Chile Earthquake of 2010

Assessment of Industrial Facilities around Concepción

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Appendixes

(The appendixes are available for download at http://dx.doi.org/10.1061/9780784413647.)

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Chapter 1
Introduction

On February 27, 2010 a magnitude (Mw) 8.8 earthquake struck off the coast of South-Central Chile (Maule, see Figure 1-1). It was the largest ground motion event in Chile since the magnitude 9.5 earthquake of 1960 and is listed by USGS as the fifth largest tectonic event ever recorded (as of February 27, 2010). The Structural Engineering Institute (SEI) of the American Society of Civil Engineers (ASCE) began planning for a reconnaissance mission to the affected zone with the intent of gathering information useful to code development. Several teams were formed, each consisting of three or more members tasked with a particular structure type or design issue.

This report, originally compiled in 2010, documents the finding of the Industrial Assessment Team, and was intended to inform code development activities connected with earthquake protection measures for industrial facilities. Accordingly, the inquiries of the Industrial Assessment Team were concentrated on identifying strengths and weaknesses in the response of industrial structures to the seismic event. It was anticipated that damage to industrial facilities had occurred, but that industrial structures designed to newer codes and standards would perform better than those designed to older codes and standards. Chile is unique in that it has a separate standard for industrial structures, NCh2369.Of 2003: Earthquake-Resistant Design of Industrial Structures and Facilities. Requirements in NCh2369 are similar to those found in the 1994 UBC for nonbuilding structures. Because of the similarity between US and Chilean standards, understanding how heavy industrial facilities performed in this seismic event is vitally important to US design practice because the design practice for nonbuilding structures has varied significantly from that employed for building structures since the publication of the 1988 UBC. Observations of the performance of industrial facilities in this seismic event provides a window to how heavy industrial facilities may perform in the next large seismic event in the western US and where US heavy industrial facilities should focus their retrofit efforts.

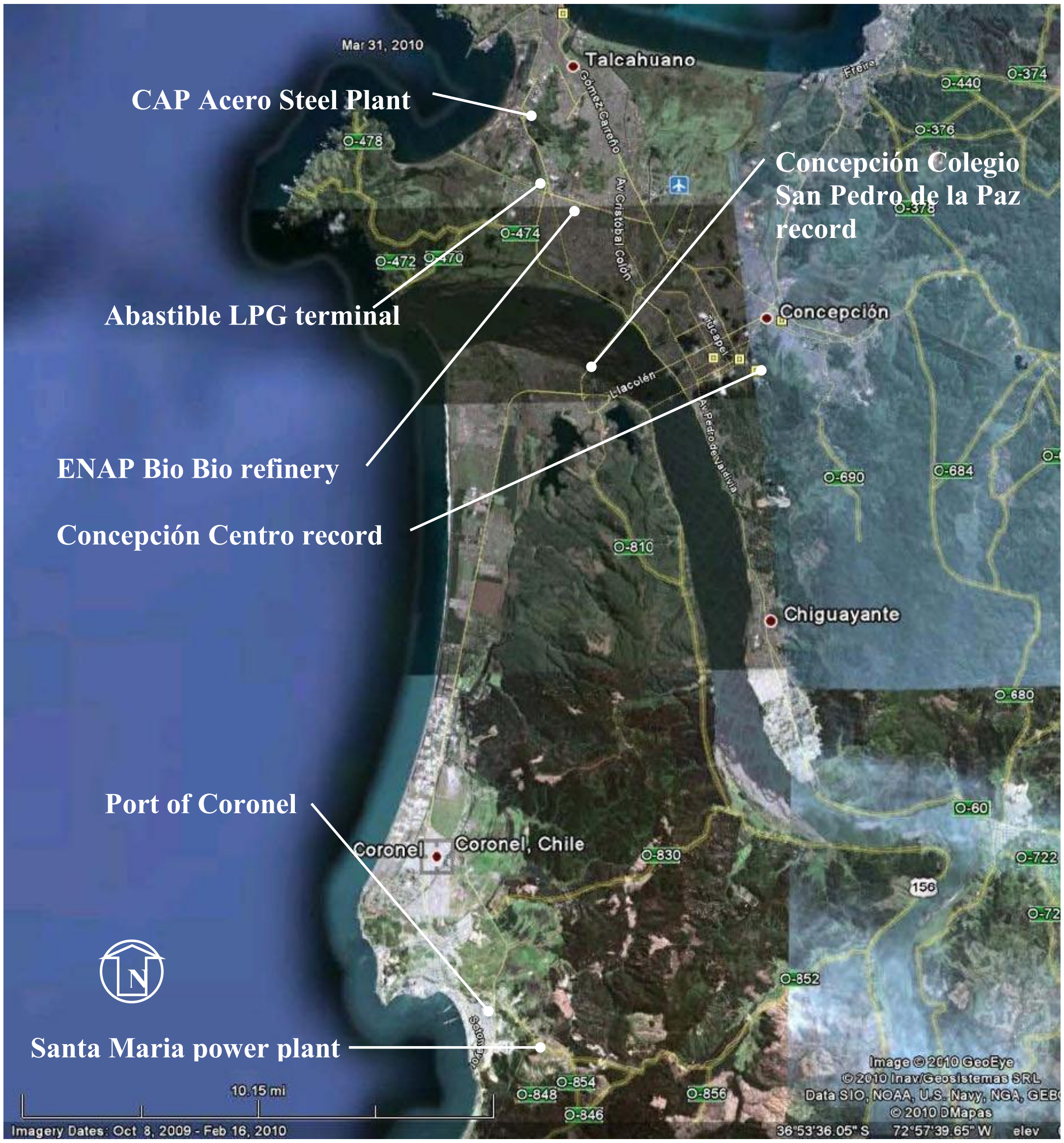


Figure 1-1: Locations of ground motion stations and of sites visited in the Concepción area (Source: GeoEye).

Chapter 2
Ground Motion Records

There are only two ground motion records for the Concepción area and no ground motion records that are particularly close to Coronel (see Figure 1-1). Figure 2-1 shows a close-up of the locations of the two ground motion records and the three facilities located near Concepción. While no ground motion records exist for the facilities visited by the Industrial Assessment Team, the two ground motion records noted above are representative of the ground motions experienced in this region. The actual ground motions experienced at the facilities visited may be different.

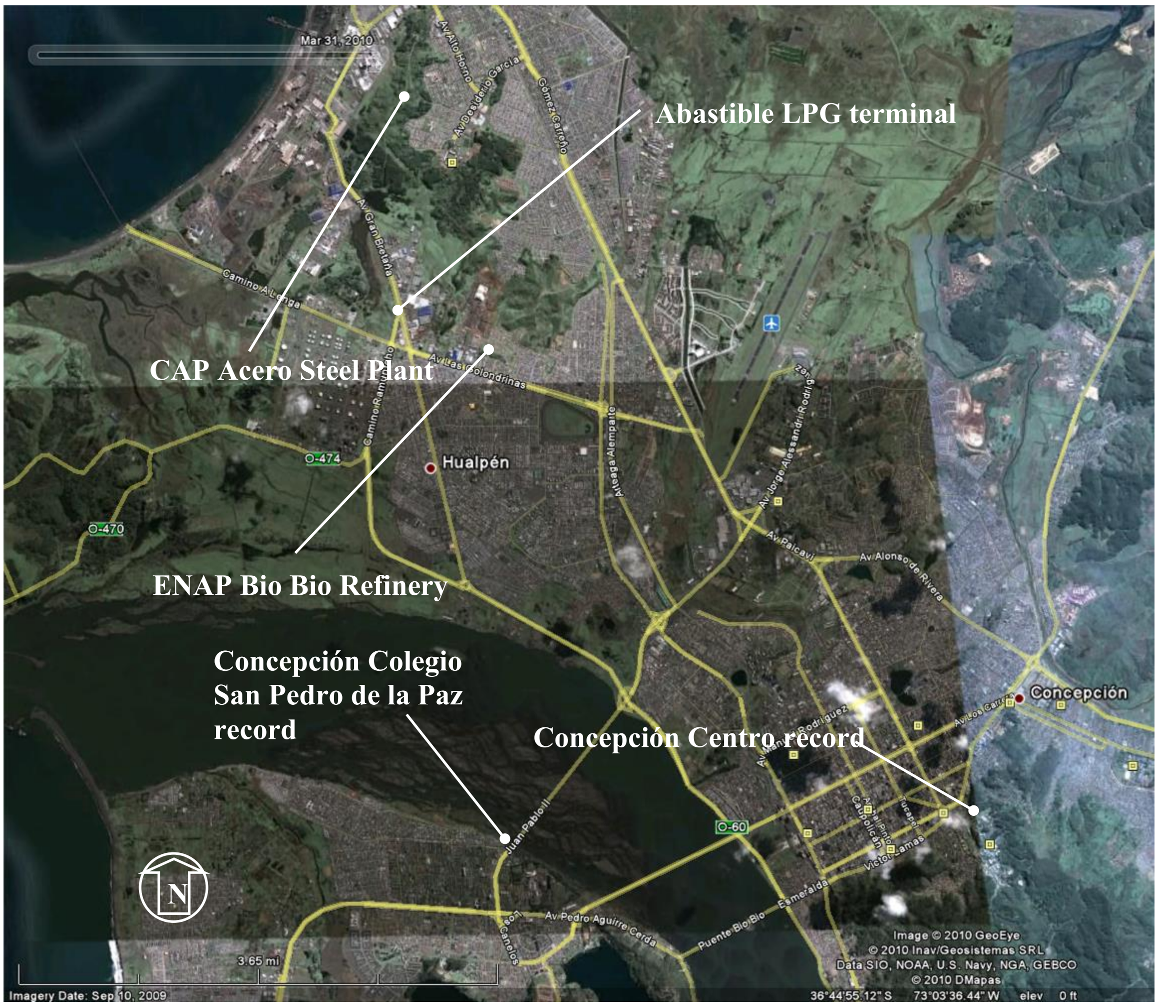
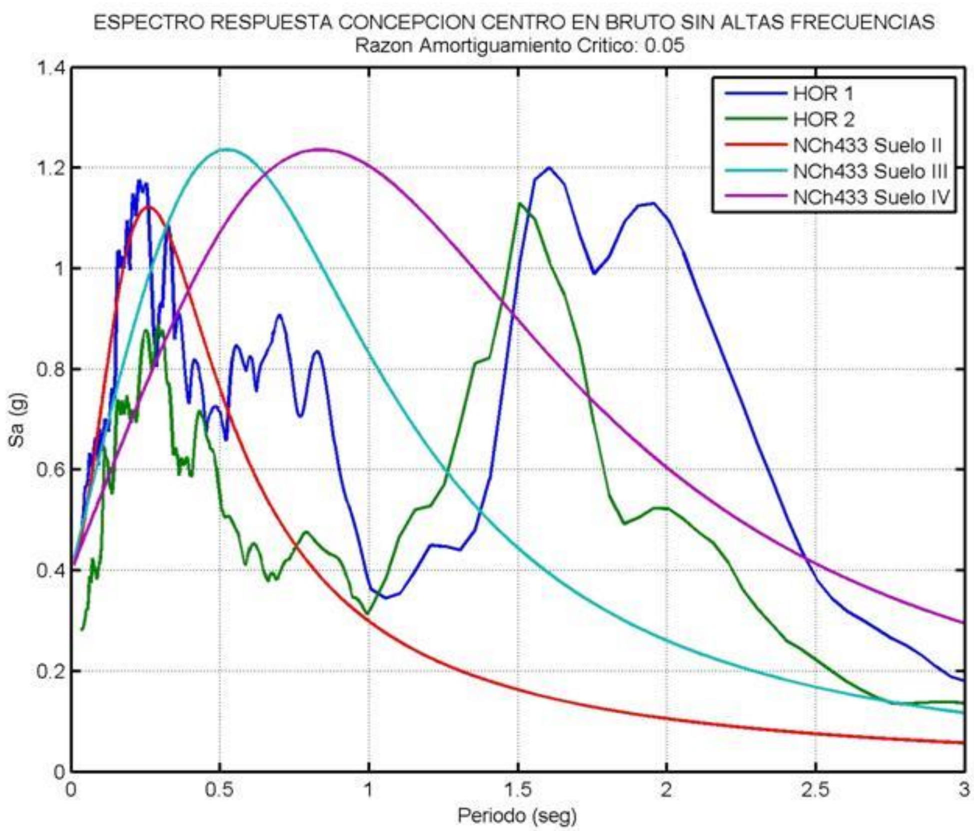


Figure 2-1: Detail of location of ground motion instruments and visited facilities in Concepción (Source: GeoEye).

The preliminarily determined response spectra, shown in Figure 2-2, were determined from data taken from a ground motion instrument located at a school near the intersection of San Martin and Anibal Pinto, 100 meters south from the main square in Concepción. The soil was identified as Site Class C as defined in ASCE/SEI 7-05. The preliminarily determined response spectra, shown in Figure 2-3, were determined from data taken from a ground motion instrument located at the Concepción Colegio San Pedro. The soil for this record was not identified. The preliminarily response spectra were developed by Mr. Rubén Boroschek and were provided to the Industrial Assessment Team by contacts in Chile.

As can be seen in Figure 2-1, the ground motion record measured at the Concepción Colegio San Pedro is slightly closer (by approximately 1 kilometer) than the ground motion measured at Concepción Centro relative to the CAP Acero Steel Plant, the Abastible LPG Terminal, and the ENAP Bio Bio Refinery sites. Unfortunately, as mentioned above, the soil type at the Concepción Colegio instrument is currently not known.

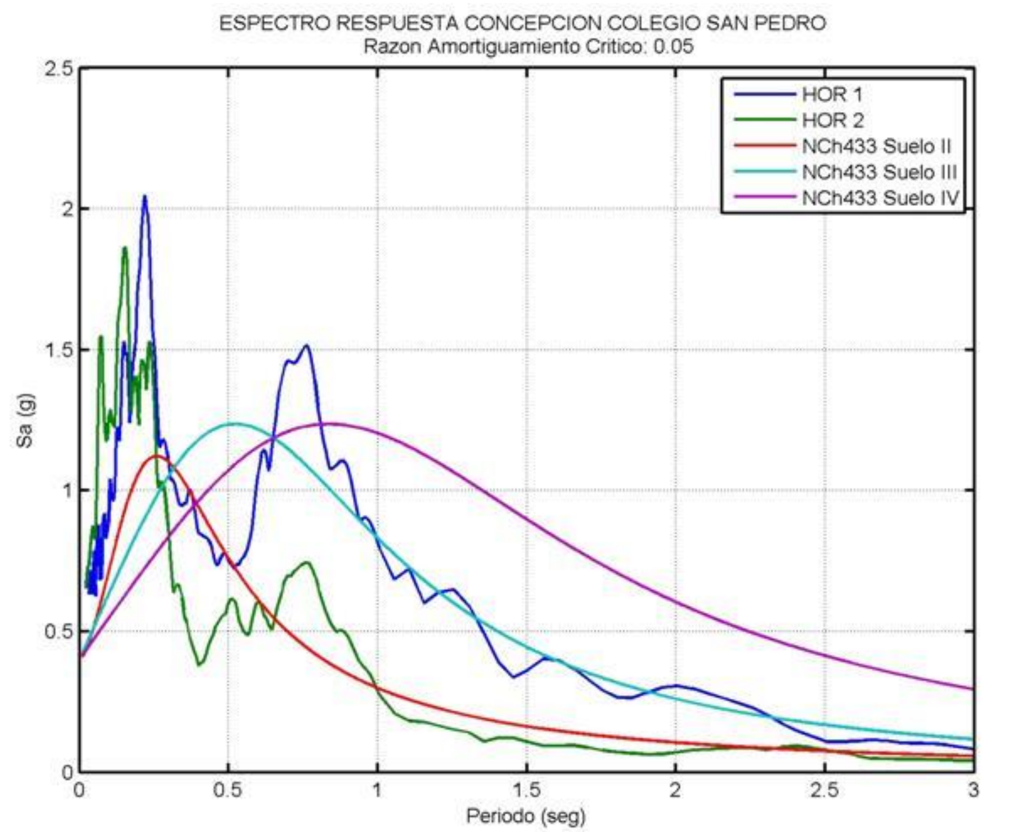


Acceleration 5 % Damping and NCh433 Elastic Spectra for Three Types of Soils.

Figure 2-2: Preliminary response spectra developed from measured data from central Concepción. (Courtesy of the University of Chile, Rubin Boroschek, Pedro Soto, and Richard Leon)

The spectra shown in Figure 2-2 and Figure 2-3 are compared against the design response spectra for various soil types as determined from the Chilean Standard NCh 433, *Earthquake Resistant Design of Buildings,* issued in 1996. Please note the "double hump" shape of the measured spectra. Structures with periods falling between the two humps would be expected to undergo increased seismic forces as the structures yield and their fundamental periods elongate (typically by as much as a factor of 1.5 to 2.0). Some structures falling in this period range could be expected to have reduced performance due to these increased seismic forces. Longer period structures would also see significantly higher ground motions than the ground motions predicted by Chilean Standard NCh 433 or Chilean Standard NCh 2369 due to the presence of the second peak in both measured response spectra. (See Appendix A for an English translation of Chilean Standard NCh433.Of96 and Appendix B for an English translation of Chilean Standard NCh2369.)

As mentioned above, no ground motion records exist for the facilities visited by the Industrial Assessment Team. For the purpose of general discussion, this report will treat the two ground motion records noted above as representative of the ground motions experienced in this region. The actual ground motions experienced at the facilities studied may be different.



Acceleration 5 % Damping and NCh433 Elastic Spectra for Three Types of Soils.

Figure 2-3: Preliminary response spectra developed from measured data from Concepción Colegio. (Courtesy of the University of Chile, Rubin Boroschek, Pedro Soto, and Richard Leon)

Chapter 3
Assessment of CAP Acero Steel Plant

Facility Overview

The CAP Acero Huachipato Steel Plant (Lat 36° 45′ 6" S, Long 73° 8′ 2" W), located in Talcahuano (see Figure 3-1) near the City of Concepción, was originally constructed in the 1950s with numerous expansions over the years. It currently occupies an area approximately three miles long by a half-mile wide and comprises a dozen very large industrial buildings and numerous support structures. The steel plant makes steel from raw materials (iron ore, coal, etc.) and produces approximately 1.45 million tons of steel a year. The CAP Acero Huachipato Steel Plant is the largest steel plant in Chile. The facility has experienced a number of significant seismic events over the past half century including the 1960 Mw 9.5 Valdivia (Chile) Earthquake.

Structures examined include those making up the Coke Plant, Blast Furnace #1, Blast Furnace #2, Steel Shop, Bar Mill (new), Bar Mill (old), Hot Rolling Mill (with water treatment plant) and the pier (see Figure 3-2). The response of these structures is discussed below. (Appendix C contains drawings for the coal bin, coal hopper, holding basins, and pier structure.)

