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The Effects of Temporary Tower Cranes on the Construction Process and Seismic Behavior of Reinforced Concrete Buildings

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ABSTRACT

When space is limited on project site to install a tower crane for the construction of high and mid-rise buildings, project managers have to deal with the challenge of assembling such crane somewhere in the middle of the building, leaving an opening in the floor slabs. Nevertheless, there is limited evidence about the effects of this type of temporary structure on the seismic behavior of buildings. The study analyzes the effect of having temporary openings in slabs of reinforced concrete buildings, when tower cranes need to be mounted inside a building. A structural intervention on the buildings' slabs was simulated to resemble the temporary openings required to install a tower crane. Results indicated that seismic deformations were almost the same with and without the slab opening, which enables the installation of tower cranes through the slabs of reinforced concrete buildings, without affecting serviceability levels and seismic behavior of buildings.

1. Introduction

Chile is a high-seismicity country, geographically located between two tectonic plates (Nazca and South American), turning it structurally vulnerable. For that reason, Civil Engineers are looking to develop procedures to mitigate the impact suffered during and after an earthquake, and to implement new techniques and design methods (Leyton et al., 2010).

Buildings, especially high-rise buildings are designed considering the seismic analysis based on the building features. Alongside, different parameters that must comply with values and minimum requirements established by local standards are assessed. This is done to reduce the vulnerability of structures and to protect people's lives (Katsanos and Vamvatsikos, 2017).

Due to its long-range, the ideal equipment for these types of construction works is the tower crane. This machine is commonly used to handle loads and materials and is capable of transporting these with accuracy. Their location depends on various factors, mainly on the available space where the building will be erected (Huang et al., 2011; Younes and Marzouk, 2018).

Said the above, when space is limited to install a tower crane, an assembly is set throughout the building, leaving an opening in the floor slabs, resulting in a building structure that should be treated as an irregular structure. This work aims to study the seismic impact caused by the installation of a tower crane inside a building, due to the limited available information on this specific topic. Furthermore, results from this study will help to verify the technical feasibility in terms of serviceability and seismic behavior, to continue implementing this construction practice.

The study considered two buildings of different geometric sections (footprint view), which were intervened on their slabs. It simulated the temporary void required to install the tower crane throughout the building.

Spectrum modal analysis was used to evaluate the seismic deformations, based on the current Chilean standard (Chilean National Institute of Standards, 2009) for the seismic design of buildings. Likewise, the Standard's upgrades performed after the earthquake of February 27, 2010, were also considered. These criteria are based on international standards (Saragoni, 2011; Wallace, 2012; Deger and Wallace, 2015). A structural analysis software was used to obtain drift values, considering building types with and without the temporary intervention on their slabs. This method allowed comparison and verification of significant

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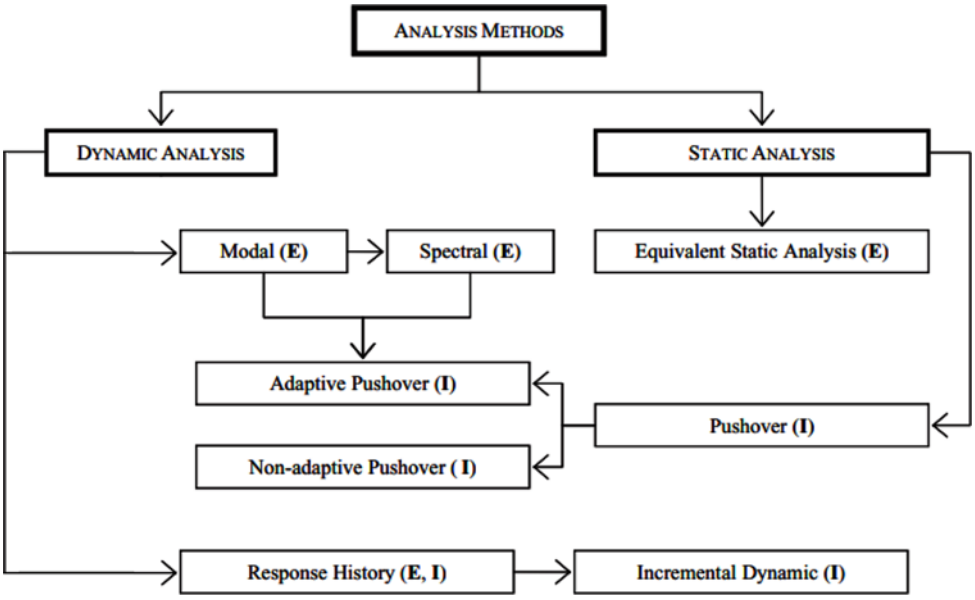

Fig. 1. Structural Analysis Methods Used in Seismic Design (adapted from Elnashai and Di Sarno, 2008)

statistical differences between the two scenarios.

Therefore, the main goal of this work is to analyze the seismic behavior on reinforced concrete buildings, subdued to structural alterations due to temporary installations of tower cranes inside their structure. This study mainly aims to: 1) determine the seismic demand, through the design spectrum for the conditions of the city of Concepción-Chile based on current local standards; 2) select the optimum location of a tower crane inside the building, to assess potential harmful effects over the building's structural response; 3) analyze the results before and after the structural modification of the slabs for the models under study, by applying statistical methods; and 4) check, based on results, the feasibility of installing a tower crane inside the building.

2. Literature Review

2.1 Seismic Analysis

Seismic analysis is a subset of structural analysis, which aims to estimate a structure's response to an earthquake. It is part of structural design processes, earthquake engineering or structural evaluation, in places where earthquakes are common (Sunil et al., 2017; Panneerselvam et al., 2017).

During the nineteenth century, several events had to occur before civil engineers and experts on the field developed various seismic-resistant analysis methods. Despite the attention caused by these extreme events, until then, engineers were not ready to provide a solid seismic solution, since expert knowledge was based on static behavior, leaving dynamic behavior far from knowledge back then. However, in 1908, a huge earthquake devastated the city of Messina (Italy) and its surroundings, causing the death of approximately 83,000 people, a consequence by far too disastrous as compared to previous events. This event was responsible for the practical design of structures to resist seismic demands (Housner, 1984).

There has currently been an increase in the use of seismic analyses, both from industry and academia, mainly due to more sophisticated computers available, as well as the increase of friendly-user software (Elnashai and Di Sarno, 2008). There are currently various structural analysis methods used for seismic engineering purposes, grouped in two main categories as shown in Fig. 1.

The seismic codes of each country generally regulate the choice of the method of structural analysis for the seismic design of a building. The current seismic regulation in Chile is the NCh433 (Chilean National Institute of Standards, 2009). This standard includes seismic design approaches similar to those used in both ASCE 7 and ACI 318 international codes (Wallace et al., 2012). Even more, the NCh433 standard was specially calibrated for reinforced concrete buildings that have a large amount of total shear wall to floor area (total shear wall density). In Chile, most reinforced concrete buildings have a large total shear wall density that ranged between 5% and 6%, which is relatively large compared with buildings of similar height in seismic regions elsewhere (Junemann et al., 2015).

The NCh433 Chilean standard allows the type of seismic analysis to be based on the linear-elastic behavior of the structure, like spectral modal analysis, since there are not yet inelastic analysis procedures simple enough to be applied in professional practice. For this reason, the emphasis of the provisions of this standard was not put into more sophisticated methods of analysis, but require strict limitations on seismic deformations for safe and proper structuring seismic behavior. For example, in order to obtain enough lateral stiffness and prevent damage to secondary elements, the inter-story drift measured in the center of mass is not allowed exceeding the value of 0.002. On the other hand, to obtain reasonably symmetrical structures and with

enough torsional rigidity, the difference between the inter-story drifts measured in the center of mass and the plant's corners is not allowed exceeding the value of 0.001. This seismic design philosophy has been very successful. During the 2010 8.8Mw Concepción earthquake in Chile, one of the biggest recorded in human history, a large majority of reinforced concrete (RC) buildings performed well. Only 2% of the estimated 2,000 RC buildings taller than nine stories suffered substantial damage during such a vast earthquake (Junemann et al., 2015).

