NUMERICAL AND EXPERIMENTAL INVESTIGATION OF A CHILEAN BRIDGE-SOIL SYSTEM

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1. *Abstract*

An actual traditional Chilean highway bridge is analyzed in order to carry out a detailed nonlinear analysis of the structure subjected to strong ground motions. The selected non-skewed bridge was built in 2008; it has five spans of 30 *m* long and is supported by four multi-column bents and two seat-type abutments. The superstructure is supported over traditional Chilean elastomeric bearing pads and is located in a particular seismic area, where several bridges experienced significant damage during the mega Chilean earthquake in 2010 (Mw=8.8). In this context, a campaign of field tests was performed, allowing to estimate the predominant vibration frequency, the dispersion curves, and the shear wave velocity profiles of the soil using surface waves and the H/V spectral ratio methods. Overall, the soil displays a predominant vibration frequency of 0.5 *Hz* and has an average shear wave velocity of 280 *m/s* throughout the first 30 meters depth. Moreover, gravimetric studies indicate that the bedrock is located at 220-280 m depth. These local soil conditions make that the structural seismic demands can be significantly amplified. Based on the geophysical campaign, the soil profile beneath the bridge was also determined. In a parallel effort, the dynamic properties of the bridge were identified from acceleration records that were measured under regular traffic excitations. Frequency analysis and the stochastic subspace identification (SSI) method were employed to estimate the modal frequencies and mode shapes of the structure associated with low amplitude excitations. Overall, the longitudinal deck frequency was 1.98 *Hz*, the transversal deck frequency was 2.4 *Hz*, the vertical deck frequencies (for each span) were 3.85-3.99 *Hz*, and the warping modes of the deck were 4.50-4.64 *Hz*. The four multi-column bents were also measured, identifying natural frequencies of 3.8-4.2 *Hz* along the longitudinal direction. A 3D bridge-foundation-ground model was made in the software OpenSees. This simulation shows that the local amplification related to a reduced soil domain of 32 m beneath the bridge can be secondary in comparison to the amplification that occurs through the single soil column when the bedrock is deep. Additionally, it was observed that the radiation damping is 1-3% for the studied traditional Chilean bridge. The soil experienced plastic deformation due to the Soil-Structure-Interaction, generating hysteretic damping and also residual ground displacements of the bridge support (implying residual structural stresses). Based on these analyses, it can be observed that the Chilean Bridge Design Code (BDM) can be ineffective to estimate the seismic demand in base of traditional geotechnical prospections when the bedrock is deep, observing that geophysical techniques should be also used to classify the site amplification. In case that the basin is deep, seismic hazard study are required to evaluate the seismic demand for design of Chilean bridges.

Keywords: nonlinear analysis of bridges; soil and structure identification; detailed bridge component models.

2. Introduction

Chile is recognized to be a highly seismic prone area due to the tectonic activity related to the subduction of the Nazca plate beneath the South American plate, a mechanism that belongs to the ring of fire of the Pacific Ocean basin. In this context, Chile has been struck by 91 earthquakes Mw≥7.0 since the year

1900 (i.e., 7.6 earthquakes Mw≥7.0 average per decade). Moreover, one-third of mega-earthquakes Mw≥8.5 that have occurred worldwide have affected the Chilean territory since the year 1500 (13 mega-earthquakes). In fact, the Valdivia Earthquake in 1960 (Mw=9.5) is the strongest known seismic event recorded in modern times [1]. Just during the last decade, Chile has been struck by ten seismic events Mw≥7.0, where the most powerful have been the Maule mega-earthquake in 2010 (Mw=8.8), the Iquique earthquake in 2014 (Mw=8.2) and the Illapel earthquake in 2015 (Mw=8.4) [2].

Before 1985, the seismic design of Chilean bridges was based on international standard codes such as DIN [3] and AASHTO standard codes [4]. After that, the bridge design has been regulated by the Chilean Highway Bridge Design Manual (BDM)[5], which is mainly based on the AASHTO LRFD Bridge Design Specifications [6]. In this context, the BDM takes into account a seismic demand in agreement with the Chilean seismicity, and it considers seismic reduction factors that have been calibrated from the satisfactory performance of bridges during the Algarrobo earthquake in 1985 (Mw=7.8) [5].

Chilean bridges have evidenced a relatively good performance during the last earthquakes taking into account that only 3% of Chilean bridges were significantly damaged during the mega-earthquake in 2010 [7] (and only 7 bridges suffered minor damage in the 2015 Illapel earthquake). That is, a significant number of bridges have required to be total or partially repaired (i.e., 211 bridges of 7000 bridges in the affected area) during the Maule mega-earthquake in 2010 [7], implying that the connectivity was reduced for several days or weeks (inclusive months or years) in the most affected areas. Moreover, 24 bridges collapsed during the Maule mega-earthquake in 2010 (Mw=8.8) [7], indicating an inadequate bridge practice and/or deficiencies in the Chilean bridge design.

