Double pushover analysis of a RC wall building subjected to earthquake and tsunami in sequence

S.J. Tagle\textsuperscript{1}, R. Jünemann\textsuperscript{1,2}, J.Vásquez\textsuperscript{2}, J.C. de la Llera\textsuperscript{1,2}

\textsuperscript{1}Department of Structural and Geotechnical Engineering, Pontificia Universidad Católica de Chile
\textsuperscript{2}National Research Center for the Integrated Management of Natural Disaster (CIGIDEN)
CONICYT/FONDAP/15110017, Vicuña Mackenna 4860, Santiago, Chile

Abstract

Chile is located in one of the most seismically active areas in the world and has an extensive coastal zone that makes it vulnerable not only to earthquakes but also to tsunamis. Additionally, Chile has a typical type of residential building which is based on reinforced concrete walls with high wall density and with a floor plan configuration that resembles a “fish-bone” structure. Research has been focused on the seismic behavior and response of this type of buildings, however there is little information on the expected behavior of the typical Chilean building subjected to both earthquake and tsunami. Thus, the objective of this research is to analyze the response of typical Chilean buildings subjected to sequential loading of earthquake and tsunami by means of nonlinear pushover analysis. A real 20 story building damaged during 2010 Chile earthquake is taken as a case-study building, and a simplified nonlinear finite element model of the building is developed using the software DIANA. These models are first subjected to earthquake pushover until different damage states are reached. Then, tsunami pushover is applied following the variable height pushover (VHPO) approach. The response of the building subjected to the sequential action of both loading conditions is analyzed in terms of typical response variables such as lateral roof displacement and base shear. The results of this research will help to understand the vulnerability of this type of structures and evaluate how critical is the previous earthquake damage in the tsunami response of this type of structures typically used in Chile and other seismic countries.

Keywords: shear wall damage; earthquake and tsunami in sequence; inelastic finite element models; simplified model
1 Introduction

Chile is one of the most seismic countries in the world and although the majority of the engineered buildings showed a good behavior after the 2010 Maule earthquake, nearly 2% of the reinforced concrete (RC) buildings with more than 9 floors suffered significant damage, typically concentrated in the RC walls at lower levels [1]. Chile is also a country with more than 4,200 km of coastline with large urban centers located in coastal sectors exposed to tsunami action. In general terms, modern structures showed good behavior after the tsunami that followed the 2010 Maule earthquake and only scour of foundations was observed [2]. For the 2015 Illapel earthquake, around ten buildings with more than 8 stories were inundated by the tsunami. Showing only damage to non-structural elements due to the wave force on the first story [3].

A tsunami is a massive wave triggered by an earthquake, landslide or other disturbances. Once the wave, or series of waves, reaches the shore with a high speed, it starts to cause damage to the different constructions and objects on its way. Nowadays, Chilean seismic codes like Nch433 [4], Nch2369 [5] and Nch2745 [6] provide guidelines for seismic analysis and design, but do not take into account the possible tsunami action. On the other hand, the Chilean code Nch3363 [7], Japanese code MLIT 2570 [8] and ASCE 7 [9] in the United States provide the minimum design requirements for structures located in zones exposed to tsunami flood. These codes consider that the main actions of the tsunami on structures can be replaced by the following loading cases: i) hydrostatic force: force caused by a static fluid over a surface; ii) buoyancy: vertical force experimented by the object submerged; iii) hydrodynamic force: force caused by a fluid at a certain speed over a surface; iv) debris impact load: force caused by a fluid in movement when it hits a surface; v) impact of floating objects: force caused by the impact of big objects, like vehicles, over a surface; and vi) uplift: vertical pressure presented over a slab after a wave hits a wall. These codes consider the tsunami actions in different ways. For example, the Chilean code calculates the lateral tsunami load considering hydrostatic and hydrodynamic loads based on the expected inundation depth obtained from inundation maps [7]. The American code takes into account the same loads, but inundation depth and velocity are required [9]. Finally, the Japanese code calculates the lateral tsunami load as an hydrostatic load, where inundation depth is taken from a tsunami inundation map. This load is amplified by a factor dependent on the distance from shore and the presence of energy dissipation structures [8].

