GEOPHYSICAL STUDY AND 2D NON-LINEAR MODELING OF SITE EFFECTS IN VIÑA DEL MAR CITY, CHILE

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ABSTRACT

The Maule earthquake of February 27, 2010 induced damage to several buildings throughout the central area of Chile. This was particularly the case in the downtown of Viña del Mar city, where an anomalous concentration of structural damage was found in several medium-rise buildings distributed along a narrow area of approximately 1 km in length. These observations suggest possible localized seismic amplification effects. A geophysical characterization using Surface-Wave based techniques and gravimetry were conducted to characterize the main dynamic properties of the soil and the depth of the basin. This data was complemented with several Standard Penetration Test (SPT) measurements to develop a geotechnical characterization of the area. Additionally, the cyclic behavior and mechanical properties of predominant materials were characterized experimentally based on remolded samples. Based on this whole description, a finite element model was developed to estimate possible singular site amplification effects explaining the concentration of structural damage in the area. The results suggest that the concentration of damaged buildings is probably most related to the basin depth (1D site amplification) than to the basin shape effects.

Keywords: Geophysical methods, Site effects, Finite elements, Non-linear behavior

1. INTRODUCTION

As it is well known, Chile is one of the most seismic countries in the world, largely due to subduction of the Nazca plate beneath the South American plate. Particularly, the central area of Viña del Mar, historically known as Población Vergara, suffered considerable damage in the 1906 (Mw 8.2) and 1985 (Mw 8.0) earthquakes; both with an epicenter in the coast of Central Chile. In contrast, the adjacent uplands areas, emplaced on Paleozoic metamorphic and granitic rocks, were relatively undamaged (Thorsen, 1999). More recently, structural damage in 9 medium-height buildings (Jünemann et al., 2015) was reported in the same area of the Maule 2010 earthquake (Mw 8.8), located at more than 400 km north of the epicenter.

In the Viña del Mar area, Chilean current seismic design code for residential buildings classifies the soil as type D (according to \(180<V_{s30}<350\) m/s and SPT blow counts). This means that the design of all buildings is subject to the same design acceleration spectrum. Nevertheless, damage concentration suggests that acceleration levels are not uniform through this zone of the city.

First-order controlling factors for damage in urban areas are related to earthquake magnitude and relative distance to the large slip zones. These two factors contribute homogeneously at a regional scale. However, if damage distribution is locally clustered, this is probably related to soil amplification effects. This effect can explain a bounded damage zone during an earthquake, where the local...
morphology and geology are crucial (Chavez-Garcia, 1998). Thus, geophysical methods were used (such as surface waves methods and gravity) to characterize Viña del Mar’s central area, with the aim to obtain a basin geometry and the dynamic parameters of the infilling sediments. These parameters will provide the elements to model complex site effects in the study area.

2. GEOLOGICAL SETTING AND DAMAGED BUILDINGS

The studied area is in the downtown area of the city of Viña del Mar, Chile (33°01’30”S 71°33’9”O WGS84). This town is crossed by the Marga Marga river. The regional geology is conformed by a Paleozoic/Jurassic intrusive basement outcropping in the southern flank of the study area. The overlying basin infilling includes semi-consolidated marine terraces of the Navidad Formation (Miocene), outcropping in the northern limit of the study area, mainly clayey sandstone, fine-grained sandstone and coarse-grained sandstone. On top of this sequence, fluvial unconsolidated sediments are found, with a widespread distribution in the central low lands of the study area, composed of sand, silt and, to a minor extent, gravel (Gana et al., 1996). The geology units are depicted in Figure 1.

A comprehensive research of the distribution of damage to buildings during the earthquake of 2010 is available in Jünemann et al. (2015). According to this study, a common plan topology of residential Chilean construction with reinforced concrete consists of walls configured to resist gravity and lateral loads. Most damaged buildings presented high average axial stresses in walls due to gravity loads. This study defined three levels of damage based on the operational conditions of the buildings immediately after the earthquake: damage level I is assigned to buildings with restricted use; damage level II to buildings declared non-habitable; and damage level III to collapsed buildings or with risk of collapse. The approximate locations of the damaged buildings in downtown Viña del Mar are shown in Figure 1. It can be noted that almost all buildings are located on a straight line in the Qf unit defining a narrow area of approximately 1 km in length. This anomalous distribution and the reiterative damage with 1985 earthquake motivates this study.

Figure 1. Geology of the valley and damage buildings. Geology modification from Muñoz (2013).
Axis in meters (UTM 19S)
3. GEOPHYSICAL SURVEY

To characterize the area that concentrates damage to buildings, a detailed geophysical survey was conducted. Details of the techniques used, and main hypotheses are presented below.