The superstructure of a traditional Chilean highway bridge is commonly made of a continuous deck slab that is supported on precast girders (which are commonly dilated over each bent). The superstructure rests on bents and abutments over bearing pads, i.e., without providing an integral connection between the deck and bents, implying that the superstructure is partially seismic isolated from the infrastructure (reducing seismic demands) [8], but it becomes prone to experience significant relative displacements, making that decks can experience pounding with shear keys (transversal and torsion movements) and abutments (longitudinal and torsion movements)[8]. According to the Chilean practice, vertical seismic anchor bars are commonly installed on each support to prevent uplifting of the deck, i.e., restraining the relative vertical movements between the deck slab and the infrastructure, but the practical experience observed in past earthquakes has shown that these seismic anchor bars have also played a significant role in the longitudinal and transversal response of the deck [9]. Based on these Chilean bridge characteristics is that the infrastructure does not experience usually damage during an earthquake [8] (excepting when lateral spreading or liquefaction occurs due to the soil-structure interaction [10]). In contrast, there are common damage patters that have been observed during past earthquakes, such as [8,10–12]: (a) excessive lateral, longitudinal and torsional displacement of the deck (involving residual displacements of bearing pads and permanent horizontal deformation of the vertical anchor steel bars), (b) collapse of segments of the deck due to the loss of vertical support on abutments, because of insufficient seat support lengths and excessive displacements, (d) damage of shear keys and precast girder due to pounding, (e) damage of abutments and backfill due to longitudinal pounding with the deck. In this context, these damage patterns have been mainly concentrated when bridges have been built on areas where the soil profile has dynamic properties that amplify the seismic demands on periods that are close to the longitudinal, transversal or torsional modes of the deck [12] (in particular when the soil period ranges between 0.5 to 2.0 s). Similarly, curved and skewed bridges have shown to be more susceptible than non-skewed bridges because of their natural susceptibility to experience an amplified torsional response [11] (because of the unsymmetrical response of abutments, unsymmetrical mass distribution and unsymmetrical pounding between deck and abutments that amplify the deck responses).

From 1996, the Chilean BDM has made mandatory that transverse diaphragms should be incorporated to connect girders at the end of each span; however, these previous versions of the BDM stated that diaphragms could be eliminated if engineers can demonstrate that they were not required for intermediate or low seismic intensity zones, i.e., seismic zones 1 and 2 according to the BDM. Similarly, the old versions of the BDM stated that ductile external shear keys should be provided in order to avoid that the deck is unseated from bents and abutments; however, specific guidelines about how to design shear keys were not explicitly given until the 2008 version of the BDM. On the other hand, a boom of highway infrastructure construction arose in Chile, when the concessions system was introduced during the mid-90s. The concessionary companies (in general foreign companies) started to eliminate the transverse diaphragms and have used weak seismic lateral stoppers (but keeping the vertical seismic anchor bars) in order to reduce construction costs in seismic zones 1 and 2 [13], taking into account that the BDM was ambiguous, and also to the fact that authorities have consequently accepted these kinds of reductions for seismic areas of intermediate and/or low seismic intensities. Nevertheless, the mega-earthquake in 2010 evidenced that several bridges experienced significant displacement and 24 bridges collapsed due to insufficient lateral restrictions [7], insufficient seat support lengths and/or the absence of transverse diaphragms (implying significant damage of girder) in particular for bridges that were built in the seismic zone 2 (where restrictions were relieved) and were placed over soils that amplify the deck responses. For these reasons, after the mega earthquake in 2010, the BDM [5] was significantly modified, including new requirements that are concisely described as follows: (1) all bridges must have transverse diaphragms independent of the seismic zone, (2) bents and abutments should include external and internal shear keys for all seismic zones (specific requirements were included to require specific strengths and geometries), (3) a site coefficient that depends on the type of the soil was introduced to takes into account the site amplification for the design loads of all structural components, (4) the seat support lengths were modified taking special care for skewed and/or curved bridges, (5) more strict specifications were included to evaluate the soil characteristics and evaluate its capacity to avoid liquefaction and lateral spreading strengths, (6) bearing pads should be anchored to girders and the infrastructure (abutments or bents). In this context, it should be observed that before the BDM 2017, girders were basically resting over bearing pads, implying that girder usually slides over the bearing pads when the friction strength was reached during strong earthquakes, allowing significant displacement of the superstructure [14].

The current version of the Chilean Highway Bridge Design Manual (BDM) [5] suggests three seismic design methods that are based on equivalent linear elastic analyses (considering a reduction seismic factor according to a ductility that should be provided for each structural component), such as (1) a static analysis, (2) a modified static analysis, and (3) a linear spectrum. On the other hand, the BDM indicates that the Chilean authorities have the faculty to request special analyses for relevant bridges such as sophisticated linear or nonlinear time-history analysis. In this context, these special analyses should be computed from a seismic demand estimated from a site-specific seismic hazard analysis. However, the Chilean BDM does not provide any specific guidelines to perform these kinds of sophisticated analyses or how to perform a proper seismic hazard study for this purpose. As a result, bridge experts use their own criteria, existing in some cases significant discrepancies between them. In agreement with the international trend of evaluating performance base design of bridges subjected to seismic excitations in a more realistic manner, and also with the objective to evaluate the deck displacements and poundings (with abutments and shear keys), the current research has the objective to perform a detailed analysis of a traditional Chilean bridge in order that this analysis can help to generate a future document that provides guidelines to carry out non-linear time-history analyses for traditional Chilean bridges in a similar manner that other international guidelines do, for example, the guidelines given by the Pacific Earthquake Engineering Research Center [15,16] or Illinois Center for Transportation [17].

