DIRECTIVITY EVIDENCE FOR THE CONCEPCION CITY DAMAGE IN THE EL MAULE, CHILE 2010 EARTHQUAKE.

R. Ramos (1), G.R. Saragoni (2)

(1) *Seismic Engineering, Master of Science Graduate Student. Dept. Civil Engineering University of Chile.* rubenramossalas@gmail.com

(2) *Principal Professor, Department of Civil Engineering, University of Chile,* rsaragon@ing.uchile.cl

Abstract

The most iconic picture up today of the El Maule, Chile 2010, MW = 8.8 earthquake is the collapse of the reinforced concrete building “Alto Rio” in Concepción City. Many studies has been performed to explain this collapse, however they has not been successful so far.

In order to better understand the reason of the collapse of this building, in this paper is analyzed the progressive failure of another reinforced concrete structure that also collapsed for the same earthquake at Concepcion City. The structure correspond to the Casino del Colegio Inmaculada Concepcion, located at the same site where was recorded the important accelerogram of Concepcion Center contributing with this new forgotten information for more than 7 years. The structure of frame and roof slab, “canopy type”, can be modeled as a simple one degree of freedom structure allowing to correlate easily the response and collapse mechanism with the characteristics of the demanding accelerogram.

The nonlinear model used allows to reproduce a damage similar to the observed in the site inspection and photography record.

The response of the model shows a clear directivity in the instant of the damage, which happens at similar time for the model and the real structure. The beginning of the damage occurs approximately at 11s and the collapse at 14s for the model. This well-defined short interval of time corresponds to first pulse released by the earthquake from the south asperity of the Nazca Plate and its corresponding Rayleigh wave.

Happening that the structure collapse before that soil response appear in the accelerogram. The time collapse match well with the one indicated by witness the night of the collapse.

In this study is also shown the existence in the record of excitation of pulse type, with evident directivity which clearly correlate with the damage.

The structural response of study is compared with the time response of the “Alto Rio building” of different studies finding the same effect of the accelerogram in the response of both structures.

Given the similarity of fundamental periods and the behavior in time of the roof drift and direction of collapse of the Canopy and "Alto Rio" building, it is inferred that Pulse 1 from seismic source asperity contains enough energy capable of damaging structures and depending of their redundancy can lead to progressive collapse.

Keywords: source directivity, reinforce concrete canopy, structure collapse, Concepcion, Chile 2010 earthquake, Alto Rio, nonlinear analysis.

1. Introduction

The collapse of the "Alto Río" building in Concepción became a worldwide icon of the El Maule, Chile earthquake of February 27, 2010. This building, analyzed by numerous authors, has shown the difficulty in explaining the shape and timing of its collapse [1, 2, 20, 21, 22, 23 and 24].

Fortunately for study purposes there is another reinforced concrete structure that also collapsed and that remained forgotten for years. Given its simplicity, assimilable to one degree of freedom, it facilitates a detailed study of its failure mode and instant of the collapse. With this analysis, it can be also determined which ground motion signals produce the greatest energy inputs to the structure to explain its damage and understand its progressive collapse. Additionally, it is questioned, understanding the mechanism of energy that causes the collapse if can be extended to other damage structures of Concepción.

2. The collapsed structure of the Immaculate Conception College casino

For the earthquake of February 27, 2010 there was a structure that collapsed and remained forgotten for about seven years. This structure is located on Aníbal Pinto 340 Street, next to the "Plaza de Armas" (Main Square) of Concepción and approximately 1.15 km east of the emblematic "Alto Río" building. This structure is very significant, since it is located at the same place of the Immaculate Conception College where the most important epicentral accelelerogram of the El Maule 2010 earthquake was recorded, Mw = 8.8 at just 10.0 meters and in the direction S30 ° W. west. The main axes of the building coincide exactly with the same directions of the accelerograms. Originally it was built with the purpose of being a covered patio for various activities, and by the date of the earthquake it was used as a casino (dining room) for the students of the College. See Fig. 1.