2.1.1 Static Analysis and Dynamics of Structures

According to historical records, the earthquake from Messina (Italy) in 1908, was the basis for the practical seismic design of structures, since a group of experts provided the first recommendations about the equivalent static method after the earthquake. Later, this method was known around the globe and used in countries with high seismicity levels. At first, the Static Method applied for structural analysis considered a certain seismic coefficient, until the Los Angeles city (USA), modified the seismic design requirements in 1943. Since then, a more rigorous value for this coefficient was obtained, which considered the vibration period. This was the first-time seismic requirements from construction codes considered the mass and flexibility of buildings, which are based on the structures' dynamics (Housner, 1984). For this reason, various authors have recognized the importance of the vibration period, and the way it determines the dynamic behavior of structures. This period is directly related to the building's height and mass, and inversely proportional to its stiffness (Crowley and Pinho, 2004; Guler et al., 2008). This type of analysis was enormously favored by computers since they made it easier to understand concepts such as response spectrum and design spectrum (Housner, 1984). In this context, Bozorgnia and Bertero (2004) define the response spectrum as the maximum response expressed in terms of displacement and velocity or acceleration, that produces a dynamic action over a structure or a one-degree of freedom oscillator.

2.1.2 Considerations for Spectrum Modal Analysis

When a seismic analysis for a building is performed, it is common to use a Spectrum Modal Analysis, (Crempien et al., 1989). The method is conceptually simple and easy to apply through commercial structural analysis programs (Barradas and Ayala, 2014). For that reason, various international standards recommend using this method; specifically, in Chile, the NCh 433 code for seismic design of buildings is used (Crempien et al., 1989; Chilean National Institute of Standards, 1996). According to this code, the structural model must consider a minimum of three degrees of freedom per floor (two translational and one rotational). Conversely, the number of modes required in the analysis must be included, to obtain at least 90% of the total mass for each seismic direction analyzed. This simplifies the analysis considerably since the first modes commonly comply with the condition previously mentioned. The code also specifies that the combination of maximum mode contributions be performed

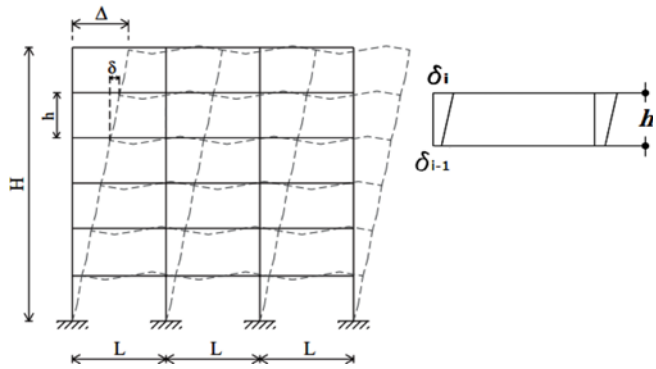

Fig. 2. Interstory Drift (adapted from Elnashai and Di Sarno, 2008)

by using the Complete Quadratic Combination (CQC) method, considering a constant buffering rate for all vibration modes equal to 5% (Chilean National Institute of Standards, 2009; Lagos et al., 2012).

2.1.3 Seismic Deformations

The drift should be obtained in order to understand the behavior of structural systems and determine the damage level that can occur in a certain element (Katsanos and Vamvatsikos, 2017). International standards specify its estimation method and the maximum allowable limits that cannot be exceeded. This, with the goal of avoiding damage on secondary elements, and obtain safe structures and adequate seismic behavior (Kose and Karslioglu, 2011).

Elnashai and Di Sarno (2008) mention that the interstory drift from a building is defined by the relative lateral displacements between two consecutive floors, divided by the interstory height, as shown in Fig. 2, and expressed in Eq. (1).

$$\text{Drift} = \frac{\delta}{h} = \frac{\delta_i - \delta_{i-1}}{h} \tag{1}$$

Where,

h = Interstory height

δ_i = Displacement of (n_{th}) floor

δ_{i-1} = Displacement of (n_{th}−1) floor

Current Chilean standards demand limitations on seismic deformations to obtain safe structures and adequate seismic behavior. Thus, Chilean code (Chilean National Institute of Standards, 2009) establishes that drift obtained between two consecutive floors, measured at the center of mass in every direction of analysis, shall not exceed 2%. Conversely, it establishes that the maximum drift between two consecutive floors, measured at any point from the floor in every direction of analysis, shall not exceed 1% as compared to the drift measured at the center of mass.

2.1.4 Seismic Response of Irregular Buildings

Real structures are almost always irregular as perfect regularity is an idealization that very rarely occurs (De Stefano and Pintucci, 2008). On the other hand, building irregularities pose

structures to more severe damages that increase the necessity of seismic evaluations (Shojaei and Benham, 2017). In this sense, several studies have been made to assess the seismic response of vertical irregular structures. In recent years, investigations on seismic behavior of irregular buildings in elevation have intensified, thanks in part to the greater availability of efficient nonlinear dynamic 3D computer code (De Stefano and Pintucci, 2008). Furthermore, nonlinear analysis methods have been used to test the consistency of seismic response for both plane and vertical irregularities in buildings. Modified pushover analysis is very promising as a simplified method for evaluating the seismic performance of irregular framed buildings (D'Ambrisi et al., 2009) and other structural and non-structural elements (Forcael et al., 2014). When a tower crane is added to a structural system during construction, openings are created or left throughout the different levels of the building, which result in building structures that should be treated as structures with plan irregularities, i.e., multi-story plan-asymmetric structures. The most common effects of this type of irregularities lead to non-negligible torsional effects; moreover, the evaluation of the exceedance probability of the assumed limit states has evidenced a significant sensitivity of the structure to the introduced irregularities (D'Ambrisi et al., 2013). Studies have been done on the use of innovative passive control technologies, aimed at mitigating the effects of the torsional response.

Despite the existence of nonlinear analysis methods to evaluate the seismic performance of irregular structures, the main objective of this research is to use a simplified but successful and effective structural analysis method (spectral modal analysis) (Jünemann et al., 2015), to study whether the perforations caused by the installation of the cranes have a negative influence on the lateral and torsional rigidity of the buildings.

2.2 Tower Crane

2.2.1 Temporary Structures in Construction

Temporary structures serve a specific purpose for a short time and are used for the execution of permanent works in a construction job site. These temporary structures are installed by a professional from the construction company, or by an outside expert that provides these services (Teizer, 2015).

As mentioned previously, a job site can have as many temporary structures as needed, in some cases; they are relevant activities to the project even though they are regarded as secondary tasks. Site offices and warehouses; scaffolding; temporary beams; shoring; perimeter's fence, and tower cranes are considered as temporary structures (Loyola and Goldsack, 2010; Teizer, 2015).

2.2.2 Tower Crane for High-Rise Structures

The tower crane is widely used for building large-scale civil engineering projects or high-rise buildings. It is also used to lift heavy elements, and to perform any work not capable of being done by other types of equipment (Shin, 2015). Its location depends on the job site, with urban areas being the most complex since any accident there can cause severe consequences for pedestrians, workers and crane operators. Visibility issues, wind loads and operator's safety are becoming more serious challenges when using these cranes during construction of high-rise buildings. Furthermore, the efficiency of vertical transport activities drops exponentially as the buildings' height is increased, modifying the costs and the project's schedule (Wei et al., 2015). Therefore, tower cranes are one of the most versatile construction equipment for a project, but also one of the most critical ones (Trevino and Abdel-Raheem, 2017).

When a construction project is being planned, it is fundamental to know: the exact number of cranes to use, the installation timing and their location. For these reasons, a careful study is crucial to avoid unnecessary costs and project delays (Huang et al., 2011; Marzouk and Abubakr, 2016). Likewise, Wei et al. (2015) mention that this issue is greater in urban areas limited by space since the number of cranes that could be installed on site is limited.