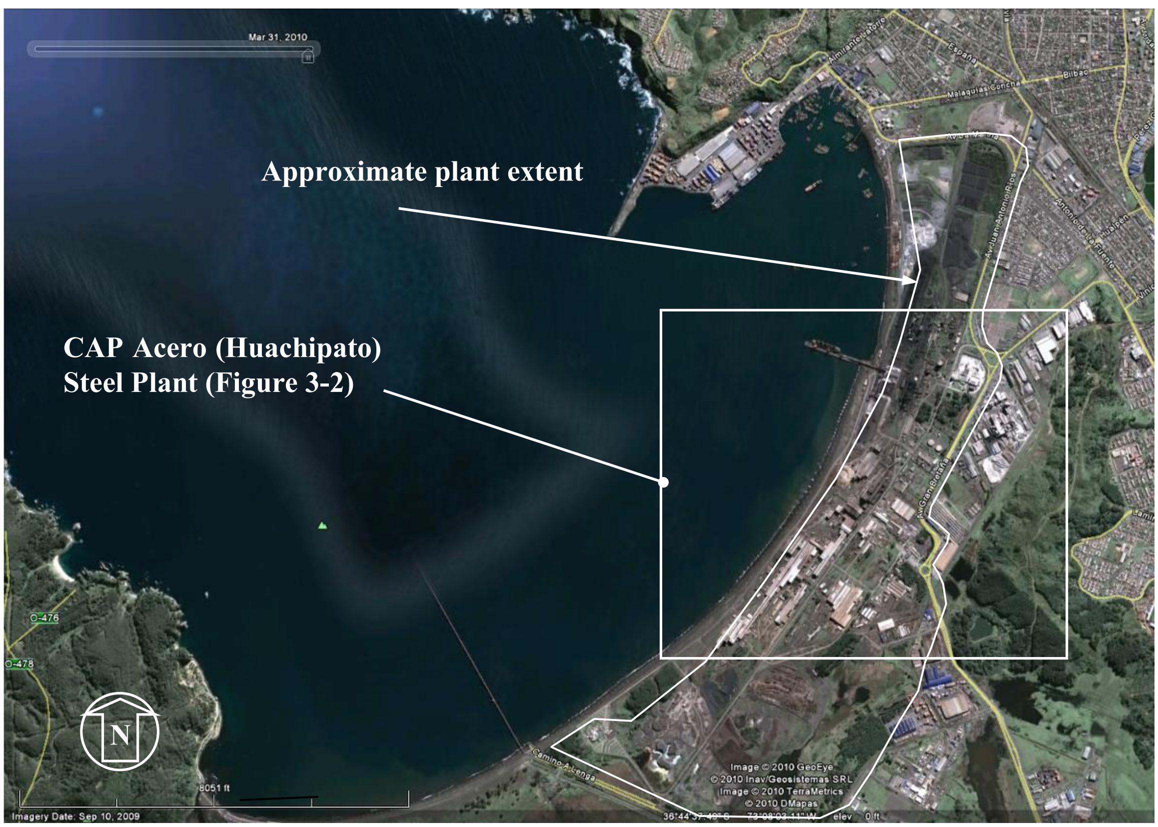


Figure 3-1: Aerial view of CAP Acero Steel Plant and harbor (Source: GeoEye).

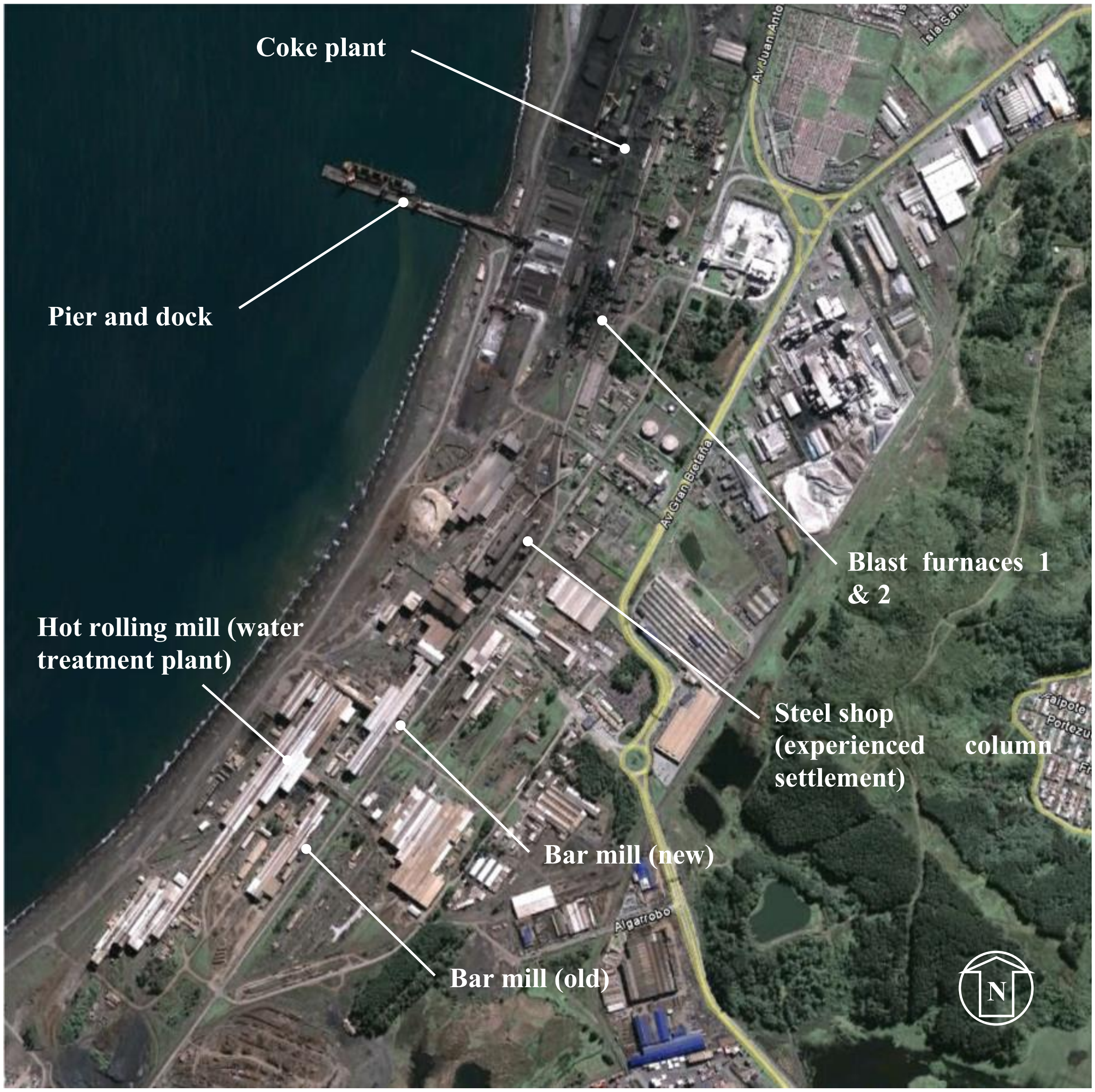


Figure 3-2: Close-up aerial view of CAP Acero (Huachipato) Steel Plant (Source: GeoEye).

Soil Conditions

The soil was observed to be a medium to dense dark sand with a loose surface layer. Localized areas of lateral spreading and settlement damage from liquefaction were observed at a few plant locations (especially near the shore line). The soil beneath Column A-3 in the Steel Shop was found to be SM (SP – SM) soil with medium to high density. The water table depth is approximately 2.5 m.

Facility Design

The facility buildings and structures utilize reinforced concrete spread footings for their foundation support. Some the structures have large elevated concrete bases. The primary superstructures are typically steel moment frames in the transverse direction and steel concentrically braced structures in the longitudinal direction. For larger spanning roofs, the spans are typically trusses that also serve as moment resisting beams. The bases of the columns frames typically used tall saddles and long anchor bolts to allow the bolts to stretch. Shear keys were typically used at the base of the columns to transfer shear. Their design is very comparable to similar structures built in coastal California during the same time frames. Today in the United States, these would typically be classified as ordinary steel systems.

Estimated Response Spectra

The response spectra from the closest ground motion instruments to this location (within 10 km) are shown in Figure 2-2 and Figure 2-3 above. Because this was a subduction zone type earthquake, the ground motions measured at Figures 2-2 and 2-3 are considered to be representative of the ground motions experienced in this region. The actual ground motions experienced at the facilities visited may be different due to local geology.

Coke Plant Assessment

The only structure investigated at the coke plant was the elevated coal bin shown in Figure 3-3 (Lat 36° 44′ 16" S, Long 73° 7′ 23" W). This large structure is composed of an elevated concrete base which is approximately 50 feet by 65 feet in plan and 100 feet tall. A 60-foot tall steel coal bin is located on top of the elevated base. The structural system of the coal bin is a combination of steel moment frames, brace frames and steel plate shear walls. The coke plant was constructed in 1990. See Appendix C for drawings.

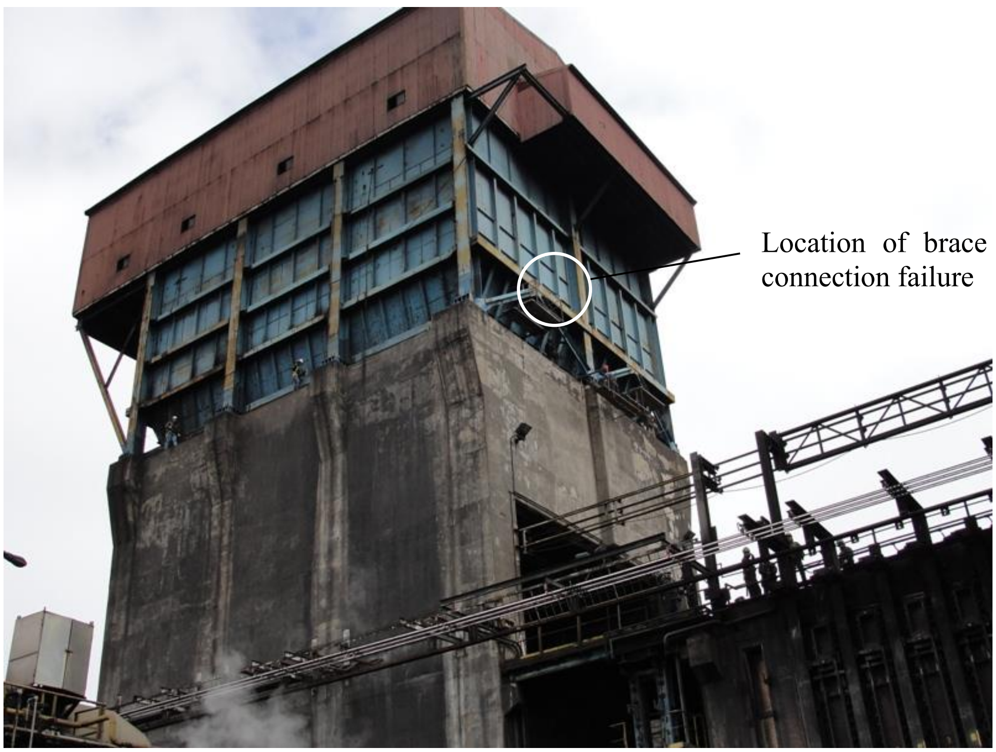


Figure 3-3: Coke plant.

Observations

The bracing connections of the coal bin failed. As can be seen in Figure 3-4, the wing plates fractured at the ends of the braces. In general, the coal bin diagonal braces and connections were less robust that than would expected in current designs. In discussions with the plant engineers, it was learned that the bins were originally designed using a seismic mass equal to the dead weight of the bin plus only 50% of the weight of the stored coal. The use of 50% of the coal weight was based on the average level of coal in the bin at any given time. The coal bins were approximately 75% full at the time of the earthquake. The plant engineers plan to design the replacement diagonal braces and connections for 100% of the weight of the stored coal. It is not currently know what lateral seismic design coefficient the coal bin was designed for originally or what the seismic design coefficient for the new braces and connections will be.

Figure 3-4: Failed brace connection at coal bin.

ACI 313, *Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials*, allows the use of a maximum 20% reduction in the weight of the stored product for seismic design. The reduction allowed in ACI 313 is based on small scale tests done in India that show a reduction in base shear due to intergranular movement of the stored bulk material during shaking. The practice of reducing the seismic mass based on the probability of the bin only being partially full during a seismic event is not the same as the reduction allowed in ACI 313.

Coal Bin Adjacent to Blast Furnaces

The coal bin associated with Blast Furnace #1 (Lat 36° 44′ 33" S, Long 73° 7′ 33" W) was investigated. This large steel structure is approximately 20 feet long by 75 feet in plan and 34 feet tall. The structural system of the coal bin is a combination of ordinary steel moment frames, ordinary steel brace frames and steel plate shear walls. Blast Furnace #1 was constructed in 1950 while Blast Furnace #2 was constructed in 1966.

Observations

The coal bin (Figure 3-5) associated with Blast Furnace #1 partially collapsed in the transverse direction and was being stabilized by series of pipe column braces. It is not totally clear what caused the failure. It appears the column bases at the left side of the columns in Figure 3-5 shifted to the left initiating a plastic hinging of the columns. There was some localized liquefaction/lateral spreading in the nearby area of the bins but there was no observed failure of the bin continuous footing. It is likely that the continuous footing shifted due to the observed liquefaction/lateral spreading. Also in discussions with the plant engineers, it was learned that the bins were originally designed using a seismic mass equal to the dead weight of the bin plus only 50% of the weight of the stored coal. The use of 50% of the coal weight was based on the average level of coal in the bin at any given time. The coal bins were approximately 75% full at the time of the earthquake. The plant engineers subsequently decided to design the replacement supports for 100% of the weight of the stored coal. It is not currently know what lateral seismic design coefficient the coal bin was designed for originally or what the seismic design coefficient for the repaired bin will be. For the repair, the coal bin structure was lifted as a whole with a 600-ton crane, stabilized, and supported with reinforced concrete columns. See Appendix C for coal bin drawings.

Figure 3-5: Blast furnace #1 coal bin.

ACI 313, *Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials*, allows the use of a maximum 20% reduction in the weight of the stored product for seismic design. The reduction allowed in ACI 313 is based on small scale tests done in India that show a reduction in base shear due to intergranular movement of the stored bulk material during shaking. The practice of reducing the seismic mass based on the probability of the bin only being partially full during a seismic event is not the same as the reduction allowed in ACI 313.

The practice of reducing the seismic mass based on the probability of the bin only being partially full during a seismic event is also common in the United States. This practice may need to be reviewed.

Recommendation regarding ASCE/SEI 7 and Retrofit of Existing Structures

Currently, the commentary to ASCE/SEI 7-05 repeats the reduction in product weight for a bulk storage silo allowed by ACI 313 for an elevated bin or silo and recommends a limit on the reduction of seismic mass for a ground supported storage silo. ASCE/SEI 7 Chapter 15 should add provisions incorporating the ACI 313 practice of allowing some reduction in seismic mass due to intergranular movement and provide more limitations on the permitted reduction of seismic mass based on the probability of the elevated storage structure only being partially full.

Also the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed.

Steel Shop

The Steel Shop (Lat 36° 44′ 50" S, Long 73° 7′ 37" W) suffered significant damage. The steel shop is a large steel industrial building where molten pig iron is transformed into basic steel billets, which are subsequently rolled and shaped into structural steel members and plates. Exterior views of the building are provided in Figure 3-6 and Figure 3-7. The steel shop is approximately 650 feet by 280 feet in plan, and 200 feet tall. The building utilized steel moment frames in the transverse direction and concentrically braced frames in the longitudinal directions. It was constructed in 1973. A large exterior conveyor transfers materials into the top of the building.

Figure 3-6: Exterior damage to steel shop and collapsed conveyor, view from east of the conveyor.

Figure 3-7: Damage to steel shop conveyor, view from the southwest.

Observations

An intermediate support of the conveyor failed resulting in collapse of the conveyor structure (Figure 3-6 and Figure 3-7). The cause of the support failure was not readily determined from our observations. In addition, a major column (A-3) that supports the crane used to transport heavy pig iron ladles (Figure 3-8), settled, probably due to localized liquefaction (see Figure 3-9). It was noted that there had been dewatering in the vicinity of this individual foundation over a long period. Column base anchor bolts also stretched (See anchorage damage observations later in this report). Damage to the Steel Shop resulted in a four-month shut down of the overall steel plant facility with production restarted in early July of 2010.

Figure 3-8: Pig iron ladle.

Figure 3-9: Column A-3 settlement.

Column A-3 experienced settlement on the order of 8 inches. As can be seen in Figure 3-9, the settlement caused significant deformation to the members framing into Column A-3. CAP conducted a soil investigation, which identified the supporting soil as sandy soil. Based on the Unified Soil Classification System, the sandy soil varied from SP to SM. The water table elevation was approximately 8 feet below grade. As a repair action, CAP has driven 28 micro piles under the Column A-3 footing. Some piles required modification due to interferences with existing foundations in the same area. Additionally, CAP increased the depth of the Column A-3 foundation by 20 inches using dowels (Figure 3-10). The dowels were added to reinforce the foundation so it would act as a solid block, minimizing possible cracking under it due to the differential pressure and to increase its flexural capacity.

An analysis of the deformed frame was made to understand the effect of the potential repairs. The originally planned repair procedure was altered after the analysis was made. In order to level the main crane rails, the main structural beam was jacked and shimmed. Afterwards, all surrounding structures (beams, braces, platforms, etc.) were released and re-attached.

Figure 3-10: Column A-3 footing repair.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

. In addition to life safety concerns, the design of an industrial facility often considers minimizing damage and downtime due to the value of the facility and the value of the output of the facility. Therefore, evaluation of liquefaction potential is acritical before an industrial structure or facility is sited or built. Also it appears the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed. No change to ASCE/SEI 7 is warranted. As discussed above, the facility changed its repair plan for column A-3 after conducting analysis of the deformed building structure. Repairing or retrofitting a settled main building support, such as Column A-3 discussed above, requires a careful assessment of the new load paths created by the settlement prior to detaching or cutting loose any of the affected structure.

Hot Rolling Mill

At approximately 2000 feet in length, the rolling mill is the longest structure in the steel plant. It is located adjacent to the water treatment plant. The high bay portion is approximately 50 feet in width and approximately 40 feet high. The total width varies between 50 and 150 feet. An aerial view of the building is provided in Figure 3-11. The building consists of steel truss moment frames in the transverse direction and concentrically braced frames in the longitudinal directions. Interior views of the Hot Rolling Mill are provided in Figure 3-12 and Figure 3-13.

Observations

Damage to the Hot Rolling Mill appeared to be confined to the column anchorages. It consisted of stretched and ruptured anchor bolts and displaced baseplates. A detailed discussion of observations relating to anchorage performance is provided later in this report.

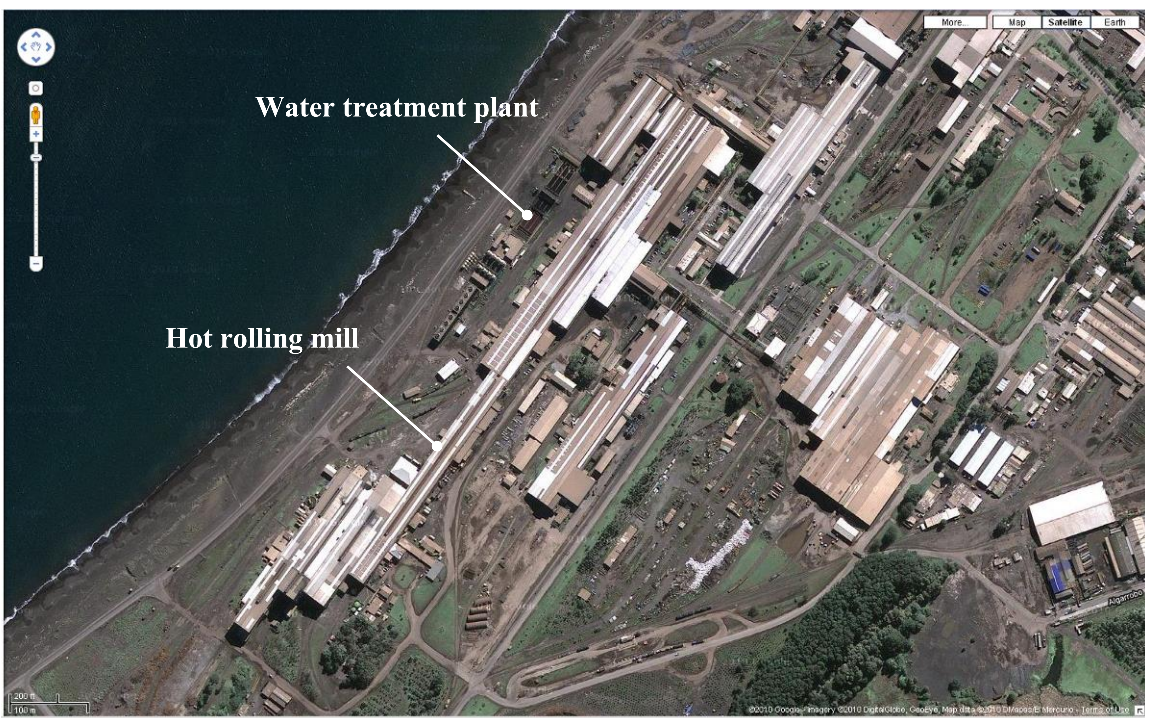


Figure 3-11: Aerial view of hot rolling mill (Source: GeoEye).

