

Seismic Assessment and Retrofitting of a Container Wharf and Container Yard following the 27 February 2010 Chile Earthquake



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SUMMARY:

Damage to the San Vicente Terminal International (SVTI) container terminal in Central Chile caused by the 27 February-2010 Maule-Region Earthquake ($M_w = 8.8$) included significant lateral displacements up to 1.5 meters and in-plane torsional distortion along the 600-meter wharf superstructure; extensive settlement in nearly half the 20 hectare container yard; and severe structural and non-structural damage to warehouses and miscellaneous support building facilities. This paper describes the result of a post-earthquake reconnaissance visit with the corresponding structural/geotechnical behavior inferences and presents both emergency and long-term repair concepts and proposed phasing to mitigate disruption of operations and prepare for the future of the terminal. The structural assessment of the wharf structure before and after the proposed retrofitting scheme is described in terms of nonlinear static analyses and on the basis of predefined limit states included in current guidelines for the seismic design/assessment of waterfront structures.

Keywords: *seismic assessment and retrofitting, waterfront structures.*

1. INTRODUCTION

At 3:30 am, on Feb 27, 2010, a Magnitude 8.8 earthquake struck central Chile. The epicenter was located about 100 km northeast of the city of Concepción. Approximately 3 hours after the earthquake, a tsunami with waves up to 3-m high swept Talcahuano (North of Concepción) and several coastal villages and islands. The earthquake severely impacted the San Vicente (SVTI) container terminal (about 90 km south west of the epicenter) causing damage to the 600-meter wharf, the 20-Ha container yard, and miscellaneous terminal buildings.

The SVTI wharf is a marginal structure consisting of three berths: Berth 1 (160 m), Berth 2 (220 m) and Berth 3 (220 m). The wharf has three expansion joints that subdivide the structure in segments (from South to North) with lengths of 185, 145, 110 and 160 m, respectively. The wharf has a reinforced concrete retaining wall at the back supported on a gravity-type concrete caisson, providing containment to a relatively loose artificial backfill with a thickness of 8 to 10 m. The fill is underlain by a layer of fine silty sands, which are in turn underlain by sedimentary rock. Subsurface data indicates that the top of rock elevation typically ranges from -18 to -20 m at the back of the wharf and from -25 to -28 m at the front of the wharf. Fig. 1 shows the site plan for the terminal including overall suggested repairs.

The original wharf structure for Berths 2 and 3 was built in the early seventies and consists of a combination of pretensioned concrete piles and steel pipe piles supporting a concrete deck (Fig. 2a). The pre-tensioned concrete piles are 0.5 m square, with 20 – 14 mm diameter strands arranged in circular pattern and 8 mm diameter spiral at 150 mm pitch. These piles are located in the four outermost seaside pile rows. The steel pipe piles are 1.05-m diameter x 9.6-mm thickness, distributed along the two landside rows. Bent spacing is 5 m. The superstructure consists of 1.0-m deep transverse pile cap, two 1.0-m deep longitudinal pile caps above the pipe piles, additional six 0.6-m deep trapezoidal beams in the longitudinal direction, and a 0.4-m thick concrete deck.

The Berth 1 wharf structure was built in the nineties and consists of 1.02 m diameter x 12.5 mm thick steel pipe piles at the two landside rows and 0.61 m diameter x 10 mm thick steel pipe piles at the two seaside rows (Fig. 2b). Bent spacing is 8.2 m. The piles are braced in the transverse direction with a system of 1.75 m deep steel trusses. In the longitudinal direction, the trusses are installed along the back two rows, in staggered fashion.

The container yard was created by reclaiming land to the sea with material borrowed from an adjacent hill. The platforms were reclaimed first, followed by Berth 2/3 construction and subsequent backfilling. The paving system is a combination of concrete pavers, and undoweled and doweled concrete slabs on grade.

Currently, container loading/unloading to/from SVTI terminal is performed through Harbor Mobile Cranes (HMC). The proposed rehabilitation program would provide the client would serve to purpose to facilitate the installation of rails for Ship-to-Shore (STS) cranes which could potentially improve the productivity of the terminal.