3.1 Shear Wave Velocity Profiles

Surface-wave based techniques were used to obtain Rayleigh-wave dispersive properties at several sites in the studied area. By solving an inverse problem, the shear wave velocity profiles at each location was obtained. In this research, combined source-controlled (active) with ambient noise (passive) techniques were performed, using sensors arranged in several arrays to capture different wavelengths. The methodology to combine different analysis techniques to find a reliable shear wave profile was previously proposed in different studies (Tokimatsu, 1997; Humire et al., 2014). Frequency-wave number method (FK) (Kvaerna and Ringdahl, 1986) has been used for active experiments and 2D arrays. The SPAC method (Aki, 1957) was also used to analyze 2D passive measurements. The Extended Spatial Autocorrelation Method (ESPAC) (Hayashi, 2008) was selected as the analysis methodology for passive linear measurements.

The instrument used was a Geometrics® Geode-24 seismograph with twenty-four channels and geophones of 1 Hz and 4.5 Hz for the linear arrays, and six Tromino® 3G for the 2D passive arrays. With the purpose of obtaining the best and deepest shear wave velocity profile, different dispersion curves were combined to extend the characterization of the site across a wide range of frequencies. An example of the combined dispersion curve is depicted in Figure 2a. The inversion process was conducted with the neighborhood algorithm (Wathelet, 2008). The inversion process minimizes iteratively the misfit between the empirical dispersion curve and an analytical model. In this study, the minimal misfit obtained was about 0.05. The combined dispersion with the best-adjusted curve is depicted in Figure 2a, and the corresponding shear-wave velocity profile is depicted in Figure 2b.

![Figure 2](image_url)

Figure 2. (a) Combined dispersion curve with the best analytical model near to 8 Norte and 2 Poniente Street in Figure 1. (b) Best inversion associated with shear wave velocity profile

3.2 Predominant Frequencies Using HVSR

Another important methodology used in this research was the horizontal-to-vertical spectral ratio (HVSR) from microtremors or the Nakamura (1989) method. A 3-component, 4.5 Hz Tromino® instrument was used to perform the measurements throughout the studied area. At each point, the velocity from ambient noise for a duration of at least 16 min was recorded. The analysis procedure was based on Leyton et al. (2012): the data were divided into 60 seconds windows to calculate the Stockwell Transform (S-Transform) of each component; both horizontal components were combined
to finally compute the horizontal to vertical ratio. A typical result of this methodology is shown in Figure 3.

Figure 3. An example of Nakamura measurement, performed at Viña del Mar, with an amplitude of 5.3 and 0.93 Hz of frequency. a) HVSR for each 1 min. time window, with the grayscale proportional to HVSR amplitude. b) minimum (10, 20, 40, 60, 80 and 90) percentiles and maximum of HVSR

3.3 Gravimetry

To obtain information of sediments’ thickness variation over the basement, a gravimetric survey was performed across the studied area. Gravity information has been obtained using a Scintrex CG5 instrument and a differential GPS Trimble C5. In terms of accuracy, for gravimetric measurements the obtained errors were about 0.1 mGal and altitude errors below 50 cm with the GPS base station in Valparaiso. Altitude errors below 50 cm provide free air gravity errors below 0.02 mGal, much smaller than instrumental errors. To estimate the regional field in a continental-scale, the density model of South America (Tassara et al., 2006) was used as the first regional field. Local regional is obtained with measurements made on basement outcrops, defining in this way gravity points directly associated with the local regional field. Providing that these basement-associated gravity points can define a plane surface (3 points at least), the local regional effects were able to eliminate by removing this plane level from the data set. Details of the results obtained are presented below.

4. GEOPHYSICAL RESULTS AND GEOTECHNICAL MODEL

The geophysical survey was initially focused on the buildings damaged during the 2010 Chile earthquake. In total, seven shear wave velocity profiles were obtained; 4 were very close to the most damaged buildings and 3 others complete the characterization of the area. Figure 4a displays the representative location of measurements and the corresponding Vs30 values. It can be noted that Vs30 values are almost uniformly throughout the downtown of Viña del Mar. HVSR measurements form a grid in the area with approximately two blocks of interspacing. The results obtained are displayed in Figure 4b with a double scale; color is proportional to predominant frequency/period (F0 or T0), while the size of the symbol is proportional to HVSR maximum amplitude. Locations with associated predominant frequencies larger than 2.5 Hz are indicated in white. It should be noted that low frequencies are in the center of the area and the high frequencies are on the edges; in addition, amplitudes are fairly homogeneous, suggesting an almost constant impedance contrast between shallow sediments and deep stiffer material.