Most research on behavior and performance of structures has been focused on either seismic or tsunami action separately, but scarce literature is found on the sequential action of earthquake and tsunami. De la Barra [10] studied the behavior of a RC building subjected to the seismic action followed by tsunami by using a pushover analysis. In this study, a shear wall building constructed in 1979 code was considered as a case study. A resisting plane of the building was modeled using fiber elements in OpenSees. One of the findings is that the capacity of the structure to tsunami is minimally influenced by the preceding earthquake damage. Latcharote et al [11] studied a reinforced concrete building under the sequential action of an earthquake and a tsunami. A wall-frame model of a 7 story building was developed and subjected to earthquake ground motion, followed by hydrostatic force as a starting point to consider the sequential action of these loads. This study recommends the use of a hydrodynamic force to predict structural damage due to tsunami in a more realistic way.

The objective of this research is to study the behavior of a typical RC wall building, subjected to the sequential action of earthquake and tsunami through double pushover analysis. The structure is first subjected to a seismic pushover until a certain damage state is reached, and then the tsunami load is applied up to the capacity of the building. A real medium rise RC wall building constructed in 2005, that
was damaged during the 2010 Maule earthquake, is selected as a case-study. The seismic performance of this building has been previously studied in detail [12][13]. A simplified model of the building is developed in this study which consists in an inelastic finite element model (FEM) of a fictitious slice of the building using the software DIANA [14].

The simplified model proposed in this research is subjected to the seismic action through inelastic pushover analysis. Four different seismic damage states are evaluated, and the tsunami loading is applied in each case. This research considers the tsunami action through the variable height pushover (VHPO) [15] formulation. This formulation is based on Qi et al [16] equations, which provide the tsunami hydrodynamic force acting on an obstacle. A set of tsunami loading cases are investigated applying different loading directions and Froude numbers, and considering different preceding earthquake damage states. Based on these results, the effect of previous earthquake damage in the tsunami capacity of the building is discussed.

1 Simplified FEM for a reinforced concrete wall building

This section describes the inelastic finite element formulation that is considered throughout this research. Nonlinear finite element models of RC walls are developed in the software DIANA 10.2 [14] since it has shown good results in previous research regarding RC wall behavior [12][13][17]. Inelastic constitutive models for concrete and steel are considered. The total strain rotating crack approach was used to model concrete behavior, in which the principal stress tensor is evaluated in the principal strain directions [14]. The parabolic model is considered to model compressive concrete behavior, while the Hordijk model [18] is considered for tensile behavior. Both are based on the fracture energy. The compressive fracture energy for the parabolic model ($G_c$) is obtained from recommendations given by Pugh et al [19]. For tensile behavior, the tensile fracture energy is obtained according to the CEB-FIP code recommendations [20]. To model steel behavior, the Menegotto-Pinto model [21] is used. This model does not include bar buckling or fracture. The type of element used to model concrete elements in the analysis is the Q20SH, as defined by the software DIANA [14]. This is a curved shell quadrilateral element with four nodes and four Gauss-point quadrature [22]. It is selected because it has shown good results in reproducing the expected behavior of RC walls [17] and is able to capture the out of plane buckling of the wall. The element used to model the reinforced steel is the "bar" element, which is an embedded element that assumes perfect bonding [14].