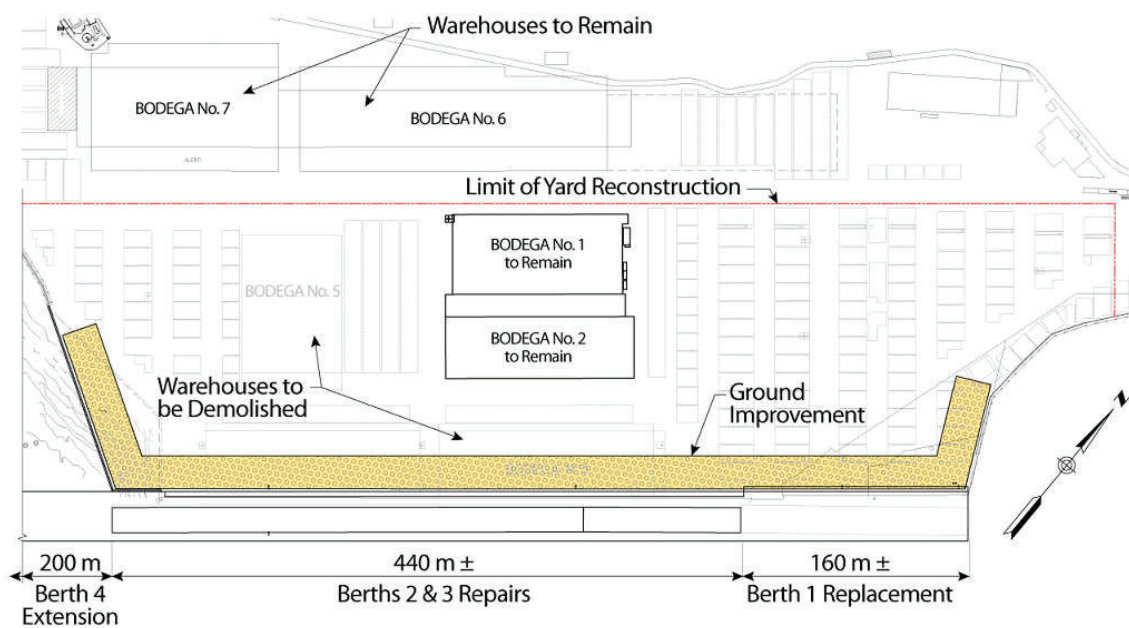


Figure 1. Overall plan view of SVTI wharf structure

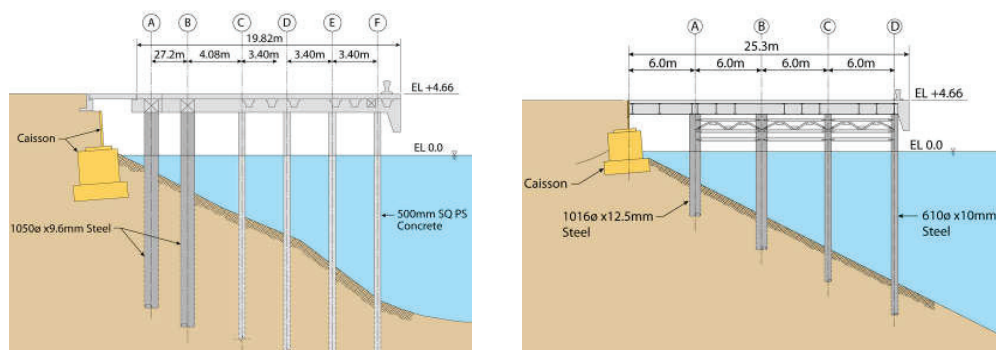


Figure 2. Typical berth sections before retrofitting

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Table 1 summarizes the maximum accelerations for uncorrected ground motion records available in the literature (Boroschek, et al. 2010, Barrientos 2010). The locations of some of the recording stations in relation to the epicenter of the main event are shown in Fig. 3, and the corresponding calculated acceleration response spectra (Boroschek et al. 2010) are shown in Fig. 4 together with the design acceleration spectrum for the SVTI site based on the Chilean code SCH2369 (2003) for assumed soil Type III (soft soil).

It is noticed that for the record obtained in the station closest to SVTI (Concepcion EW) the amplified accelerations can be well in excess of 1.0g (where g is the acceleration of gravity) even outside of the short period range. These are much higher than the code spectral acceleration for the short and intermediate period range.

