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GEOTECHNICAL OBSERVATIONS FROM THE 2014 IQUIQUE EARTHQUAKE

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Abstract

The 2014 Iquique earthquake struck off the coast of Chile on April 1st with a moment magnitude of 8.2, as a result of thrust slip at shallow depths. The earthquake's epicenter was located 34 km northwest of Iquique within a well-known seismic gap, and it generated a tsunami with a maximum run up of 4.4 m which affected local fishermen's facilities in the cities of Arica and Iquique. While the overall response of the structural and geotechnical systems was satisfactory, there was evidence of liquefaction and lateral spreading on the quay walls of Iquique's port and on the stream channel flowing under the Tana bridge. Overall, the seismic performance of the inspected bridges was satisfactory. Likewise, natural slope failures were observed along the coastal bluffs and interior creeks, and a large slope failure blocked the only route between the cities of Iquique and Alto Hospicio, causing mayor traffic congestions in the following days. Researchers from GEER-NSF and CIGIDEN-FONDAP deployed in the area to document the geotechnical aspects of this extreme natural event. This article summarizes the main findings and observations conducted by the reconnaissance teams with emphasis on the effects and consequences of the ground motion on the built infrastructure such as ports, bridges, retaining structures and cut slopes.

Keywords: earthquake reconnaissance, mega thrust earthquake, liquefaction, landslides

1. Introduction

A reconnaissance team from the Geotechnical Extreme Event Reconnaissance (GEER) Association and the National Research Center for Integrated Natural Disaster Management (CIGIDEN) and local universities visited the area to document the geotechnical effects of the earthquake and to perform field testing at key locations. This article summarizes the most relevant aspects of the field observations; for more details, the reader is referred to the GEER Association report, [1]. While no major failures were reported, this earthquake sequence is a great opportunity to learn about the field performance of different engineered geotechnical systems.

2. Background

An offshore mega thrust subduction earthquake of moment magnitude (M_w) 8.2 occurred on April 1st, 2014 at 20:46:45 local time, with hypocentral location 19.572°S, 70.908°W and depth 38.9 km, as reported by the Chilean National Seismological Center (CSN). The United States Geological Survey (USGS) located the hypocenter at 19.642°S 70.817°W and depth of 20.1 km. The ground shaking intensity issued by the USGS is shown in Fig.1. The earthquake triggered a tsunami that struck the coast with maximum run-up depths of 3.1 m in Iquique and 4.4 m in Puerto Patache. The event ruptured a known seismic gap at the interface between the Nazca and South American plates; the fault rupture dimensions were 240 km wide and 270 km long, and the maximum slip was approximately 8 m according to a finite fault solution [2]. The main event was preceded by an earthquake sequence in the same rupture plane with maximum magnitude M_w 6.7 [3] on March 16th. Several aftershocks, including a M_w 7.5 at 20:49:25 and a M_w 7.0 at 20:57:59 on the same night, and a M_w 7.6 on April 3rd 2014 at 23:43:15, further increased the extent of the damage.