Figure 3-12: Interior view of hot rolling mill.

Figure 3-13: Interior view of hot rolling mill.

Bar Mill (New)

The New Bar Mill (Lat 36° 45′ 10" S, Long 73° 7′ 54" W) is the newest structure in the steel plant, having been constructed in 2008. An interior view of the building is provided in Figure 3-14. The building has a second floor that occupies its full plan area. The building is 1000 feet by 100 feet in plan and 80 feet tall. The building utilizes steel moment frames in the transverse direction and concentrically braced frames in the longitudinal directions. It was designed to the requirements of Chilean Standard NCh 2369-2003, *Earthquake Resistant Design of Industrial Structures and Facilities*. This standard is similar in its approach to the 1994 Uniform Building Code. The steel plant is located in an area where the seismicity is similar to that of UBC Zone 4.

Figure 3-14: Interior of New Bar Mill (built about 2005) at second level.

Observations

No significant damage occurred to the new Bar Mill. The performance of the structure was outstanding. The only noticeable damage was due more to workmanship than to design. Several of the interior column bases were grouted all of the way to the top of the base plate instead of the bottom of the base plate. Slight movement of the column bases during the seismic event caused the grout to crack and spall off (see Figure 3-15). Structurally, this is considered to be cosmetic damage that could have been avoided if the grouting had extended only to the bottom of the base plate.

Figure 3-15: New Bar Mill column base grout spalling.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The performance of the new Bar Mill highlights the success of Chile's current seismic code for industrial facilities. Because of the similarities between Chilean and U.S. seismic codes, no code changes appear to be warranted based on the observed performance of the new Bar Mill.

A common column grouting problem due to poor workmanship was highlighted above. Proper grouting of columns to avoid incidental damage to the grout cap should be highlighted in code commentary or training materials.

Water Treatment Plant

Adjacent to the Hot Rolling Mill is a water treatment plant consisting of two in-ground basins (Lat 36° 45′ 6" S, Long 73° 8′ 2" W) comprised of a number of reinforced concrete pits (see Figure 3-16). One of the basins is 200 feet by 70 feet in plan and 40 feet deep. The other basin, which was affected by liquefaction, is 150 feet by 60 feet in plan and 12 feet deep. Intermediate basin walls are located in the longitudinal direction. The pit is supported on a mat footing. It is estimated that the holding basins were constructed in the 1950s to 1960s. See Appendix C for drawings.

Figure 3-16: Water treatment plant in-ground basin.

Observations

A number of vertical oriented cracks developed in the walls of the water treatment plant pits (holding basins) due to liquefaction settlement at one end as shown in Figure 3-17. The cracks were significant, extended to the base of the structure, and compromised the ability of the structure to contain liquid. The repair consisted of installation of micro piles under the end of the basin that experienced settlement, and simple repair of the cracks. See Appendix C for repair drawings.

Figure 3-17: Cracks in pit wall due to liquefaction settlement.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

In addition to life safety concerns, the design of an industrial facility often considers minimizing damage and downtime due to the value of the facility and the value of the output of the facility. Therefore, evaluation of liquefaction potential is acritical before an industrial structure or facility is sited or built. Also it appears the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed.

Pier

The pier (Lat 36° 44′ 27" S, Long 73° 7′ 42" W) used by the CAP Steel Plant was pile supported and equipped with two large cranes (Figure 3-18) that moved on rails embedded in the concrete deck of the pier. The pier deck is approximately 60 feet wide and 1500 feet long in plan.

Figure 3-18: Pier with damaged crane (jumped tracks, wheel dolly supports failed).

Observations

The older of the two cranes suffered a failure of its wheel supports. The Industrial Assessment Team discussed the failure of the older crane with the plant engineers. The plant engineers noted that the older crane was not equipped with any mechanism designed to grab the rail to resist seismic movements (see Figure 3-19) and derailing and that the failure was a result of the lack of rail grabs. The second crane was of new construction (including a crane rail grab mechanism) and did not suffer any direct seismic damage. The new crane did, however, suffer indirect seismic damage. The crane was being used to load a container ship at the time of the seismic event. The container ship's crew headed to sea immediately after the earthquake to avoid a possible tsunami wave after they realized that a seismic event occurred (this is standard operating protocol). The ship's crew failed to detach the crane cable from the load and the crane's boom buckled as the ship left the pier. The cable eventually snapped as the ship left the pier.

Figure 3-19: Failed crane wheel.

The original pier was constructed in 1950 to a length of 880 feet and was extended in 1972 to reach a length of 1210 feet. The pier was supported on vertical piles with battered steel piles designed for the lateral loads. The piles have a concrete cover for corrosion protection. The battered piles were only embedded in the pile cap approximately 4 inches. All of the battered piles failed at the pile-to-cap connection and pulled out of the pile cap (see Figure 3-20). The pile-to-cap connection was only designed for axial load and did not consider the shears and moments developed at this fixed connection. See Appendix C for drawings.

Figure 3-20: Failed battered piles.

It was also reported that the sea floor moved vertically upward approximately 0.8 meters to 1.0 meters during the seismic event. The failure that occurred was in the new portion of the pier where the new crane was located. It is uncertain whether the connection failure was caused by earthquake forces and displacements or the ship leaving the pier while still connected to the new crane. The force generated by the release of the cable connecting the crane to the ship was large enough to buckle the crane boom. It is planned, pending issuance of permits, to dredge the harbor adjacent to the pier to restore the water depth to previous levels.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The scope of ASCE/SEI 7 does not include the design of industrial piers which are not open to the general public. However it does include special requirements for battered piles, which include designing the connection for overstrength forces and designing for shears and moments in addition to axial forces. Based on our observations, these considerations are very important.

General Conclusions and Recommendations

Because the steel plant contained structures constructed over a period of almost 60 years, different levels of damage were observed. Damaged concrete pedestals, damaged or failed connections, and support failures of a conveyor structure and coal bins were observed in the older structures. No damage (other than cosmetic grout damage) was observed in the newest structure built in the plant (New Bar Mill) designed and constructed to the latest Chilean codes and standards. The performance of the New Bar Mill is a strong indication that current codes provided adequate seismic designs for this event. No change to ASCE/SEI 7 is warranted. Liquefaction damage to a holding basin and significant settlement of a major crane column in the Steel Shop building were observed. Also the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed. Battered piles also failed at their connection to the underside of the concrete pier slab similar to failures observed in older construction during US seismic events. No change to ASCE/SEI 7 is warranted.

Later in this report, the seismic performance of anchorages in the CAP Steel Plant is presented.

Chapter 4
Assessment of ENAP Bio Bio Refinery

Facility Overview

The ENAP Bio Bio refinery (Lat 36° 46′ 38" S, Long 73° 7′ 9" W) located in Talcahuano was originally constructed in the 1960s with a major expansion in the 1990s (Figure 4-1 and Figure 4-2). The refinery covers a plan area of approximately 3/4 miles by 3/4 miles (See Figure 4-3). The refinery's capacity is 116,000 barrels per day. The refinery contains numerous significant industrial structures. The structures studied included various process units (including a coker unit), fin fan units, a wooden cooling tower, control buildings (for nonstructural damage) and a tank farm. In general, the assessment was approached by evaluating damaged structures in a given process unit or area. There were no reported fires during or after the earthquake which was quite fortunate given the level of damage. At the time of our visit, the refinery was expected to be out of operation for 6 to 9 months. We subsequently learned that the refinery was restored to operation by the end of July of 2010. Retrofit work continued after July on the derrick (drilling rig) on the coker tower, the water intake at the Bio Bio River, and the pump station for the return of treated water to the river.

Figure 4-1: ENAP Bio Bio refinery.

The location of the ENAP Bio Bio Refinery relative to the city of Concepción can be seen in Figure 2-1. Figure 4-3 shows the major areas of the refinery visited. Figure 4-4 provides a close-up view of the north end of the refinery, where most of the process (hydro cracking) equipment is located. Figure 4-5 shows the southern end, where the coke processing facility is located and where the derrick structure on top of the coker structure detached from the cutting deck. Figure 4-6 shows the western portion of the tank farm where large tanks with floating roof structures were investigated. A plot plan of the facility is shown in Figure 4-7.

Figure 4-2: ENAP Bio Bio refinery (view of typical process units).

Figure 4-3: Aerial view of ENAP Bio Bio refinery (Source: GeoEye).

Figure 4-4: Close-up aerial view of northern end of ENAP Bio Bio refinery (Source: GeoEye).

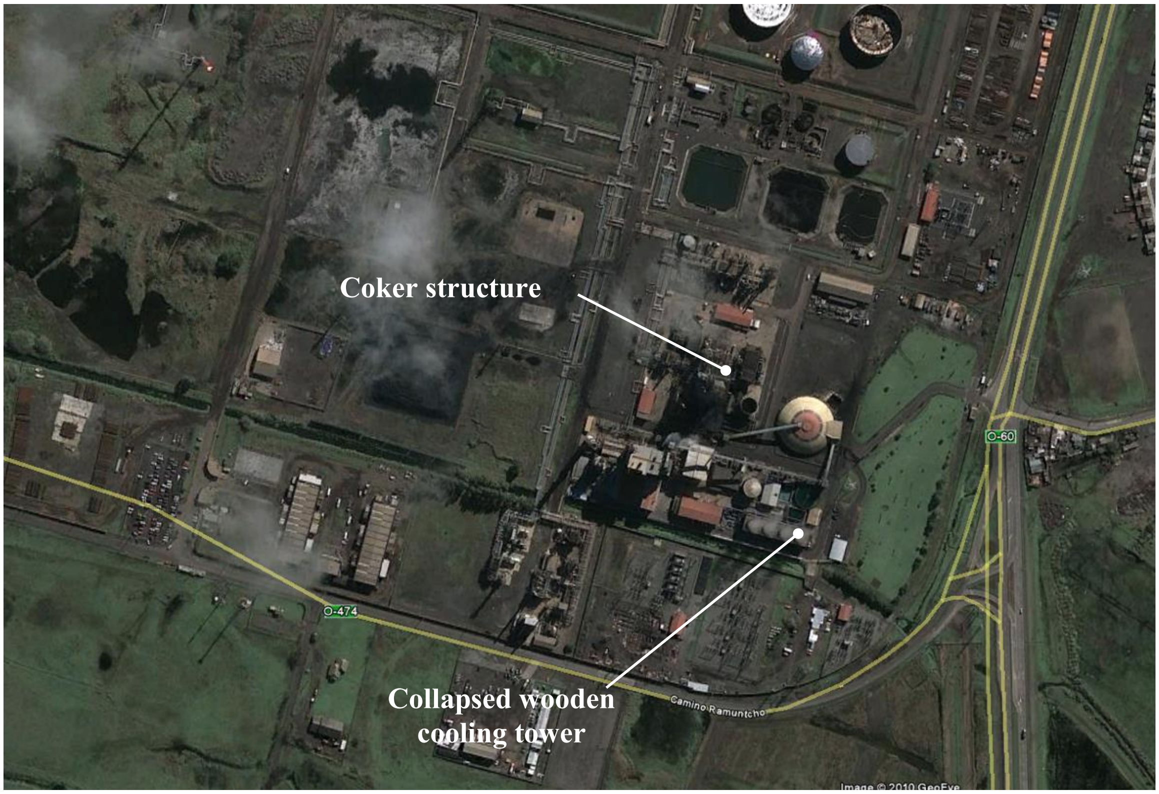


Figure 4-5: Close-up aerial view of southern end of ENAP Bio Bio refinery (Source: GeoEye).

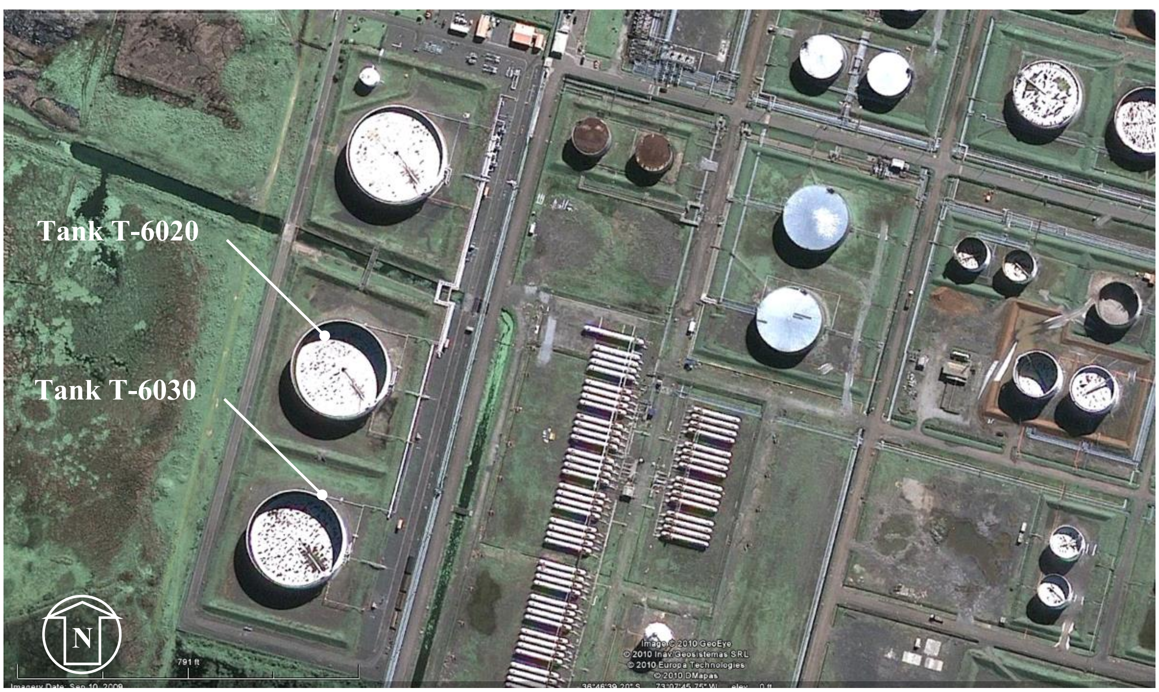


Figure 4-6: Close-up aerial view of western-most tanks at ENAP Bio Bio refinery (Source: GeoEye).

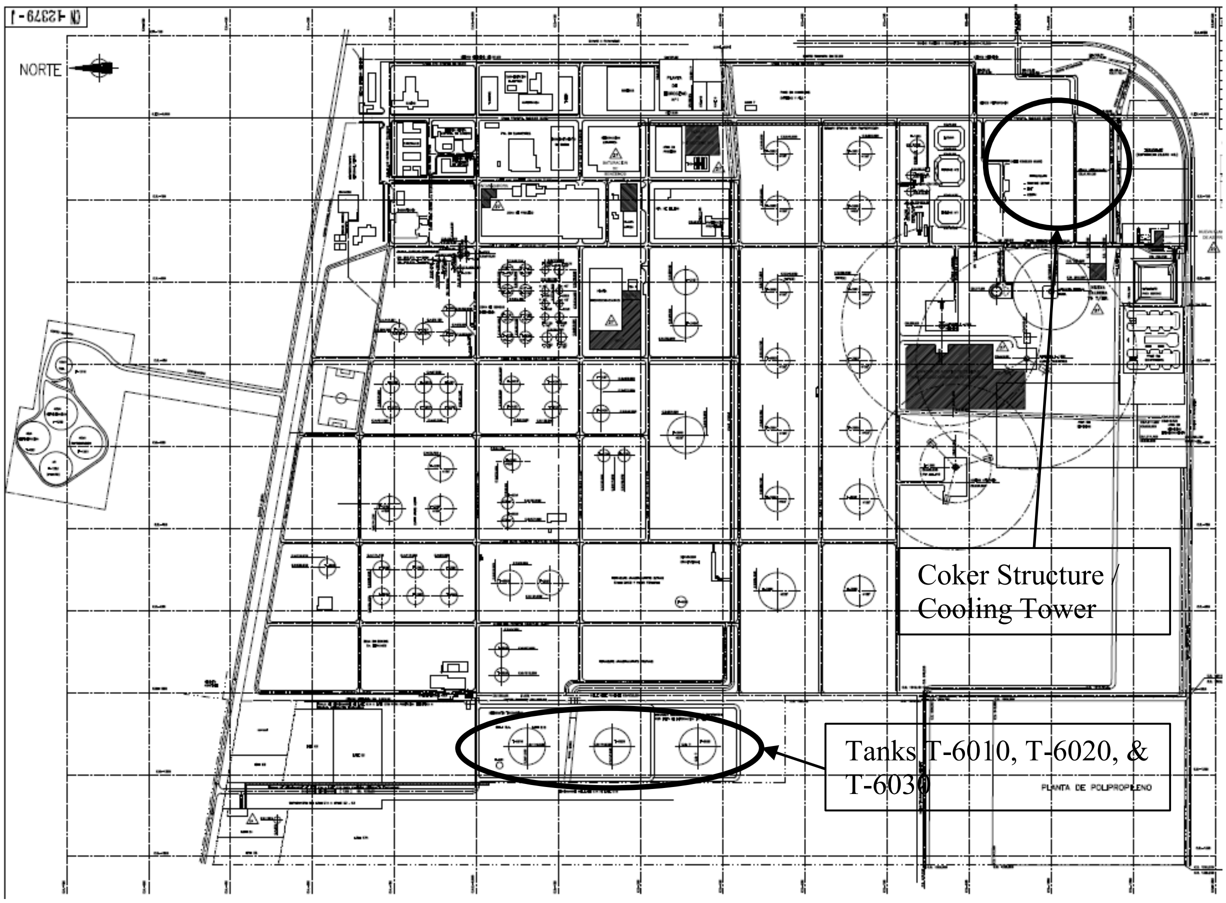


Figure 4-7: Plot plan of ENAP Bio Bio refinery (Source: Courtesy of ENAP).

Soil Conditions

The soil was observed to be a medium to dense sand with a loose surface layer. Areas of localized lateral spreading and settlement damage from liquefaction were observed primarily in the tank farm areas. No specific soil information (reports or borings) were provided to the Industrial Assessment Team.

Facility Design

The facility structures utilize reinforced concrete spread footings for their foundation support. Some the structures have large elevated concrete bases while concrete moment frames are used for others. A majority of the superstructures are a combination of steel moment frames and steel braced frames. Today in the United States, these would typically be classified systems as ordinary steel systems. There are also large number of vertical vessels and horizontal vessels interconnected with welded steel piping. The piping is supported by a variety of structural framing systems. The bases of the large structural columns, tanks and vertical vessels typically use tall anchor chairs and long anchor bolts to allow the bolts to stretch. Their design is very comparable to similar structures built in coastal California during the same time frames. (Appendixes D, E, F, and G contain English translations of ENAP engineering documents for selected facility design criteria.)

Estimated Response Spectra

The response spectra from the closest ground motion instruments to this location (within 10 km) are shown in Figure 2-2 and Figure 2-3 above. Because this was a subduction zone type earthquake, the ground motions measured at Figures 2-2 and 2-3 are considered to be representative of the ground motions experienced in this region. The actual ground motions experienced at the facilities visited may be different due to local geology.