1. Summary of available earthquake records for the offshore Maule (2010) earthquake

$\frac{1}{n} \sum_{i=1}^n \left(\frac{\partial f}{\partial x_i} \right)^2$	$\frac{1}{n} \sum_{i=1}^n \left(\frac{\partial f}{\partial x_i} \right)^2$	$\frac{1}{n} \sum_{i=1}^n \left(\frac{\partial f}{\partial x_i} \right)^2$
Universidad de Chile ₁	0.17	0.14
Estación Metro Mirador, Santiago ₁	0.24	0.13
CRS Maipú RM ₁	0.56	0.24
Hospital de Tisné RM ₁	0.30	0.28
Hospital Sótero de Río RM ₁	0.27	0.13
Hospital de Curicó ₁	0.47	0.20
Hospital de Valdivia ₁	0.14	0.05
Vina del Mar, Marga Marga ₁	0.35	0.26
Vina del Mar (Centro) ₁	0.33	0.19
Colegio San Pedro, Concepción ₂	0.65	0.60

Source: 1: Boroschek, et al. 2010, 2: Barrientos 2010

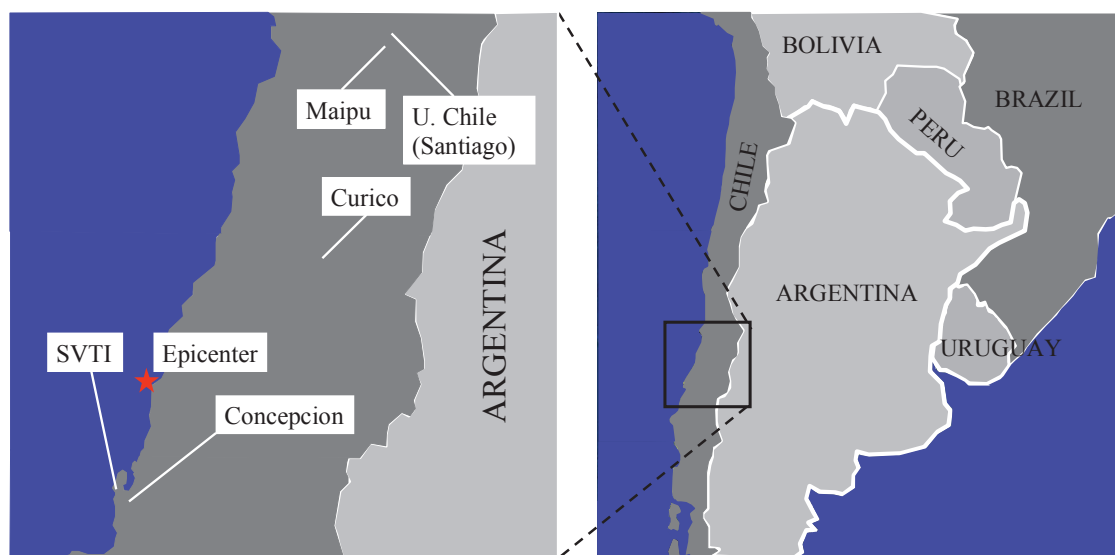


Figure 4 Location of some of the recording stations and epicenter

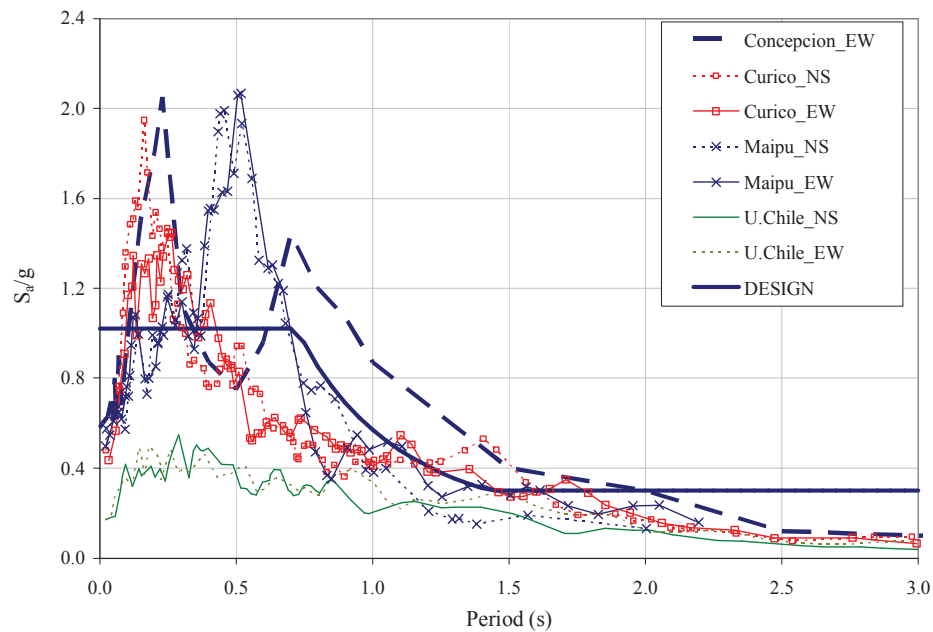


Figure 1 Calculated spectral acceleration and SVTI design spectrum

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Lateral seaward movement of the wharf was observed not only as a result of embankment subsidence, possibly associated to insufficient compaction of both the fill itself and the silty sand layer, and also twisting of the deck in plan. The former is a problem that may occur in terminals that have been reclaimed to the sea, whereas the latter occurs because the center of stiffness of a marginal wharf does not coincide with its center of gravity. The fact that the embankment failure was more pronounced at the extreme north and south ends forced a clockwise twisting of Berth 1 and a counterclockwise twisting of Berth 3 and thus the full marginal wharf system formed a C-shape in plan after the earthquake. At Berth 1, the twisting was accompanied by significant seaward tilting of the wharf that was as large as 1.5 meters at the north end, which suggested that the slope may also have experienced a slip-circle rotational failure below the pile tip elevation.

Embankment subsidence ran along the shoreline of the entire platform (Fig. 5a). The twisting of the deck at the extreme wharf's ends also triggered loss of support for the approach slab (Fig. 5b), which severely limited operations of the terminal. Both the lateral wharf movement and the deck twisting in plan led to a relative lateral displacement at expansion joints varying from 100 mm to 200 mm (Fig. 5c). The maximum observed sway of Berths 2 and 3 was about 300 mm and took place at the southeast end of the wharf. The presence of the expansions would have critically crippled operations had SVTI had STS cranes. It is the authors' opinion that the use of expansion joints in marginal wharf applications is usually driven by tradition rather than actual need, because the temperature gradients would not typically induce significant induced forces even for long segments.