3. Description of the Traditional Chilean Bridge Study Case

A traditional Chilean bridge was selected as a study case considering several features that make that this case could be an interesting case for extensive analysis. First of all, the selected bridge has more of the features that the current version of the BDM [5] suggests. For example, the bridge has transverse diaphragms and strong external shear keys on bents and abutments (different to a significant number of bridges that were built by the concessionary companies during mid-90 to 2010); however, it does not have internal shear keys connecting the transverse diaphragms such as the current version of the BDM suggests [5], but it can be considered to be the most closer typology to the current recommendations, between bridges that were built before 2010. Second, this bridge was subjected to the Maule mega-earthquake in 2010 (Mw=8.8) and is located in the seismic area 2 of the BDM [5] (i.e., related to an intermediate seismic demand and where several bridges showed an inadequate performance due to relieve of some seismic design recommendation by concessionary companies). Overall, the bridge experienced low damage during the 2010 earthquake (i.e., only external shear keys of intermediate bents have suffered limited damage). Moreover, near to the bridge, several overpasses and bridges experienced significant damage or collapsed, indicating that the area should be experienced a significant local site effect. In this context, Fig. 1 shows that in a radius of 3 km around the Aguila Norte bridge (the selected one), two skewed bridges collapsed (the Romero bridge and the railway Overcrossing Hospital) due to insufficient seat support lengths and poundings, and other three bridges (the Chada overpass, the Azufraderos overpass, and the Champa overpass) experienced significant damage related to pounding between girders and lateral stoppers, significant transverse displacements, and failure of backfill soil (and impacts between the deck and abutments).

Fig. 1 – Bridges that have collapsed near to the Aguila Norte Bridge in the 2010 earthquake

The selected bridge was the Aguila Norte bridge (Fig. 2 and Fig. 3). This is a straight bridge of 148 m length consisting of 5 equal spans of 29.6 m, located in the route G-550 joining the Aguila Sur and Hospital streets and crossing the Angostura River in the Maipo province of the metropolitan region (just 47 km south of Santiago City). The deck is made of a continuous RC slab 20 cm thickness of 10.7 m width (allowing 2 ways for traffic), an asphaltic layer of 5 cm is placed over the deck slab. The deck also has one pedestrian passageway at both edges of the deck. The deck slab is anchored and supported over 4 precast prestressed concrete girders of 29.4 m length and 1.4 m height, which are spaced at 2.6 m from center to center (Fig. 2). Girders are supported over two abutments and four bents by bearing pads 60^{o}Shore A of 66 mm thickness on abutments and 46 mm thickness on bents (with intermediate metallic plates, Fig. 3), observing that girders are resting over bearing pads allowing sliding by friction. A longitudinal gap of 20 cm was provided between girders, but the deck continuity is partially provided by the continuous deck slab. Similarly, a 10 cm longitudinal gap is used between the deck and the abutment backwall that is covered by an expansion joint TRANSFLEX BRIDGE 400 USL. An RC transverse diaphragm of 25 cm thickness is placed at both ends of each span to join the four girders (over the four bents and both abutments). Twelve seismic vertical anchor bars of 22 mm diameter of ductile steel (A44-28H) are used to join the deck slab to the bent cap (over bents) and six vertical anchor bars are used to join the deck to abutments (these anchor bars are installed inside of a PVC pipe of 3 inches diameter that avoids initial interaction between the anchor bars and the transverse diaphragms).

The infrastructure is made of four bents and two abutments. Bents have three RC piers of circular section 1.2 m diameter, which are joined at the top by an RC rectangular bent cap of 1.5 m height and 1.7 m width. The bent caps have external RC shear keys of 30 cm height and 70 cm width to avoid the excessive transverse displacement of the deck, (a transverse gap of 5 cm is used between external shear keys and girders). The four bents have different heights (according to the river topography) ranging between 3.3 m to 4.9 m. A square pile cap of 1.7 m is employed to joint piers at its lower level, and it is also employed to connect piles. Three RC piles of 1.2 m diameter are embedded 16.75 m depth into the soil under each bent (each pile can be understood as a continuity of the piers). Abutments retain compacted backfill soil. An RC backwall 30 cm thickness, 10.70 m width, and 4.83 m height is used for both abutments. The backwall is embedded in a foundation mat of 1.5 m thickness; consequently, the foundation mat is supported over 2 rows of RC piles 1.2 m diameters (a total of 5 piles are used under each abutment). Abutments have two wingwalls 3.1 m length and 35 cm thickness to retain the backfill soil.