A real RC wall building representing the typical Chilean residential building was selected as a case-study. The building under study has 18 floors and 2 basements and has a “fish-bone” type floor plan configuration, as shown in Fig. 1. Thickness of all the walls and slabs is 20 cm and 15 cm, respectively. This building suffered brittle failures in some RC walls after the Maule 2010 earthquake, as shown in Fig. 2a. Most of the damaged walls presented a flag-shape configuration in height (Fig. 2c) and the damage was typically concentrated at the basement, specifically in the wall irregularity. The crack was propagated horizontally where concrete crushing and buckling of rebars was observed [12].
A simplified model of the building is proposed in this research with the aim of reproducing the response of the building at a lower computational cost than the previously developed 3D inelastic models of the entire building [23]. The wall at axis Q (shown in red in Fig.1) is selected as a case study to be modeled since it was one of the most damaged walls. This wall has a flag-shaped configuration, as shown in Fig.2c, and has a different cross section in the basements (Fig. 2b) and from first story and up (Fig.2d). In the basements, the flange goes along all the building width. However, from the first floor and up the length of the flange is reduced to 2.96 m, as shown in Figs. 2b and 2d.

Previous studies have shown that considering only the isolated cantilever wall does not represent the expected behavior [12]. Thus, in order to capture the 3D interaction between the wall and the rest of the building, it is necessary to include a complementary portion of the building. Since the building is not exactly symmetrical and a real "slice" of the building is complex (Fig.2), the selected wall is replicated as in a mirror, as shown in Fig.3. The wall separation $S_W = 1.6 \, m$ is considered, which is based on the width of the corridor in the real building. The slab width $W_s = 3.9 \, m$ was selected equal to the wall flange width at the typical story plus half of the door width at each side of the Q axis. Finally,
the wall flange at the basements $F_{bw} = 8 \ m$ was considered equal to half the distance of each wall located next to the Q axis in the basement. These variables were selected based on a sensitivity analysis developed elsewhere [24].

Nominal material properties specified in the project are used. A yield stress of 420 MPa is considered for steel, and concrete compressive strength of $f' \ c = 25 \ MPa$. Only unconfined concrete is considered. For computational economy, nonlinear concrete and reinforcing steel rebars are considered only until the 5th floor and from 6th floor and up, only linear concrete is considered. Slabs are modeled with linear elastic behavior at all stories. The translation in X, Y and Z directions are restricted at the base nodes of both walls. Additionally, the translation in the X direction is restricted at the free borders of the slab. No rigid diaphragm is considered.

![Center of gravity](image)

Identical replicated wall

Original wall

$F_{bw}$

$S_{nw}$

Fig. 3 – Simplified model and their variables

2 Earthquake loading

An earthquake pushover analysis is developed on the simplified model with load pattern of roof displacement for different damage states. Damage state I ($D_I$) corresponds to the first crack in concrete, which occurs at 2.7 cm roof displacement. Damage state II ($D_{II}$) corresponds to boundary steel yielding in compression at 10.3 cm roof displacement. Damage state III ($D_{III}$) corresponds to the peak strength of the structure at 13.9 cm. Finally, Damage state IV ($D_{IV}$) corresponds to the final state after a brittle failure occurred due to concrete crushing in the critical section. The different damage states are shown in Fig.4a.

The model is loaded up to a specific damage level, and then is unloaded until zero base shear condition, which corresponds to the final state after earthquake loading and the initial state for tsunami loading. Fig. 4b shows the loading and unloading process of the earthquake pushover for the different damage states in terms of the total base shear, i.e., considering both walls. Fig.4b shows that for damage states $D_I, D_{II}$ and $D_{III}$ the behavior of the structure is essentially elastic. For damage state $D_I$, the loading-unloading occurs through the same curve and there are no residual deformations. For damage states $D_{II}$ and $D_{III}$, the unloading occurs through almost the same loading curve and the residual deformations are very small (0.1 cm and 0.15 cm for $D_{II}$ and $D_{III}$, respectively). In contrast, for damage state $D_{IV}$, significant inelastic excursions occur, and residual deformations of 4.7 cm are observed.
3 Tsunami loading

In order to adequately represent the effect of the tsunami on the structure for assessment purposes, this research focuses on the variable height pushover (VHPO), which correspond to a hydrodynamic loading approach proposed by Petrone et al. [15]. To implement the VHPO, the equations for the tsunami force proposed by Qi et al. [16] are considered. In this approach, a dense area of buildings is assumed and additional effects as flooding at the back of the building, buoyancy, uplift and debris impact are neglected. The location of the tsunami load is presented in Fig.6.