Approximately the same spatial distribution of HVSR data is used for gravimetric measurements (Figure 4a). A reasonably good correlation between both quantities can be noted, suggesting that a simple geotechnical model composed of an almost homogenous soil over a stiffer material (apparent bedrock) could explain both sets of data satisfactorily. A similar conclusion has been reported in previous studies (Thorson, 1999) based only on geotechnical boreholes and shear-wave profiles of about 30m depth. Based on this interpretation and the relative uniformity of shear-wave profiles, a curve (Equation 1) was fitted to the available shear wave velocity profiles up to one half of the maximum characterized wavelength (Figure 5a):
\[ V_s(z) = 158.62z^{0.2} \text{ [m/s]} \]  

(1)

where \( z \) is the depth from the surface in meters. The adjusted Pearson correlation coefficient was 0.88.

Figure 4. (a) Approximate location of shear wave velocity profiles with \( V_s30 \) values and gravimetric measurement with contours of residual values. (b) Nakamura measurements (\( T_o \) and \( F_o \)) and buildings’ fundamental periods (\( T_b \)) according to the total number of stories rule suggested by Jünemann et al. (2015)

Because it was not possible to obtain the exact location of stiffer material (engineering bedrock) with the shear wave profiles, it was decided to use the measured predominant frequencies to extend a soil column iteratively down to the longest period of the analytical elastic transfer function that matches the measured value. The shear wave velocity function of Equation 1 was used during this process. The soil’s density was fixed at 1700 kg/m³ according to SPT estimations. The apparent bedrock levels obtained are shown in Figure 5b, with the selected profiles for the model (explained in section 5.2.2). The gravimetric data were used only as a qualitative comparison, because direct inversion of residual is highly sensitive to density contrast between soil and apparent bedrock and reliable information for these values were not available. In this figure, black triangles indicate the location of the most damaged buildings and the red triangle correspond to the location of the available record (Viña Centro or VC station). It can be noted that the most damaged buildings are located in areas with depths in the range of 85 to 100m approximately. Only one building is located over thinner sediments thickness of about 50m.

Figure 5: (a) Shear wave profiles and adjusted curve from Equation 1 and (b) basement depth and selected profiles from extended profiles based on predominant frequencies, axis in meters (UTM 19S)
5. **FINITE ELEMENT MODEL**

To study the seismic amplification at the area, a Finite Element Model (FEM) was developed using the software PLAXIS 2D® (Plaxis, 2015). The soil was modeled using the Small Strain-Stiffness (HS-small) constitutive model to reproduce its non-linear behavior due to cyclic loading. Two kinds of models were created based on the 3D basin characterization: 1D soil columns and 2D sections. 2D sections will be used to estimate basin shape effects, while 1D columns will be used as reference to compare 2D results. 2D sections were located at records are available (B-B’ in Figure 5b), and across the most damaged area of 2010 earthquake (A-A’ and C-C’ in Figure 5b).

5.1 **Material Parameters**

To characterize main geotechnical properties of predominant materials, two types of laboratory test were conducted on remolded soil samples from a borehole near VC station. Samples were extracted at 17 m depth. They were classified as SW-SM according to USCS. To characterize the non-linear cyclic behavior shear modulus degradation and damping curves were obtained from combined resonant column (RC) and torsional shear tests (TS) performed at an initial confinement pressure of 100 kPa. The maximum shear modulus obtained was 84.13 MPa, i.e. a shear wave velocity of 217 m/s. This result differs in about -20% from those obtained with Equation 1, probably because of some fabric modification due to remolding process and because confinement is perfectly isotropic in the laboratory. However, it is assumed that this difference is acceptable for the purpose of describing the cyclical behavior of the soil. The obtained values are shown in Figure 6 as black crosses, as well as reference curves for sands from Seed and Idriss (1970) and low plasticity soils from Vucetic and Dobry (1991). The elastic threshold is a little larger than reference curves ($\gamma \leq 2 \times 10^{-5}$), but degradation is very similar to sand’s upper limit proposed by Seed and Idriss (1970).