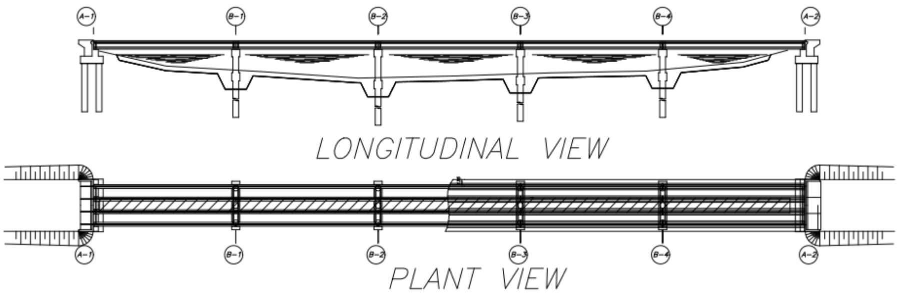


Fig. 2 –Aguila Norte bridge, general longitudinal and plant drawings

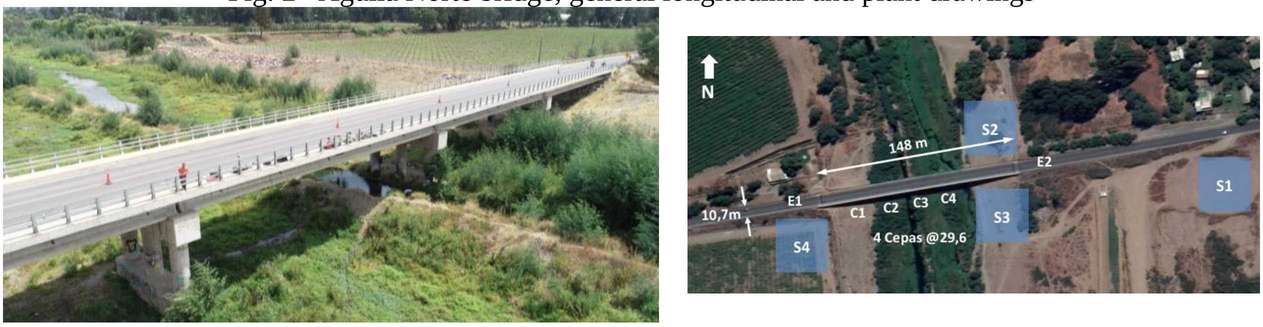


Fig. 3 – General Aguila Norte bridge view and sites of geophysical studies

Fig. 4 –View of bent cap and connection between bets and the superstructure

The bridge was built in 2008 and designed considering the AASHTO 2002 specifications, considering a design truck HS20-44 +20%. In term of the seismic demand, the bridge was initially classified to be in the seismic zone 2 (related to a peak effective acceleration A_0=0.3g, i.e., an intermediate seismic demand zone) and over soil Type II (according to the Chilean BDM [5]), observing that the soil was classified to be a sandy gravel from borehole samples and results from standard penetration tests (SPT). In this context, a Type II soil should display a shear velocity of 400 m/s (or higher) through the first 10 meters beneath the soil surface according to the BDM [5], as a consequence, the BDM spectrum that was employed in the original design has a plateau for natural periods lower than 0.3 s. In contrast, the average shear velocity along the first 30 m was 277 m/s and the natural period of the deposit was 2.0 s according to geophysical measurements described in section 5, allowing to conclude that the soil deposit cannot be appropriately classified as one of the four types of soils that are suggested in the Chilean BDM [5].

4. Dynamic properties identification from OMA

Three campaigns of ambient vibration (AV) tests were carried out in order to identify the dynamics properties of the Aguila Norte bridge from Operational Modal Analysis (OMA). The equipment employed for the measurements includes twelve Episensor ES-U2 force balance accelerometers and a DAQBOOK/2005 acquisition system, a signal filter, and a portable computer. The Chilean ministry of public works (MOP) facilitates a highway bridge inspector truck to install the accelerometers temporally during the third AV campaign (Fig. 7). The acquisition system consisted of a portable 16-bit analog-digital converter and was configured with x10 gain and an analog low pass filter of 100 Hz; the DAQBOOK and the portable computer were set to measure at a sampling rate of 500 Hz. The identification of the modal properties was made by using the Subspace System Identification (SSI) technique [18] and verified from traditional frequency analyses (Power Spectral Density, Transfer Functions, Coherence) [19], further details about its application for the study case can be found in the reference [20].

The first AV campaign was focalized in estimating the general response of the superstructure by using special arranges of sensors to capture the longitudinal, transversal, and vertical movements of the deck. In the longitudinal direction, it was found that a rigid body longitudinal mode was identified related to a natural frequency of 1.95-2.47 Hz (which experiences a high variation during the time due to the change of the traffic loads) (Fig. 5); moreover, it was identified several dominant frequencies in the range of 3.8-4.2 Hz that was initially associated with the local response of bents (confirmed in the third AV campaign). In the transversal direction, it was found three transverse modes that were related to the global deck response (Fig. 6); in a similar way to the longitudinal response, it was observed several predominant frequencies between 4.1-4.5 Hz, which were consequently related to torsional mode shape of the deck. In the vertical direction, it was simultaneously measured the first three spans of the bridge using a spatial arrangement of sensors distributed longitudinally along the bridge. Similarly, the first span was measured at both sides of the bridge to detect warping responses. Overall, it was found that all spans were dominated by a vertical mode related to a 3.85-3.99 Hz; similarly, it was identified a warping mode associated with the first span related to a frequency of 4.64 Hz; however, it was found that channels associated with different spans show a very low coherence, despite that vertical movements, were described for similar frequencies in each span. Therefore, it was concluded that every span could be hypothetically related to an independent mode shape, based on the physical fact that precast girders were dilated over each bent (observing that spans are connected by the deck slab only).