Tank Farm Assessment

Much of the tankage (see Figure 4-8) in the ENAP Bio Bio Refinery tank farm was built in the 1990s. Because this construction predated the Chilean Standard NCh 2369-2003, *Earthquake Resistant Design of Industrial Structures and Facilities*, most if not all of the tanks in the tank farm were designed to the requirements of Zone 4 of API 650 Appendix E. The seismic requirements of API 650 Appendix E at the time of the tank farm construction paralleled those of the 1991 Uniform Building Code. The tanks utilized concrete ring walls to support the tank shell walls. They are typically self-anchored (no anchor bolts).

Figure 4-8: ENAP Bio Bio refinery tank farm.

The ENAP Bio Bio Refinery personnel who accompanied the Industrial Assessment Team noted that only minor damage occurred in the Tank Farm area. Specifically, three external single deck pontoon floating roof tanks experienced some problems. No elephants foot buckling was noted.

Two of the floating roof tanks were operating with no seismic freeboard when the seismic event occurred. Crude oil sloshed out approximately 270 degrees around these two tanks. One of the floating roofs was completely undamaged and the other floating roof sustained some damage to pontoons. Neither roof was in danger of sinking. No fires occurred in these tanks. Only two of the three tanks were accessible to the Industrial Assessment Team.

Observations

Figure 4-9 and Figure 4-10 show Tank T-6020, a 220-ft diameter by 48-ft high external floating roof tank built in 1993 to the provisions of API Standard 650 8th Edition. The tank was designed to the seismic provisions of Appendix E of API 650 based on Zone 4 with an importance factor of 1.0.

Figure 4-9: Evidence of sloshing in tank T-6020.

Figure 4-10: Tank T-6020.

The tank was operating at its maximum liquid level of 45'-0. The actual height of the sloshing wave was not observed. The convective period of the tank is approximately 11 seconds, which significantly exceeds the period range included in the preliminary response spectra shown in Figure 2-2 and Figure 2-3. Using the measured PGA of 0.4g, the procedures in current API 650 Appendix E, and a range of T_L of 4 to 8 seconds, a very approximate range of sloshing wave heights is 3.5 to 7 ft.

Figure 4-11 indicates that the floating roof rolling ladder stayed on its tracks during the seismic event. This may be an indication that the sloshing wave height was in the lower end of the estimated range. The floating roof's performance was judged to be excellent.

Figure 4-12 shows the condition of the floating roof rim space and weather shield. The condition of the weather shield is excellent. The condition of the floating roof seal in the rim space could not be determined. The original seal installed in 1993 was a vapor-mounted foam-filled seal. It could not be determined if the seal in the tank was the original type installed in 1993.

Figure 4-13 shows the approximate 2 inch tilting settlement of Tank T-6030 due to soil liquefaction. Figure 4-14 shows the lateral spreading near Tank T-6030 due to the soil liquefaction. Tank T-6030 is a 220-ft diameter by 48-ft high external floating roof tank built in 1995 to the provisions of API Standard 650 8th Edition. The tank was designed to the seismic provisions of Appendix E of API 650 based on Zone 4 with an importance factor of 1.0.

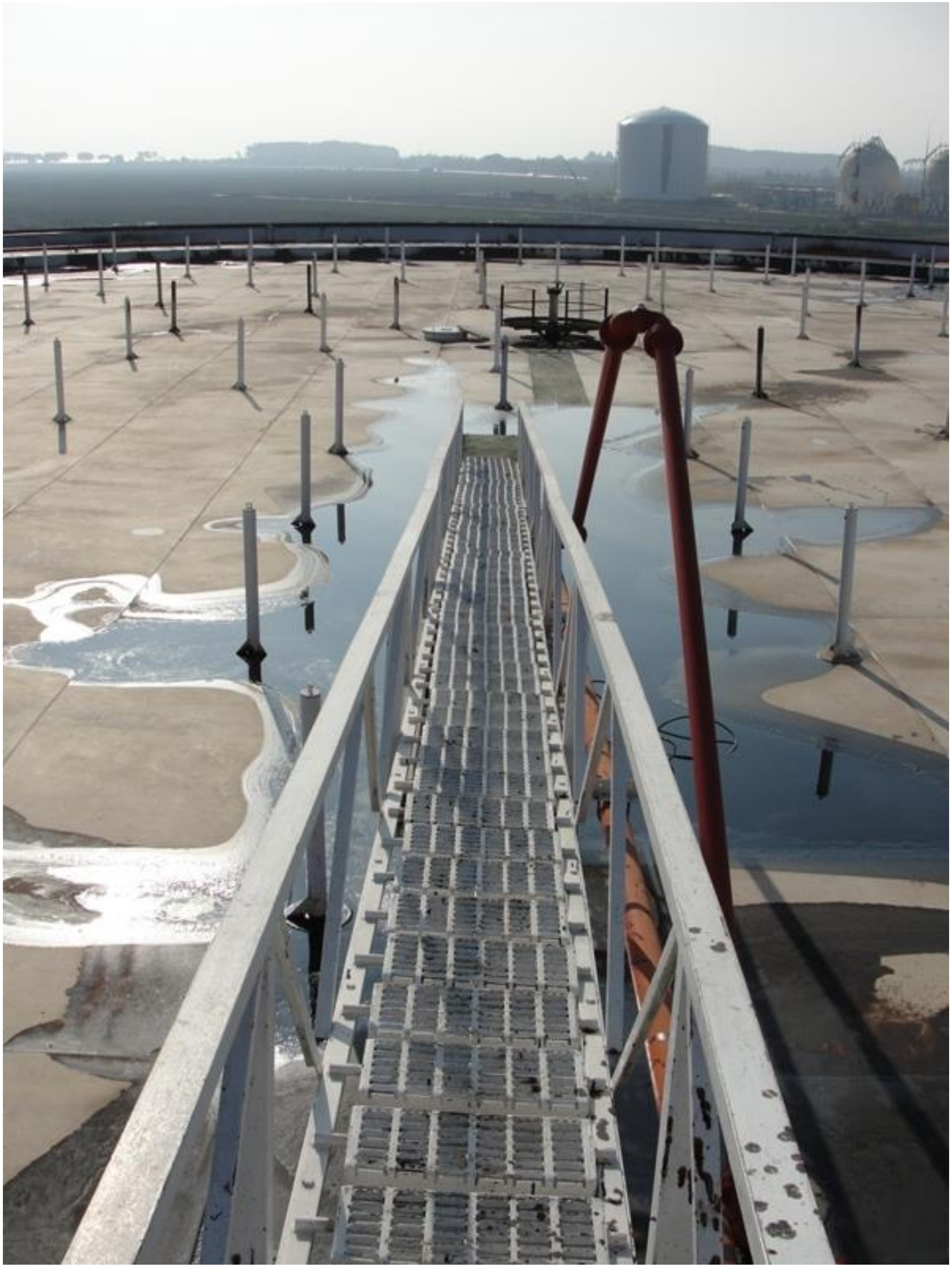


Figure 4-11: Floating roof rolling ladder.

Figure 4-12: Tank floating roof seal damage.

Figure 4-13: Liquefaction settlement adjacent to tank (tilting).

Figure 4-14: Lateral spreading.

Tank T-6030 was only 25 percent full at the time of the seismic event. Unfortunately, the deformation of the tank bottom due to settlement from soil liquefaction resulted in a leak in the bottom plate. The leak was most likely from a cracked fillet weld in a bottom weld joint but this could not be confirmed.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The observations in the Tank Farm area seem to confirm the provisions used in ASCE/SEI 7 for tanks. Proper seismic freeboard and flexible piping connections reduce or eliminate damage to tanks during seismic events. Ground-supported storage tanks designed using the procedures in API 650 seemed to perform quite well.

Evaluation of liquefaction potential is critical before a structure is sited or built. Also it appears the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed.

Coker Unit

The primary structure in the coker unit is the coke drum structure. The coke drum structure, shown in Figure 4-15 (Lat 36° 47′ 7" S, Long 73° 7′ 8″ W), is made up of two coke drums (pressure vessels) supported on an extremely heavy elevated concrete pedestal and stabilized by a surrounding steel platform structure. A steel derrick is attached to the top of the steel platform structure (the cutting deck). The derrick and supporting structures together (shown in Figures 4-16 and 4-17) have a height in excess of 300 feet. The coke drum structure was constructed in 1995-1996. The coke drum structure and the drilling derrick are the tallest structures in the refinery.

Figure 4-15: Coke drum structure.

Figure 4-16: Derrick on top of coke drum structure.

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Figure 4-17: Derrick leg attachment.

Observations

The very heavy wall coke drums and the supporting concrete pedestal appeared undamaged by the seismic event. The drilling derrick atop the coker unit experienced failure of the 16 approximately 1-1/2 inch diameter ASTM A325 bolts (four per leg, see Figure 4-18) that connected it to the pedestal. The bolts were reported to have failed due to thread separation from the bolt shank (see Figure 4-19), which may have been precipitated by corrosion in combination with overload. The derrick continued to move (from either wind loads or aftershock loads) for a period of time following the earthquake and workers bravely attempted to secure it by jamming pieces of the fracture bolts into the bolt holes. Eventually, the baseplates were welded to the supporting structure but not before the derrick had rotated counterclockwise some three to four inches. Had it rotated another 6 inches it would have completely lost support on one end and toppled over onto the process unit. The resulting damage would likely have been catastrophic. It is our understanding from plant personnel that another refinery in Chile suffered similar derrick anchorage damage to its coke drum structure.

The permanent shift of the derrick resulted in a significant misalignment between the coke drum and the drill attached to the derrick (Figure 4-20). This misalignment put the coker unit out of operation since the drilling of the coke is an essential part of the operation. As part of the repair, the derrick structure was retrofitted to the NCh 2369 Chilean code and supplemental A325 bolts were added to the baseplate anchorage for each of the derrick legs.

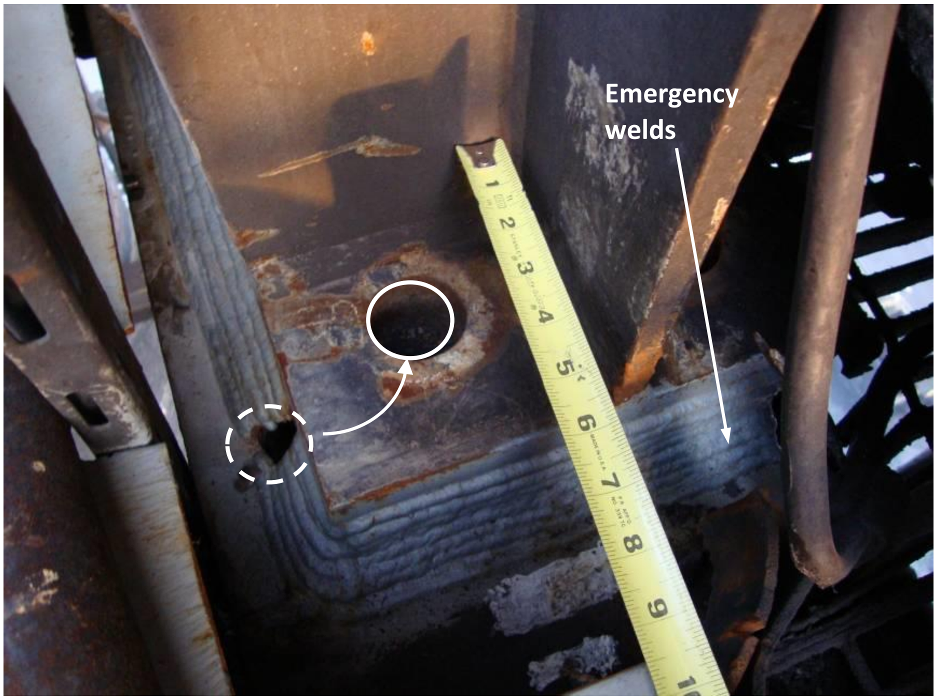

Figure 4-18: Shift of derrick leg counterclockwise.

Figure 4-19: Failed derrick attachment bolt.

Figure 4-20: Resulting misalignment of drill and coke drum.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The cause of the bolt failures should be closely investigated since this structure represents a structure type common in U.S. refineries and because derrick toppling could have catastrophic consequences. It is likely that the increased forces on the derrick resulting from being supported on top of the coker unit, as defined in ASCE/SEI 7 Equation 13.3-1 or 13.3-4, were not considered in the derrick attachment design. Since this requirement and a similar requirement in the 1997 UBC had probably not been introduced into the building code at the time the coker unit was designed and constructed. It is recommended that this structure be evaluated in detail to determine the likely range of forces that caused failure of the anchor bolts and that Equation 13.3-1 or Equation 13.3-4 be assessed to determine its adequacy for this situation. Derrick anchorages are obviously highly vulnerable to strong earthquake ground motions. Also, existing refineries need to be aware of their vulnerability and retrofit actions taken because of the serious outcome of derrick toppling.

If end plate connections are used to connect the derrick to the tower as was the case here, the connection should be designed for an amplified load recognizing the limited ductility associated with this type of connection. Alternatively, use of a ductile chair-type connection with adequate stretch length and ductile bolts (ASTM A36) would not necessarily require amplification of the demand. (For a typical example of this, see Figure 8-21 in Chapter 8.)

Cooling Tower

As seen in Figure 4-21 and Figure 4-22, a wooden cooling tower (Lat 36° 47′ 13" S, Long 73° 7′ 17″ W) adjacent to the coker unit collapsed during the seismic event. The assessment team was not able to get close enough to the tower to determine what precisely initiated the collapse. We understand that this structure was subsequently replaced with a fiberglass unit.

Figure 4-21: Collapsed wooden cooling tower (from coker unit).

Figure 4-22: Collapsed wooden cooling tower (ground level).

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Pictures of the collapsed wooden cooling tower are included in this report to highlight the fact that this is a typical failure in refineries during major seismic events. Wooden cooling towers have not performed well in seismic events. This may be due to poor design, specifically, non-concentric bracing that induces bending in the columns, but also to lack of maintenance and the subsequent deterioration associated with continuous operation over many years. Table 15.4-2 of ASCE/SEI 7 assigns an R value of 3.5 to steel, concrete, and wooden cooling towers. In contrast, the CAP Acero Steel Plant uses a reinforced concrete cooling tower. The concrete cooling tower experienced no damage. The refinery cooling tower design should be assessed and its seismic capacity determined and compared with estimated demand. Based on this assessment, it appears that the values in Table 15.4-2 of ASCE/SEI 7 for wooden cooling towers should be reviewed during the next code cycle with the goal of possibly reducing the R value or prohibiting their use for situations where they would be assigned to SDC D, E or F.

Bolted Stack

A self-supporting bolted stack (Lat 36° 46′ 42” S, Long 73° 7′ 10” W) shown in Figure 4-23 suffered anchorage failure and toppling of the upper 40-foot bolted portion during the earthquake. The bolts joining the stack sections were specified to be ASTM A325. However, an investigation revealed that DIN EN ISO 898 Grade 8.8 bolts were used instead.

Figure 4-23: Bolted stack (top portion of stack is missing).

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Observations

The ENAP Bio Bio Refinery contains many stacks. All but one of the stacks are welded. None of the welded stacks suffered any damage other than stretched anchor bolts during the seismic event (anchor bolt performance is discussed separately later in this report).

The connection bolts of the end-plate bolted connection of the upper section of the only bolted stack in the ENAP Bio Bio Refinery failed causing the upper section to fall, crushing an unoccupied pickup truck and blocking a road. Fortunately, the stack section did not hit any other structure in the refinery when it fell. Investigation of the failure revealed that the bolts (Figure 4-24) in the failed connection had suffered severe corrosion and thread stripping. The corrosion contributed to the failure of the bolts under load.

Figure 4-24: Failed ISO 898 8.8 stack bolts exhibiting thread stripping.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Corrosion is a common problem in most industrial facilities. This is especially true in refineries. The Industrial Assessment Team recommended to ENAP personnel that if the bolted stack is repaired, the bolted connections should be inspected on a regular basis. Retrofit details, just as with details used for new construction, used in industrial facilities must consider the possibility of corrosion.

Process Units (Hydrocracking, Isomerization, and Catalytic Cracking Units, Ethylene Plant)

The process units in this refinery are typical of those found in U.S. refineries. They consist of primary process vessels, interconnected piping, and reinforced concrete and steel support structures. The oldest units were erected in the 1960s while the most recent construction dates from the early part of this decade (see Figure 4-25).

The primary structure in the Hydrocracking Unit is the large hydrocracking vessel, which is supported on concrete pedestal columns. Also included are fired heaters (furnaces) which in this installation suffered pedestal damage (see Figure 4-26, Figure 4-27, and Figure 4-28). (Appendix H contains a soil report for the Hydrocracking Unit.)

Figure 4-25: Process units.

Figure 4-26: Fired heater (part of topping and vacuum I).

Figure 4-27: Fired heater unit supports.

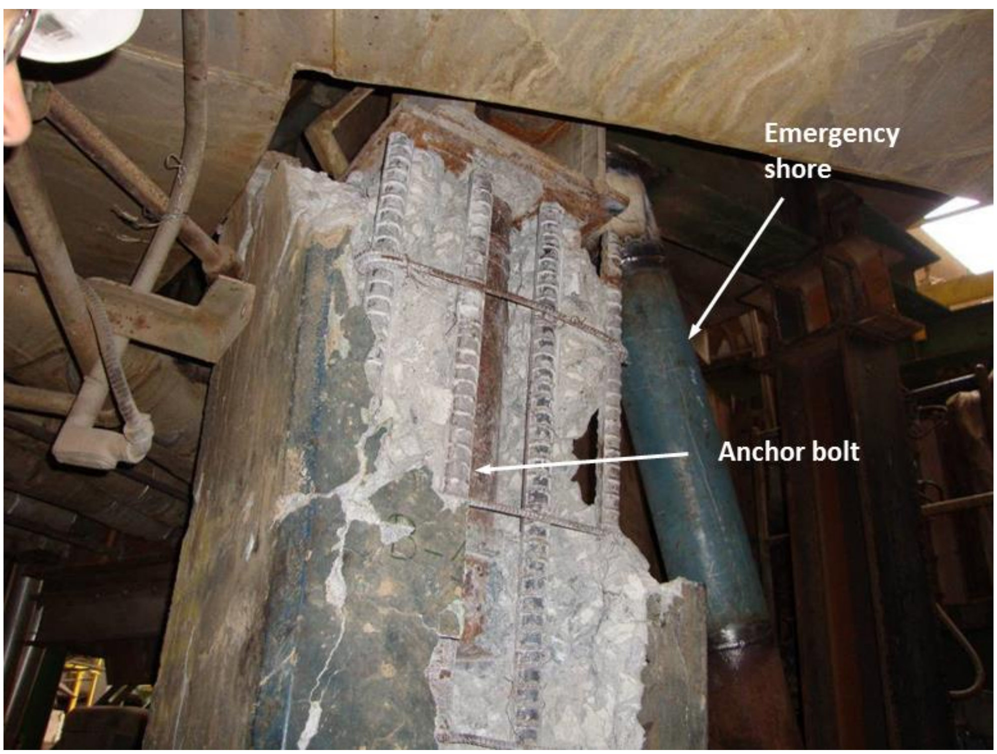

Figure 4-28: Damage to fired heater supports.

Observations

Many of the pedestal supports showed evidence of having sustained large compression forces resulting in spalling of the concrete below the baseplate (see Figure 4-28). This structure was likely designed in the 1960s or 1970s and the column pedestals were sized with four bolt diameter edge distances and provided with confinement steel consisting of #4 hoops at one foot on center, typical of detailing in that era. Pedestals designed later with larger bolt edge distances and closer tie spacing performed well. Similar damage was observed in many heater vessels supports of older vintage.