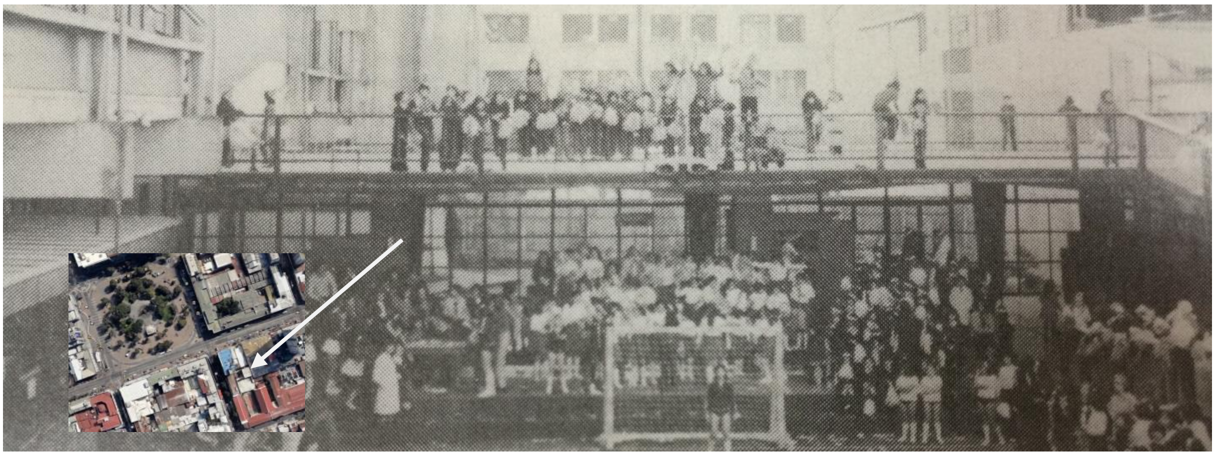


Fig. 1 - Covered patio (casino) and its context in the Immaculate Concepción College (Enlarged image [3], reduced image Google Maps showing main square).

3 Background Compilation

3.1 Geometric Background

The architectural plans of the archives of the College [4] allow to have an exact geometry of the structure. From the point of view of seismic engineering, it is very interesting, since it is a structure that is very close to the "ideal" of a one of degree of freedom. A structure based of frames of a span of 7.85 m in the cross section and two spans of 6.60 m longitudinal respectively as shown in Fig. 2. The reinforced concrete structure with six columns of 3.20 m of free height, geometry pyramidal trunk, of square section: 30 cm edge at the base and 65 cm at the ceiling. The roof is a slab also of pyramidal trunk geometry, began with a thickness of 45.0 cm in the head of the columns which is reduced to 15 cm as its free edge.

There is no information regarding the system of foundations in the documents consulted. However, since the purpose of the present study seeks to reproduce the seismic response of the structure, the relevant thing is to know the support condition that existed at the base of the columns. It must be noticed the great relative difference in moments of inertia between the base and the ceiling, which allow that for modeling purposes it can be considered as pin at the base. The geometry of the structure, transferred from the original drawing, is plotted in Fig. 2.

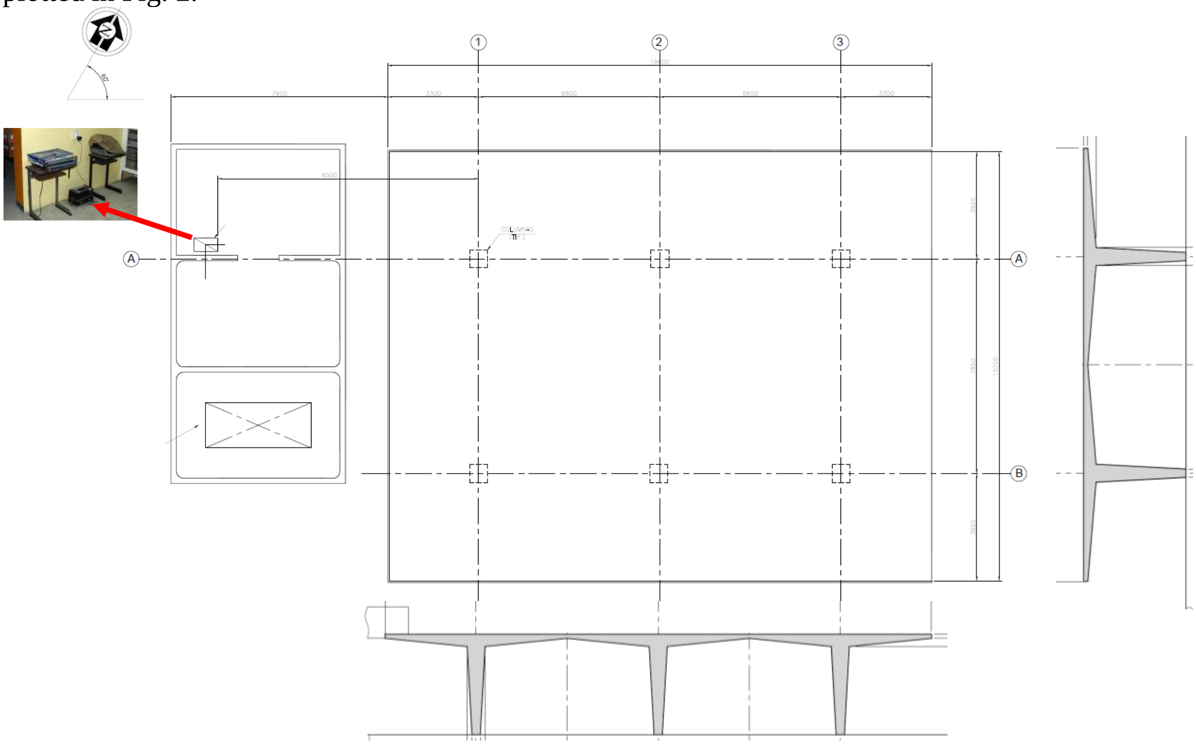


Fig. 2 – Canopy structure. Plan view, Elevations and plant showing accelerometer distance 10 m.