An underside inspection of the marginal wharf revealed different damage levels in the structural components. The top of pile/deck connection showed no evidence of non-prestressing reinforcement projecting from the pretensioned piles into the pile caps (pinned connection). Typical pile damage at these locations consisted of significant concrete bottom cover spalling in the transverse cap (Fig. 6a). Unlike the pretensioned piles, the large steel pipe shafts did develop curvature at the pile/cap

connection as evidenced by the diagonal tension (shear) cracking shown in the cap (Fig. 6b). The extent of cracking appears to reflect the significant resistance exerted by the landside steel piles to prevent the lateral movement of the rather flexible seaside concrete piles.



a) Embankment settlement

b) approach slab seat loss at wharf ends

c) Deck displacement at expansion joint

Figure 6 Observed wharf surface damage



a) typical seaside concrete pile-to-cap damage

b) typical landside concrete pile-to-cap damage

Figure 7 Observed underside wharf damage

The extent of damage in the container yard pavement was reported to cover a few hectares initially but increased to as much as 8 Ha due to the occurrence of nearly 20,000 aftershocks recorded within six months after the main event. Fig. 7a show zigzag cracks in concrete pavers at the South end of the terminal which follow the shoreline and thus suggesting that these were triggered by slope failure. Unreinforced slabs suffered brittle cracking separating entire segments of concrete as shown in Fig. 7b. The presence of dowels in reinforced slabs attenuated the damage extent as these did not suffer much transverse settlement but rather opened up at joint locations. A particular form of distress consisted of cracks outlining the container stacks (Fig. 7c), which may be the result of the slamming of heavy container stacks into the ground resulting from vertical accelerations of the ground.

of pipe piles –readily available at the project site- and a doweled connection, were found to be the best alternatives to ensure the desired stiffness, strength, and ductility of the wharf structure.