![Figure 6: Degradation curves for material.](image)

Additionally, the sample was subjected to a set of isotropically consolidated drained (CID) monotonic triaxial tests to obtain their friction angle ($\phi'$) and cohesion ($c'$) at confinement pressures of 100 and 200 kPa, representative of the soil pressures in the field. The parameters for the Hardening Soil model with Small Strain-Stiffness (HS-small) available in Plaxis 2D® were calibrated based on these laboratory results. Figures 6 and 7 show calibrated paths against laboratory data. The model can appropriately reproduce the contractive to dilatative phase transformation of the material, as well as the shear strength.
Figure 7: Calibration and laboratory results for HS-small, at confinement pressures of 100 and 200 kPa.

The model includes a stiffness dependency on the stress level by a reference effective stress (100 kPa) and a power-law variation described by a parameter \( m \). A sensitivity analysis was performed to adjust the shear wave velocity profile of Equation 1 to this rule. Seven control points were selected along the soil column to perform a cyclic loading at very low amplitude to ensure elastic behavior. From the secant shear modulus obtained at each depth, the FE model’s shear wave velocity was calculated and compared to Equation 1 (Figure 8).

Figure 8: Calibrated and reference (Equation 1) shear wave velocity profile.

Apparent bedrock was assumed to behave linear elastic. To estimate the impedance contrast, an additional geophysical inversion was followed to adjust the amplitude of the HVSR ratio to the theoretical ellipticity curve. We selected for this inversion the profile located at Quillota/4Norte (in the southeast area of Figure 1), were damaged buildings were found. The results are shown in Figure 9. For a bedrock shear wave velocity of about 900 m/s the fit to the amplitude of the ellipticity curve, as well as the basement depth compared to the value estimated from the 1D elastic transfer function are very similar.
5.2 Available records

In the studied zone, there are two seismic stations separated by 4 km approximately. The USM station (located at University Santa Maria) is on geotechnical rock and the VC station (Viña Centro) is located on soil deposits of about 30 m according to our geophysical estimations. These stations recorded the earthquakes of 1985 (Mw 8.0) and 2010 (Mw 8.8), unfortunately the record of 2010 event at USM is not reliable. For that reason, we used only the 1985 records in our models. Figure 10a and 10b displays the two components (EW and NS) of the available accelerations and the corresponding 5\% damped elastic response spectra (PSa).

5.3 Developed Models

5.3.1 Soil Column

Firstly, a soil column was modeled at the VC station to reproduce the available records and validate the modeling approach. The geometry is shown in Figure 11a, where the column was 1-meter wide, with a maximum soil depth of 30 m and a total height of 45 m. Two major calculations phases were conducted: (i) Initial phase: initialization of stresses by K0 procedure; (ii) Second phase (seismic simulation): this phase includes the dynamic loading, and it has the same duration of the seismic input.
Boundary conditions at lateral limits were tied degrees of freedom, and at the bottom of the model, it was used the compliant base condition. Tied degrees of freedom connect the nodes on the same elevation and force them to have the same vertical and horizontal displacement. The compliant base condition is used to minimize wave reflections at the base and to introduce seismic loading.

The size of the mesh elements was selected according to Laera and Brinkgreve (2015) who recommend that the average size cannot be greater than one-eighth of the wavelength associated with the maximum frequency with a significant energy content of the seismic signal. For the analyzed case, the maximum frequency with significant energy content is around 20 Hz, while the minimum shear wave velocity is close to 210 m/s. Using this criterion, the average element size should not exceed 2.6 m. The soil column FEM model had 256 6-nodes triangular soil elements with an average element size of 0.42 m. The water table was located at 3m below the surface and soil is assumed to behave as perfectly undrained during the dynamic analysis.

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Table 1: General mesh information for selected profiles.

<table>
<thead>
<tr>
<th>Profile</th>
<th>Number of elements</th>
<th>Number of nodes</th>
<th>Average size (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-A’</td>
<td>22960</td>
<td>47280</td>
<td>2.58</td>
</tr>
<tr>
<td>B-B’</td>
<td>25060</td>
<td>51958</td>
<td>2.56</td>
</tr>
<tr>
<td>C-C’</td>
<td>18909</td>
<td>38993</td>
<td>2.30</td>
</tr>
</tbody>
</table>

3.2 2D sections

Three 2D profiles were generated, all of them with an orientation of 36°N as it is shown in Figure 5b. Two profiles were selected (A-A’ and C-C’ in Figure 5b) to analyze and estimate the response at the most damaged area (black triangles). Profile B-B’ was selected to reproduce the field measurement in VC station (red in Figure 5b). The same boundary condition and calculation phases of the 1D column were used, except at lateral limits where we used free-field condition. To simulate the propagation of waves to the far-field, two dashpots are added at each node, following the normal and tangential direction to the lateral boundaries. The number of elements, nodes, and the average size of the three profiles are shown in Table 1, and the mesh for C-C’ section is shown in Figure 11b.