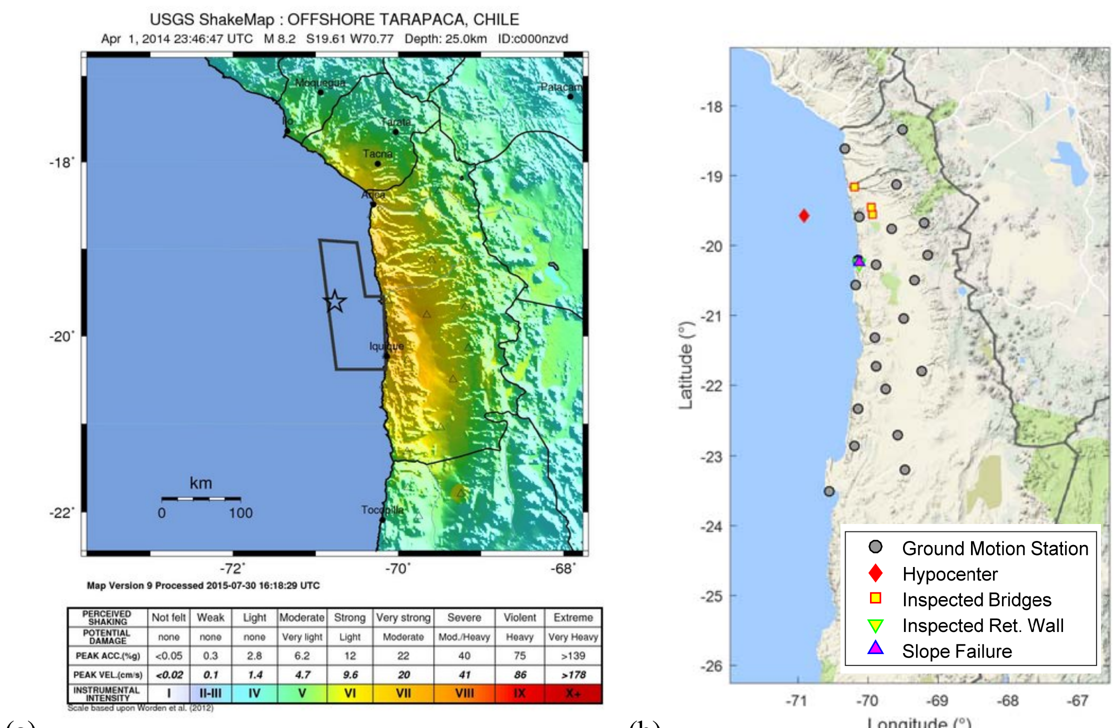


Fig. 1 (a) USGS ShakeMap for the M_w 8.2 event, (b) plan view of the main deformation zone

3. Recorded Ground Motions and Ground Response

Available ground motions include more than 20 digital recordings of accelerometer stations and a number of broadband stations [4]. Table 1 presents a summary of ground motion intensities for the main event M_w 8.2 and the April 3rd aftershock of M_w 7.6. Measured peak ground acceleration (PGA) and spectral ordinates values were compared against recent ground motion prediction equation [5], and 5% damping response spectra were compared against the Chilean standard for earthquake resistant design of buildings [6]; in general, these models are in reasonable agreement with the intensities recorded. In some cases, however, (e.g. station T03A, near Cavancha Beach), the spectral ordinates for high frequencies are well above design standards.

Table 1 – Ground motion intensities of recorded motions

Earthquake	Station	Peak Ground Acceleration PGA (g)			Peak Ground Velocity (cm/s)			Epicentral Distance (km)
		N-S	E-W	Vert.	N-S	E-W	Vert.	
Main event M_w 8.2, April 1st 2014	T10A	0.659	0.781	1.046	3.149	3.782	4.827	128.4
	PB11	0.493	0.752	0.457	4.773	6.366	3.790	132.9
	T07A	0.519	0.613	0.329	2.519	3.000	1.557	139.9
	T03A	0.576	0.606	0.218	2.754	2.950	1.034	108.3
	T09A	0.409	0.576	0.307	1.945	2.688	1.346	73.1
	MNMCX	0.308	0.458	0.281	2.769	4.141	2.335	146.3
	T08A	0.446	0.392	0.320	2.179	1.877	1.559	115.2
	GO01	0.242	0.361	0.162	2.210	3.485	1.485	179.9
	T13A	0.273	0.340	0.229	1.326	1.660	1.102	193.7
	T05A	0.305	0.272	0.255	1.482	1.322	1.179	106.4
	T06A	0.267	0.217	0.154	1.304	1.054	0.737	107.7
	HMBCX	0.239	0.259	0.173	2.060	2.432	1.597	132.5
	TA01	0.174	0.196	0.076	1.705	1.888	0.673	134.2
Aftershock M_w 7.6 April 3rd 2014	T10A	0.598	0.637	0.443	2.863	2.871	2.169	91.0
	T07A	0.534	0.342	0.300	2.611	1.676	1.469	74.0
	T08A	0.327	0.455	0.236	1.592	2.228	1.141	45.3
	T13A	0.384	0.369	0.284	1.863	1.803	1.357	114.8
	T03A	0.295	0.230	0.106	1.433	1.111	0.506	44.2
	GO01	0.254	0.206	0.164	2.432	2.005	1.438	160.7
	HMBCX	0.249	0.228	0.205	2.376	2.083	1.896	63.3
	PB11	0.211	0.214	0.178	1.996	2.090	1.600	117.4

3.1 Microzoning

Prior to the earthquake, a seismic microzoning had been proposed for Iquique and Alto Hospicio cities [7] based on geophysical data and the geological background. This study identified rock outcrops and different zones susceptible to site amplifications, including eolian and marine sand deposits with depths of more than 25 m, and artificial fills. Recorded ground accelerations showed to be consistent with the proposed microzoning; for instance, a PGA of 0.27g was measured on bedrock, and a PGA of 0.6g was measured on marine deposits of moderate thickness, which indicate the effects of relatively soft soil layers on the ground amplification. In the city Alto Hospicio, the recorded PGA values of 0.44g are in agreement with the stiff soils identified in reference [7].