2.2.3 Unforeseen Issues at Construction Sites

Problems and unforeseen issues may appear during the execution of construction works. These issues must be analyzed thoroughly to establish corrective actions that enhance productivity and reduce time and project costs (Giménez and Suárez, 2008; Enshassi et al., 2013).

Some cases might require more exhaustive planning since it is common that projects are not executed as designed. Furthermore, it is fundamental to investigate the reasons that caused the interruption of construction activities and their causes (Giménez and Suárez, 2008).

Among the large number of problems encountered at a construction job site, there are lack of materials, tools, and equipment; adverse climate; natural disasters; loss of materials; temporary shutdowns; human losses; work absenteeism; among others (Toor and Ogunlana, 2008; Larsen et al., 2016). Furthermore, space limitations should be considered. This last issue becomes a fundamental one since it restricts storage and movement of materials, as well as the installation of temporary ancillary structures and the amount of heavy equipment (Loyola and Goldsack, 2010).

2.2.4 Tower Crane Problems Due to Space Limitations

The tower crane's location shall be defined prior to construction so that site-related problems can be solved (Marzouk and Abubakr, 2016). These include site limitations, building shape and size, amount and type of materials required, and site obstacles. In other words, it is necessary to solve problems promptly when space is limited to install the crane (Huang et al., 2011; Abdelmegid et al., 2015). Conversely, Younes and Marzouk (2018) add that in some cases the solution involves installing the tower crane inside the structural system.

Muñoz (2006) and Kim et al. (2018) mention the great responsibility engineers have, for analyzing the location of tower cranes for any project, which requires technical knowledge and

(a) (b) (c)

Fig. 3. Location of Tower Crane inside the Building: (a) Base of a Tower Crane inside a Building, (b) Hole in Slab (bottom view), (c) Hole in Slab (top view)

on-site experience before making the best decision. Among the first things to observe, is to identify if there is enough space around the site. Muñoz (2006) evaluated four buildings that had this problem. An external solution was sought to install the crane on three of them. First, an area close to the job site was rented. Then, a site across the street from the site was used to install the crane; and lastly, a section from the curb duly authorized by the city's Municipality was used. The last building studied was not limited by space, since various options were available to install the tower crane. However, the installation was moved forward to meet the deadlines established in the master schedule, deciding to embed the crane within the building's surface (see Fig. 3(a)). Figs. 3(b) and 3(c) correspond to projects performed by the company "Sudamericana de Equipos" (SAE).

Furthermore, considering working with a tower crane inside the building would help to solve space and scope problems, and becomes a good option, if the amount of heavy equipment required is reduced (Muñoz, 2006). If the crane is located inside the building, the structure must have temporary openings in each slab for proper handling of the crane. Another option would be to erect the crane through the elevator or escalator box (Gray and Little, 1985). However, this last alternative is not recommended since access to upper levels is restricted. Likewise, the elevator box might suffer damage when the crane is being uninstalled, due to the difficulty of controlling the works inside the opening. In addition, it is recommended to install the tower crane as soon as possible after finishing the construction of the building structure. This practice will enable the elevator's operation for moving personnel and materials throughout the execution of the finishing works (Gray and Little, 1985).

2.3 Slabs in Reinforced Concrete Buildings

2.3.1 Generalities

Slabs are plane elements. Their thickness is considerably lower compared to its other two dimensions. They are supported in all or some edges, and mainly resist loads perpendicular to its plane. They are mainly subdued to torsion outside the plane (Park and Gamble, 2000; Zineddin and Krauthammer, 2007).

In a building, a slab corresponds to the element where human activities are developed. The strength transfer is sent from the slab, through the beams (or walls) supporting them, passing by the columns until they reach the foundations (San Bartolomé, 1998).

2.3.2 Slab Openings

Solid slabs are widely used in construction. In some cases, small openings must be performed in buildings to let mechanical and electrical systems pass through them. By contrast, larger openings are required for escalator and elevator boxes (Koh et al., 2009).

Koh et al. (2009) mention that the stringency of the analysis depends on the properties of the slab opening and the structural engineer's criteria. On the other hand, the effect of the structural calculation is not considered for small openings. Constructively, in this case, extra reinforcing rebar would suffice in every direction adjacent to the opening sides and must be equivalent to the amount of reinforcing steel removed. This reinforcement shall extend beyond the opening. Furthermore, rebars in the corners installed at 45° are needed to control slab cracking produced at the vicinity of the opening (Pico et al., 2010).

The strength of a slab may be affected due to various reasons. This includes design errors, construction problems and inclusion of an opening. Moreover, the opening will cause a reduction in the slab's stiffness (Floruț et al., 2014; Khajehdehi and Panahshahi, 2016).

Until now, a broad literature review has been made. However, what happens in buildings remains unanswered. To answer these questions, it is important to know if a building will react well to an earthquake if the plane's geometry and height configuration are regular and symmetrical, which means to have uniform mass distribution, and that stiffness is gradually distributed on the building's height.

Furthermore, asymmetry leads to a response torsion component, produced by the eccentricity generated when the center of mass (CM) does not match the center of rigidity (CR), thus negatively affecting building performance due to demand increase from both structural and non-structural elements (Fox et al., 2017). Furthermore, it is important to consider the effect caused by having slab openings on each floor, over the CM and CR. Accordingly, the CM corresponds to the geometric place where the concentrated mass in each floor is located. Conversely, the CR corresponds to such point where the slab is moved without

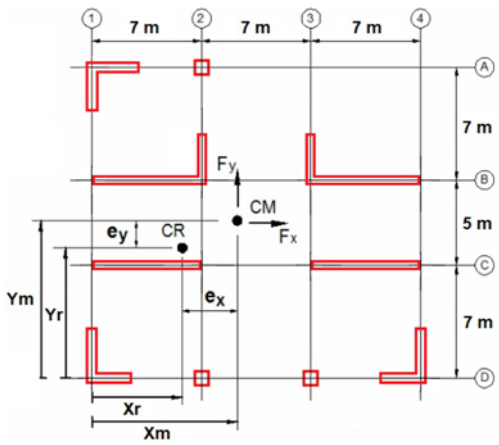

Fig. 4. Center of Mass, Center of Rigidity and Eccentricity (adapted from Aguiar, 2008)

rotation, with respect to the previous floor, after applying a horizontal shear force over it. Furthermore, the eccentricity corresponds to the distance between the concepts previously defined, when these do not match (Aguiar, 2008; Almazán et al., 2012).

Figure 4 shows the previous parameters when the CM does not match the CR. The horizontal distance between these two parameters generates the X-eccentricity (e_x), and the vertical distance generates the Y-eccentricity (e_y).

3. Methodology

3.1 Models Used

First, it is important to mention that in countries with high-seismicity like Chile, most reinforced concrete buildings have a large amount of total shear wall to floor area (total shear wall density) that ranged between 5% and 6%, which is relatively large compared with buildings of similar height in seismic regions elsewhere (Jünemann et al., 2015). These structural walls provide the required strength and rigidity, and behave well during earthquakes (Deger and Wallace, 2015; de la Llera et al., 2017).

This distinctive characteristic of Chilean buildings makes construction processes much slower than in other construction systems, so the time of use of the cranes is at least six months.

Fig. 5. Building Types: (a) Type A Building, (b) Type B Building

The above, together with the high seismicity present in countries like Chile, make the seismic design regulations specify to use the elastic design spectrum reduced by the R factor as the only seismic action, during the construction and future operation stages of the buildings. Besides, the regulatory limits for controlling inter-story drifts are calibrated for this particular construction typology, subject to the level of seismic demand mentioned above.

Saragoni (2011) indicates that between 1985 and 2009 there were 11.913 buildings erected in Chile, duly authorized by local construction permits. From those, 9,974 have between 2 and 9 floors, representing more than 80% of the total amount until that year. Furthermore, Rojas et al. (2011) and Lagos et al. (2012) mention that the typical reinforced concrete building consists of slabs between 14 and 19 cm thick, with walls between 15 to 25 cm thick and average interstory height of 2.5 m.