A discussion of anchorage performance is provided later in this document.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Existing installations may need to be retrofitted either by jacketing (e.g., fiberglass wrap) or augmentation of the pedestal with new reinforced concrete.

HDT Unit Pipe Rack and Supported Fin Fan

The Fin Fan provides air cooling for the Hydrotreating Unit (Lat 36° 47′ 7" S, Long 73° 7′ 15″ W). The pipe rack structure supporting the Fin Fan was built in 1996. It is a reinforced concrete frame. The Fin Fan is supported on steel column with dense concrete fireproofing.

Observations

The Fin Fan support columns at each end of the support structure shown in Figure 4-29 collapsed during the seismic event. The support columns were structural steel encased in concrete fire proofing. They were bolted at the base with an end plate connection to the concrete frame and at the top to the Fin Fan frame (Figure 4-30). The bolted connections of these supports failed in many locations allowing the supports to rotate and displace off of their supporting beams. This was a particularly difficult structure to repair because of the large crane cantilever distances and heavy weights. It was necessary to import a very large crane to the site to make the repairs.

Figure 4-29: Collapsed HDT unit fin fan.

Figure 4-30: Displaced fin fan unit support.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

It is likely that the increased forces on the Fin Fan resulting from being supported on top of the concrete support frame, as defined in ASCE/SEI 7 Equation 13.3-1, were not considered in the Fin Fan attachment design. Since this requirement and a similar requirement in the 1997 UBC had not been introduced into the building code at the time the Fin Fan was designed and constructed. It was typical practice to design these elevated Fin Fan structures without consideration for the amplification caused by response of the supporting pipe rack structure. In addition, the detailing of the column baseplate connections did not seem to be sufficiently robust. It is recommended that this structure be assessed analytically and Equation 13.3.1 be checked to determine if the predicted demand is adequate.

Consequential Damage Caused by Piping Inertial Forces and Relative Displacements

One area of concern in the design of refinery facilities is relative displacements between points of attachment of large piping systems. These relative displacements can cause very large reaction forces at restraint points, which must be accounted for.

Observations

In some cases, large piping runs between vessels and other structures sustained significant distortions due to the relative movement of the attachment points (see Figure 4-31). This caused consequential damage to the piping supports. Figure 4-32 shows damage to a support anchorage, probably resulting from movement of the supported piping and the short column length of the supporting frame. The horizontal vessel shown in Figure 4-33 is supported high in a concrete structure. Damage to the vessel support was caused by forces induced in the vessel by movement of a large pipe attached with a flanged connection. The pipe in question is attached at its other end near the top of an adjacent tall vessel.

Figure 4-31: Large pipe supports with significant lateral displacement.

Figure 4-32: Damage to pipe support anchorage.

Figure 4-33: Damage to vessel support caused by piping-induced lateral forces.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Relative displacement effects need to be considered in the design of support structures for distributed systems. More definitive guidance on how these effects should be taken into account in design is needed.

General Conclusions and Recommendations

Stretched anchor bolts of many vessels, concrete pedestal failures, sheared bolts, tank sloshing, and liquefaction settlement were observed. At least two major failures may have been related to the connection design forces not being amplified due to one structure being supported by another. There were indications of significant pounding of supports from vertical acceleration. Current industrial practice is to add additional reinforcing hoops in the top of the pedestal to prevent shattering of the pedestal from pounding. It also appeared that many older concrete support pedestals were too small for the large shear forces imparted by the supporting structures based on the experience of the authors.

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Chapter 5
Assessment of Abastible San Vicente Liquefied Petroleum Gas (LPG) Terminal

Facility Overview

The Abastible San Vicente LPG Terminal (Lat 36° 46′ 22" S, Long 73° 8′ W), located near Talcahuano (Figure 5-1), specializes in the management of Liquefied Petroleum Gas (LPG). The terminal receives bulk liquids from tankers through its modern pipeline system mounted on piles in a pier that extends approximately 2200 meters into the Bay of San Vicente. A plot plan of the facility is shown in Figure 5-2. Construction of the facility began in 2003.

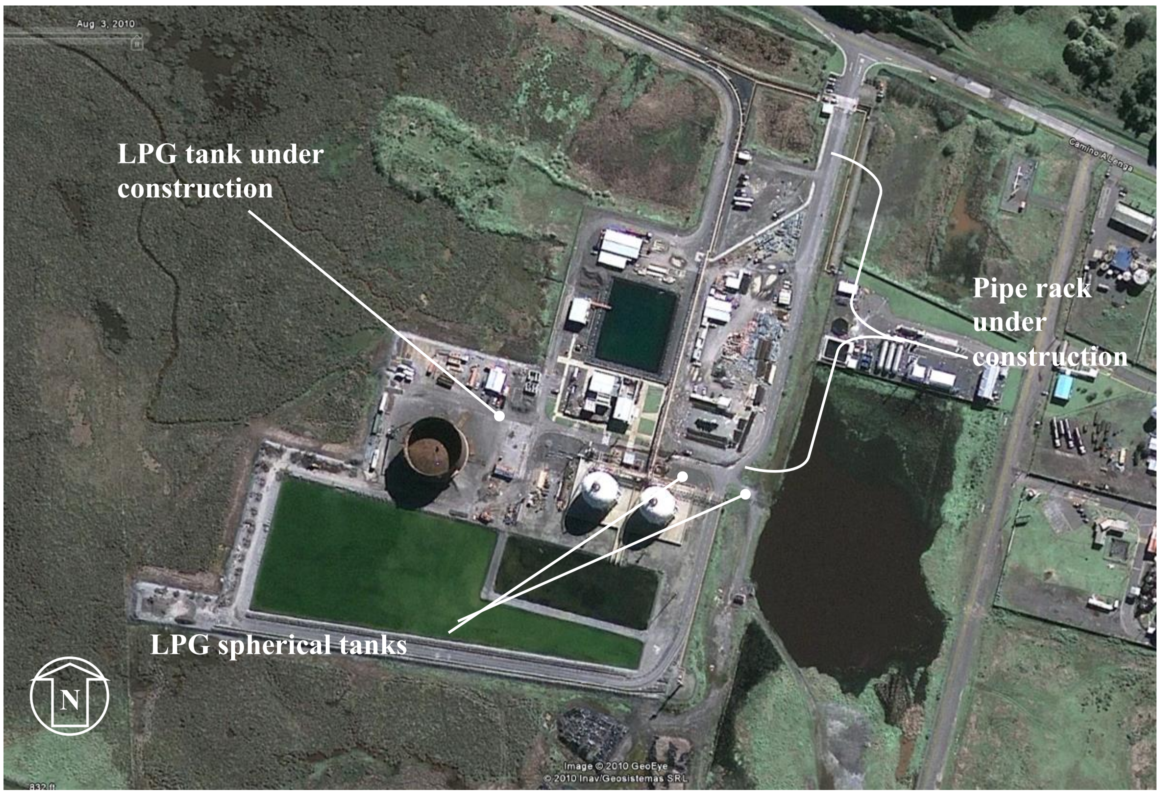

Figure 5-1: Aerial view of Abastible San Vicente LPG facility (Source: GeoEye).

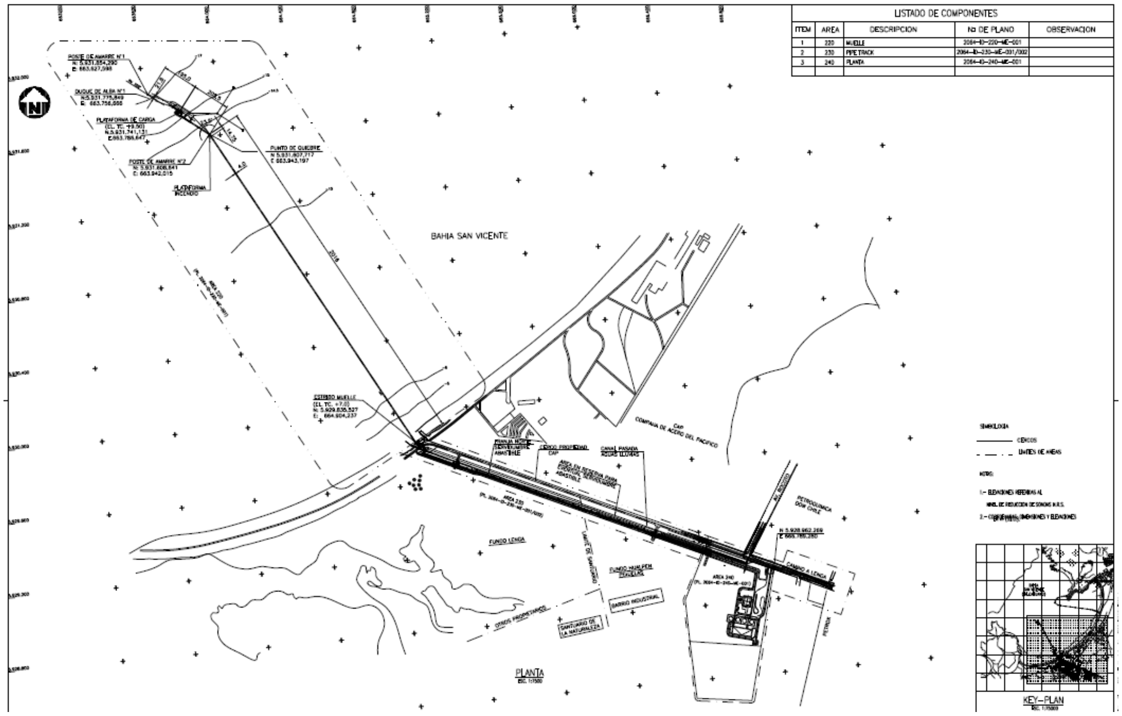

Figure 5-2: Plot plan of Abastible San Vicente LPG Terminal. (Courtesy of Abastible)

Soil Conditions

Soil information and the soil improvement plan used are only available for the LPG tank location. The original soil at the tank site is described in Table 5-1. The elevation of the water table is unknown. The partial soil report made available to the Industrial Assessment Team indicates that the soil beneath the LPG spheres was improved in a manner similar to that used for the LPG tank location.

Table 5-1. LPG Tank Soil Description

Soil Layer	Depth	Description
I	0.3 m	Topsoil, brown silty clay, moderate plasticity with abundant roots
II	0.3 m – 3.1 m	Grayish brown, high plastic clay
III	3.1 m – 16.5 m	Dark gray coarse sand with gravel
IV	16.5 m – 18 m	Saturated dark gray medium sand

The soil improvement consisted of removing the soil beneath the site of the LPG tank to a depth of approximately 3.1 m (depth of sand). The removed soil was replaced by sand taken from Soil layer III (Table 5-1) and compacted to a relative density of 80 percent.

Estimated Response Spectra

The response spectra from the closest ground motion instruments to this location (within 10 km) are shown in Figure 2-2 and Figure 2-3 above. Because this was a subduction zone type earthquake, the ground motions measured at Figures 2-2 and 2-3 are considered to be representative of the ground motions experienced in this region. The actual ground motions experienced at the facilities visited may be different due to local geology.

LPG Spheres Assessment

The Abastible LPG facility contains two 31,472 barrel (bbl) LPG spheres (Figure 5-3) constructed in 2004 when the plant was originally built. Both spheres were full of product at the time of the seismic event. (Appendix I contains drawings of the LPG spheres and tank.)

Figure 5-3: 73.5-ft diameter LPG sphere.

Structure Description

The two LPG spheres (Lat 36° 46′ 22″ S, Long 73° 8′ W) are 73.5-ft in diameter and were designed to the ASME BPVC Section VIII, Division 1. The design pressure of the spheres is 110 psig and the specific gravity of the stored product is 0.625. The thickness of the vessels ranges from 1.237 in. to 1.496 in. The fundamental period of the spheres is 0.355 seconds. Other dimensional information is shown in Figure 5-4. The sphere, supporting columns, and bracing were designed to the seismic requirements of Zone 3 of the Chilean Standard NCh 2369-2003 using a response modification factor R of 3 and an importance factor of 1.2. Zone 3 of the Chilean Standard NCh 2369-2003 roughly corresponds to Zone 4 of the 1997 UBC without near field factor increases.

The bracing system is an ordinary concentrically braced frame. The bracing (Figure 5-5) is made up of built-up 12.8 in. x 5/8 in. thick square tubes fabricated from A572 Grade 50 steel plate. The wing plates are 1.25 in. thick and penetrate the tubular columns. The wingplates are welded to internal circular diaphragms in the columns (Figure 5-6).

Observations

No damage of the LPG spheres, columns, bracing, or foundations were observed.

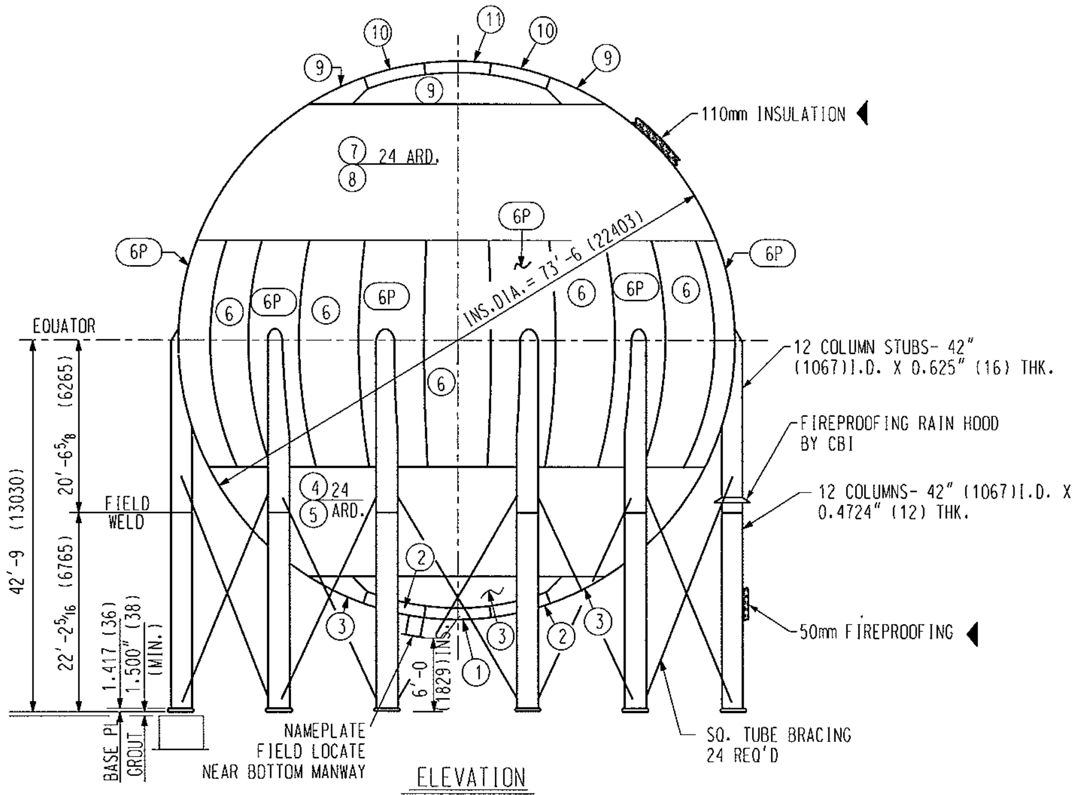

Figure 5-4: Sphere geometry. (Source: Courtesy of CB&I Inc.)

Figure 5-5: Support columns and bracing.

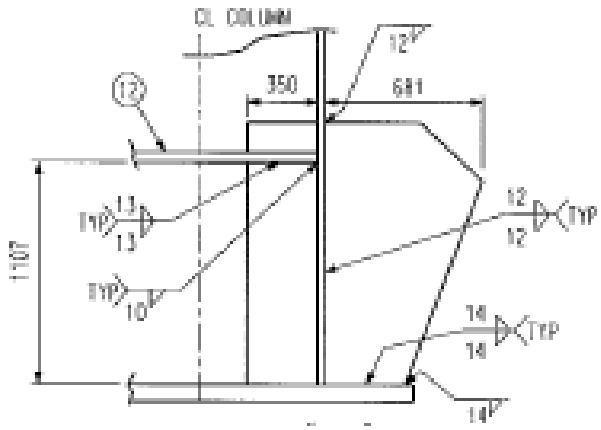

Figure 5-6: Wingplate attachment detail. (Courtesy of CB&I Inc.)

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The seismic design forces and detailing used in the design of the LPG Spheres is very similar to what is currently specified in ASCE/SEI 7. Ordinary concentrically braced frames performed well during the seismic event. Therefore no code changes appear to be warranted based on the observed performance of the LPG Spheres.

LPG Tank Assessment

The Abastible LPG facility contains one 50,000 m^3 refrigerated LPG tank (Figure 5-7) constructed in 2010. The LPG tank had just completed testing and was being insulated at the time of the seismic event. The LPG tank was empty at the time of the seismic event.

Figure 5-7: 123'4″ diameter x 99'2″ LPG tank (under construction).

Structure Description

The LPG tank is a 123′4″ diameter by 99′2″ high single wall dome roof refrigerated storage tank (Lat 36° 46′ 21" S, Long 73° 8′ 4" W) with a suspended deck designed to the requirements of API 620 Appendix R. The design pressure of the tank is 1.5 psig and the specific gravity of the stored product is 0.582 (propane) and 0.601 (Butane). The tank is designed to store LPG at –49 °F. The thickness of the tank ranges from 0.315 inches to 0.935 inches. The fundamental period of the tank is 0.30 seconds and the sloshing period of the tank is 6.44 seconds. Other dimensional information is shown in Figure 5-8. The tank was designed to the seismic requirements of Zone 3 of the Chilean Standard NCh 2369-2003 using a response modification factor R of 4 and an importance factor of 1.2. Zone 3 of the Chilean Standard NCh 2369-2003 roughly corresponds to Zone 4 of the 1997 UBC without near field factor increases.

Observations

Because the LPG tank was empty at the time of the seismic event, no conclusions can be drawn on the possible seismic performance of the tank. However, the performance of the improved soil beneath the LPG tank can be evaluated. The Abastible facility sits on liquefiable soil. As can be seen in Figure 5-9, lateral spreading due to soil liquefaction did occur near the tank. The lateral spreading stopped at the point where the improved soil began. The soil improvement must be viewed as a success.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The seismic design provisions in ASCE/SEI 7 and its referenced documents for nonbuilding structures, such as tanks and spheres, appear sound. Soil improvement to prevent damage from liquefaction performed well during the seismic event. Therefore, soil improvement should be one of the options considered to mitigate the effects of liquefaction.