Besides, three monitors were selected at the surface of each 2D section. They were selected to estimate and compare the amplification along the profile where the most damaged buildings are located (A2 and C2 monitors). For comparison purposes, the basin depth at each location was obtained from Figure 5b to create 1D soil columns to propagate the USM 1985 record from the base.
6. RESULTS

Figure 12 shows the 5% damped response spectra (PSa) at VC station from the developed models. The EW component shows a better agreement with the available record for all the models (Figure 12a). For this component, the results from soil column and the 2D section are very similar for frequencies below 7 Hz, with some moderate differences at peaks values. 2D results have a better shape agreement with the available record, but maximum values tend to be about 20% larger in the range of fundamental frequency of the profile. The second peak is closely reproduced by both 1D and 2D models in the range of 3 to 7 Hz, but spectral ordinates are overpredicted for higher frequencies, especially by the 1D model. The 2D model provides a very accurate prediction for frequencies large than 10 Hz. Both models reproduce very well the recorded PGA value. The agreement with the NS component is not satisfactory probably because this component does not show amplification at the soil fundamental frequency range (1.5 to 2.0 Hz), according to Figure 4b. Nevertheless, results from the 2D model at low-frequencies (<1.5Hz) and high-frequency (>10 Hz) are reasonably close to the record. Adjustment of 1D results for this component is very poor, even PGA value is largely overpredicted.

To quantify the degree of non-linearity developed in the soil, the maximum cyclic shear strain (γ) obtained from the column models are indicated on the G/G₀ curve at different depths (Figure 6). The maximum shear modulus degradation varies with depth, reaching a minimum value of about 30% of the initial stiffness at 20 m depth. At 5m depth, the maximum degradation is in the range of 20% to 40% depending on the motion. The results indicate that the role of the inelastic behavior is considerable for the studied case.

![Figure 12: 5% damped response spectra for VC station from registered data (in black) and FEM results. In blue the results from the soil column, an in red the results from 2D section at VC monitor.](image)

![Figure 13: 5% damped response spectra at selected control points for A-A’ section. On top, the section with the selected monitors is shown, all values in meters.](image)
Figure 14: 5% damped response spectra at selected control points for B-B’ section. On top, the section with the selected monitors is shown, all values in meters.

Figure 15: 5% damped response spectra at selected control points for C-C’ section. On top, the section with the selected monitors is shown, all values in meters.

The results from 2D profiles are shown in Figures 13 to 15. The computed response spectra for the columns and 2D sections shows significant differences in shape and peak values, but PGA values are very similar except for the C3 monitor. In this last case the steeper soil-to-bedrock transition seems to increase significantly the intensity of the shaking at the surface, nevertheless some boundary effects due to model truncation could also affect the results in this area. A2, B2 and C2 monitors located in the deepest part of the sections produced very similar results compared to soil column propagation because the soil deposits are “almost 1D” in these areas. According to Jünemann et al. (2015), the most damaged buildings in Viña del Mar have between 11 to 18 stories (N), which should correspond to fundamental frequencies (F₀) in the range of 1.1 Hz to 1.8 Hz using to the N/20 rule (for periods) suggested by the same authors. This range is represented in Figure 13, 14 and 15 by the shaded area in the spectrum. Monitors A2 and C2 are approximately located close to the most damaged buildings. The A2 monitor shows a moderate increase of spectral ordinates in the 2D model compared to soil columns, while at B2 spectral ordinates are almost identical. In the case of C2, the increase of spectral ordinates in the 2D case compared to 1D propagation is very modest. The comparison of spectral ordinates and shaded area describing the range of existing buildings indicate that the concentration of damage close to A2 monitor could be partially explained by 2D effects, nevertheless concentration of damage close to C2 is probably explained because predominant period of the site is very close to fundamental period of the building, i.e. a probable site-to-structure resonance situation.
7. CONCLUSIONS

Determination of shape of the predominant impedance contrast using geophysical methods matches with the information available in the zone and the collected data. The noninvasive geophysical methods used during the research have demonstrated a great value, for its economy and versatility in urban spaces. According to the comparison between columns and profiles, the damage of the buildings is more related to the basin depth (1D site amplification) than to basin shape effects. Indeed, the predominant frequency of the soil and the fundamental frequency of the most damaged buildings are in the same range. According to our preliminary results, only the northwestern concentration of damaged buildings could be affected by 2D site amplification effects. Nevertheless, additional studies are required to assess in detail the possible basin-shape effects.

8. REFERENCES