Two models were generated through ETABS software, to analyze structural behavior produced by adding temporary openings in slabs. ETABS is an extended tri-dimensional analysis and building design program that enables the use of the finite element method to obtain more reliable results. One of the special features of this software is the ability to incorporate, when necessary, the effect of the construction sequence on the structural response (staged construction analysis). This type of structural analysis is especially useful for studying the redistribution of stresses and deformations in reinforced concrete buildings that suffer axial shortening, such as special reinforced concrete moment frames (Afshari et al., 2018). However, the buildings studied in this work are both very regular in height and axially rigid, due to the substantial restrictions of seismic design in Chile. Besides, the areas of the slab openings produced by the cranes are relatively small to the total floor area. For these

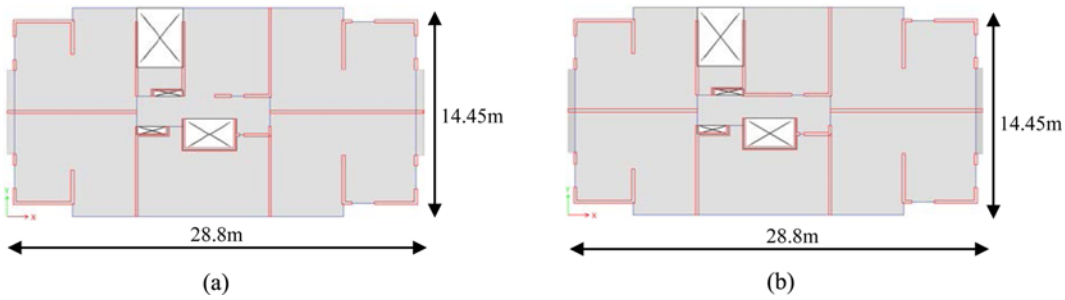

Fig. 6. Ground Floor and Standard Floor – Building Type A: (a) Ground Floor - Type A Building, (b) Standard Floor - Type A Building

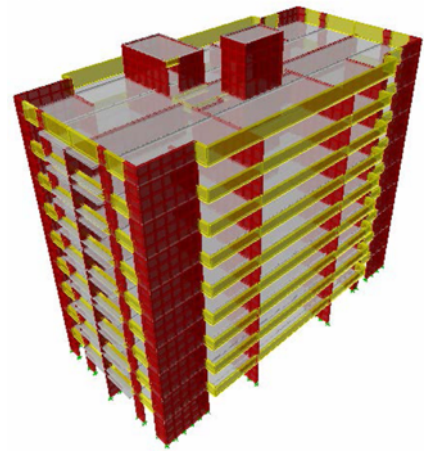

Fig. 7. 3D Model - Type A Building

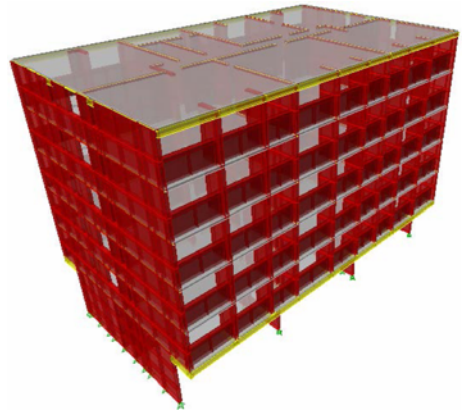

Fig. 9. 3D Model – Type B Building

reasons, a traditional method of structural analysis was used, where the dead and live loads of all floors are applied instantaneously, and it is assumed that the structure does not bear any load before the end of construction.

The models used in the study are based on information previously mentioned and correspond to two building types (A and B), as seen in Fig. 5.

3.1.1 Type A Building

Figure 6 (ground floor-plan and floor type-plan respectively), along with the 3D model shown in Fig. 7, show that type A consists of 9 floors with identical geometries. They reach the escalator and elevator box at the upper part of the building. Each level has a height of 2.45 m (floors 1 to 9), resulting in a total height of 25.93 measured from the ground to the elevator box's roof. At a footprint view, it has a length of 28.8 m, a width of 14.45 m and slabs 15 cm-thick.

The building's structure consists of walls, beams, and reinforced concrete slabs, conforming a structural configuration based on walls.

3.1.2 Type B Building

Figure 8 (ground floor-plan and floor type-plan respectively), along with the 3D model shown in Fig. 9, show that type B consists of 6 floors. The first floor has a height of 3.7 m and a height of 2.55 m from the second to the sixth floor, resulting in a total of 16.45 m. At a footprint view, it has a length of 22.5 m, a width of 13.75 m and slabs 15 cm-thick. The building's structure has walls, beams, and reinforced concrete slabs.

3.2 Design Spectrum

The spectrum's pseudo accelerations were defined to estimate the seismic loads supported by the buildings, which depend on the soil type and seismic zoning.

These considerations were based on data obtained from the specifications of Chilean standards (Chilean National Institute of Standards, 2009), which were improved after the February 2010 earthquake. It is important to mention that these criteria are based on international standards (Saragoni, 2011; Wallace, 2012). Eq. (2) shows how the spectrum S_a was estimated.

$$S_a = \frac{S \times A_0 \times \alpha}{\frac{R^*}{I}} \tag{2}$$

Where,

A_0 = Effective acceleration

I = Building's importance coefficient

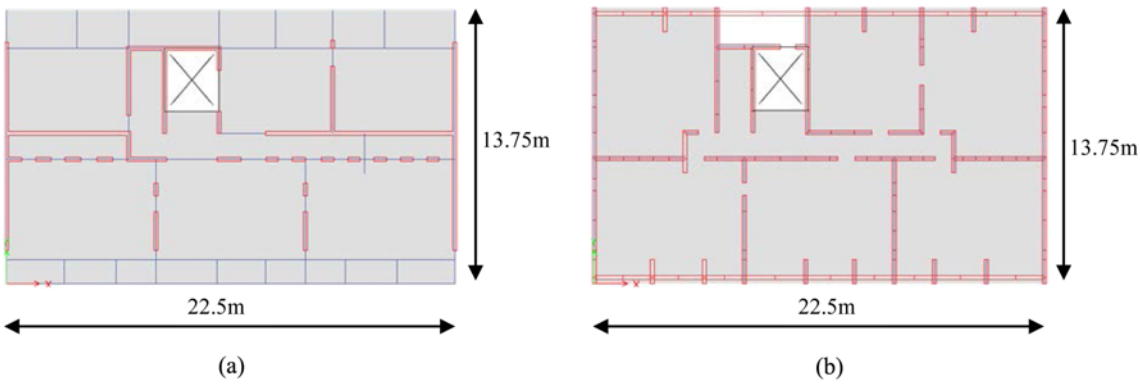

Fig. 8. Ground Floor and Standard Floor - Type B Building: (a) Ground Floor - Type B Building, (b) Standard Floor - Type B Building

R^* = Spectral acceleration reduction factor (response modification factor)
S = Parameter that depends on the soil type
α = Amplification factor of the maximum effective acceleration

Data used to estimate spectrum S_a (See Eq. (2)) and amplification factor α (see Eq. (3)), was obtained according to criteria established by the Chilean standards (Chilean National Institute of Standards, 2009).

$$\alpha = \frac{1 + 4.5 \times \left(\frac{T_n}{T_0}\right)^p}{1 + \left(\frac{T_n}{T_0}\right)^3} \tag{3}$$

Where,
T_n = Vibration period for n mode
T_0 and p = Parameters relative to the foundation soil type
α = Amplification factor established for each n vibration mode

3.3 Opening Size

The opening sizes for building types A and B were defined once the structural models were ready, with the corresponding load combinations and seismic demand through a design spectrum. First, it was necessary to do some research about the type of tower cranes used in Chile, and the various companies worldwide that would provide such services. The research was focused on Liebherr cranes, a company recognized as one of the biggest suppliers of construction equipment in the world. It was found that one of the most frequently used crane models in Chile was Liebherr 85 EC-B 5 (as shown in Fig. 10). SAE Company confirmed that this model would be ideal for our specific case study since it may be installed in places limited by space and has been currently used for the similar types of projects in Argentina and Brazil.