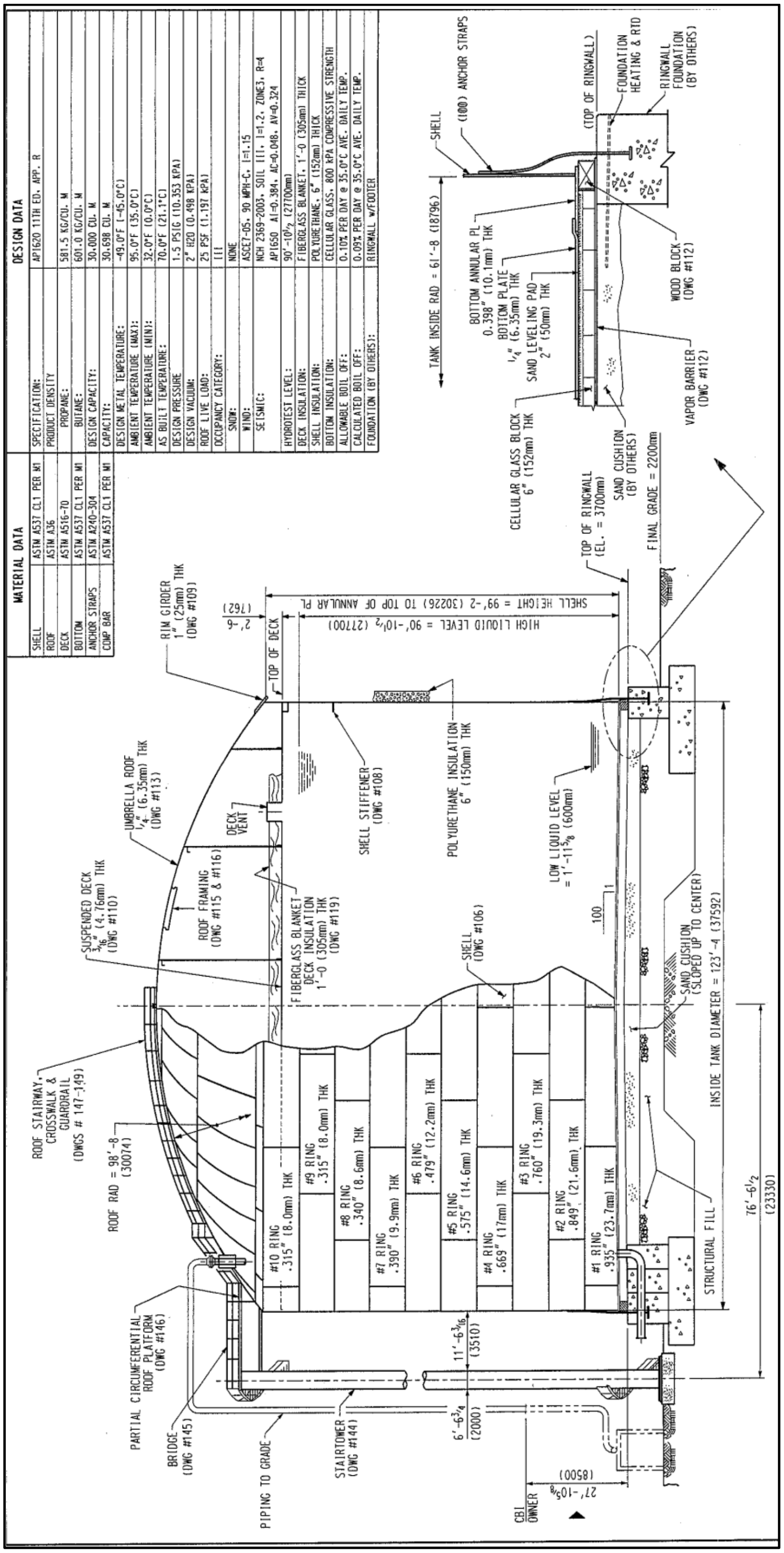

Figure 5-8: LPG tank geometry. (Source: Courtesy of CB&I Inc.)

Figure 5-9: Lateral spreading near LPG tank arrested by improved soil. (Courtesy of CB&I Inc.)

Pipe Rack Assessment

The pipe rack shown in Figure 5-10 was under construction at the time of the seismic event.

Figure 5-10: Pipe rack (under construction).

Observations

The pipe rack was constructed just beyond the outside edge of the improved soil of the tank area. The spread footing foundations of the support columns (Figure 5-11) of the pipe rack were observed to have laterally displaced several inches due to lateral spreading of the soil toward the adjacent

marsh areas. The lateral displacement was greater nearer the marsh, creating significant horizontal relative displacements between the pipe rack footings. Care must be taken to ensure that soil improvements cover the entire area of potential liquefaction that the structure is constructed upon.
.

Figure 5-11: Effect of lateral spreading on pipe rack column.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Evaluation of liquefaction potential is critical before a structure is sited or built. Also it appears the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed.

Wall Assessment

The wall shown in Figure 5-12 was under construction at the time of the seismic event. The wall also serves as a foundation for one side of a steel pipe support structure. The lightly loaded steel pipe support structure shown in Figure 5-12 used an adjustable double-nut ungrouted base plate support detail commonly used in industrial facilities.

Observations

The wall was constructed near the outside edge of the improved soil of the tank area. Although not obvious in Figures 5-12 and 5-13, the wall was observed to have settled a few inches due to liquefaction of the soil. The pipe support structure settled with the wall.

Figure 5-12: Settlement of wall (under construction).

Figure 5-13: Settlement of wall (additional view).

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Evaluation of liquefaction potential is critical before a structure is sited or built. Also it appears the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed.

Control Room Assessment

The control room was located on the second floor of a combined control room/administration building. The structural system used in the design of the building was not known but it appeared to be either shear wall (concrete or masonry) or braced frame construction.

Observations

The control room suffered no structural damage and only minor nonstructural damage. According to the plant personnel, the only nonstructural issue they had during the seismic event was toppling of the computer monitors (Figure 5-14).

Figure 5-14: Control room computers.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Nonstructural elements of the control room performed well during the seismic event. Therefore no code changes appear to be warranted based on the observed performance in the control room.

Electrical Room and Mechanical Room Assessments

The electrical and mechanical rooms were located in one story at grade buildings. The structural systems of these building appeared to be shear walls (concrete/masonry) with a steel beam roof system. The buildings appeared to be about 25 feet by 50 feet in plan. All equipment appeared to be anchored. Elevated electrical conduit and piping was supported by suspended threaded rods that were not laterally braced. These elements appeared to perform well.

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Observations

The electrical and mechanical rooms did not have any structural damage and suffered only minor nonstructural damage occurred in the electrical room as shown in Figure 5-15 and Figure 5-16. In Figure 5-15, hanging threaded rods that supported an item of suspended equipment were significantly bent. The equipment was not laterally supported. In Figure 5-16, a suspended light fixture was damaged by swinging and impacting an adjacent item.

Figure 5-15: Bent support rod of suspended equipment support.

Figure 5-16: Damaged light fixture.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Nonstructural elements of the mechanical and electrical rooms performed well during the seismic event. The only damaged items were suspended equipment and lighting that were not laterally braced. These items would have required bracing per ASCE/SEI 7. Therefore no code changes appear to be warranted based on the observed performance in the mechanical and electrical rooms.

Pump Room Assessment

The pump room (Figure 5-17) is located in one story building that is about 30 feet by 50 feet in plan. The floor slab is about 15 feet below grade and the roof is about 15 feet above grade. The structural system of this building appeared to be a shear wall (concrete/masonry) with a steel beam roof system. The building appeared to be about 25 feet by 50 feet in plan. All equipment appeared to be anchored. Electrical conduit and piping was typically attached to the walls using a series of inverted steel bents (Figure 5-18) or were suspended with threaded rods that were not laterally braced. The electrical conduit and piping appeared to perform fine. It should be noted that at the time of the earthquake, the pump room was undergoing expansion to accommodate the new LPG tank.

Observations

The pump room did not show evidence of any significant structural damage. Because of the relatively high water table (only 4 to 6 feet below grade), there was leakage around the piping connection that goes through the wall. There was no other apparent damage to either the structural system or nonstructural components.

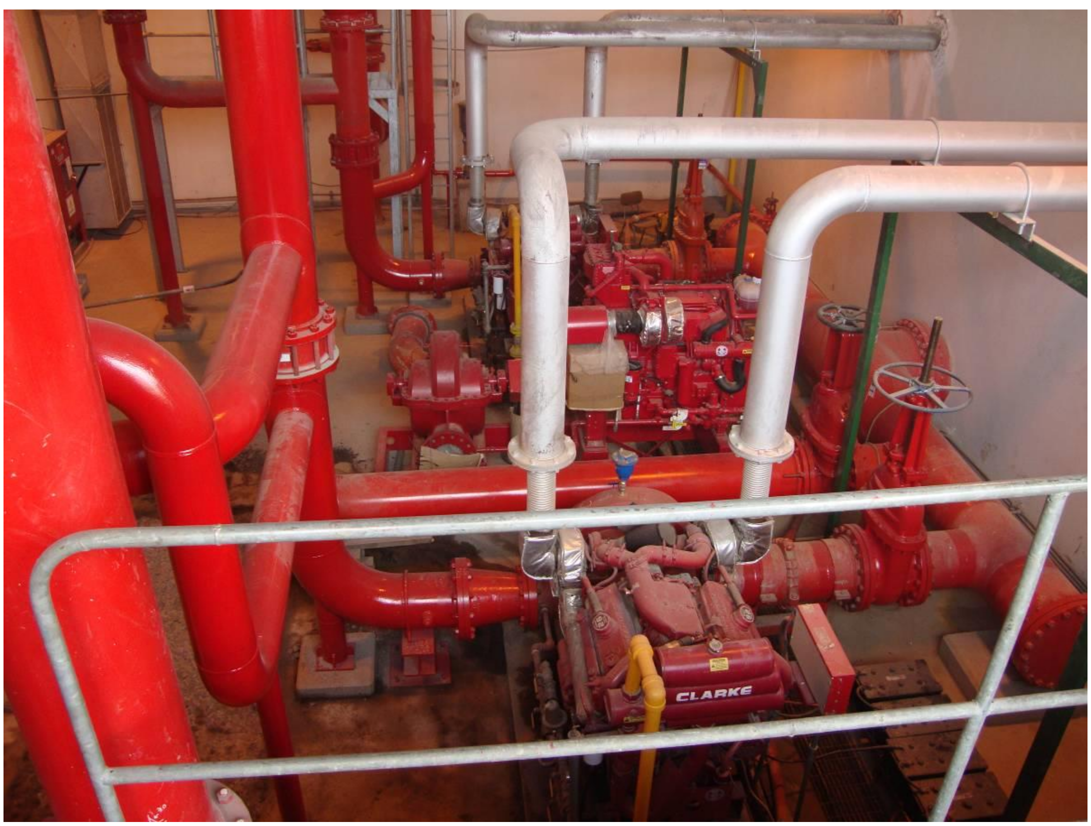

Figure 5-17: Pump room.

Figure 5-18: Pump room piping.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Nonstructural equipment, piping and electrical conduits performed well during the seismic event. The only damage was to a piping connection though the below grade wall. Perhaps some type of requirement should be added for piping penetrations that are below the water table to consider effects of earthquakes in designing the penetration details. No other code changes appear to be warranted based on the observed performance in the pump room.

Pier Assessment

The approximately 2200 meter long pier (Figure 5-19) is used to transfer LPG from ships to the storage spheres and tank. The pier is supported on steel pipe piles. The superstructure of the pier is composed of steel trusses with a cable tray hung with rods from the overhead superstructure. The cable tray and cable tray supports did not appear to be laterally braced.

Observations

The crane at the end of the pier (not shown - photography not permitted at end of pier) performed poorly. The bracing connections failed. The failure was likely due to the extreme torsional irregularity built into the crane support frame (one complete side was left unbraced). It should also be noted that the sea floor moved vertically upward approximately 0.8 meters to 1.0 meters during the seismic event, which may have also created additional loading on the structure.

As can be seen in Figure 5-20, the cable tray supports hung from the pier superstructure failed for a significant length of the pier and dropped to the deck of the pier. The cable tray was supported using fiberglass U-shaped channel-type beam members. The channel beam members were in turn supported by rods that penetrated the web of the channels with nuts placed below the web. Fiberglass channels were used to support the cable tray because of their corrosion resistance.

Unfortunately, the fiberglass material did not have sufficient resistance to pull-through of the nuts through the web of at least one connection (see Figure 5-21 and Figure 5-22). It is likely that adjacent connections became overloaded and the tray support system suffered a progressive collapse. It is suspected that a combination of significant horizontal and vertical seismic loads in addition to dead load caused the failure.

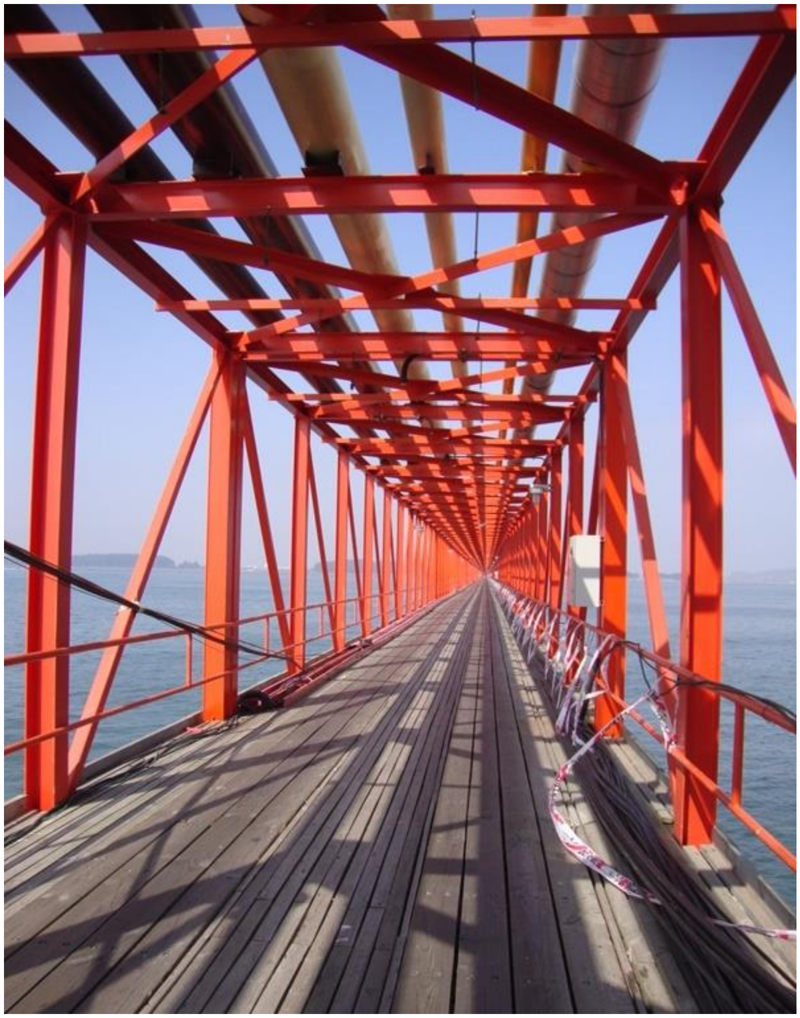

Figure 5-19: LPG terminal pier.

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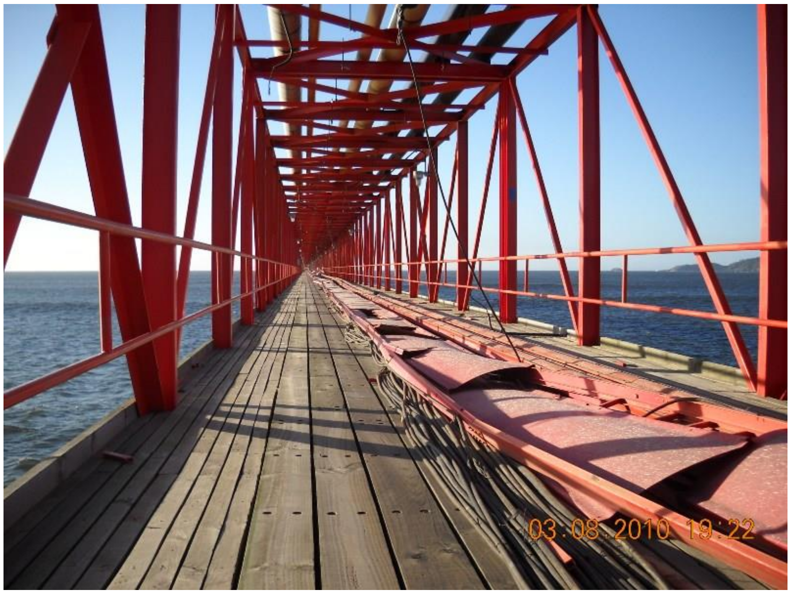

Figure 5-20: Dropped cable tray.

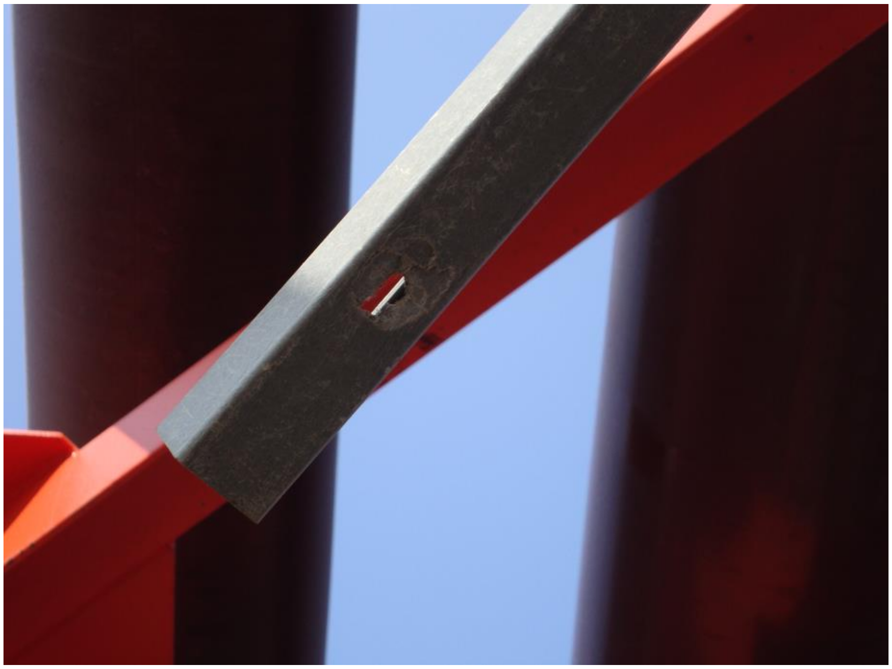

Figure 5-21: Damaged fiberglass channel.

Figure 5-22: Fragment of channel on hanger.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

It is suspected that the highly irregular configuration of the crane support structure in torsion was the primary cause of the failed support structure connections. It is our opinion, that if torsion had been treated properly or avoided, the connections would not have failed.

It is suspected that the failure of the fiberglass channels could have been avoided if proper design checks and detailing has been performed for seismic loads. Chapter 13 of ASCE/SEI 7 needs to consider providing commentary on hanging and bracing materials made of alternate materials like fiberglass.

General Conclusions and Recommendations

The Abastible LPG facility performed well during the seismic event. The soil improvement carried out under the LPG spheres in 2004 and under the LPG tank in 2010 performed well. The outer edge of the soil improvement did not perform as expected by the operator of the facility resulting in excessive movement of the pipe rack and wall adjacent to the LPG tank.

Nonstructural components performed reasonably well.

The cable tray at the pier performed poorly as described above. Chapter 13 of ASCE/SEI 7 needs to consider providing seismic design requirements for hanging and bracing materials made of alternate materials like fiberglass. The crane at the pier performed poorly as described above. Significant torsional irregularities should be avoided if at all possible.

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Chapter 6
Assessment of Santa Maria Power Station

Power Station Overview

The Santa Maria Power Station (Lat 37° 2′ 32" S, Long 73° 8′ 1″ W) is a 350 MW coal-fired plant located near Coronel, Chile approximately 500 kilometers south of Santiago (Figure 6-1). The plant was under construction at the time of the earthquake. The unit under construction represents the first phase of a 700 MW power plant. The first stage of the Santa Maria power plant involves a 350 MW pulverized coal combustion plant with a seawater desulfurization system, electrostatic particulate removal and low NOx burners. The large structures were supported on drilled concrete caisson foundations. Some smaller secondary structures and the coal tunnel were supported on improved soils. Please note that operational loads caused by the weight of coal for these large structures are significant. Since none of these structures were loaded with coal at the time of the earthquake and none were in operation, no real conclusions can be drawn.