3.2 Description of the failure

To complete the background of the collapsed structure, there is the valuable account of Mr. Salvador Sáez Ruiz [5] who was present at the College the night of the mega earthquake (He worked in maintenance at the College at the date of the earthquake and currently as head of the Central of Notes). It should be noted that he was safe in the personnel casino at a distance of about 20 m from the structure under study, and that he remained there until the earthquake ended. From that privileged position he could first listen and then see the damage to the College caused by the earthquake. His observation includes views of the College during his evacuation there, as well as visits in details the following days. His account with very valuable information for the interpretation of the failure is described herewith:

- Heard what he identify as the collapse of the structure of the roofed patio due to the direction of origin of the sound and the burst of glass (Only this structure had widespread damage to glass). This noise he described "as the closest thing to war, due to the sound of frequent explosions"
- Once the main earthquake ended, and during his escape, describes that in this place "there was a lot of dust in the air" caused by the pulverization of the columns.
- For the “explosions” of the columns, his estimate is that “it is not more than 15 seconds from the beginning of the earthquake”. This precision, although not timed, corresponds to a kind of "count" that he performed waiting when the main shock would arrive from the initial tremor.
- He describes that the structure was dilated around 5.0 cm around, so there was no interaction with other structures before this displacement.
- He delivered a significant amount of photos of the damages recorded in the College.

Mr. Eduardo Cuevas (General Services Head of the College) was also interviewed [6] who was present during construction, since he has been linked to the College since the mid-1960s, adding among other things that the demolition work lasted much longer time initially estimated by the Fabbro construction company responsible of the demolition due to the hardness of the concrete and the abundance of the steel reinforcement.

While the photographs are not a rigorous engineering survey of the damage. They deliver very valuable information due to the numerous views and details. Fig. 3 shows the image of the collapsed structure and a summary of the main damages displayed there.

Fig. 3 - Collapsed structure and main demages. (Main photo taken from video https://www.youtube.com/watch?v=QzJFbhYWmbw. Small photographs Salvador Sáez)

It can be observed:

- - Columns failed only at the base.
- - The central columns suffered damage in a wider length than the lateral ones. The centrals descended approximately 120 cm, while the lateral ones between 70 and 80 cm.
- - The structure descended less in the north axis given the presence of steel profiles of a window that they provided additional resistance.
- - The fault in the columns was of the fragile type: produced by a mechanism of failure of shear, or by compression of the concrete and lack of confinement.
- - There is evidence of a large displacement in the positive direction of N60 ° E due to the collision with the surrounding chapel, and at some point it must reach a displacement of about 35 cm.

- The “explosive” detachment of the south-east railing seems to account for a clash between the canopy and the main building in the negative direction of N60 ° E. Since, the railing was well anchored and it does not have enough mass to generate such a detachment.

- In the other directions there is no evidence of crashes or large displacements.

The description of these damages is valuable and preponderant for the validation of the structural model to be used and in the conclusions that can be obtained there.

4 Structural model

4.1 Geometry

From the geometry shown in Fig. 2, a model based on prismatic bars for the columns and two-dimensional finite elements, shell type, for slabs is generated. To model the variable section of the columns it was divided into 10 spans, using the average section in each of them. In similar way the variable section of the slab was considered by dividing it and using the average section.

The CSI Perform-3D software was used for structural analysis. The fiber model is used for compression bending interaction behavior. On the other hand the shear is modeled with a non-linear spring located at a distance equal to the useful column height (“d”) at the base, decoupled from the other efforts due to software limitations, with the capacity provided by the dominant compression of 42 tons in the columns.

The inertial mass is distributed in each node to capture the effect consistently.

Since the structure is prior to 1970, before the concrete confinement was a standard in construction, the behavior is fragile. For this type of structure ASCE 41-13 [7] recommends to use a modal damping equivalent to 2% of the critic.

4.2 Relevant boundary conditions

A preliminary study of the structure produced the collapse in the longitudinal direction of the structure and the ground record, but in the opposite direction. Also the time of occurrence and failure modes were similar to the observed. The study did not pay much attention to the boundary conditions attributed the canopy due to the secondary structures.

On the other hand, an initial study of the contactless or isolated canopy of the surroundings provides a similar failure mode in all columns. However, in the field it is observed a greater decrease in the central columns (Axis 2), if it is observed longitudinally. It is also notorious as well as an inclination towards East observing it transversely.

a) b) c)

Fig. 4 - a) Partition type “Waffle”. b) Steel link in column "1-A". c) Aluminum corner in column "1-B".

The photographic background shows secondary elements that affect the behavior of the structure. The following elements that affected the response were detected:

- The "north" facade had a near and parallel partition "Axis 3" of type “Waffle” (Commercial "Covintec" or "Impac Panel") with steel box profiles that prevented a further fall of that axis (See Fig. 4 – a)..

- Around the column "1-A" there was a steel link of an old window between the main building of the school and the Canopy. The photos show that this link prevented a major fall from this corner (See Fig. 4 – b).

- Around Column "1-B", "South East", there was a corner conformation of four aluminum profiles of the window. This configuration allowed a certain degree of bracing between them, giving it axial resistance, which prevented a greater fall of the concrete column there located (See Fig. 4 – c).

The inclusion of these restrictions and precisions in the model achieves the fall in the right direction and sense, with failure modes and time according to what was observed in the field.