Figure 10 shows the square section (1.2 × 1.2 m) of the crane. A free "clearance" zone of 50 cm around the perimeter was considered based on a recommendation from an expert from the field, resulting in a slab opening of 2.2 m by 2.2 m. These dimensions correspond to the minimum dimensions required since the opening size should permit oscillation and possible displacements due to seismic demands that the tower crane might face. Recommendations of a free "clearance" in every direction are also proposed by Gray and Little (1985).

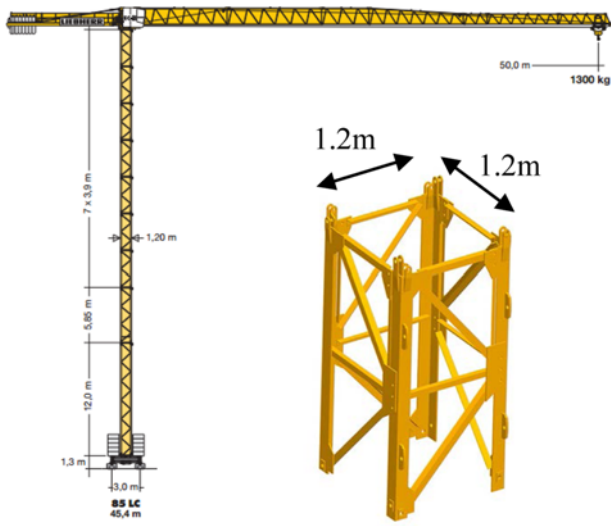

Fig. 10. Tower Crane LIEBHERR 85 EC-B 5

3.4 Location of Tower Crane (Opening)

The location of the crane was found once the tower dimensions and the opening size were clearly established. For that purpose, the following aspects were considered: a) the maximum radius of the crane; b) the intervention of structural elements; c) the relationship between CM and CR.

Five openings were established in both models (see Fig. 11). First, the maximum radius of the crane was found based on the technical sheet from crane model Liebherr 85 EC-B, which said that it could reach up to 50 m. This model has a rotation mechanism for more accuracy and convenience of the rotation movements; therefore, the rotation radius was 50 m.

This shows that locating a good place for the crane was not an issue since it could be installed at any place from both buildings. Only slabs could be affected by the crane's location since

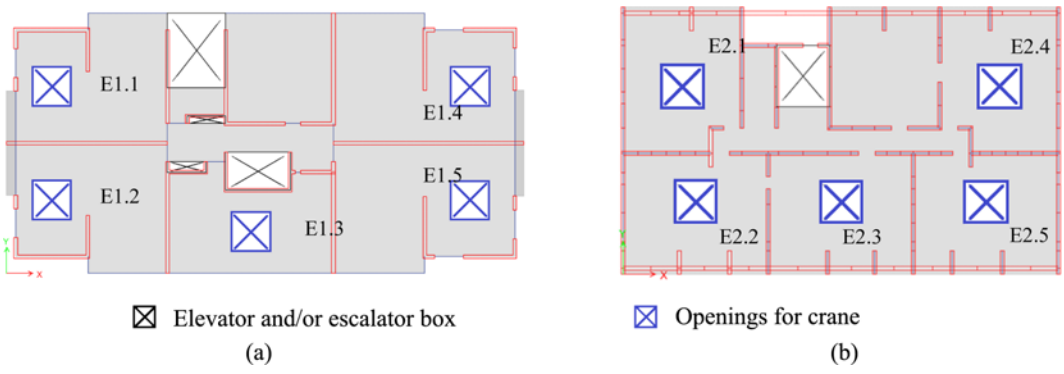

Fig. 11. Openings' Location of Tower Crane: (a) Openings' Location of Tower Crane for Type A Building, (b) Openings' Location of Tower Crane for Type B Building

affecting walls and beams could damage the structural integrity of the buildings. Finally, the relationship between the CM and CR were key factors in establishing the openings' location. ETABS was used to obtain these parameters for each floor diaphragm.

The distance between the CM and CR (eccentricity in X and Y) were evaluated based on the information obtained from ETABS. The location that had the highest values was selected, with the goal of assessing the most critical scenario in terms of torsion-produced effects.

4. Analysis and Considerations

4.1 Static Loads

Gravitational loads acting over structural elements were determined in accordance with Chilean standards for permanent loads and live loads (NCh 1537 of 2009). Permanent loads correspond to dead loads from structural elements (obtained automatically through ETABS), and dead loads (*DL*) from partition walls, floor finishes, among others. A total load of 100 kgf/m^2 was considered for the latter. A common load of 200 kgf/m^2 was considered a live load (*LL*). Tables 1 and 2 detail the loads applied to the slabs in both models, where *LL*: live load, *DL*: dead load, *Lr*: roof load.

4.2 Seismic Mass

Seismic mass is considered as 100% of dead load (permanent load) + 25 – 50% of the live load. The standard requires 25% for private housing buildings or public buildings where an agglomeration of people is uncommon and 50% where agglomeration is common. The buildings under study fall into the first category; therefore, the seismic mass was estimated by Eq. (4).

$$\text{Seismic mass} = DL + 0.25* \tag{4}$$

Table 1. Acting Loads over Slabs of Type A Building

Floor	LL [kgf/m^2]	DL [kgf/m^2]	Lr [kgf/m^2]
SLAB escalator and elevator box	-	100	100
SLAB 9	-	300	100
SLAB 8	200	100	-
SLAB 7	200	100	-
SLAB 6	200	100	-
SLAB 5	200	100	-
SLAB 4	200	100	-
SLAB 3	200	100	-
SLAB 2	200	100	-
SLAB 1	200	100	-

Table 2. Acting Loads over Slabs of Type B Building

Planta	*LL* [kgf/m^2]	*DL* [kgf/m^2]
SLAB 6	200	100
SLAB 5	200	100
SLAB 4	200	100
SLAB 3	200	100
SLAB 2	200	100
SLAB 1	200	100

4.3 Spectrum Modal Analysis

4.3.1 Accidental Torsion

Chilean standards (Chilean National Institute of Standards, 2009) recommend two ways for estimating accidental torsion effect. In this case, static torsion moment on each slab was considered as the product of the slab's shear strength caused multiplied by the accidental eccentricity on each level. Eccentricity is established by Eq. (5) (x-direction) and Eq. (6) (y-direction).

$$e_{kx} = \pm\frac{0.1 \times b_{ky} \times Z_k}{H} \tag{5}$$

$$e_{ky} = \pm\frac{0.1 \times b_{kx} \times Z_k}{H} \tag{6}$$

Where,

b_{kx} = Longer dimensión of level k in x-direction
b_{ky} = Longer dimensión of level k in y-direction
H = Total height of building
Z_k = Height from basal level to floor from level k

4.3.2 Response Modification Coefficient, "R", (According to ASCE)

Chilean standards show that the design spectrum is obtained by dividing the Y-axis coordinates of the elastic spectrum, from the response modification coefficient R^*, associated to the period with the greatest equivalent translational mass in the analysis direction (x or y). The coefficient R* was estimated by Eq. (7).

$$R^* = 1 + \frac{T^*}{0.1 \times T_0 + \frac{T^*}{R_0}} \tag{7}$$

Where,

R_0 = Structural response modification factor (Spectral modal analysis)
T^* = Mode period with the greatest equivalent translational mass in the analysis direction. Values were determined by analyzing both models. R_0 was set as 11, due to the similarities in both building's structural configuration (wall-based reinforced concrete type)

4.3.3 Input Data

Concepción was studied for two main reasons: it was the place where buildings under study were built; and its location. Located in the central-southern part of Chile, the region of Bío-Bío was one of the most affected ones after the earthquake of February 2010. This earthquake had a magnitude M_w of 8.8 (Westenenk et al., 2012; Montalva et al., 2016). The region of Bío-Bío suffers earthquakes