The tsunami net force \( F \), experimentally obtained by Qi et al. [16], is presented in Eq (1), where \( C_d \) is the drag coefficient; \( u \) is the flow velocity; \( g \) is the acceleration of gravity; \( \rho \) is the density of the fluid; \( h \) the height of the wave; \( \lambda \) is a leading coefficient; \( b \) the tributary width where the tsunami is applied; \( F_r = u/\sqrt{gh} \) is the Froude number and \( F_{rc} \) is the Froude number threshold [15].

\[
F = \begin{cases} 
0.5C_d\rho u^2h & \text{if } F_r < F_{rc} \\
\frac{1}{4}4^{1/3}\frac{1}{4}\frac{1}{4}4^{4/3} & \text{if } F_r \geq F_{rc} 
\end{cases}
\] (1)

The Froude number is a parameter depending on the height and speed wave, thus each tsunami event can be characterized by a specific Froude number. However, there is no recorded information on wave speed in past tsunami events in Chile. Thus, Froude numbers estimated by T. Asai et al. [25] in different locations after 2011 Great East Japan earthquake, will be used to perform a sensitivity analysis of this parameter. The values considered for the Froude number in the present investigation are 0.6 and 1.27, that may represent low and high Froude number limits for areas prone to tsunami inundation.
4 Earthquake and tsunami in sequence

A double pushover analysis of earthquake and tsunami in sequence is applied in this section to the simplified model. The schematic loading sequence is described in Fig. 7. First, the loading of the seismic pushover is developed by applying lateral displacement to all nodes at the roof level until a specific damage state is reached (Fig. 7a). Second, the seismic pushover is unloaded by applying lateral displacement to all nodes at the roof level in the opposite direction until zero total base shear is obtained (Fig. 7b). Finally, tsunami pushover is applied from ground level following the VHPO approach (Fig. 7c). A positive tsunami pushover means that the direction is the same as the seismic pushover and a negative tsunami pushover means that this load is applied in the opposite direction of the seismic pushover.

As was mentioned before, two values of Froude number are considered: $F_r = 0.6$ and $F_r = 1.27$. Additionally, two different directions of the tsunami action are evaluated: positive and negative. Finally, the four different damage states $D_I$, $D_{II}$, $D_{III}$ and $D_{IV}$ are considered, as well as the case with no previous earthquake ($D_0$). This produces a total of 18 double pushover sequential analysis, since the case without previous earthquake damage ($D_0$), the direction of the tsunami has no effect due to the symmetry of the simplified model. Results for each analysis are presented in terms of base shear versus top displacement in Fig. 8. Fig. 8a shows tsunami pushover (positive or negative) without previous earthquake damage ($D_0$) for Froude numbers $F_r = 0.6$ and $F_r = 1.27$. The results show that the behavior in terms of the pushover curve shape is similar for both Froude numbers, but the peak values are different. For $F_r = 1.27$ higher total shear force is obtained compared to $F_r = 0.6$, while higher top displacement is observed for $F_r = 0.6$.