Based on this information, the damage reconnaissance campaign after the earthquake focused on zones where ground motion amplification was likely to occur. The sites of most interest are located in the cities of Iquique and Alto Hospicio, which are described next.

3.2 Site effects

The tax exempt zone of Iquique, known by the acronym ZOFRI, is an industrial zone composed of mainly 3- to 5-stories confined-masonry warehouses. Most of this zone is characterized by a moderate to thick layer of marine deposits. Towards the east, the layer thickness decreases to about 20 m and mixes with eolian deposits. The observed effect of the ground motions in the area include the collapse of roof parapets, moderate to severe non-structural damage, and a landslide, which may have been aggravated by the topographic effects on the escarpment to the east side of ZOFRI. Although no site specific records are available, the measured H/V ratios [8] suggest a significant impedance contrast between the soil layers and the bedrock, which probably led to amplified shaking at the site. The slopes located to the east of ZOFRI have terraces, for vehicles' circulation and parking, built with shallow compaction and no reinforcement. Consequently, the landslides shown in Fig.2(a) were somewhat anticipated. The peak residual deformation was estimated in 1.3 m by adding the cracks openings across straight lines perpendicular to the direction of sliding.

On the other hand, Alto Hospicio is characterized by rigid gravels with a matrix of halite, also known as rock salt [9]. Most of the damage in this area was concentrated in masonry houses of up to 2 stories near the center of the city, Fig.2(b). It is hypothesized that partial dissolution of salts molecules may have opened voids that became unstable during the earthquake, since near the houses several cavities were identified in the surface as well as seismic induced cracks on the pavement.

(a)  (b) 

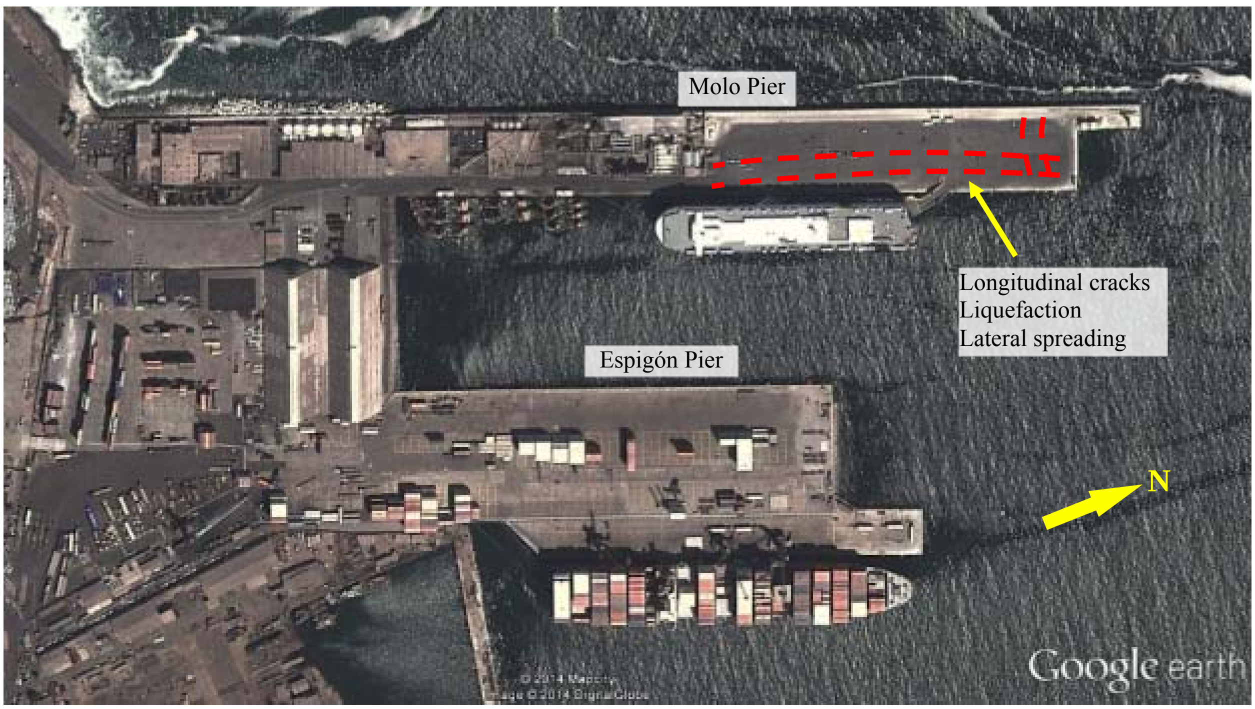
Fig. 2 (a) landslides in the ZOFRI area, (b) partial collapse of masonry house in Alto Hospicio