4.3 Materials and design

There are no structural drawings available; however, we have the background described herewith. The permits and construction plans of the College date from 1964 and 1965, the courtyard was built in 1968 [5, 8]. During that period, CAP released bars with projections 44-24 H, the previous A37-24H were smooth without projections [9]. The photos show reinforcements with projections for both stirrups and longitudinal reinforcements steel. Since it is defined the quality of the steel A 44-28 H, therefore F_y = 280 MPa and F_u = 440 MPa. The elongation of the bars was 16%.

Interest in the seismic performance of old structures has led to the development of a preliminary draft standard for interventions in heritage structures [10]. In this document and ASCE 41 recommend a cylindrical resistance of f`c = 15.2 MPa for old concrete in which the resistance could not be obtained by testing. The resistance that corresponds to a concrete "Class C" of the norms NCh 429-1957 and NCh 430-1961 [11, 12], usual quality for the time of construction of the covered patio.

ASCE 41 and the proposed draft standard described in the previous paragraph recommend increasing the strength of concrete and reinforcing steel. The characteristic strength of concrete by a factor of 1.5, to take into account the hardening of the concrete made more than 40 years ago. The Mander curve is used to model the nonlinear behavior of unconfined concrete [13], ($f`_{co}$= 15.2 MPa, ε_{co}= 0.002). On the other, the strength of steel is increased by a factor of 1.25, both for creep and for rupture, to take into account the statistical value of the strength of the steels of that time. To describe the curve, the Priestley-Calvi model [14] is used. The ASCE 41 standard also indicates the method of design and evaluation of the strength of the structure.

The diameters of the bars are obtained by proportions with other elements of known dimensions that surround them. From the photographs of the damaged structure it appears that the vertical reinforcements corresponded to 3 bars of ϕ25 mm per side from the base and up to 1.20 m high. After that height and up to the ceiling the reinforcement doubled to 6 bars per face. No damage is observed in the columns from the steel change up. On the other hand the stirrups corresponded to bars of ϕ8 mm at 20.0 cm spacing, and with hooks at 90 °.

4.4 Seismic records

The accelerograms for Concepción Centro obtained in the College are shown in Figs. 5 [15]. Fits note that the significance of these records is that they correspond exactly to the energy received by the structure under study, without the need for axis rotations, or distance corrections, or modifications by type of soil.

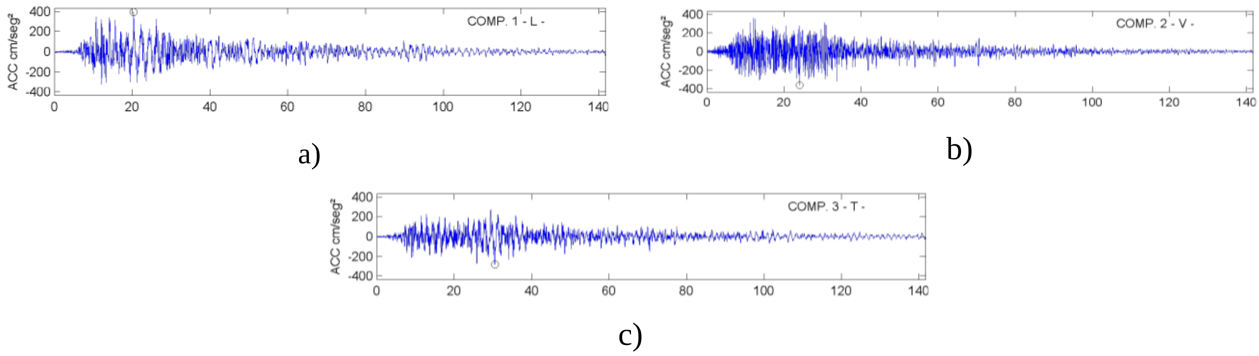
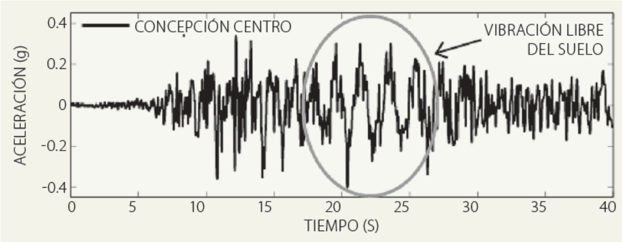
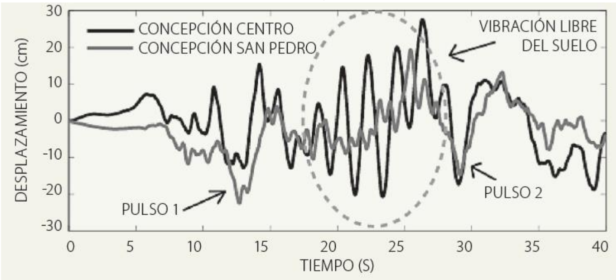


Fig. 5 – a) COM. 1-L, corresponds to N60°E direction. b) - COM. 2-V, corresponds to the vertical direction. d) COM. 3-T, corresponds to S30°E direction.