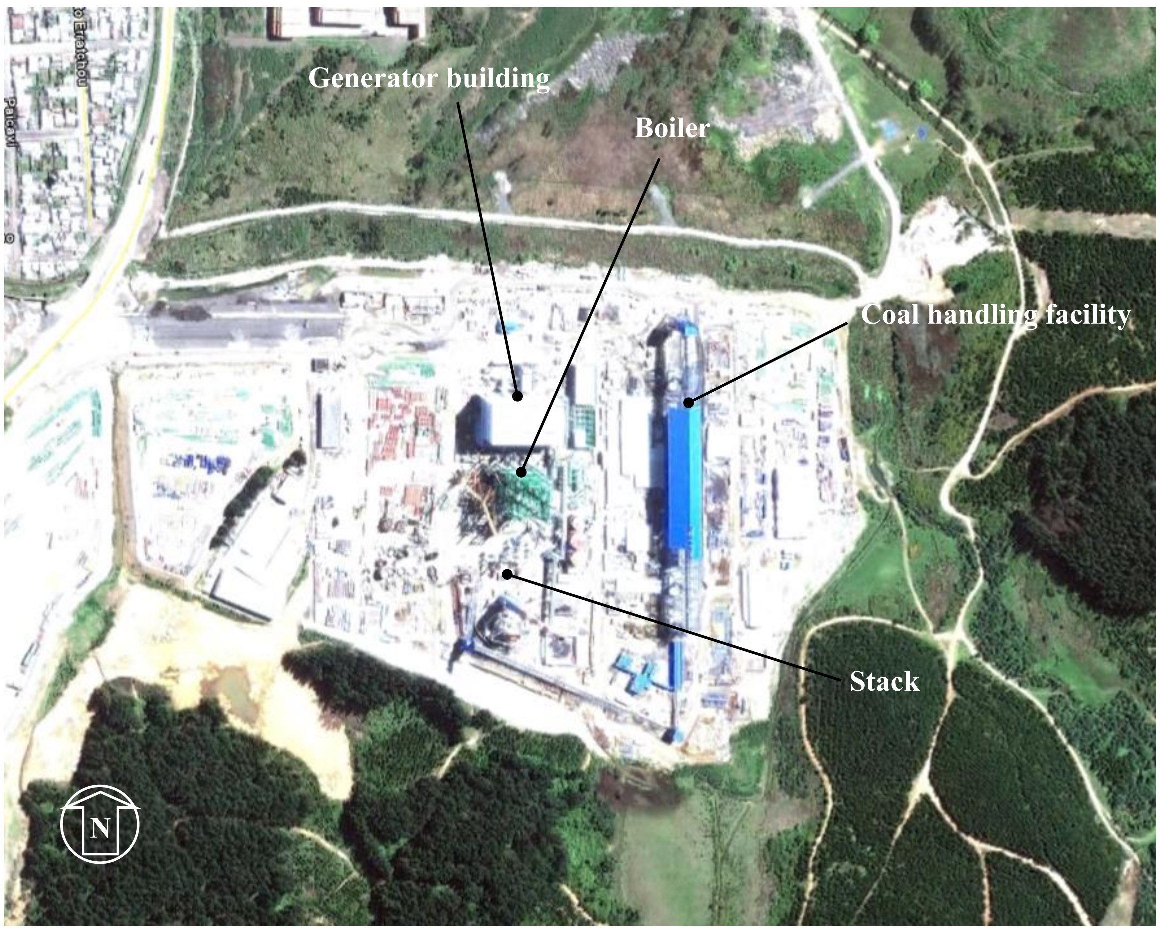

Figure 6-1: Aerial view of Santa Maria Power Station (under construction) (Source: GeoEye).

Each large structure in the Power Plant was designed by separate international contractors. Some of the structures were designed by Chilean structural engineers. All structures were independently peer reviewed. All structures were designed to meet the Chilean Standard NCh 2369-2003. Given that the design of the structures were based on the same seismic criteria and the designs were all independently peer reviewed, the difference in design details especially in the area of bracing connections and anchorage is remarkable.

The Santa Maria Power Station sustained some damage to its coal handling facility and some significant seismic settlement and tilting of secondary plant structures. The overall damage, however, was fairly minor.

Soil Conditions

No soil information was made available to the Industrial Assessment Team.

Estimated Response Spectra

As discussed above, neither of the two response spectra shown in Figure 2-2 and Figure 2-3 are located near Coronel. No ground motion records exist for this site.

Boiler Structure Assessment and Observations

The coal fired boiler was under construction when the earthquake occurred (Figure 6-2). No damage was observed. The boiler support structure was a steel braced frame (comparable to an Ordinary Concentrically Braced Frames or OCBF in the United States.). The bracing connections used a very large number of bolts and massive structural members. Chairs were used with the anchor bolts.

Coal Bin Assessment

The coal bin was under construction when the earthquake occurred (Figure 6-3). No damage was observed. The bracing used connections joined by end plates and bolts. Chairs were used with the anchor bolts. The anchor bolt configurations did not seem consistent with members and connection bolts above the anchor bolts. The coal bin was not loaded with its normal operating load during the seismic event. Therefore, no assessment of how the structure in normal operation would behave in a similar seismic event could be made.

Coal Pile Conveyor and Support Structure Assessment and Observations

Coal used as fuel for the power plant is offloaded at the Port of Coronel about one mile from the facility and transferred to the plant by conveyor belt. At the plant, the conveyor rises on a structural steel support system to a height of 80 – 100 feet above the ground and is transferred to a second conveyor supported on a steel braced frame structure (see Figures 6-4 and 6-5). This steel braced frame structure is in turn supported on massive circular concrete columns supported on drilled concrete caissons. This structure is approximately 1,000 feet long. The braced frame structure has a roof and an unloading conveyor that will deposit coal in piles. The large steel and concrete structure did not appear to be damaged during the earthquake. However, the transfer car, which unloads the coal from the second conveyor and transfers it to unloading chutes, was badly damaged at a moment connection (see Figure 6-6). It is apparent from the photo that the transfer car moment connection was not designed for seismic loads. It is fortunate that the conveyor was not loaded with coal at the time of the earthquake (Figure 6-7).

Figure 6-2: Boiler structure (under construction).

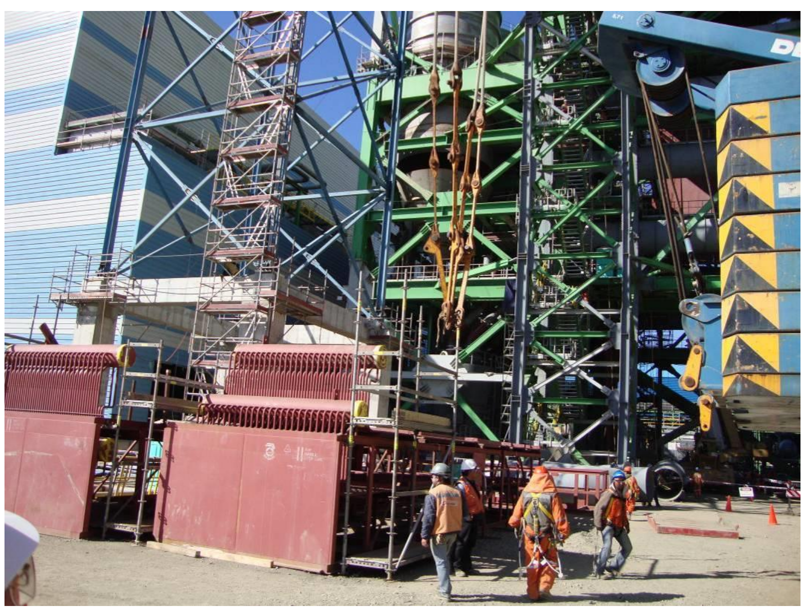

Figure 6-3: Coal bins and supporting structure.

Figure 6-4: Coal-handling facility.

Figure 6-5: Upper coal conveyor.

Figure 6-6: Damaged transfer car wheels (notice extremely poor transverse moment connection detail).

Figure 6-7: Upper coal conveyor.

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Conveyor Tunnel Assessment

A concrete rectangular tunnel runs under the location of the coal pile. The tunnel is approximately 1000 feet long with a cross section of approximately 15 feet by 15 feet. The tunnel is supported on improved fill. There are expansion joints built into the design to allow for relative displacements. Inside the tunnel, there is a conveyor and vibrators installed in openings located in the roof of the tunnel (Figure 6-8). The concept is that the coal is unloaded from the piles above and is mixed (since coal is received from various sources) to obtain the optimum fuel. The vibrators help assure that the coal will loosen from the pile and go downward onto the conveyor. The conveyor is supported on adjustable bolts (Figure 6-9). There was some minor settlement of the tunnel, which was accommodated by the expansion joints. No damage was observed to the tunnel or conveyor (Figure 6-10). It should be noted that coal had not yet been unloaded.

Figure 6-8: Lower coal conveyor.

Figure 6-9: Support, lower coal conveyor.

Figure 6-10: Lower coal conveyor at conveyor tunnel exit.

Stack Assessment and Observations

The plant has a large reinforced concrete stack approximately 400 feet tall (Figure 6-11) with a large rectangular breach opening (Figure 6-12). The exterior shell of the stack was completed at the time of the earthquake. It did not appear that the stack experienced any damage during the seismic event.

Figure 6-11: Reinforced concrete stack.

Figure 6-12: Large breach opening in stack.

Tank Assessment and Observations

Some tanks and some other secondary equipment and structures were supported on mat footings located on improved soils. The mat footings sank and tilted during the earthquake (Figure 6-13). It was not clear what repair measures would be taken for these tanks, equipment and their foundations. The tanks themselves appeared undamaged. It should be noted that the tanks were empty when the earthquake occurred.

Figure 6-13: Tilted tank mat footing resulting from liquefaction.

Removable Anchor Bolts Assessment

One of the more innovative design approaches used in the design of some of the large structures, was removable anchor bolts. The concept is to provide bolt details where the large diameter anchor bolts can be removed and if necessary replaced after a large earthquake. These bolts have Tee Heads (Figure 6-14) that are connected to special sockets in the concrete with large sleeves. These bolts can be simply loosened, twisted and removed. Several of the bolts were checked after the earthquake and none had permanently stretched. Therefore, none of the bolts needed to be replaced.

Figure 6-14: Removable tee head anchor bolts.

Miscellaneous Connections Assessment and Observations

The Santa Maria power plant presents an interesting snapshot of different interpretations of the Chilean seismic code. Many structures were designed by different subconsultants from different countries. Many different bracing and anchorage configurations were observed (Figures 6-15 through 6-18).

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

There are no specific recommendations regarding needed changes to ASCE/SEI 7 based on the observations made at the Santa Maria Power Plant because the plant was not operational at the time of the seismic event.

Figure 6-15: Multiple small anchor bolts.

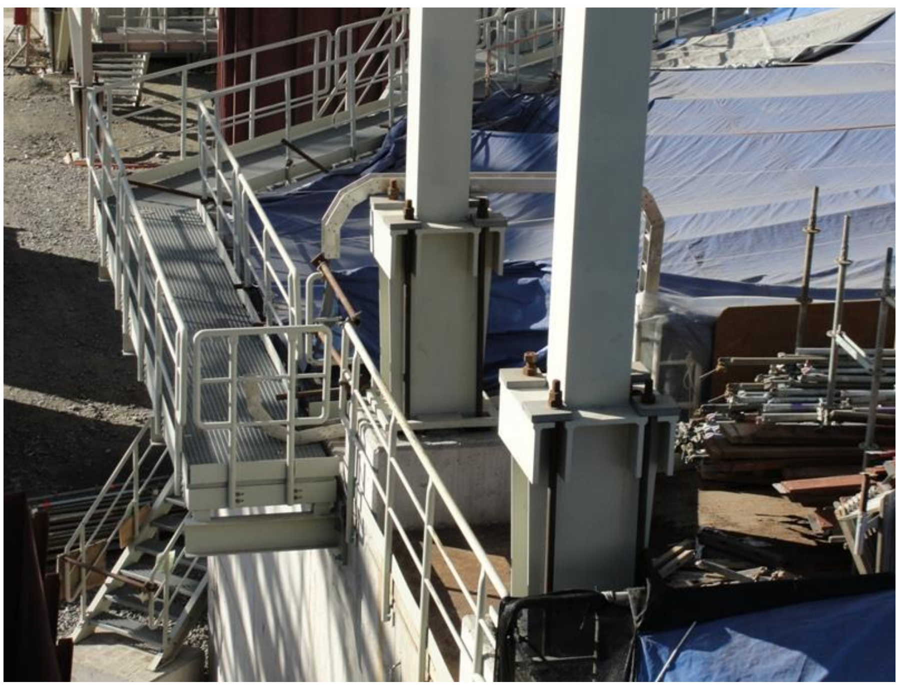

Figure 6-16: Large anchor bolts with 1-meter stretch length.

Figure 6-17: Bracing connection.

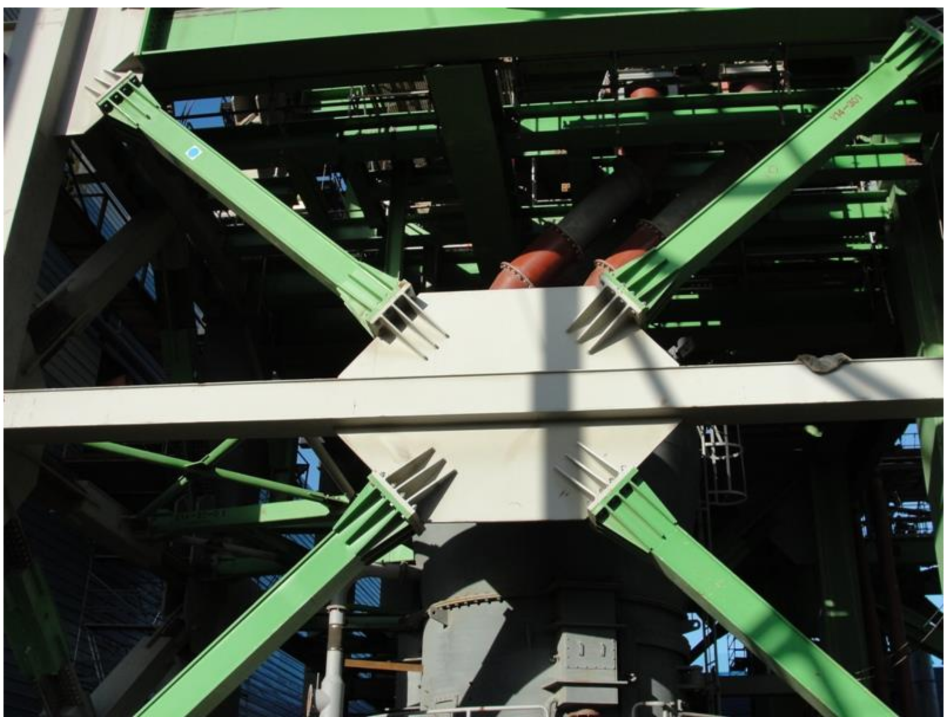

Figure 6-18: Bracing connection.

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General Conclusions and Recommendations

There are really not specific conclusions that can be made regarding the performance of these power plant structures since the structures were not operationally loaded. The few conclusions that can be made are:

1. It is important that appropriate approaches be used when using improved soil rather than using deep foundations in areas of liquefaction. Unlike the LPG facilities, which performed well when fully loaded and sitting on improved soils, the empty tanks in the power plant performed poorly on the improved soils at the power plant. It is not clear if this was a design problem or an implementation problem with the improved soil.
2. The concept of removable large diameter anchor bolts should be considered and encouraged as a best practice in the United States along with tall chairs on large steel structures in seismic areas. These also would require the use of shear keys.

Chapter 7
Assessment of Base Isolated Pier

Base Isolated Pier Overview

The base isolated pier (Lat 37° 2′ 1", Long 73° 9′ 7" W) at the Port of Coronel (Figure 7-1) suffered only minor damage at the isolation joint, which was located at about the midway point from the shore where the roadway portion transitions to the pier portion of the pier. The container crane suffered no damage.

Figure 7-1: Aerial view of base isolated pier in Coronel (GeoEye).

Soil Conditions

No soil information was made available to the Industrial Assessment Team.

Estimated Response Spectra

As discussed above, neither of the two response spectra shown in Figure 2-2 and Figure 2-3 are located near Coronel. No ground motion records exist for this site.

Base Isolators Assessment and Observations

The rubber isolators could not be directly observed since they were below deck. It was reported to us by others they were not damaged and that they performed as expected.

Isolation Joint Assessment and Observations

Evidence of significant movement in both longitudinal and traverse directions was observed in the isolation joint between the pier and land (Figures 7-2 through 7-5). It appears there was some lateral spreading at the shoreline causing a permanent reduction in the isolation gap in the longitudinal direction of several inches. Significant movement, such as that observed, would normally be expected.

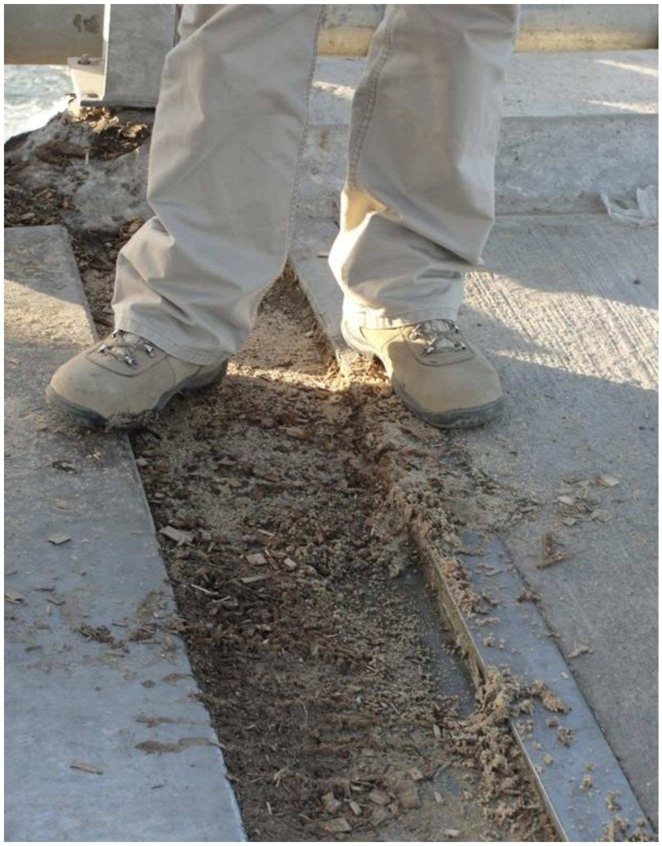

Figure 7-2: Isolation joint.

Figure 7-3: Isolation joint at edge.

Figure 7-4: Isolation joint.

Figure 7-5: Damage to railing due to transverse lateral movement.

Crane Assessment and Observations

The pier container crane shown in Figure 7-6suffered no damage during the earthquake.

Figure 7-6: Container crane on isolated pier.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

The pier and the container crane performed well. The isolation joint area could have been better detailed to avoid damage. Current ASCE/SEI 7 requirements in Chapter 17 are currently adequate in this area and do not warrant any modification. However, for long linear pier type structures, perhaps some wording should be added to Chapter 17 indicating, that where non-isolated near shore portions may shift location relative to isolated portions, effects of lateral spreading should be considered in sizing the width of the isolation joint.

General Conclusions and Recommendations

The base isolated pier performed well. This type of system should be considered along with other properly designed fixed base systems.