![Fig. 7 – a) Loading seismic pushover; b) Unloading seismic pushover; c) Loading positive tsunami](image)

Fig. 8b shows the double pushover analysis for the earthquake at damage state $D_I$ for negative and positive tsunami direction. The earthquake loading-unloading is shown with a black continuous line. Results show that previous damage state $D_I$ has no significant influence in this case, since the peak base shear, peak top displacement and curves in Fig. 8b are very similar to the case without a previous earthquake (Fig. 8a). It is also observed that the tsunami direction has no influence on the results. The different behavior for each Froude number is also observed in this case. In the same way, Fig. 8c shows the double pushover analysis for the earthquake at damage state $D_{II}$ for negative and positive tsunami directions. In this case, the earthquake pushover reaches a higher base shear and higher top displacement than in the case of damage state $D_I$. The curve shape, peak base shear and top...
displacement are again very similar to the cases without previous earthquake damage \( (D_0) \) and case \( D_I \). Fig. 8d shows the same trend for damage state \( D_{III} \).

Finally, Fig. 8e shows the results for the earthquake damage state \( D_{IV} \). In this case, the earthquake unloading produces a condition of significant residual displacement where the tsunami loading starts. As a consequence, the maximum base shear is smaller in this case than in previous ones for both Froude numbers. Additionally, the positive tsunami loading is the worst-case scenario where lower tsunami base shear is reached for both Froude numbers.

Based on these results, it is possible to conclude that the effect of previous earthquake damage in the tsunami capacity of the building is negligible for damage states \( D_I, D_{II} \) and \( D_{III} \) for both tsunami directions, which is consistent with the results obtained by De la Barra [10]. However, the effect of previous earthquake is considerable for damage state \( D_{IV} \) followed by a positive tsunami, where a reduction in maximum total base shear and an increase of maximum top displacement is observed respect to the other damage states. This is due to the significant residual displacement of the earthquake pushover.

Fig. 8 – Earthquake and tsunami in sequence on simplified model for \( F_r = 0.6 \) and \( F_r = 1.27 \): a) Tsunami without previous earthquake; b) Tsunami from \( D_I \); c) Tsunami from \( D_{II} \); d) Tsunami from \( D_{III} \); e) Tsunami from \( D_{IV} \)
Fig. 9 shows the crack pattern at the maximum capacity of the structure under a tsunami pushover for the case with no previous earthquake \( (D_0) \). Most of the cracks are located in the basements of wall 1. There are normal crack strains with a 45° angle observed.

![Crack pattern at maximum capacity for tsunami without previous earthquake](image)

**Fig. 9** – Normal crack strains (Eknn) at maximum capacity for tsunami without previous earthquake with \( F_r = 0.6 \)

Fig. 10 shows the crack pattern for earthquake damage state \( D_{III} \) (Fig. 10a), the peak of negative tsunami after the earthquake pushover with damage state \( D_{III} \) (Fig. 10b), and peak of positive tsunami after earthquake pushover with damage state \( D_{III} \) (Fig. 10c). For damage state \( D_{III} \) (Fig. 10a), maximum normal crack strains (Eknn) are observed in wall 1 at the first floor and also wall 1 presents more cracks than wall 2. In the case of a negative tsunami after an earthquake with damage state \( D_{III} \) (Fig. 10b), higher cracks are located in the first basement of wall 1, in an area where little damage is observed in Fig. 10a. Most of the cracks are located in wall 2, in a zone not cracked by the earthquake (Fig. 10a). In the case of a positive tsunami after an earthquake with damage state \( D_{III} \) (Fig. 10c), crack strains presented on the first floor of wall 1 are reduced compared to the cracks strains presented in Fig. 10a. Additionally, higher crack strains are now located in the first basement on wall 2. According to the observations previously presented, it is possible to conclude that the areas with higher damage due to the tsunami (positive or negative) are different than the areas damaged by the earthquake for damage state \( D_{III} \).

The magnitude and location of cracks caused by a negative or positive tsunami starting from damage state \( D_{III} \) (Fig. 10b and Fig. 10c) are very similar to the ones obtained in the case without previous earthquake \( D_0 \), as shown in Fig. 9. This supports the preceding conclusion that the previous earthquake damage has no significant influence on tsunami behavior for damage states \( D_1, D_{II}, \) and \( D_{III} \).

