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BEHAVIOR OF THE MAIN WHARF IN SAN PEDRITO PORT IN MANZANILLO, MEXICO

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SUMMARY

The earthquake of 9 October, 1995 ($M_s=7.3$, $M_w=8.0$) in the Mexican port of Manzanillo damaged a Container Terminal at the San Pedrito harbor. This is a descriptive paper which reviews succinctly the damages observed in Manzanillo during this earthquake and then presents a discussion of the behavior of the docking facilities at the Container Terminal. The authors conclude that constructive methods influenced detrimentally the performance of the piles in that wharf, that more research is required to specify design spectra for docks in Manzanillo and that further studies must also be undertaken to account properly for pile-soil interaction effects in these structures.

INTRODUCTION

Manzanillo is the second most important port in Mexico. It is located in the Pacific Coast, some 510 km north of Acapulco and about 200 km south of Puerto Vallarta, fig 1. On 9 October, 1955 a large earthquake hit it, producing considerable damage in the port, the city and many towns and villages along the coast and inland.

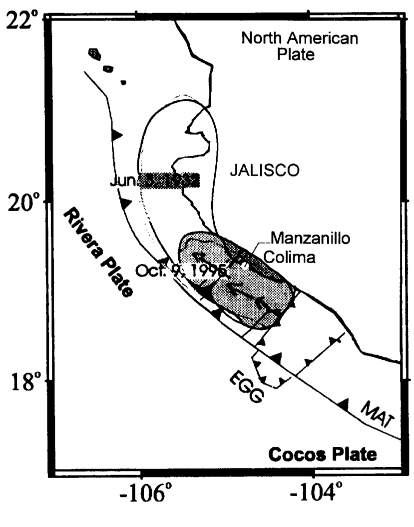

Fig 1. Location of the epicenter and rupture zone for the 1932 and 1995 earthquakes

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Damage distribution in Manzanillo was influenced by the existence of granular fills covering large areas. The earthquake produced the collapse of a nine story building in the city of Manzanillo, where more than 30 people died, and of two low rise structures. No structures collapsed or overturned on account of the settlements induced by liquefaction but several two and three story buildings had to be condemned. Rigid small rise dwellings founded on shallow masonry footings settled differentially but only a few experienced structural damage.

Many of the facilities in the San Pedrito harbor were built on reclaimed land, which liquefied extensively during the earthquake. Materials used for land reclamation were dredged from the neighboring lagoons and are deeper than in urban areas, typically their depth reaches 10 to 15 m. Settlements in yards produced the rupture of water supply and electricity lines; fissuring, cracking and dislocation of pavements were extensive; some buildings settled differentially and some others tilted, in many cases severely. Differential settlements in the yards of the Container Terminal exceeded 80 cm and lateral displacements in unconfined slopes were more than 2 m.

SEISMOLOGICAL BACKGROUND

Manzanillo has been hit by strong tremors at least 8 times before the 1995 shock, starting in 1611. The epicenter of the largest instrumentally recorded earthquake in Mexico (3 June, 1932, M_S=8.2) was located off Manzanillo (Singh et al, 1985).

It was established from several seismological studies performed after the 1995 earthquake that the epicentral zone, 17 km deep, was located on the interface of the Rivera and North American plates, some 24 km south of Manzanillo (e. g., Domínguez-Rivas *et al*, 1997). Surface wave and seismic moment magnitudes were $M_S=7.3$ and $M_W=8.0$, respectively. From these studies it has been concluded that the 1995 event was not a repetition of the 1932 earthquake. Consequently, the occurrence of another large earthquake along the coast of Colima and Jalisco in the next few decades can not be ruled out.

Strong motion records

There were no accelerographic records in Manzanillo itself or in the San Pedrito harbor. A network of several three-component accelerographs was installed in a thermoelectric power house, some 5 km from Manzanillo but only one of these was placed in the free field. It recorded a maximum horizontal acceleration of 380 gals. The duration of the intense portion of the accelerogram at this site was about 50 s; the ratio of vertical to horizontal acceleration maxima turned out to be 0.8. The apparatus was lain over dense sands of undetermined depth. Fourier and response spectra show that site effects influenced ground motion at this recording station.

Seismic hazard

A temporary network of portable seismographs installed some weeks after the October 1995 earthquake at 14 sites having different soil conditions, recorded microtremors as well as some of the replicas that followed it. Together with ambient vibration measurements, the information gathered during this study allowed for the quantification of site effects by means of dynamic amplification factors at specific sites (Lermo et al, 1997). These data and the previously known seismological information were used to assess seismic hazard in Manzanillo, assuming that large earthquakes, i.e. characteristic earthquakes, are generated following a renovation process in which the times of occurrence follow a lognormal rather than a poissonian distribution (Ordaz, 1996).