An important characteristic of this record is the existence of pulses from the northern and southern plate asperities. In Fig. 6 Saragoni and Ruiz [16] identified seismic pulses at intervals in which no ground-free vibrations are observed in the records of Concepción Centro and San Pedro. The pulses identified before and after the ground-free vibrations show similar shapes in both stations, confirming that they are related with the seismic source. This aspect of the records is highlighted to investigate their influence on the occurrence of collapses and the implications that this would have.



a)



b)

Important phases of the record

- Start of record from 0 to 10 s.
- Start of Pulse 1 from 10 to 10.9 s.
- Rayleigh type wave from 10.9 to 14.6 s.
- Rest of first pulse from 14.6 to 16 s.
- Free soil vibration up to 28 s.
- Start Pulse 2 at 28 s.

c)

Fig. 6 - a) Accelerogram of Concepción Centro, free soil vibrations are observed. b) Displacements, comparative records of Concepción Centro and San Pedro, showing the same two Pulses 1 and 2 and the free ground vibrations (Saragoni y Ruiz [16]). c) Important phases of the record.

5 Analysis of results

With the structural model and seismic records of the Immaculate Conception College, a rigorous analysis was carried out taking into account the three components of the ground motion simultaneously, which allows to study the complete seismic demand. In addition, alternative analysis were performed that considering only one horizontal direction at a time plus the vertical action of gravity and earthquake.

5.1.- Dominant modes.

Twenty modes of vibration were calculated, of which the first three mobilize practically the entire seismic mass (See Table 1). The main modes turn out to be transverse the first, torsional the second and longitudinal the third. The contribution of the following modes is negligible, confirming that the behavior correspond to a structure of one degree of freedom. The site effect is discarded as a cause of its seismic demand, since the structure modes are far out from the natural frequency of the soil [17,18].

Table 1. Structure modal periods and masses

Mode Nº	Períod (s)	N60°E (mass)	S30°E (mass)	Vertical. (mass)	N60°E Accumulate	S30°E Accumulate
1	0.453	0.000	0.872	0.000	0.000	0.872
2	0.452	0.000	0.093	0.000	0.000	0.964
3	0.431	0.974	0.000	0.000	0.974	0.964
4	0.188	0.000	0.000	0.009	0.974	0.964

5.2.- Analysis of structure drift and damage.

The main results of the analysis are summarized in Fig. 7. In this, the roof displacement (CG) is plotted in the longitudinal direction N60 ° E, shown by a blue segmented line. The non-linear behavior that models the displacement of each column at 1.20 on where it is postulated the plastic hinge. For identification in this Figure, each performance curve has the name of the intersection of the axes that cross it. The start and end of Pulse 1 have been identified with reddish arrows, and the start and end of the Rayleigh wave with gray arrows; In addition ground displacement is included (scaling).

As a reference and ease of reading, the structure behaves as elastic up to a drift 2.0 cm, after which the non-linear incursion begins, resulting in failure by shear when the roof drift greatly exceeds the previous displacement due to the Pulse 1.

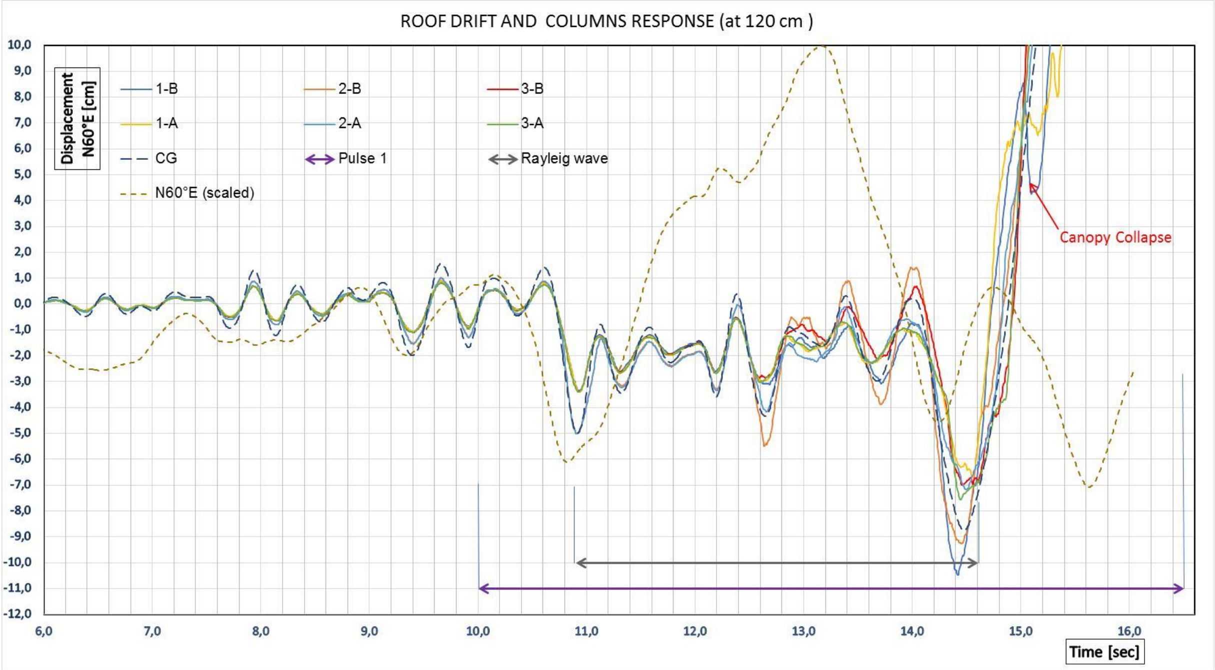


Fig. 7 - Roof offset in the N60°E direction and the columns at h=120cm.