4. Performance of Ports

An aerial view of the port of Iquique is shown in Fig.3(a), which consists of the Molo pier, a passenger terminal 590 m long, and the Espigón pier, a cargo terminal 400 m long. The piers built between 1928 and 1932 are made of concrete quay walls supported on top of gravelly mounds with boulders placed around the perimeter. In both piers the backfill materials were likely dumped inside the quay walls with little compaction effort. The underlying soil conditions instead, are quite different. While the Molo pier is underlain by a ~5 m layer of medium dense sand, a ~10 m layer of dense silty sand, and a layer of cemented sand, the Espigón pier is supported directly on rock. Moreover, a seismic retrofit was performed in the Espigón pier after a lease to a private operator in 2002.

These contrasting subsurface soil conditions and the current state of the infrastructure led to a dissimilar seismic performance. After the earthquake, the Espigón pier was relatively undamaged and there was no apparent evidence of significant lateral spreading or distress to the pier. As a result of the seismic upgrade, the Espigón pier continued in operation following the earthquake. In contrast, differential settlements up to 1.1 m, liquefaction, and lateral spreading were observed in the Molo pier, as shown in the photographs of Fig.3 (b) and (c). Longitudinal cracks developed across the entire length and most of the lateral spread occurred towards the east side of the pier, with a cumulative displacement that varied between 1.0 m and 1.5 m. In addition, lateral spreading developed in the transverse direction at the far northern end of the pier, creating a complicated cracking pattern on the surface. Observations showed that the lateral displacements increased an additional 0.5 m near the middle section of the pier in the weeks following the earthquake. In June 2014, five cone penetration tests (CPT) soundings were performed in the Molo pier to depths of 8 to 23 m. The CPT logs suggested that the dense sand layers were gravelly sands to sandy gravels, but most of them also revealed the presence of relatively loose liquefiable sand layers.

The port of Arica, located 130 km Northeast of the epicenter sustained no significant damage based on reports from the Chilean Army Corps of Engineers. The Ports Department of the Ministry of Public Works (MOP) informed that the facility remained operational after the earthquake.



(a)

(b)

(c)

Fig. 3 (a) Google earth view of the Molo and Espigón piers, Port of Iquique, (b) and (c) photos showing typical settlement of the backfill and lateral spreading on the Molo pier

4. Performance of Bridges

Three highway bridges along Route 5 between Iquique and Arica were subjected to strong ground shaking: Tiliviche (19.55196ºS, 69.94092ºW), Tana (19.45449ºS, 69.94683ºW), and Camarones (19.15712°S, 70.18761°W), all of which pass over river channels flowing westward through the desert and into the Pacific Ocean.

4.1 Tiviliche Bridge

The Tiliviche bridge is a curved steel girder bridge consisting of three spans, each about 22 m in length, with piers made of solid concrete walls. The curve is laid out along a radius of about 130 m and transects an arc of about 20º. An examination of the bridge deck indicates that there was no visible lateral or vertical offset at any of the bridge joints located at the abutments and pier supports. Very little water was flowing in the stream channel under the bridge at the time of the earthquake and there were no sand boils in the river bed. The only sign of distress observed in the vicinity of the bridge was a series of cracks, about 20 to 30 mm wide, which ran parallel to a stream channel for a distance of about 10 m.