Chapter 8
Anchor Bolt Performance

Anchor Bolt Behavior – Stacks and Vessels

The stretching (yielding) of anchor bolts is a primary source of ductility in the seismic response of tanks, vessels and large one story moment frame structures. Figure 8-1 depicts a large stack at the CAP Acero Steel Plant. Figure 8-2 shows the amount of inelastic deformation experienced by the 30 1.75-inch anchor bolts arrayed around the circumference of the stack. Figure 8-3 shows the shimmed bolts. The amount of permanent stretch in the bolts tends to be uniform around the base indicating oscillation of the stack around the circumference of the base, much like the precession of a spinning coin on a flat surface. This is a phenomenon observed with stacks in past earthquakes. Anchor embedment design at the CAP Acero facility typically used the procedures found in Appendix B of ACI 349-76, which are somewhat less conservative than those contained in current codes. The bolts are reported to have a specified yield of 40 ksi and an ultimate strength of 60 ksi and are threaded only at their upper end. Their performance was generally quite good and intermediate stabilization of the stack could be achieved by the simple use of shims. There was also some damage to the foundation in the vicinity of the upper embedded portion of the bolts, probably caused by the oscillation of the stack (see Figure 8-4).

Figure 8-1: Large Stack at CAP Acero steel plant.

Figure 8-2: Stretched anchor bolts on stack. (Source: Courtesy of S. Otero, CAP Acero)

Figure 8-3: Stretched anchor bolts on stack with shims.

The stretch length afforded by the anchor chair for these anchors is on the order of eight bolt diameters. The actual stretch length was probably longer due to significant strain penetration into the foundation. Figure 8-5shows the typical remedial procedure for intermediate stabilization used. If the threads were not damaged and the bolt shank was not necked down, shim washers were added under the nuts and the bolts retightened. However, since the stretch of the bolts (6-7%) exceeded the value considered by CAP Acero to be acceptable from the standpoint of energy dissipation (5%), and since replacement of the bolts in situ was deemed to be quite complicated and expensive, it was decided to construct a secondary anchor ring around the foundation with the new foundation doweled back to the original structure (see Figure 8-6). See Appendix C for drawings.

Figure 8-4: Damage to stack foundation.

Figure 8-5: Stretched anchor bolt with shims.

Figure 8-6: Repair to stack foundation. (Source: Courtesy of S. Otero, CAP Acero)

Anchor Bolt Behavior – Column Baseplates

Yielding was widely observed in baseplate anchors in the Hot Rolling Mill (Figure 8-7) at the CAP Acero facility. The anchors in the moment frames predated the 1960 Valdivia earthquake and many experienced yielding in both that event and this one (Figures 8-8 and 8-9). Many of these anchors must now be replaced either because the degree of necking is extreme or because the anchors experienced fracture below the baseplate (Figure 8-10).

Figure 8-7: Hot rolling mill.

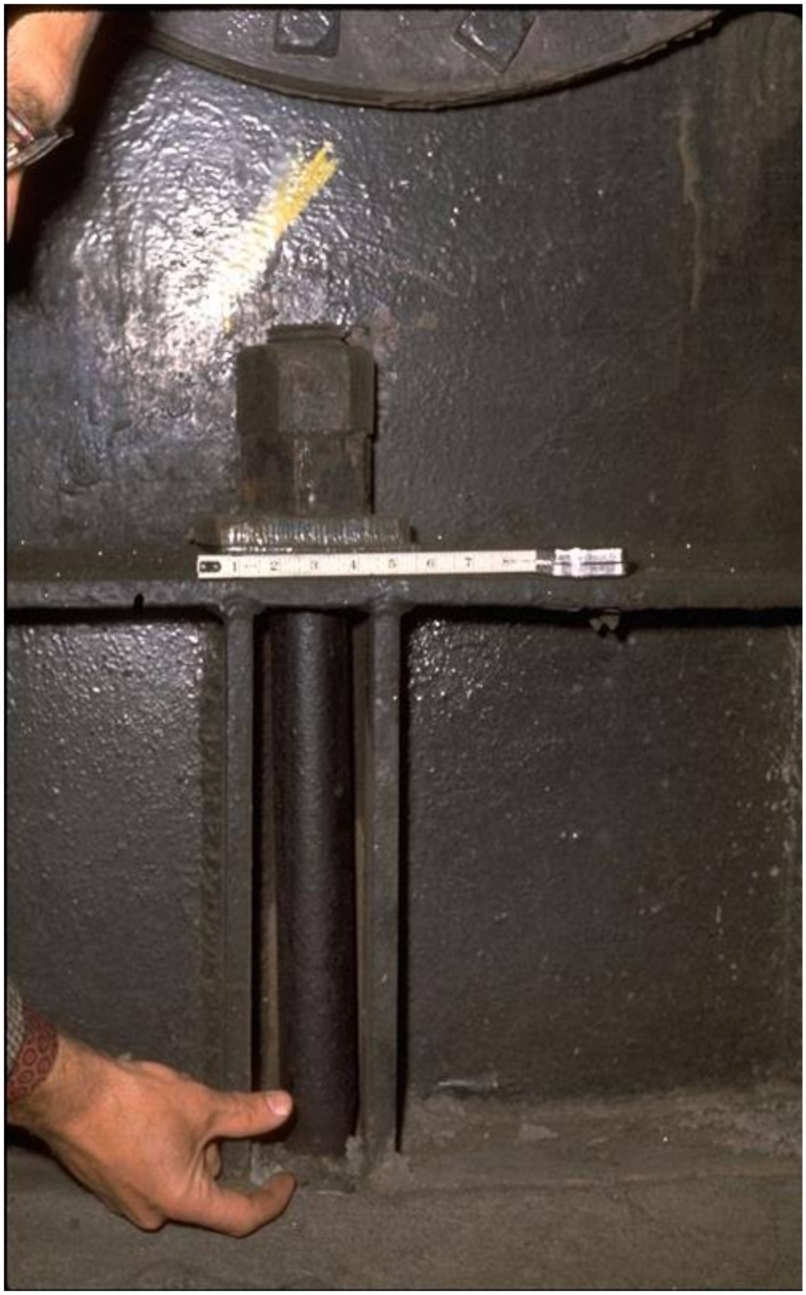

Figure 8-8: Anchor bolt stretch in 1960 Mw 9.5 earthquake, Huachipato Steel Plant. (Source: Courtesy of K. Steinbrugge, NISEE)

Figure 8-9: Yielded frame anchor bolts in hot rolling mill.

Figure 8-10: Remedial work on anchor bolts in hot rolling mill.

A review of investigation notes provided by engineers at CAP Acero indicates that, where provided, a stretch length of eight bolt diameters is considered a minimum value. This was confirmed by visual observations of the Industrial Team. With few exceptions, where stretch length was provided, failure of the anchors was avoided. In contrast, where column baseplates did not incorporate stretch length, anchor failure was often the result. See Table 8-1.

Table 8-1. Anchor Bolt Parameters (Data Courtesy of CAP Acero)

Location descriptor	*Total anchor length*		*Height of anchor chair*		*Anchor diameter (d)*		*Stretch length*	*Figure*
	mm	*in.*	*mm*	*in.*	*mm*	*in.*		
Nave 210 L.B.T.	1220	48	305	12	38.1	1.5	8.0d	8-11
Nave Maenstranza	1480	58	500	19.7	31.8	1.25	15.7d	8-12
Nave Myre	1500	59	300	11.8	31.8	1.25	9.4d	8-13
Nave Aceria Conox	3200	126	800	31.5	89	3.5	9.0d	8-14
Nave Colada Continua	3620	143	1480	58.3	89	3.5	16.6d	None
Nave Ex-Desmoledora	2060	81.1	860	33.9	76.2	3	11.3d	8-15
Nave des Tubulares	600	23.5	NA	NA	16	0.625	NA	8-16
Nave des Tubulares	600	23.5	NA	NA	16	0.625	NA	8-17
Nave 216 Distribucion	760	30	NA	NA	31.8	1.25	NA	8-18

Figure 8-11: Yielded anchor bolt, baseplate without shear lug. (Source: Courtesy of CAP Acero)

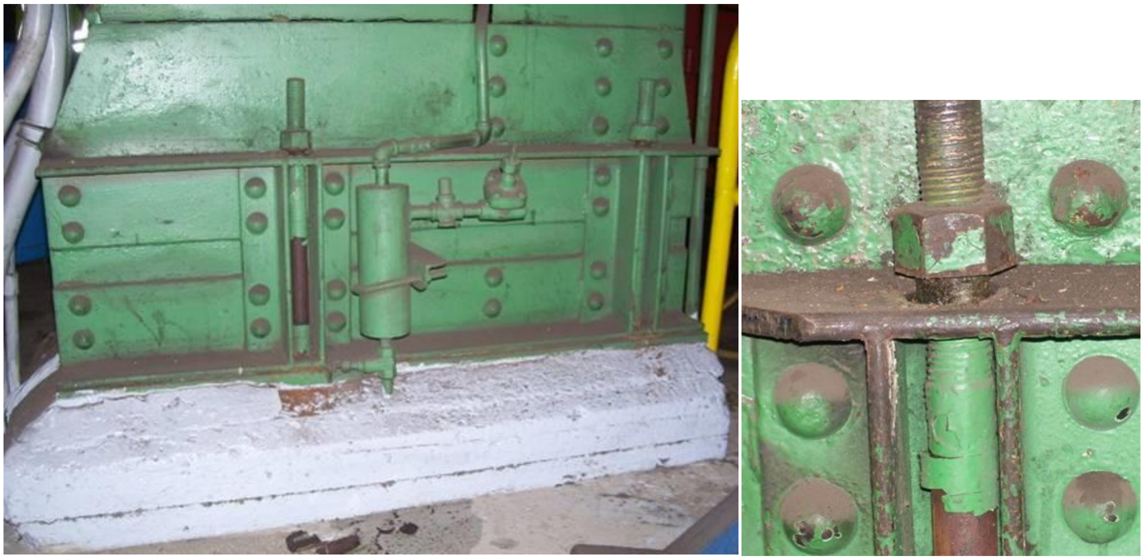

Figure 8-12: Yielded anchor bolts, estimated stretch of 20 mm (7/8 in.).(Source: Courtesy of CAP Acero)

Figure 8-13: Yielded anchor bolts, estimated stretch of 18 mm (11/16 in.). (Source: Courtesy of CAP Acero).

Figure 8-14: Yielded anchor bolts, estimated stretch of 10 mm (3/8 in.) (Source: Courtesy of CAP Acero).

Figure 8-15: Yielded anchor bolts. (Source: Courtesy of CAP Acero)

Figure 8-16: Stretch/failure of baseplate anchors (no anchor chair). (Source: Courtesy of CAP Acero)

Figure 8-17: Failure of baseplate anchors (no anchor chair). (Source: Courtesy of CAP Acero)

Figure 8-18: Fracture of baseplate anchors (no anchor chair). (Source: Courtesy of CAP Acero)

Anchor Bolt Behavior – Pedestals and Saddles

The anchorage failure of the coal bin (Figure 8-19) represents a typical weakness in older pedestal anchor bolt designs where significant shear is present and shear lugs are not used. In Figure 8-20, the edge failure likely led to the bending and rupture of the near-edge anchor. The small edge distance afforded these bolts indicates that they may not have been designed for this condition.

A common practice, whereby the grout cap is placed to the surface of the baseplate is represented by Figure 8-21. This practice almost uniformly led to premature spalling of the grout cap.

Figure 8-22 shows a sketch provided by one of the engineers from the ENAP Bio Bio Refinery of the typical older pedestal anchorage design for hydrocrackers and other vertical vessels. These pedestals suffered extensive damage in some locations as shown in Figure 8-23. While it is possible that this is the result of shear on anchor bolts with insufficient edge distance, there is some evidence that the primary loading of these anchorages was axial tension and compression (vertical pounding). Note that the vertical pedestal reinforcing is not bent or laterally displaced. The transfer of tension loads from a headed anchor into a narrow pedestal, such as this, results in significant bursting forces at the anchor head that must be accommodated with transverse reinforcing. In the case of these older pedestals, not only were the edge distances provided relatively small, but hoop spacing was large and was not concentrated at the anchor load transfer area.

Further evidence of vertical pounding was seen in failed saddle anchorages (Figure 8-24) which showed little lateral displacement. Anchors extracted from these saddles (Figure 8-25) provided clear evidence of extreme vertical compression (bearing plate cupped upward).

Figure 8-19: Coal bin. (Source: Courtesy of CAP Acero)

Figure 8-20: Shear failure of anchorage at coal bin.(Source: Courtesy of CAP Acero)

Figure 8-21: Damage to column base cap in bar mill (new).

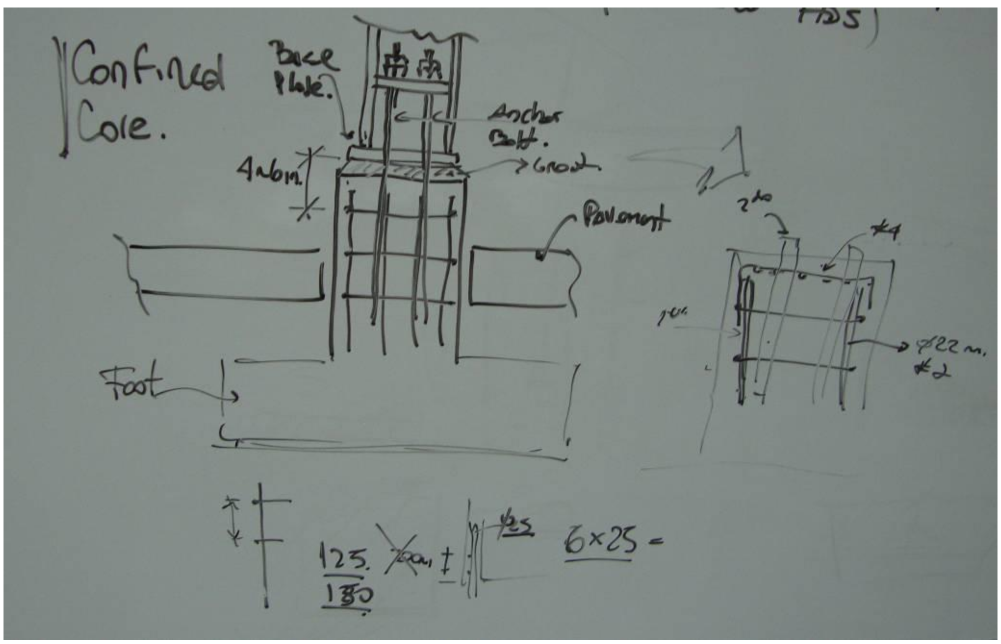

Figure 8-22: Sketch of typical pedestal anchorage design.

Figure 8-23: Failed pedestal anchorage.

Figure 8-24: Failed saddle anchorage.

Figure 8-25: Failed anchor from saddle. Note cupping of bearing plate.

Recommendation Regarding ASCE/SEI 7 and Retrofit of Existing Structures

Current Chilean practice is to provide stretch length through the use of anchor chairs coupled with extensive use of cruciform shear lugs in almost all of their anchor designs. This practice appears to be very successful.

Added design requirements for anchorage in ASCE/SEI 7-10 Chapter 15 appear to be justified by the experience in Chile. The ASCE/SEI 7-10 requirements force the bolt to yield by designing the embedment for the strength of the bolt and by providing a minimum stretch length of eight bolt diameters.

Many of the anchorage designs that appeared to successfully withstand the earthquake were based on earlier anchor design approaches, such as the 45-degree cone method contained in earlier versions of ACI 349. There is some evidence that the 45-degree cone method for determining concrete breakout capacity in tension becomes more representative with increasing embedment depth, so this is not entirely unexpected. The edge distances resulting from this approach tend to be insufficient, however. The subject of large anchor design continues to be studied intensively and appears to indicate that while tension designs may be overly conservative in some cases, shear designs that rely on the concrete breakout strength of the anchors may not be.

While Chilean anchorage designs largely behaved as intended during the seismic event, the plant personnel now face the daunting task of repairing hundreds of anchor bolts. Use of removable anchor bolts, while initially more expensive, would have decreased both the repair cost and down time of the facility. This best practice should be considered in the design of these facilities in the United States. Also the use of anchor chairs with large one story moment frame industrial building along with the use of shear lugs for transferring shear should also be encouraged.

Chapter 9
Industrial Assessment Team Recommendations

Based on the discussion presented above, the Industrial Assessment team recommends the following:

1. Currently, the commentary to ASCE/SEI 7-05 repeats the reduction in product weight for a bulk storage silo allowed by ACI 313 for an elevated bin or silo and recommends a limit on the reduction of seismic mass for a ground supported storage silo. ASCE/SEI 7 Chapter 15 should add provisions incorporating the ACI 313 practice of allowing some reduction in seismic mass due to intergranular movement and but greatly limiting the reduction of seismic mass based on the probability of the elevated storage structure only being partially full.

2. Wooden cooling towers do not perform well in seismic events (most likely due to a combination of highly eccentric connection design and extensive connector corrosion in older wooden cooling towers). Table 15.4-2 of ASCE/SEI 7 assigns an R value of 3.5 to steel, concrete, and wooden cooling towers. In contrast, the CAP Acero Steel Plant uses a reinforced concrete cooling tower. The concrete cooling tower experienced no damage. It is recommended that the values in Table 15.4-2 of ASCE/SEI 7 for wooden cooling towers be reviewed during the next code cycle with the goal of possibly reducing the R value or prohibiting their use for situations where they would be assigned to SDC D, E, or F.

3. The requirements of ACI 318 Appendix D may be overly conservative for the design of deep anchor bolt embedments with low bearing stresses and other approaches (e.g., the 45-degree cone method of Appendix B of ACI 349-76) may be more appropriate for deep anchor embedment design in reinforced concrete foundations. Similarly, shear design provisions for large anchor bolts should be reconsidered to avoid premature edge breakout failures in pedestals as discussed in Chapter 8.

4. While the anchor bolts behaved as intended during the seismic event, the plant personnel now face the task of repairing hundreds of anchor bolts. Use of removable anchor bolts, while initially more expensive, would have decreased both the repair cost and down time of the facility. This practice should be encouraged in the US. Also the use of anchor chairs along with the use of shear lugs for transferring shear should also be strongly encouraged for non-building structures and industrial buildings.

5. Older reinforce concrete pedestals that support boiler and furnace vertical vessels and are provided with small (e.g., 4 bolt diameter) edge distances did not perform well and could fail during strong earthquake ground motions. These could be retrofitted in various ways; through enlargement of the pedestal or provision of supplementary confinement in the form of steel plate carbon fiber wrap.

6. Evaluation of liquefaction potential is critical before a structure is sited or built. Also the evaluation requirements of structures associated with the effects of liquefaction/lateral spreading need to be developed.

7. It is recommend that the derrick at the top of the coker structure be evaluated in detail to determined the likely range of forces that caused failure of the anchor bolts and that Equation 13.3-1 or Equation 13.3-4 be assessed to determine its adequacy for this situation. Derrick anchorages are obviously highly vulnerable to strong earthquake ground motions. Existing refineries should become aware of the vulnerability of these types of constructions and retrofit actions taken because of the serious consequences associated with derrick toppling.

Appendixes

The appendixes are available for download at [*http://dx.doi.org/10.1061/9780784413647*](http://dx.doi.org/10.1061/9780784413647)

Appendix A	Chilean Standard NCh433.Of96: Seismic Design of Buildings
Appendix B	Chilean Standard NCh2369.Of2003: Earthquake-Resistant Design of Industrial Structures and Facilities
Appendix C	Drawings for Coal Bin, Coal Hopper, Holding Basins, and Pier Structure, CAP Acero Steel Plant
Appendix D	ENAP Engineering Standard EI-001: Structural Steel Design and Fabrication
Appendix E	ENAP Engineering Standard EI-002: Foundations and Elevated Structures
Appendix F	ENAP Engineering Standard EI-004: Wind, Earthquake, and Snow Loading
Appendix G	ENAP Engineering Standard EI-005e: Anchor Bolts
Appendix H	Soil Report for Hydrocracking Unit, ENAP Bio Bio Refinery
Appendix I	Drawings for LPG Spheres and LPG Tank, Abastible San Vicente LPG Terminal

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