Table 3. Summary of Data for Analysis

City	Concepción
Seismic zone	3
Soil type	D
Category	II
I	1.0
A_0	0.4 g
S	1.2
T_0	0.75 sec
p	1.0
R_0	11

Table 4. Response Modification Coefficient of Spectral Acceleration for Type A Building

	Without opening		With opening	
	Earthquake X	Earthquake Y	Earthquake X	Earthquake Y
T*	0.224 [s]	0.334 [s]	0.223 [s]	0.333 [s]
R*	3.348	4.170	3.341	4.162
1/R*	0.299	0.240	0.299	0.240

Table 5. Response Modification Coefficient of Spectral Acceleration for Type B Building

	Without opening		With opening	
	Earthquake X	Earthquake Y	Earthquake X	Earthquake Y
T*	0.176 [s]	0.183 [s]	0.175 [s]	0.183 [s]
R*	2.931	3.001	2.923	2.996
1/R*	0.341	0.333	0.342	0.334

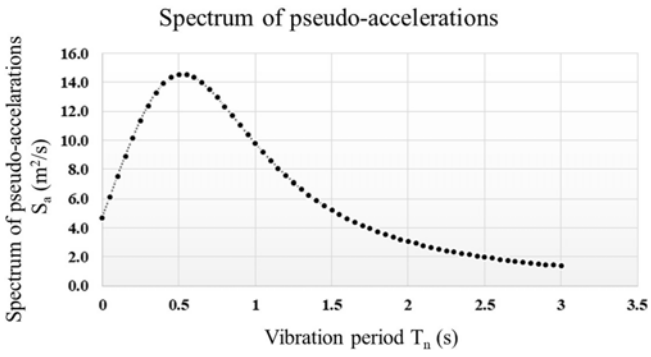

Fig. 12. General Spectrum of Pseudo-Accelerations for Type A and Type B Buildings (S_a vs T_n)

regularly and has earthquakes with $M_w > 8.8$ every 50 to 100 years. In fact, Concepción was highly devastated by the earthquake of Valdivia in 1960, the strongest earthquake ever registered, with a magnitude M_w of 9.5 (Montalva et al., 2016).

Table 3 summarizes data used to obtain the elastic spectrum of pseudo-accelerations, based on the Chilean standards (Chilean National Institute of Standards, 2009). Likewise, Tables 4 and 5 show R^* values used to obtain the seismic demand (design spectrum). Besides that, Fig. 12 shows the elastic spectrum of

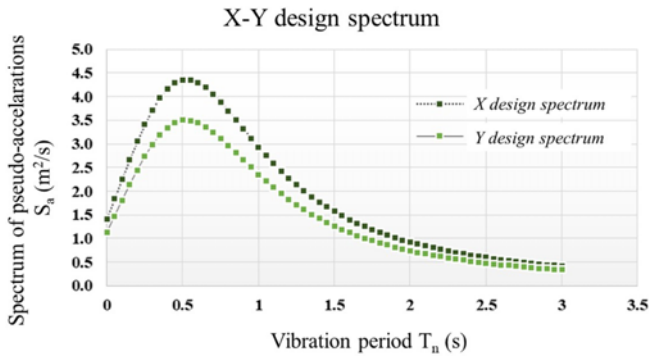

Fig. 13. Design Spectrum Graph of Type A building without Opening, X and Y Direction

Table 6. Summary of Values [m] CM and CR for Openings in Type A Building

	Center of Mass		Center of Rigidity		Eccentricity	
	CMx	CMy	CRx	CRy	e_x	e_y
E1.1	14.103	7.220	14.302	7.023	0.199	0.197
E1.2	14.104	7.212	14.302	7.023	0.198	0.189
E1.3	14.116	7.212	14.299	7.023	0.183	0.189
E1.4	14.107	7.218	14.296	7.023	0.189	0.195
E1.5	14.113	7.214	14.297	7.023	0.184	0.191

Table 7. Summary of Values [m] CM and CR for Openings in Type B Building

	Center of Mass		Center of Rigidity		Eccentricity	
	CM_x	CM_y	CR_x	CR_y	e_x	e_y
E2.1	11.324	6.782	10.703	8.179	0.621	1.397
E2.2	11.318	6.831	10.703	8.179	0.615	1.348
E2.3	11.257	6.831	10.703	8.179	0.554	1.348
E2.4	11.195	6.831	10.703	8.179	0.492	1.348
E2.5	11.189	6.782	10.703	8.179	0.486	1.397

pseudo-accelerations, which is common for both building types. Finally, Fig. 13 shows the design spectrum for Type A building, without a slab opening (Likewise, the design spectrum is obtained for building types with and without an opening in the slab).

4.4 Seismic Deformations

Seismic deformation was analyzed based on the drift. For this case, the drift evaluated was obtained between the center of mass of each floor (Drift CM), and the difference between the center of mass's drift and the maximum drift measured at any point inside the floor (ΔDrift CM-Point), both for each direction of analysis.

Two acting-seismic directions were established, one for the longitudinal direction of the building (earthquake in X), and the other one for the transverse direction (earthquake in Y).

Chilean standards (Chilean National Institute of Standards,

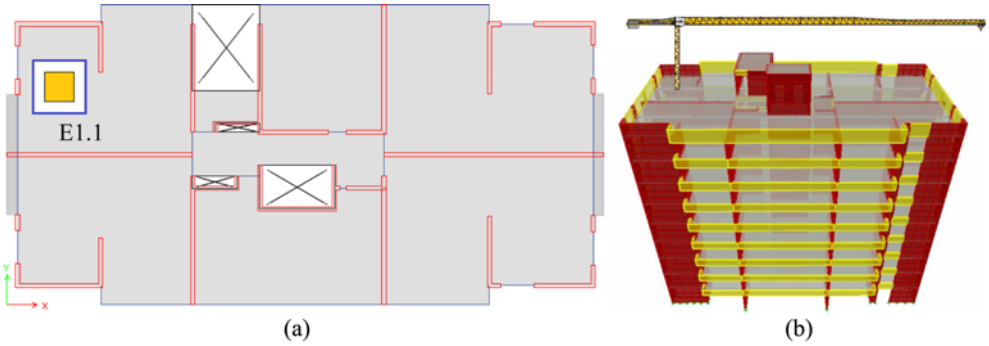

Fig. 14. Location of Tower Crane for Type A Building (with opening): (a) Plan View, (b) 3D View

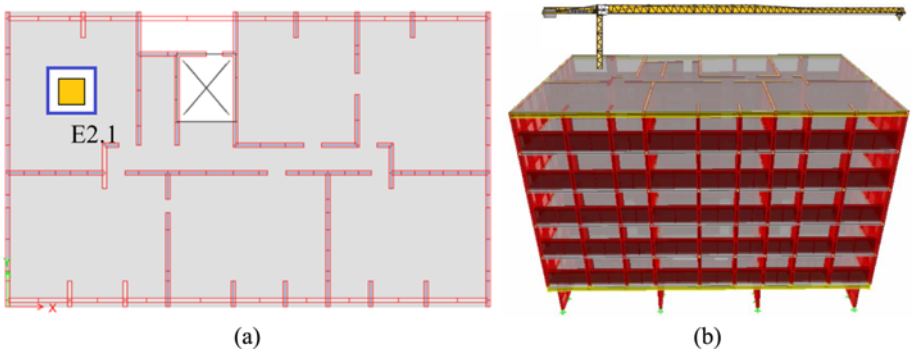

Fig. 15. Location of Tower Crane for Type B Building (with opening): (a) Plan View, (b) 3D View

Table 8. Number of Results per Analysis

	Type A building		Type B building		
	With opening	Without opening	Without opening	With opening	Total
Drift CM X	4	4	4	4	16
Drift CM Y	4	4	4	4	16
Drift Pnt. X	4	4	4	4	16
Drift Pnt. Y	4	4	4	4	16
Total	16	16	16	16	64

2009) establish that horizontal displacements and floor diaphragm rotations must be estimated for seismic actions arranged through one of the analysis methods. For this case, Spectrum Modal Analysis and accidental torsion effects were used.