The recommended structural rehabilitation of Berths 2 and 3 comprised the installation of an integrated apron structure on the waterside and a link structure on the land side of the existing wharf. The main function of the apron structure is to enhance local stability of the waterside portion of the wharf, whereas the link structure provides most of the lateral stiffness, strength, and deformation capacity of the rehabilitated wharf. The apron structure required the installation of two 600mm diameter x 25mm thick pipe piles every 6.0 meters, whereas the link structure required the installation of two 900mm diameter x 20mm thick steel piles every 5.0 meters (Fig. 8). Waterside piles of the link structures were proposed to be aligned with the shafts of the existing structure so that pile spacing in that gridline became only 2.5m and thus allows for a future installation of land side crane rail for future ship-to-shore (STS) terminal operations and upgrades.

A moment connection consisting of a circular confined reinforced concrete plug reinforced with dowels (24 #8 in the case of the 900mm pipe piles and 18 #8 in the case of the 600mm pipe piles) was proposed because it offers ample deformation capacity. The beneficial effect of the pipe pile in providing confinement to the concrete plug is reflected in the allowable high performance levels listed in design guidelines such as the Port of Long Beach Wharf Design Criteria (POLB 2009) for this type of connection; for a contingency-level ground motion (CLE) level ground motion, POLB allows strain limits as high as 0.025 for confined concrete and 0.06 for dowels.

Jet grouting, albeit expensive, would have probably been the most effective improvement method for the deeper sands and it may be suitable within the fill as well. However, the construction of stone columns was finally recommended in a 25-meter zone adjacent to the wharf because this was more readily available for the local conditions of the project site (Fig. 8). The southern slope under the wharf, where subsidence was also significant, could be relieved by removing unsuitable loose material, but even with ground improvements in place, the stability of the ground was still found to be marginal. Therefore, an anchored sheet pile bulkhead, driven to the underlying bedrock, was proposed to provide a positive means to stabilize this area without extending further into the bay and without extensive underwater construction of the usual rock dikes and compacted fill. Ground improvement and well compacted backfill will be necessary to protect the anchored sheet pile wall. Given the ability to control this fill placement, stone columns also proved to be effective in the 25-meter region adjacent to the wharf.

The stability at Berth 1 slope and pile system was a concern even with ground improvements in the fill behind the wharf. As such, it was recommended Berth 1 be used as a platform for new construction work; including the removal of the liquefiable soils and soft silt/clay, the construction of a replacement rock dike, and the driving of new piles and installation of a new superstructure.