The second AV campaign was focused on the analysis of the vertical response of the bridge deck. In this context, each span was sequentially measured by using an arrangement of six sensors (three sensors at each side of each span of the bridge), and at the same time, one sensor was kept at the center of each span and one sensor at each side of the central span (Fig. 8). The analysis shows that each span has an independent vertical and warping mode shape (summarized in Table 1), i.e., sensors related to different spans show an almost null coherence; in contrast, sensors that were placed on the same span show a high coherence indicating that each span was related to an independent movement, whose mode shapes are similar to a classical sinusoidal mode shape of a simply supported beam. On the other hand, it was also found two warping modes of 4.34 and 4.99 Hz that display a high coherence on all spans (Fig. 9) indicating that these modes were effectively related to a global warping response that could be explained in some degree due to the connection provided by the deck slab.

Table 1 – Vertical and Warping frequencies related to each span

Frequency	Span 1 [Hz]	Span 2 [Hz]	Span 3 [Hz]	Span 4 [Hz]	Span 5 [Hz]
Vertical	3.93	3.99	3.86	3.9	3.85
“Warping”	4.64	4.60	4.50	4.55	4.51

The third AV campaign was focused on the measurement of the local bent dynamic properties. In this context, three sensors were placed over the top of each bent (two longitudinal and one transverse, Fig. 7). Because of time limits, it was only possible to measure the first three bents and the east bent was unfortunately not measured. The identified local longitudinal frequencies associated with the measured bents were 4.14 Hz, 3.95 Hz and 3.99 Hz for bents 1, 2 and 3 respectively. It is worth noting that a torsional response of high coherence was observed for all bents related to a natural deck frequency of 4.55 Hz. On the other hand, along the transverse direction was not identified a local bent frequency, i.e., the bent transverse frequencies were described by the global deck transverse modes (and the torsional mode).

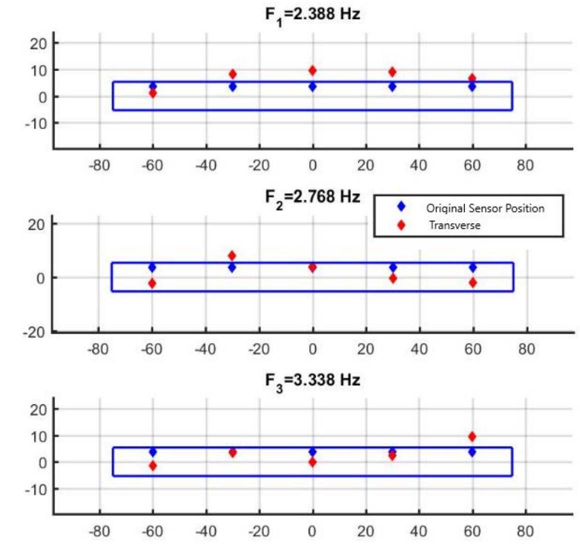


Fig. 6 – Transverse Mode Shapes

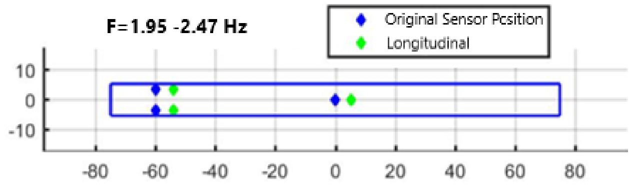


Fig. 5 – Longitudinal Mode Shape

Fig. 7 – Sensor arrange during the third campaign

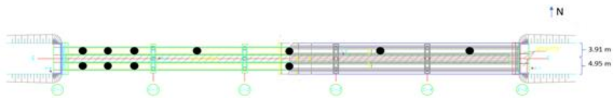
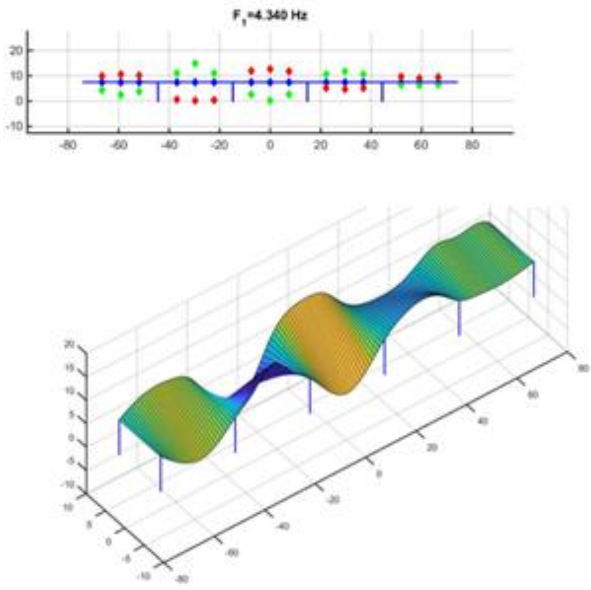


Fig. 8 – Sensor arrange during the second campaign



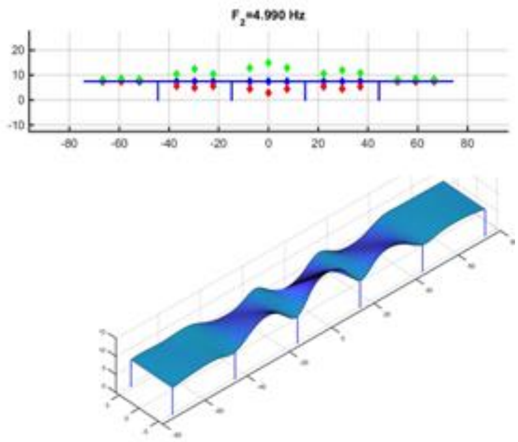


Fig. 9 – Global warping modes (in red north deck edge, in green south deck edge)