The location selected was evaluated based on three criteria: the maximum radius of the crane, the intervention of structural elements and relationship between CM and CR.

Tables 6 and 7 summarize the values obtained for each floor type and for each building floor. The openings were placed in areas with the highest X and Y eccentricities (e_x and e_y). This helped to evaluate the most unfavorable case due to the torsion effect caused by the CM and CR not matching. Finally, the highest eccentricity values were marked, resulting in location E1.1 for Type A building (see Table 6 and Fig. 14), and location E2.1 for Type B building (see Table 7 and Fig. 15).

Finally, seismic deformation was obtained based on the drift, resulting in 64 calculation tables. Table 8 shows the breakdown for such calculations.

Number 4 represents the number of load combinations for each analysis, obtaining values for DEFX1, DEFX2, DEFX3 and DEFX4 for x-direction. Likewise, values for DEFY1, DEFY2, DEFY3 and DEFY4 were obtained for y-direction. Combination details are shown next:

- DEFX1 = DL + Sx + TMx
- DEFX2 = DL + Sx – TMx
- DEFX3 = DL – Sx + TMx
- DEFX4 = DL – Sx – TMx
- DEFY1 = DL + Sy + TMy
- DEFY2 = DL + Sy – TMy
- DEFY3 = DL – Sy + TMy
- DEFY4 = DL – Sy – TMy

The combinations analyzed consisted of dead loads (DL), earthquake (S_x, S_y) and torsional moment on each direction of analysis (TMx, TM_y). The latter was a requirement for estimating seismic deformations based on local standards.

5. Results

5.1 Statistical Analysis

Statistical analysis was performed using InfoStat software, allowing the research team to decide if including the tower crane inside the building brought seismic consequences.

Information was organized for both buildings, based on tables that had all drift results (see Tables 9 and 10). Numbers “1” and “2”, inside the “Direction” column, represent the earthquake acting in “x” and “y” direction respectively. “1, 2, 3, 4 and 5” from the “Combination” column, represent each of the load combinations used to obtain the seismic deformations. Number “1” from the

Table 9. Drift Results for Type A Building

No Exp.	Direction	Combination	Opening	Floor	*Drift CM* [‰]	*Δ Drift Pnt-CM* [‰]
1	1	1	1	1	0.1633	0.0807
2	1	1	1	2	0.2041	0.1969
3	1	1	1	3	0.2449	0.2421
.	.	.	.	.	.	.
.	.	.	.	.	.	.
.	.	.	.	.	.	.
142	2	4	2	7	0.6531	0.7379
143	2	4	2	8	0.6531	0.6349
144	2	4	2	9	0.6939	0.4951

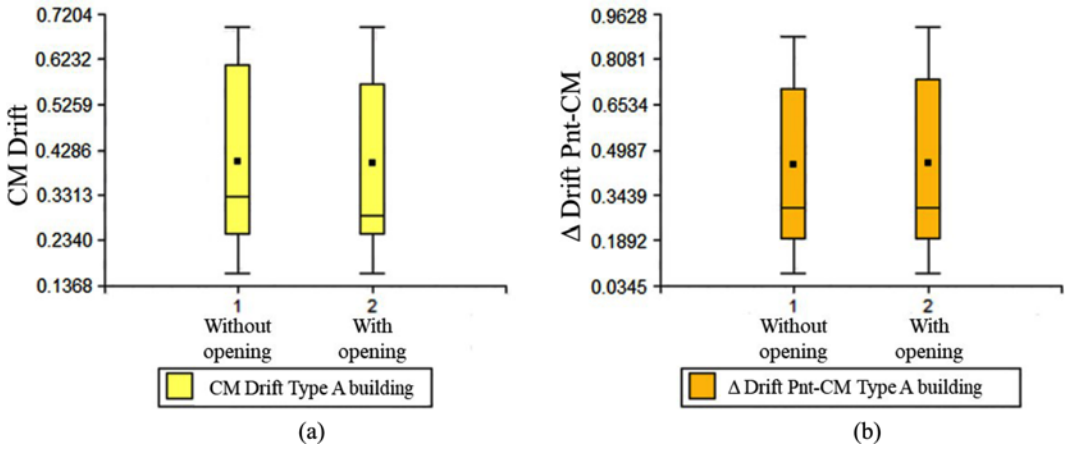

Fig. 16. Box Plot Chart - Drift CM and Δ Drift Pnt-CM for Type A Building: (a) CM Drift Type A Building, (b) D Drift Pnt-CM Type A Building

Table 10. Drift Results for Type B Building

N Exp.	Direction	Combination	Opening	Floor	Drift CM [‰]	Δ Drift Pnt-CM [‰]
1	1	1	1	1	0.1081	0.0449
2	1	1	1	2	0.1961	0.0959
3	1	1	1	3	0.2745	0.0665
.	.	.	.	.	.	.
.	.	.	.	.	.	.
.	.	.	.	.	.	.
94	2	4	2	4	0.2353	0.1367
95	2	4	2	5	0.1961	0.1599
96	2	4	2	6	0.2353	0.1067

"Opening" column, represents the building model without the opening, and "2" represents the one with the opening. The "Floor" column shows value ranges based on the number of floors. Drift values (CM and ΔPnt-CM) are the result of the combinations of each of the variables explained previously. Results are shown in Figs. 16 and 17.

5.1.1 Comparison of the Two Samples

Normality was tested at first to compare the two samples, and then, based on results obtained, the T-student or Wilcoxon test was applied. In this case, the null hypothesis (H_0) was that the sample was normally distributed. Conversely, the alternative hypothesis (H_1) stated that the sample was not normally distributed. If *p-values* were greater than 5%, then there was not enough evidence to reject the null hypothesis and the T-student test would be applied. On the other hand, if *p-values* were lower than the significance value, then the null hypothesis was rejected and the Wilcoxon test would be used.

Normality was tested from drift results for both opening cases for Types A and B buildings. This was done through the normality test (Shapiro-Wills modified) for a significance level of 5%.

Table 11 shows the normality test results for Type A building.

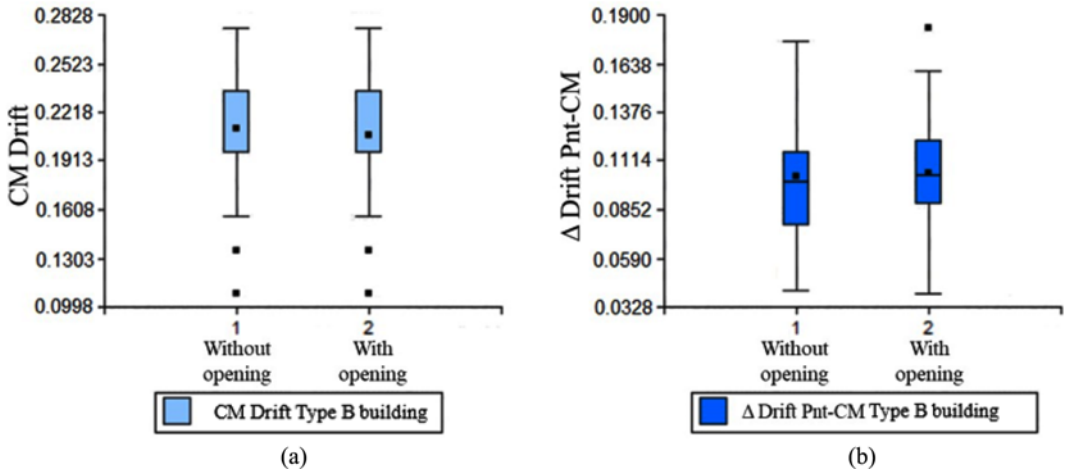

Fig. 17. Box Plot Chart - Drift CM and Δ Drift Pnt-CM for Type B Building: (a) CM Drift Type B Building, (b) D Drift Pnt-CM Type B Building