An analysis of this graph indicates the beginning of the structure failure occurs in the central columns (A2, B2), failing these by shear and flexocompression during the start of the Rayleigh wave at 11 seconds. A decrease of the stiffness of the structure is also generated, which increases the period and decreasing therefore the seismic demand. However, the progress of the Rayleigh wave leads the edge remaining columns to the flexocompression failure at approximately 14 seconds.

Therefore, the graph shows a clear relationship between the arrival of energy during the Pulse1 from the seismic source and the Rayleigh wave associated as the reason of the collapse of this structure without participation of soil response.

In Fig. 8 the existence of the pulses in the N60 ° E direction for the Concepción stations is plotted for (Rec CC_N60E) and San Pedro (Rec SP_N60E), together with the response of the model studied to demand variants. Horizontal arrow and vertical lines are included that indicate start / end of the main events in time: Pulse1, Rayleigh Wave, Free Soil Vibration and Pulse Start 2.

The study of the response of the model with alternative records, in which its components are separated, leads to the following:

- The analysis model that includes the N60 ° E component (Mod N60E) produces a similar collapse than in mode and direction, dominated by flexocompression at the base and mid-height.

- On the other hand, the model that considers only the S30 ° E component (Mod S30E) does not achieve the collapse of the structure. This model shows a small non-linear incursion with the arrival of the "Pulse 2", coming from the northern asperity of the seismic source, which explains the instant of time and direction of the observed damage.

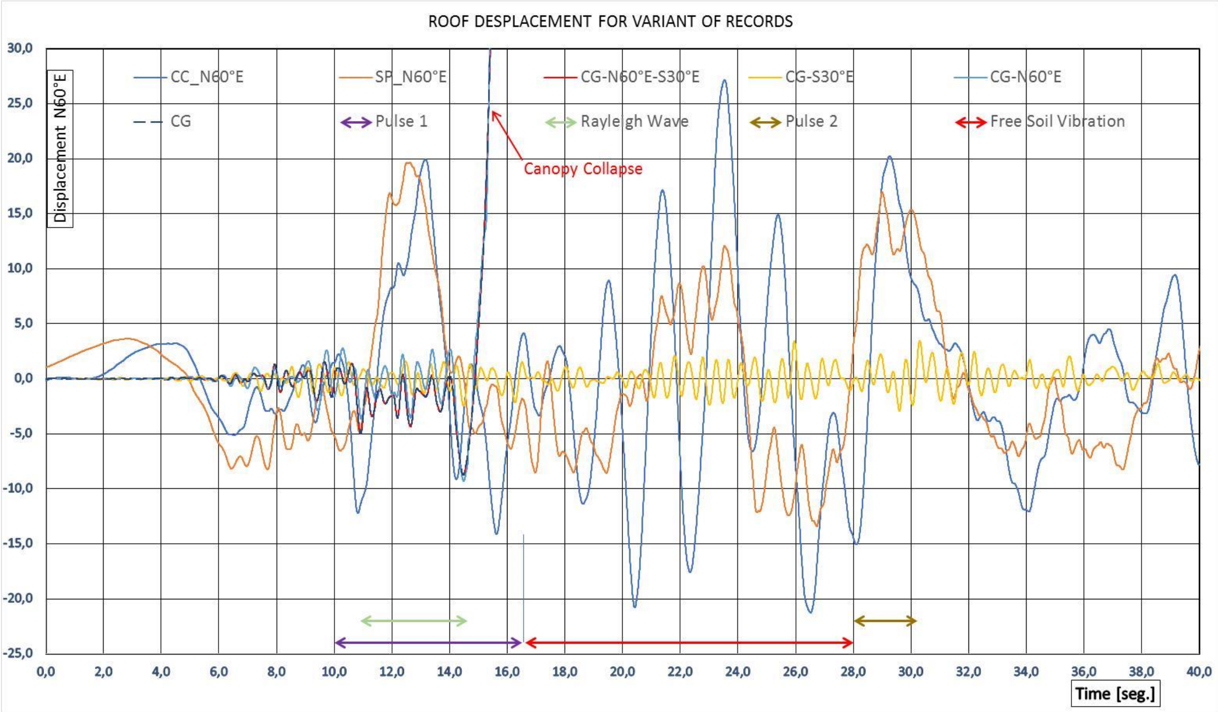


Fig. 8 - The existence of pulses in displacement record N60-E for Concepción Centro and San Pedro de la Paz compared with structure response to different records.

The response that includes the three components, is included in Fig. 7, it is called N60E+S30E in Fig. 8.