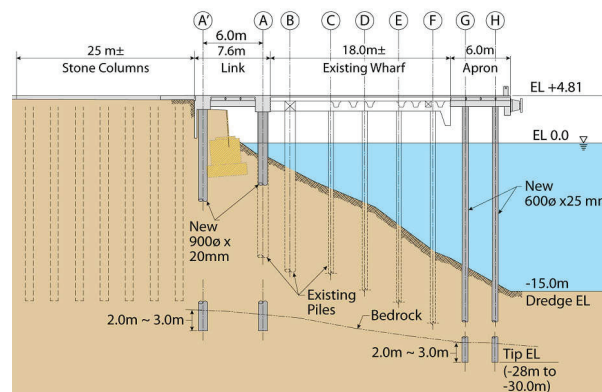


Figure 8 Proposed repair concept for Berths 2 and 3

Figure □ Structural modeling of typical Berth 2/3 bent pre-earthquake

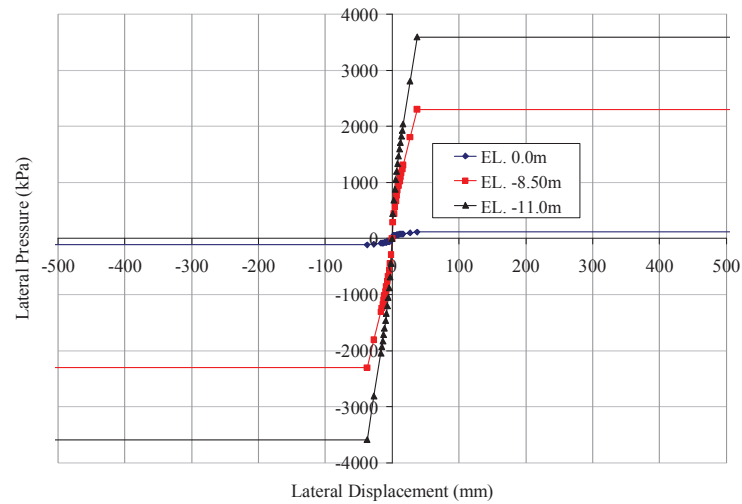


Figure 1 □ Soil p-y curves for landside link piles

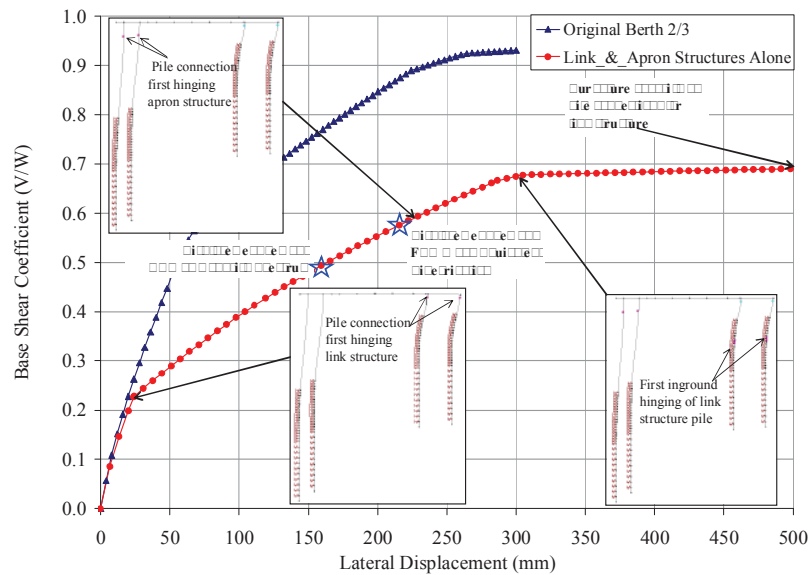


Figure 11. Lateral load response of original Berth 2/3 and new apron/link structure

As observed from Fig. 11, the combined new apron and link structure provides a system with comparable elastic lateral stiffness and base shear capacity, and significantly more ductile (by more than 60%) than the original structure. An improved performance under an event as extreme as the 2010 ground motion may be expected with the relatively simple repair scheme proposed.

Nonlinear displacement demands of the retrofitted Berth 2/3 structure were estimated (neglecting the contribution of the existing damaged structure to the overall lateral load resistance of the system) under the design acceleration spectrum (Sch2369, 2003). The lateral displacement demand was calculated to be 160mm based on the Capacity Spectrum procedure described in ATC 40 and 220mm based on the equivalent linearization procedure in FEMA 440 (2005). As can be observed from Fig. 11, the lateral displacement capacity of the combined apron/link structure –including the mass of the entire structure- is more than twice the demand obtained by either procedure and thus suggesting that the proposed retrofitting scheme provides seismic performance levels well in excess of what would be implicitly required by the current Chilean design code.

From Fig. 11, it can also be observed that the initial lateral stiffness of the structure is about:

$$K_{lat} = 0.2W/0.02m = 10 W/m \quad (1)$$

where, W is the weight of a typical bent. Then, neglecting torsion, the fundamental period of an equivalent single degree of freedom system can be estimated as:

$$T = 2\pi \sqrt{\frac{W/g}{K_{lat}}} = 0.63s \quad (2)$$

The elastic base shear demand for a period of about 0.6 seconds for the record obtained near to SVTI (Concepción EW) (Fig. 4) is around 1.0W. Such base shear demand exceeds the elastic limit in Fig. 11, which is consistent with the nonlinear response and structural damage evidenced after the February 27, 2010 Chile ground motion at SVTI. The upgraded structure would also experience damage under such extreme event; however, its ability to deform and dissipate energy would be greater than the original structure even when neglecting the contribution of the pre-earthquake structure.

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A relatively simple retrofitting scheme of a marginal wharf structure heavily damaged during the 2010 Chile earthquake was presented. The proposed solution involves the installation of two smaller enclosing structures in front and behind the existing wharf. The structure in front (apron) improves the stability, whereas the structure behind (link) is shown to provide a similar lateral stiffness and base shear capacity and a considerably greater displacement capacity than the original structure. The repair scheme also offers potential improved terminal performance as it allows for future installation of crane rails for ship-to-shore (STS) operations.

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