4.2 Tana Bridge

The Tana bridge is a curved steel girder bridge with five spans of about 16 to 18 m in length, supported on piers made of solid concrete walls. The curve is laid out along a 90 m radius and transects an arc of about 50º. Gabion walls were used to stabilize the steep slopes running west from the north abutment, as shown in Fig.4(b). No evidence of lateral or vertical offsets were observed in the bridge deck, nor significant cracks in the support piers or abutments. However, surface cracks developed at the top of the north slope adjacent to the gabion wall. Crack widths suggest a gabion displacement on the order of 30 to 60 mm. Tension cracks were visible at the interface between the bridge abutment and soil mass. Several cracks were also observed running perpendicular to the abutment wall in the slope adjacent to the north abutment.

(a)  (b) 

Fig. 4 (a) View of the bridge from the southwest showing gabion walls against the abutment in the foreground, and (b) tension crack behind gabion wall in the north abutment.

The original drawings for the bridge indicate that the soil profile consists primarily of poorly graded gravel (GP) to a depth of about 25 m with a surface layer of silty sand (SM) about 1.5 m thick. These drawings also show that each pier is supported by six 150 cm diameter piles (drilled shafts) that extend 15 m below the pile cap into the gravel layer. The abutments are supported by eight 150 cm diameter piles which extended 12 m below the base of the pile cap into the gravel layer.

Although there was no evidence of distress to the bridge, a number of liquefaction features were evident in the alluvial soil deposits beneath the bridge. Several sand boils developed just downstream of the first pier south of the north abutment; while the ejecta was primarily sand, it was unclear at that point whether or not the gravel in the profile also liquefied. Liquefaction induced settlement produced an offset of approximately 15 cm adjacent to one of the bridge piers as shown in Fig.5(a). Liquefaction on the north side of the river running southeast from the bridge abutment led to significant lateral spreading along an ~80 m strip as shown in Fig.5(b). Although the lateral spread impinged upon the north bridge pier, the ground movement was insufficient to cause distress to the pier.

(a)  (b) 

Fig. 5 (a) Settlement of soil adjacent to bridge pier as a result of liquefaction, (b) lateral spreading running parallel to stream channel flowing under the Tana bridge

4.3 Camarones Bridge

The Camarones bridge is a steel girder bridge consisting of eight spans each about 30 m in length, supported on steel frame portals, as shown in Fig.6(a). At the time of the reconnaissance, the bridge was closed to traffic due to a lateral offset of approximately 10-15 cm between two of the spans as shown in Fig.6(b), preventing a closer inspection of the superstructure. Relatively little evidence of soil liquefaction was observed in the alluvial sediments beneath the bridge; only a sand boil approximately 60 cm in diameter observed within ~10 m of the third bent from the northwest abutment. Small settlements of approximately 5-10 cm were evident in the fill surrounding the northwest abutment.

(a)  (b) 

Fig. 6 (a) View of Camarones Bridge from the north end, (b) 10-15 cm lateral offset

5. Earth Retaining Structures

The overall performance of retaining structures in Iquique was acceptable and in good agreement with the measured ground accelerations. Retaining walls with inadequate footing dimensions underwent severe damage at two locations: Cerro Dragon (20.26239°S, 70.12325°W), and the Municipal Stadium (20.24385°S, 70.13262°W). Likewise, a mechanically stabilized retaining wall failed in the south end of Cerro Dragón (20.27361°S, 70.12418°W) as a result of corrosion in the steel bars used as reinforcement. A number of non-engineered retaining walls tipped over. Many retaining walls with no reinforcement performed well. No damage was observed in basement walls or temporary braced excavations.

5.1 Retaining walls of Cerro Dragón

Several retaining walls in Los Algarrobos street at Cerro Dragon were damaged due to the insufficient size of the footing, the lack of restraining elements, and the low strength of the backfill. The observed failure mode was overturning, with measured tip displacement varying between $\Delta/H=8\%$ at the north end and $\Delta/H<1\%$ at the south end, as shown schematically in the Google earth photograph of Fig.7. The extent of the damage was significant considering that PGA at the site was less than 0.25g, and that retaining walls adequately designed for static loads may well resist ground shaking with PGA in the order of 0.3g [10].

Fig. 7 Google earth photograph of the housing development of Cerro Dragón

These retaining walls are made of reinforced concrete, have an average thickness of 20 cm, a height of 240 cm, and spans between 3 m and 15 m. The walls are part of a development built on the dunes of Cerro Dragon, where the terrain is made primarily of loose marine sands, poorly graded particles (C_u=0.50, C_c=0.80) of medium to small size (d_{50}~0.2 mm), and a friction angle between 30°-35°. Additionally, these soils have significant amount of calcareous sediments and soluble salts, which increases the risk of bearing capacity failures and large settlements induced by water infiltrations. The backfill soil was flat and the material was loose dune sand mixed with construction debris.

