms of the story in buildings, some of these are the following: geometry of the building, type of structural system, material and characteristics of the slab, among others which can be evidenced also in this study.

6. Conclusions

The results obtained from the computational modeling of two buildings (A and B) with geometric characteristics of plan dimensions 16 x 24 m and 16 x 32 m and a structural system of load-bearing walls with diaphragms built by connected panels show a non-rigid behavior in some structural elements with flexibility indices greater than 0.5.

A first estimation of the flexibility indices obtained when the maximum displacements are generated in the walls of a panel or of the complete diaphragm (maximum forces in elements), could detail a rigid or flexible behavior of these elements; However, a more detailed analysis taking into account the displacement vs. time history, throughout the measurement (or action) intervals of each earthquake, confirms the fact that the diaphragms of the floors of the study buildings (A and B) and the panels associated to these diaphragms behave in a flexible manner. The flexibility indices show rigid, semi-rigid and flexible behavior. Some semi-rigid result can be observed in figures 14, 15 and 18 and flexible behavior are detailed in Figures 16 and 17. These figures show the variation of the structural response in proportion to the characteristics of each earthquake.

Also, the flexible behavior is supported by the results presented in Tables 5 to 9, where it is evident that for different values of time, displacements with flexibility indices greater than 2.0 are generated.

The variability of the indices indicates a high sensitivity of the response to different patterns of earthquake accelerations for the diaphragm performance [24]. Therefore, it is the duty of the structural engineer to evaluate the degree of stiffness of the diaphragm in each of the cases where this structural system is implemented, with the purpose of providing the structure with the necessary capacity to adequately resist the requests of a seismic event, having a correct distribution of internal loads in elements due to the condition of rigidity or flexibility of the diaphragm.

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