Fig. 11 shows the crack pattern for the earthquake damage state \( D_{IV} \) (Fig. 11a), the peak of negative tsunami (Fig. 11b), and peak of positive tsunami (Fig. 11c). Maximum crack strains at damage state \( D_{IV} \) (Fig. 11a) are nearly 3.8 times the maximum crack strains observed in the earthquake at damage state \( D_{III} \) (Fig. 10a). As observed, in the positive tsunami, crack strains are increased in the critical section of wall 2 (Fig. 11c), respect to damage state \( D_{IV} \) (Fig. 11a). This behavior is different from the crack strains shown in Fig. 10c for a positive tsunami, where no cracks are observed in the critical section of wall 2. Thus, the structure with damage state \( D_{IV} \) followed by a positive tsunami will present a concentration of cracks located in the same area previously damaged by the earthquake. This makes a positive tsunami loading from damage state \( D_{IV} \) the worst case scenario.
Fig. 10 – Normal crack strains (Eknn): a) damage state $D_{III}$ due to earthquake pushover; b) maximum capacity of the structure under a negative tsunami after an earthquake pushover with damage state $D_{III}$; c) maximum capacity of the structure under a positive tsunami after an earthquake pushover with damage state $D_{III}$.

In the case of a negative tsunami starting from damage state $D_{IV}$ (Fig. 11b), crack strains located in the zone damaged by the earthquake (critical section of wall 2) reduce their magnitude, because of the direction of the tsunami loading. The new cracks with higher magnitude are located at the bottom of wall 2, in a zone that was not previously damaged by the earthquake. Based on this, the tsunami loading direction presents a different behavior only for an earthquake with damage state $D_{IV}$.

Fig. 11 – Normal crack strains (Eknn): a) damage state $D_{IV}$ due to earthquake pushover; b) maximum capacity of the structure under a negative tsunami after an earthquake pushover with damage state $D_{IV}$; c) maximum capacity of the structure under a positive tsunami after an earthquake pushover with damage state $D_{IV}$. 
5 Conclusions

This paper analyzes the behavior of a typical Chilean building subjected to an earthquake and a tsunami in sequence through a case study building damaged after 2010 Chile earthquake. The software DIANA was used to develop inelastic finite element models of a damaged wall of the case-study building. A simplified model of the building based on a representative fictitious slice was developed and used for the earthquake and tsunami pushover analysis in sequence. In a first stage, earthquake pushover was developed with roof displacement pattern until a certain damage state was reached ($D_1$, $D_{II}$, $D_{III}$ or $D_{IV}$). Then, the earthquake was unloaded by applying roof displacement in the opposite direction. Finally, the tsunami load was applied until the capacity of the structure was reached. The tsunami load was modeled through the variable height pushover (VHPO) approach, which is based on a constant Froude number. In this study, additional effects such as flooding at the back of the building, buoyancy, uplift and debris impact were neglected. A sensitivity analysis was carried out for different Froude numbers and different tsunami directions. Results showed that a RC structure is very sensitive to the Froude number. For example, the maximum base shear without previous earthquake for $F_r = 1.27$ is $6.5 \cdot 10^3$ kN, while a tsunami with $F_r = 0.6$ the maximum base shear is $5.8 \cdot 10^3$ kN, which means an increase of 12%.

Results consistently show that the tsunami capacity of the structure is not significantly affected by previous earthquake damage for damage states $D_1$, $D_{II}$ or $D_{III}$. For these cases, the maximum base shear and displacement at peak are independent of the previous damage state, and independent of the tsunami direction as well. However, when an earthquake damage state $D_{IV}$ is considered, the tsunami capacity is significantly reduced, and the tsunami direction has also a significant effect. In particular, a positive tsunami from a damage state $D_{IV}$ turned out to be the most unfavorable case. This is because the tsunami increased the crack strains in the area previously damaged by the earthquake, so lowest tsunami heights were achieved in this case for both Froude numbers.

References