The largest measured overturning was approximately 20 cm ($\Delta/H=8\%$), as shown in Fig.8(a). Based on Street View© photographs taken before the earthquake, it is concluded that most of the wall rotation occurred during the 2014 earthquake sequence. Large rotations were observed on other walls along Los Algarrobos street, however these walls had already overturned, presumably after the 2007 Tocopilla earthquake (M_w 7.7) and 2005 Tarapacá earthquake (M_w 7.8), and had not been retrofitted since. Large rotations on a retaining wall induced backfill displacements and differential settlements on the house shown in Fig.8(b). Following the earthquake, people blocked the streets and many abandoned their houses for several weeks.

5.2 Retaining walls at Municipal Stadium

A 20 m section of a masonry wall (h=150 cm) located at the Municipal Stadium was severely damaged as a result of ground shaking. The wall was sitting atop of a 2:1 slope 3 m high, and had a flat backfill made of dry loose sand. The failure mode was complete rocking and displacement caused by the absence of a footing or other restraining elements. No signs of a global slope instability were observed.

(a)  (b) 

Fig. 8 (a) wall displaced (Δ/H=8%) in Los Algarrobos Street, (b) settlements in the front porch of a house and rotation of a parapet due to failure of a retaining wall

5.3 Mechanically Stabilized Earth Structures (MSE)

The seismic performance of mechanically stabilized earth structures was acceptable throughout the region, including overpass approaches and bridge abutments. No major damages were observed, with the exception of the retaining walls that collapsed near the south end of Cerro Dragón, at the intersection of Los Algarrobos and Cuatro Sur streets. These MSE walls formed a three level terrace to hold a parking garage and a public road, as seen in the photograph of Fig.9(a) taken before the earthquake.

As a result of the advanced corrosion in the soil reinforcement, added to the seismic loads on the backfill, approximately 60 m of wall in the 3rd level collapsed. Localized failure of the walls in the 1st level was also observed, as shown in Fig.9 (b) and (c). The failure mode was the rupture of the steel reinforcement and internal instability of the reinforced soil mass. The backfill material was the marine sands of Cerro Dragón, which have a great corrosion potential due to the presence of chlorides and sulfates, and the free exchange of moisture between the soil and the air. The steel reinforcement was made of hot-dip galvanized ϕ10 mm bars. In non-aggressive media, this reinforcement would have performed well, but the highly corrosive soils present at the site, led to a rapid reduction of the zinc coating of the rebar cross section, affecting its strength capacity.

6. Slope Stability and Rockfall

Natural slope failures along the coastal bluffs and interior creeks were observed throughout the region affected by the ground motion, from Arica in the north to Calama in the south. In most cases, however, they caused no damage to engineered systems. Several cases of rockfall and raveling developed on cut slopes of Route 1, Route 5, and interior roadways, blocking traffic temporarily. No significant damage was observed on steep slopes stabilized with wire meshes and shotcrete. A massive slope failure affected route A-16 between the cities of Iquique and Alto Hospicio, which is a critical road for the regional commerce. This failure delayed the emergency response operations and caused major traffic congestions in the days following the earthquake.

(a)

(b)

(c)

Fig. 9. Photograph of retaining walls before the earthquake. Source: Google Street View. (b) failure of retaining walls in the upper level and backfill displacements, (c) local failure of MSE wall. Photographs (b) and (c) courtesy of Ramon Verdugo.

6.1 Slope Failures along Route A-16

On Route A-16 between Iquique and Alto Hospcio, a cut slope failed 2.5 km south of the El Pampino roundabout (20.24337°S, 70.12389°W). The failed section, shown in Fig.10, extended over 400 m on the west lane, with the largest displacements concentrated on a 100 m strip. Vertical offsets of 20–40 cm and horizontal displacements of 10–30 cm were measured at the slope scarp, and the estimated volume of displaced soil was approximately 3000 m^3.