The time intervals with energy to produce damage are the same for the records of Concepción Centro and San Pedro de La Paz, indicating that the fault is initiated and produced by the effect of the earthquake mechanism and not by the soil response of Concepción. The pulse and the Rayleigh wave have a clear directivity, which is reproduced in the existing failure. The implications of the analysis could be extended to other building failure observed in the same direction at the Gran Concepción (Concepción Centro, San Pedro de la Paz, Hualpén, Talcahuano, Coronel).

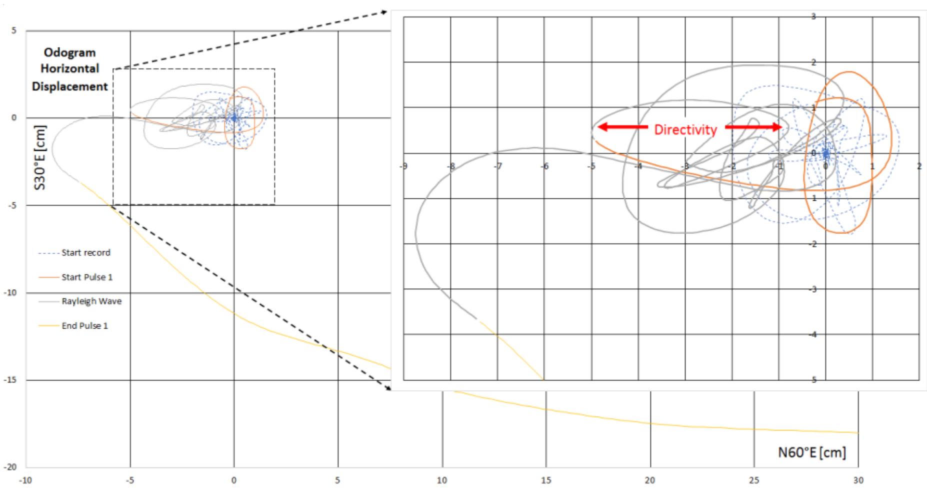


Fig. 9 - Odogram of the structure response to the Concepción N60-E record showing the same polarization from the beginning of seismic source Pulse 1.

In Fig. 9 shows the odogram that relates the horizontal offsets of the model for the full record. This shows a clear polarization with the start from the beginning of Pulse 1, remaining and accentuating during the domination of the Rayleigh wave.5.3. Horizontal Displacements

5.3. Comparison of analysis result with field observation of the damage.

The failure modes delivered by the structural model, shown in Fig. 10, are consistent with what was observed in the field, in terms of the types of failure and most damaged elements. The main aspects are summarized below:

- The central columns result in greater damage, including failure at 11 seconds due to shear and flexo-compression before the increase in height reinforcements to 1.20 m.

- The lateral columns collapsed due to the shear and flexocompression failure from 11.9 s to approximately 14 s.

- The structure descends less in the northern columns, by the support given by the metal columns of the window.

- The predominant direction in the model for the displacements in the failure is the positive direction N60 °

E, and a small component perpendicular to the west.

Fig. 10. Comparative of damage time response for the analysis with three-components.

The coincidence between observed and the results are adjusted satisfactorily in direction, failure mode and direction; concluding that the failure mode of the real structure has been well captured by the model. With these results it can be examined which part of the seismic signal is the one that produces the failure.

5.4 Comparison of this study with the collapsed "Alto Río" building

The collapse of the "Alto Rio" building in Concepcion became a worldwide icon of the El Maule 2010 Chile earthquake. This building, analyzed by numerous authors, has shown the difficulty to explain the shape and timing of its collapse (1, 2, 20, 21, 22, 23 and 24).

Due to the good results obtained in the simple structure object of this study, it proceeds to compare with the results of other authors for the "Alto Rio" building. The initial or elastic natural periods are practically identical for the Canopy and "Alto Rio" 0.49s and 0.45s respectively, so it is foreseeable that a similar initial response will occurs, before it starts a degradation of their structural capacities, with this objective in this section the results of roof drift of both structures are compared.

It should be noted that the digitized files of the data of the printed graphics were digitized. In addition, slight corrections of fractions of a second, were included in the starting time of the record (t=0) for each studies.

The comparison is shown in Fig. 11, showing a similar response behavior before the arrival of Pulse 1 of the seismic source and associated Rayleigh wave in both structures. Structural degradation or progressive collapse is also seen, manifested as an increase in the vibration period of both structures. This allows to postulate that the energy input of the Rayleigh wave in both structures was similar. The low structural redundancy of the Canopy produces a rapid collapse. On the other hand in the "Alto Rio", the energy input causes damage, but its greater redundancy of structural elements allows a slower degradation which would delayed its collapse. The comparison is limited to the collapse time of the first structure.

Finally, it is important to note that collapse in both structures occurs in the same direction and sense, which is approximately N60°E.

Fig. 11. Comparative damage graph for record with three components.

6 Conclusions and comments

The present study allows showing evidence of directivity in the damage in the city of Concepción produced by the earthquake of “El Maule”, on February 27, 2010, which is associated with Pulse 1 of the seismic source associated with the southern asperity of the earthquake.

A successful survey of the geometry and the structure of the "forgotten" Canopy was done, allowing a structural model with damage similar to the observed at field and consequent with the witness account (19). The damage occurs in time intervals corresponding to the entry of Pulse 1, and its associated Rayleigh wave, in the interval between 10 and 14 seconds of the seismic record.