Table 11. Normality test for CM Drift and Δ Drift Pnt-CM BldgA (Shapiro-Wilks modified)

Variable	n	Mean	S.D.	p-value
CM Drift Bldg A	144	0.40	0.19	< 0.0001
Δ Drift Pnt-CM BldgA	144	0.45	0.29	< 0.0001

Table 12. Wilcoxon's test (Mann-Whitney U) for CM Drift and Δ Drift Point-CM Type A building

Rating	Variable	Groups	*n* (1 and 2)	p-value
Opening	CM Drift Bldg A	1 and 2	72 and 72	0.8276
Opening	Δ Drift Pnt-CM Bldg A	1 and 2	72 and 72	0.9554

Table 13. Normality Test (Shapiro-Wilks modified) for CM Drift and Δ Drift Point-CM Type B Building

Variable	n	Mean	S.D.	p-value
CM Drift Bldg B	96	0.21	0.05	< 0.0001
Δ Drift Pnt-CM Bldg B	96	0.10	0.03	0.0128

Table 14. Wilcoxon's test (Mann-Whitney U) for CM Drift and Δ Drift Point-CM Type B Building

Rating	Variable	Groups	*n* (1 and 2)	p-value
Opening	Drift CM Bldg B	1 and 2	48 and 48	0.5815
Opening	Δ Drift Pnt-CM Bldg B	1 and 2	48 and 48	0.7031

As shown, *p-values* were significantly lower than 5% in both samples, which meant that there was enough evidence to reject the null hypothesis. Therefore, Wilcoxon's test was used (Mann-Whitney U).

Wilcoxon's test was used once it was proven that drift samples in relation to their CM and ΔDrift CM-Point were not normally distributed. This test enabled the team to compare data, considering the null hypothesis (H_0) that the drift without the opening is equal to the drift with the opening in both cases (CM and ΔPoint-CM), with a significance level of 5%. Table 12 shows the results obtained, where *p-values* were significantly larger than 5%. There was not enough evidence to reject the null hypothesis and it was assumed that data from samples were equal as shown next:

Ho: CM Drift without opening - Type A building = CM Drift with opening - Type A building

Ho: Δ Drift Pnt-CM without opening - Type A building = Δ Drift Pnt-CM with opening - Type A building

As shown in Table 12, the drift depends on the "Rating" (opening). Conversely, "Groups" refers to data: 1) without the opening and 2) with the opening. Finally, "*n*" shows the number of results used in the analysis, which were 1) 72 without the opening and 2) 72 with the opening for Type A building. This sequence also explains the analysis performed on Building B.

Results for the normality test obtained from Building B are shown in Table 13. Here, *p-values* for the samples with and without the openings were considerably lower than 5%, which meant that there was enough evidence to reject the null hypothesis. Therefore, Wilcoxon's test was used (Mann-Whitney U).

Wilcoxon's test was used to compare data, after checking that drift samples in relation to the CM and ΔDrift CM-Point were not normally distributed. The null hypothesis in this case was that drift without the opening was equal to the drift with the opening for both cases (CM and ΔPoint-CM), with a significance of 5%. Results obtained are shown in Table 14. As shown *p-values* on both cases were significantly larger than 5%, therefore there was not enough evidence to reject the null hypothesis and it was assumed that data from samples were equal, as shown next:

Ho: Drift CM without opening - Type B building = CM Drift with opening - Type B building

Ho: Δ Drift Pnt-CM without opening - Type B building = Δ Drift Pnt-CM with opening -Type B building

5.2 Discussion

The simulation was run once the models from ETABS were established, and an adequate crane type was chosen for both

building types. Through this, it was possible to obtain drift parameters based on the CM, and the difference between the CM Drift and the maximum drift measured in a point from each floor (ΔDrift CM-Pnt), defined earlier in the study. These results were statistically analyzed, enabling the comparison of two samples based on a rating variable, which was, for this study, the inclusion of an opening through the slab. This process showed that, statistically, there was not a significant variation after the openings were applied into the building, since *p-values* were considerably larger than the 5% significance level required on Wilcoxon's test (Mann-Whitney U). Thus, there was no evidence to reject the null hypothesis, which leads to assume that the samples compared were equal.

In summary, the statistical analysis showed that there was no significant variation after the tower crane was installed inside the building. A reason for this could be that the opening did not have much influence over torsion effects caused since the variation between both cases was almost null. Furthermore, the area removed from the slab was small compared to the total area of the slab. The area of the opening was 4.84 m^2 (2.2 m × 2.2 m) and the total floor area of Types A and B buildings were 368 m^2 and 292 m^2 respectively. The former did not reach 2% of the total area on any of the cases. Moreover, the structural behavior of both building types did not suffer any variation. Finally, the seismic demand through the design spectrum remained almost the same. This also occurred with the rigidity, since the study did not modify relevant structural elements. The opening was located in a place that modified slabs only.

Furthermore, the analysis considered an additional space of 1m around the perimeter of the opening to simulate similar conditions to the ones found in an earthquake where great displacements of structural elements are expected. Results obtained with the new condition (3.2 m × 3.2 m) were similar to the ones obtained with the original opening size. There were no significant statistical variations in seismic deformations (Drift CM and ΔDrift CM-Point) after the opening sizes were increased in both buildings.

6. Conclusions

The study described in this work enabled the analysis of the effect caused by temporary openings in building slabs for installing tower cranes, due to space limitations.

The type of tower crane selected complied with the study's requirements. This resulted in choosing a city crane type, which is ideal in reduced spaces. This way, it was possible to establish the opening size considering the dimensions of the tower crane's cross-section and the clearance required to prevent impact with the slab borders or with other structural elements. Finally, it was possible to select an optimum location for the tower crane to study the temporary harmful effect over the building's seismic behavior where the decisive parameter was the relationship between CM and CR. Therefore, the most adverse location in terms of torsion effect was selected, since the maximum radius of the crane was long enough to consider options that only affected the slab.

Seismic deformations were determined, in terms of drift, for both cases, considering seismic demands through the design spectrum in accordance with local standards. This way, the seismic analysis requirements for buildings were fulfilled, as deformations were estimated for design seismic actions. Resulting in values for n = 144 of Building A of: CM Drift Mean = 0.40, Std. Dev. = 0.19, p-value < 0.0001 and Δ Drift Pnt-CM Mean = 0.45, Std. Dev. = 0.29, p-value < 0.0001. While values for n = 96 of Building B of: CM Drift Mean = 0.21, Std. Dev. = 0.05, p-value < 0.0001 and Δ Drift Pnt-CM Mean = 0.10, Std. Dev. = 0.03, p-value < 0.0128. Concluding that deformations were not normally distributed.

Drift results were analyzed in terms of the CM and the difference between CM drift and Point drift, for building simulation models with and without openings in their slabs, to assess the influence on seismic behavior buildings due to such openings. Statistical analysis helped to conclude that the openings did not affect seismic deformations, thus there was no structural difference in using cranes or not, since the variation was statistically nonsignificant. As for Building A with p-value for CM Drift Bldg A equal to 0.8276 and p-value for Building B equal to 0.9554.

After the various analyses were performed throughout the work, it was feasible in terms of serviceability and seismic performance, to recommend installing a tower crane inside a building's structural system. As mentioned earlier, the opening had almost no influence over the determination of seismic deformation, which means that contractors may continue performing similar construction practices. This can be done once an optimum location has been found for the tower crane. In addition, intervention should be performed in slabs only and not in any other structural elements. Furthermore, it is necessary to consider the clearance around the opening's perimeter, which must be at least 50cm to protect the building's structure.

Future research involving the installation of tower cranes inside a building may consider performing a static (local) analysis, considering a standard type of slab, for the various buildings selected, with and without an opening. This will enable the research team to verify how the building strength would be affected when openings are included in the study, due to the structural discontinuity generated where the tower crane would be placed. Furthermore, a deeper study could be conducted about the shear stress distribution in the floor plane, and how these could be affected when temporary openings are incorporated.

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