5. Soil profile characterization and bridge-foundation-ground model

The dominant frequency of the site was determined using HVRS Nakamura technique [21], and was equal to 0.47-0.53 Hz (Fig. 10 (a)) (related to a peak H/V ratio of 3.4-4.1). The geophysical measurements were performed in four sites near to the bridge (Fig. 3), showing consistent results. The shear wave velocity profile (Table 2) was determined by the combined inversion of the H/V curves and the dispersion curves, which were consequently obtained from MASW, cross-correlation of ambient noise and SPAC methods (Fig. 10). In this context, the average shear velocity related to the first 30 m depth was V_{S30}=277 m/s. The result of the inversion indicates a soil deposit depth of 240 m over the basin, which is coincident with the study of Gonzalez [22], who indicates that the bedrock is approximate to 210-250 m depth from gravity measurements. Other geotechnical parameters related to the soil were taken from Gonzalez [23].

Table 2 – shear wave velocity profile

Layer	Depth (m)	Thickness (m)	Specific weight (kN/m^3)	Shear wave velocity (m/s)
1	4	4	19	170
2	14	10	19	240
3	55	41	20	360
4	87	32	20	420
5	240	153	21	560

From the estimated soil profile, a 3D finite element (FE) was built in OpenSees [24,25] in order to study the nonlinear seismic response of the bridge-foundation-ground system (Fig. 11). The model was built following the procedure described by Elgamal [26] for the analysis of the Humboldt Bay Middle Channel bridge. In this context, a large enough soil domain was considered around the bridge, i.e., a soil domain of 253.4 m length, 70.8 m with and 38 m depth (i.e., 32 m down of the pile cap, which is double of the pile lengths) with a horizontal mesh of 5m length and a vertical mesh of 1.6 m around piles and 2.5m under them (implying 20892 brick elements).

Some features of the Aguila Norte bridge were ignored to simplify the bridge model, such as the nonlinear response of shear keys and the vertical seismic anchor bars. I-girders made of H40 concrete were modeled considering a linear elastic-isotropic material with a Young modulus of 28000 MPa, a Poisson ratio of 0.20 and a density of 2.5 ton/m^3. The bridge deck slab was modeled using elastic shell elements and linear-elastic material related to concrete H30 (Young modulus of 23500 MPa). Similarly, the transverse diaphragms were modeled as linear elastic beams of H30 concrete. The bearing pads were modeled using Zerolength elements whose elastic properties were computed assuming a shear modulus of 13 kgf/cm^2 (in agreement with the BDM [5]) and Steel01 material to capture its perfect-elastic plastic response. In the vertical direction, linear elastic behavior was assumed. Piers and piles were modeled using fiber sections with force-based beam-column elements. This non-linear section allows modeling the formation of plastic hinges. Concrete01 material was used to model the core and the covering concrete. The reinforcement bars were modeled using Steel01 material. The bent cap beams were modeled using linear elastic material related to concrete H30. Rigid links without mass were used to connect I-girders and the bridge deck.

For simplicity, the pile group under abutments was replaced by an equivalent linear elastic-isotropic block (nD ElasticIsotropic) that considers a Young weighted modulus between piles and soil (E=2700 MPa, ν=0,25, and ρ=2,2 ton/m^3) (Fig. 11 (b)). The abutment was modeled using standard bricks with linear elastic-isotropic material (concrete H30). The geometry of the approach ramp was simplified, considering a slope of 40 degrees.

The soil layers were modeled with eight-nodes brick elements, considering the Pressure Dependent Multi Yield material (PDMY) [27]. Therefore, a hysteretic hyperbolic shear response is considered to evaluate the soil responses according to parameters that were deduced from elastic properties [23] and others that were taken from default values of the PDMY material model. An equal DOF constraint (horizontal and vertical) was used for perimeter nodes that are the same depth, as a result, this boundary condition reproduces a conventional shear beam, assuming a semi-infinite half-space [26]. At the bottom of the model, Lysmer-Kuhlemeyer dashpots [28] were considered in the longitudinal and transverse directions, and vertical movements were restrained.

The longitudinal and transverse input base forces were included according to the assumptions suggested by McGann and Arduino [29] (an Opensees example of the site response analysis of a layered single soil column), i.e., the input forces were computed as the product between the Lysmer-Kuhlemeyer dashpots and the velocity that the column soil layer has at 32 m depth (level of the model base). The seismic record of Santa Lucia station (placed on a shallow rock in Santiago City, i.e, 47 km from the Aguila Norte bridge), which was measured during the Maule 2010 mega-earthquake, was considered as an outcrop signal. The within rock record (i.e., 240 m under the surface) was obtained using the DEEPSOIL software [30]. Then, the seismic wave propagation was computed considering a single shear soil column in OpenSees [29] (using the freeFieldDepend.tcl tool) according to the soil profile deposit (Table 2) and the PDMY material model assumed for each soil layer. Finally, the seismic motion at 32 m depth was consequently employed to compute the input force that was used for the 3D bridge-foundation-ground model. Additionally, it was verified that the shallow response of the soil column (considering the soil profile) that was obtained from OpenSees and the PDMY material was reasonably similar to results that were obtained with DEEPSOIL (with a constant damping ratio Dmin=2%) and a single soil column model simulated with OpenSees considering isotropic linear elastic material (nDMaterial ElasticIsotropic) and a Rayleigh damping ratio of 2% for the first and second mode of the soil deposit. The input in the 3D model considered the strong motion defined between the 5 and the 95% of the Arias Intensity to reduce the time length and improve computational efficiency, which corresponds to a time length of 46 s.