The failure on route A-16 is located on the steep cliffs of Iquique, at an elevation of 215 m above sea level. The slope runs north-south, dips west at an angle of 24°, and is made primarily of marine sands underlain by Tertiary sedimentary rocks. The depth to bedrock is about 10–15 m and the estimated peak ground accelerations for the main shock were 0.3g-0.4g. Following the earthquake, traffic was permanently closed on the west lanes. The east lanes opened for emergency operations within a day and for regular traffic within three days. Shallow failures occurred at different locations along Route A-16, but they concentrated at higher elevations, near Alto Hospicio. Rockfalls on the hillside were pervasive, but the loose material was plowed off before the reconnaissance team arrived to the site on April 4th. While raveling failures of non-stabilized slopes occurred repeatedly, road cuts protected with shotcrete and wire mesh performed well and only minor damages were observed.

Fig. 10 Slope failure on Route A-16 (-20.243028°, -70.124056°).

6.2 Rockfalls

Rockfalls and raveling failures of disaggregated material were common on over-steepened cut slopes in the Regions of Tarapacá and Antofagasta. In most cases, the rock fragments and soil were plowed off the roads within a few days after the earthquake and the traffic flow was restored quickly. The extent and volume of the rockfall was in direct proportion to the slope angle, and decayed with increasing site-to-source distance. For example, slopes of sedimentary rock on route A-16 (96 km from the hypocenter) were significantly damaged, whereas steep loose-gravel slopes on Route 565 located 130 km east from the hypocenter, experienced only minor rockfall as seen in Fig.11.

7. Closing remarks

The Iquique earthquake sequence and tsunamic of April 1st of 2014 caused minor to moderate damage to geotechnical systems. Reconnaissance teams from the GEER Association and CIGIDEN visited the area and documented the consequences of this earthquake, including shallow slope failures, several landslides, liquefaction at Iquique's port, possible site-effects in Alto Hospicio, and the response of several bridges and earth retaining structures. The extent of the damage was limited and consistent with the levels of ground shaking, as the earthquake ruptured only a portion of the mega-thrust zone. The evidences gathered in this study are key for preparing for a much greater earthquake.

(a)  (b) 

Fig. 11 (a) Disaggregated rock failures on Route A-16 (photo courtesy of prensalibre.com), (b) minor raveling along Route A565, unimpeded traffic.

8. Acknowledgements

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9. Copyrights

10. References

[1] Rollings, K. Ledezma, C., Montalva, C. (2014). Geotechnical Aspects of the April 1sr, 2014, M_w 8.2 Iquique, Chile Earthquake, GEER Association Report No. GEER-038. Version 1.2: October 22, 2014.

[2] Hayes, G. (2014). Updated Finite Fault Results for the Apr 01, 2014 M_w 8.2 99 km NW of Iquique, Chile Earthquake (Version 2). National Earthquake Information Center (NEIC) of United States Geological Survey. http://comcat.cr.usgs.gov/earthquakes/eventpage/usc000nzvd#scientific_finite-fault. Last accessed on 5/25/2014

[3] Barrientos, S. (2014). Informe Técnico Terremoto de Iquique, M_w=8.2. Report Centro Sismológico Nacional Universidad de Chile, April 2014.

[4] Centro Sismológico Nacional de la Universidad de Chile (CSN). http://www.sismologia.cl/

[5] Abrahamson, N., Gregor, N., Addo, K. (2014). BCHydro ground motion prediction equations for subduction earthquakes, Earthquake Spectra, submitted.

[6] Oficial, N. C. NCh 433 Of. 1996, Diseño Sísmico de Edificios. Instituto Nacional de Normalización, INN, Chile.

[7] Becerra, A., Podestá, L., Monetta, R., Sáez, E., Leyton, F., and Yañez, G. (2014). Seismic microzoning of Arica and Iquique, Chile. Manuscript submitted for publication.

[8] Nakamura, Y. (1989). A method for dynamic characteristics estimation of subsurface using microtremor on the ground surface, Q.R. Rail. Tech, 25–30.

[9] Marquardt, C., Marinovic, N. and Muñoz, V. (2008). Geología de las ciudades de Iquique y Alto Hospicio, Región de Tarapacá, Carta Geológica de Chile, Serie Geología Básica, 113, 33.

[10] Sitar, N., Mikola, R. G., & Candia, G. (2012). Seismically induced lateral earth pressures on retaining structures and basement walls. Proceedings of Geotechnical Engineering State of the Art and Practice, GeoCongress, 335-358.