Given the similarity of fundamental periods of the Canopy and "Alto Rio" and their behavior in time of their roof drift and the direction of their collapses, it is inferred that Pulse 1 contains energy capable of damaging structures and according to structural redundancy of these lead them to a progressive collapse.

On the other hand, given the similarity of the seismic pulses detected in the records of Concepción Centro and San Pedro de La Paz, it is estimated that the conclusions obtained in this structure can be extended to others in the Gran Concepción.

7 Acknowledgments

To Eduardo Vega, director of the Immaculate Conception College, who allowed access to the archives of the College to gather background information. Also to Salvador Sáez and Eduardo Cuevas, who shared their experience and knowledge during construction, as well as during and after the earthquake. To Rodrigo Retamales guidelines in structural analysis for seismic performance.

8. References

[1] Alimoradi A, Naeim F. Did the large coseismic displacement cause the global overturning collapse of the Alto Río Building during the 27 February 2010 offshore Maule, Chile Earthquake? The Structural Design of Tall and Special Buildings, 19, 2010. P. 876 – 884

[2] Zhang P, Conte J, Restrepo J, Ou J. Detailed Nonlinear Modeling, Pushover and Time Historiy Analysis of Alto Rio Building Using Beam-Truss Model. 16th World Conference on Earthquake Engineering. Paper No. 4571. Santiago, Chile. 2017.

[3] 1978 and 1980 Graduation Books. Immaculate Conception College. Internal edition.

[4] 1978 and 1980 Duhart E, Goycoolea R. Immaculate Conception College Extension. Architecture Plan. 1965.

[5] Sáez S. Personal communication. 2018

[6] Cuevas E. Personal communication. 2018

[7] ASCE. ASCE41-13 Seismic Evaluation and Retrofit of Existing Buildings. 2013

[8] Book 90 years Immaculate Concepción Externado School. Internal edition. 1989.

[9] Saragoni R. Personal communication. 2018

[10] Sarmiento R, Retamales R. Development of a Proposed Draft Standard for Preliminary for Structural Interventions in Patrimonial Building. Memory to qualify for the title of Civil Engineer. Universidad de Santiago, Chile. 2018.

[11] INN. NCh 429 – Reiforced concrete – Part 1. 1957

[12] INN. NCh 430 - Reiforced concrete – Part II. 1961

[13] Mander J B, Priestley M J N, and Park R. Theoretical Stress-Strain Model for Confined Concrete. Journal of Structural Engineering, Vol. 114, No. 8, August,1988.

[14] Priestley M, Seible F, Calvi G. Seismic Design and Retrofit of Bridges. Wiley Interscience, 1996.

[15] Boroschek R, Soto P, Leon R. Terremoto Maule. 27 de Febrero de 2010 Mw = 8.8. Civil Engineering Department , Faculty of Physical and Mathematics Sciences, University of Chile. 2012.

[16] Saragoni R, RuizS. Implications and New Challenges for the Seismic Design of the 2010 Earthquake. Mw - 8.8 Earthquake in Chile, 27 February 2010. Civil Engineering Department, Faculty of Physical Sciences and Mathematics, University of Chile. 2012. (In Spanish)

[17] Sandoval M, Saragoni R. Analysis of Seismic Demand in the Collapse of the Alto Rio Building, Considering Wave Propagation, During the Earthquake of February 27, 2010. Memory to qualify for the title of Civil Engineer. University of Chile. 2017

[18] Vivallos J, Ramírez P,. Fonseca A. Seismic Microzoning of the City of Concepción. Bio-Bío Region. Geological Chart of Chile National Subdirectorate of Geology. 2010.

[19] Ramos R. Analysis of the Apparent Directivity of Damage in Conception in the Earthquake of February 27, 2010. Thesis in development to qualify for the Master's degree in Engineering Sciences. Mention Seismic Engineering. Faculty of Physical and Mathematics Sciences. Civil Engineering Department. University of Chile.

[20] Bonelli P. Bonilla K. Boroschek R. Assessment of Ground Motion and Correlation with Damage In Buildings, 2010 Chile Earthquake 16th World Conference on Earthquake Engineering, Santiago Chile, January 9th to 13th 2017. Paper N° 3614

[21] Restrepo J. Conte J. Dunham R. Parker D. Wiesner J. Dechent P. Detailed Nonlinear FE Pushover Analysis of Alto Rio Building.16th World Conference on Earthquake Engineering, Santiago Chile, January 9th to 13th 2017.Paper N° 3904.

[22] NIST GCR 14-917-25. Recommendations for Seismic Design of Reinforced Concrete Wall Buildings Based on Studies of the 2010 Maule, Chile Earthquake. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering March 2014 U.S. Department of Commerce National Institute of Standards and Technology Engineering Laboratory.

[23] Zeynep T. Wallace J. Collapse Assessment of the Alto Rio Building in the 2010 Chile Earthquake. Earthquake Spectra December 2014.

[24] Song Ch. Pujol S, Lepage A. M.EERI. The Collapse of the Alto Río Building during the 27 February 2010 Maule, Chile, Earthquake. Earthquake Spectra June 2012.