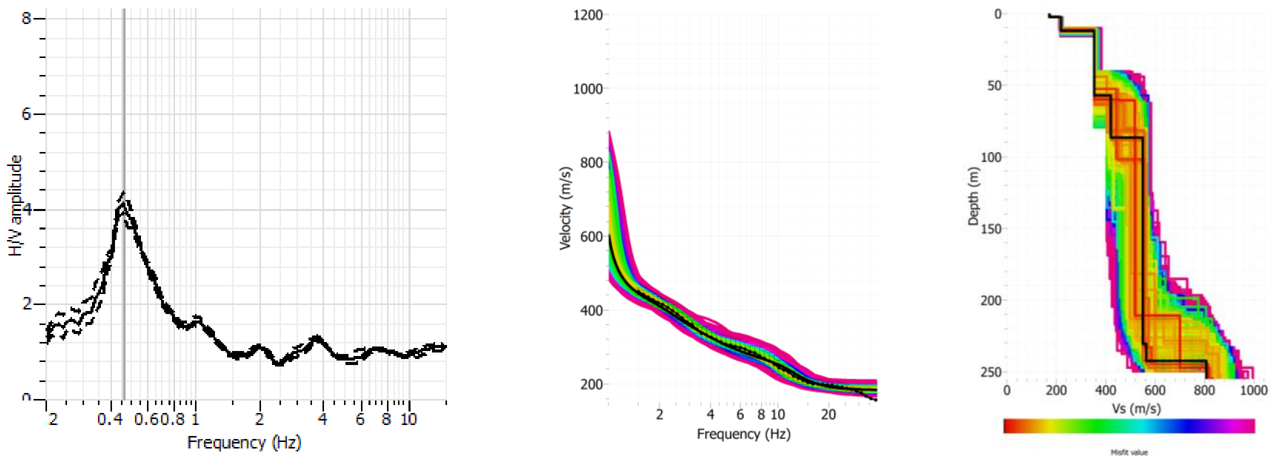


Fig. 10 –(a) HVSR, (b) dispersion curve adjustment, (c) shear wave velocity profile inversion

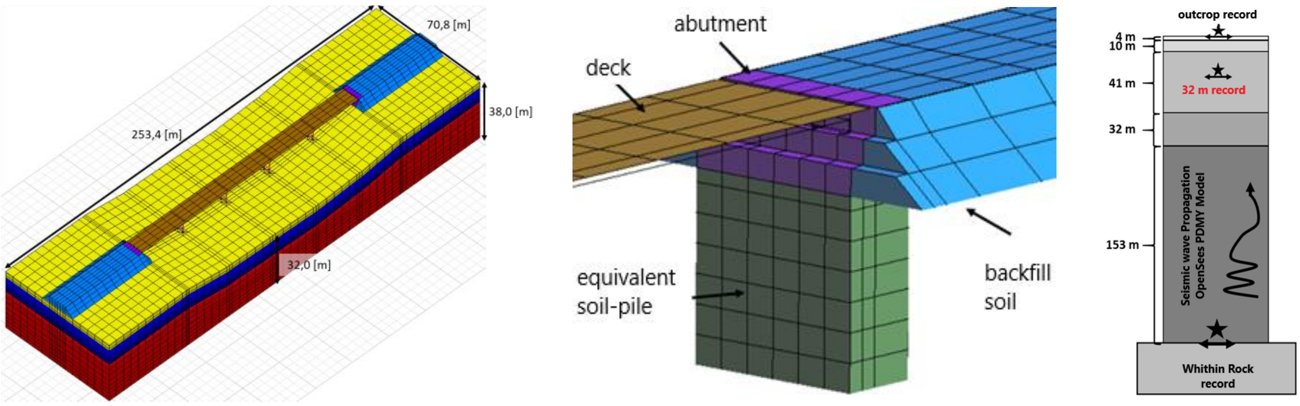


Fig. 11 – (a) 3D Model View with SSI (GiD pre and post processor), (b) soil-pile equivalent model, (c) soil wave propagation model for input estimation

In order to solve the numerical simulation, three solution steps were employed. First, the weight load of the soil was initially applied, considering linear elastic properties for the soil layers. Second, the weight of the bridge was subsequently applied. Then, the soil material was updated to be the PDMY material model using the updateMaterialStage command. Finally, the dynamic seismic analysis was solved using a sparse system of equations (Umfpack), RCM DOF numbering, constraint handler transformation method, algorithm Krylov-Newton and numerical Newmark integration (β=0,25 y γ=0,5). The time step was the same as the input motion (0,01s). A Rayleigh damping of 5% for the first and second modes of the bridge was applied to the model. The seismic load was applied by using a plain pattern load in both longitudinal and transverse directions.

The modal analysis of the 3D bridge-foundation-ground model (OpenSees SSI) shows that the computed natural periods related to the longitudinal, transverse and vertical mode shapes (Table 3), were 25-50 % larger than the identified from OMA (Section 4). These differences are attributed to the fact that bearing pads display longer stiffness when they are subjected to low deformations that the assumed in the 3D model (i.e., a shear modulus near to 18-20 kgf/cm^2>13 kgf/cm^2) and also because of extra masses that can be accumulated over the deck could have been underestimated. Additionally, Table 3 shows the natural periods result of the bridge model when the Soil-Structure-Interaction (SSI) is simplified by considering equivalent elastic springs along piles according to the elastic properties inferred from the shear velocity of soil layers (OpenSees Simplified SSI), and when piers are clamped at the soil surface level (OpenSees No SSI). Results indicate that the SSI increases the natural periods between 3-8% of the soil-bridge system in comparison to the model without SSI (due to the flexibility of soil), indicating that the soil generates an almost clamped condition for low amplitudes vibrations when linear-elastic properties are used.

Table 3 – Natural periods comparison

Mode	**Description**	**OpenSees SSI**	**OpenSees Simplified SSI**	**OpenSees No SSI**	**Identified OMA**
1	Longitudinal	0.68	0.79	0.63	0.53
2	Transverse	0.61	0.64	0.59	0.42
3	Vertical	0.38	0.38	0.38	0.26

The 3D bridge-foundation-ground model shows that the seismic response of different nodes at the soil surface was not significantly modified near or far from the bridge. The response obtained from the 3D model was similar to the response that was obtained considering an equivalent shear column in OpenSees with PDMY material model, validating the reduction of domain that was employed. Moreover, the soil input movement 32 m depth was only 10-20 % amplified (in terms of its RMS value) in comparison to the shallow soil response. In this context, one can conclude that soil domain was small enough in comparison to the soil deposit depth (32m of 240 m), verifying that the amplification that occurs through the 32 m shallow layer is secondary in comparison to the seismic wave amplification that occurs through the entire soil deposit.

The numerical solution of the 3D model evidenced that pier columns does not experience significant seismic demands because the superstructure is partially isolated over the infrastructure according to the Chilean bridge typology. On the other hand, the simulation has evidenced that soil can experienced plastic deformation, generating residual displacement of the bridge supports. In particular, the shear stress vs shear strain plots has shown that permanent deformations occurred toward the middle of the river bed. This deformation results in permanent loads of the structure (an effect that is not captured from simplified SSI models). In this context, it was also observed that the natural frequencies of the system changes during the time due to the non-linear soil response and the Soil-Structure-Interaction, e.g., the 3D model evidenced that the natural frequency related to the first longitudinal mode was modified from 0.68 Hz to 0.78 Hz during the strong motion phase. In parallel, the 3D model was simulated considering only Lysmer-Kuhlemeyer [28] dashpots around its boundaries, and an initial displacement was applied to the deck (longitudinal and transverse); the bridge and soil material models were considered linear-elastic and not damping was considered for soil and the bridge. As a result, it was observed from the damped response that the damping ratio related to the radiation damping (provided by the boundaries conditions) was 1.5-2.5%. Finally, it was also observed that the transverse displacement that was obtained from the 3D numerical model evidenced that the deck has exceeded the 5 cm, indicating that the deck should be pounded external shear keys over bents (a fact that was effectively observed in the mega-earthquake in 2010).

6. Conclusions

The dynamic properties of a traditional Chilean bridge were identified from Operational Modal Analysis. The structural modes of Chilean bridges are basically described by the deck vibrating over bearing pads (and vertical seismic anchor bolts), generating a partial seismic isolated superstructure. This structural configuration reduces the seismic demand of the infrastructure; however, the superstructure displacements can be significantly amplified making that phenomena such as poundings (between deck and shear keys, or deck and abutments) or deck unseating become more likely than the observed for other bridge typologies used in other seismic areas around the world; in special when the soil profile amplifies seismic movements with frequencies close to the longitudinal or transverse deck modes. The soil profile was also estimated from geophysical analyses. Results indicate that the bridge was built in a site that amplifies seismic movements of low frequencies (0.5 Hz), whose dynamic properties cannot be captured from simple geotechnical prospections related to the study of the superficial soil deposit only (e.g., the first 30 meters), i.e., the site response was mainly described by the basin response of 240 m depth. In this context, several bridges have experienced an unsatisfactory seismic performance during the mega-earthquake in 2010 (Mw=8.8) near to the Aguila Norte Bridge. The main reason of these inadequate seismic performance was that certain design criteria were ignored for intermediate seismic zones in order to reduce construction costs. On the other hand, the geophysical study has also evidenced that geotechnical prospections, which were used to classify the soil type according to the Chilean BDM recommendations (before 2010), were inadequate to estimate the seismic demand for soil profiles that present a deep bedrock because the structural demands for low frequencies could be not properly captured. A 3D bridge-foundation-ground model shows that the shallow soil amplification can be secondary for deep soil basin, because of the main amplification is generated through the entire soil deposit (for long natural periods). Additionally, it was also observed that the plastic response of the soil (due to SSI) can induce residual ground displacements generating permanent stresses into the structure.

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