Design spectra

The design spectrum adopted for Manzanillo corresponds to an earthquake having a 100 year return period, with a maximum ground acceleration of 0.4 g in sites having soft or loose soils. Special considerations were required to specify the design spectra for docks. Accepting that reductions to the ordinates of elastic design spectra can not always be attributed to ductility alone, an additional reduction factor was incorporated, given that docks are highly hyperstatic structures and the collapse or total failure of a dock requires that a large number of piles fail. Pending further research, a value of 2 was assigned to this reduction factor, which can generically be thought of as one that accounts for the inherent overstrength in these structures. Design spectra finally adopted for the docks in San Pedrito is given in fig 2, for ductility factors equal to 1 and 2.

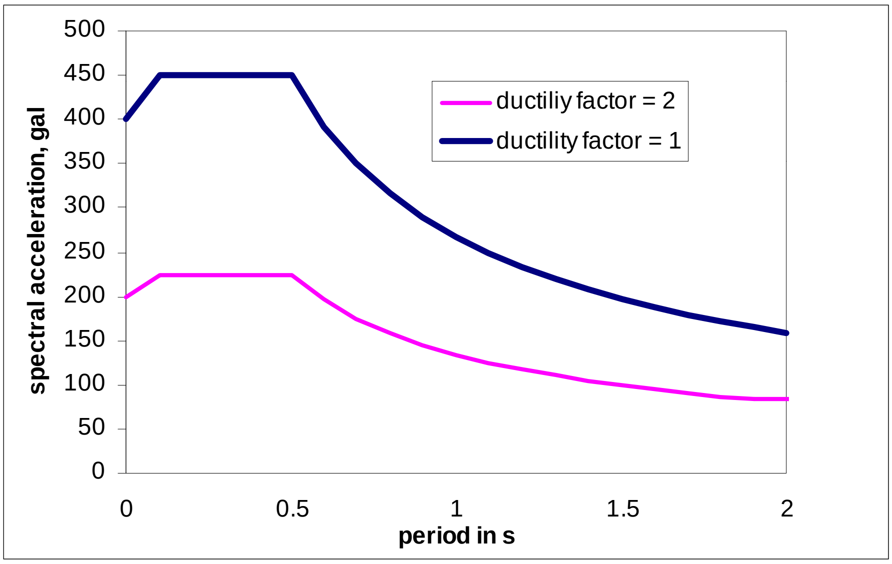

Fig 2. Design spectra for docks and wharves in San Pedrito, including an overstrength reduction factor

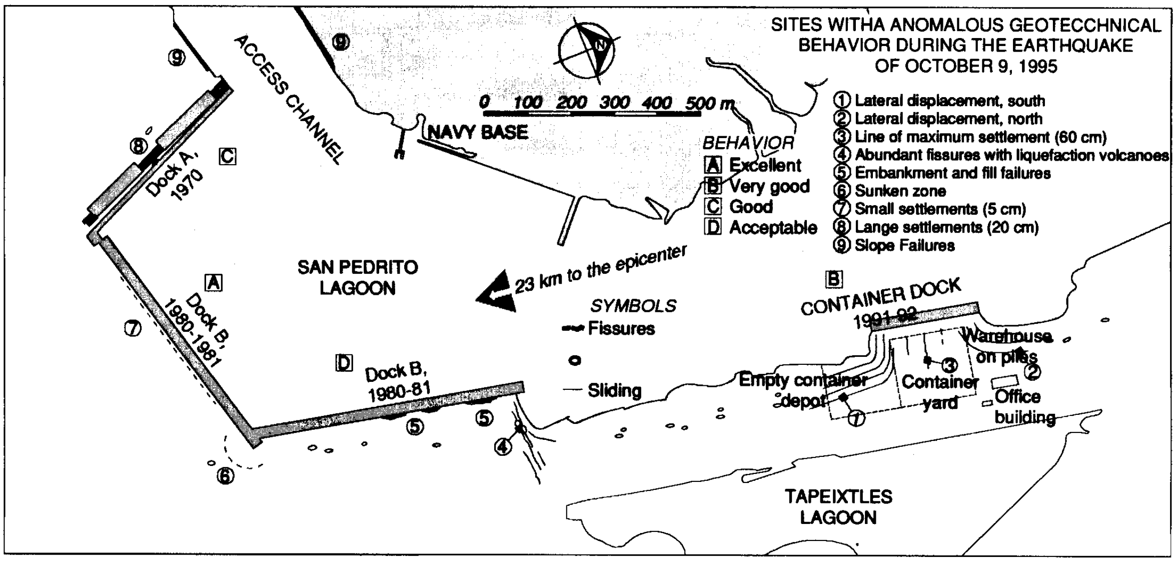

Fig 3. Damage and damage distribution in the San Pedrito Harbor

SAN PEDRITO HARBOR FACILITIES

Background

The first dock in San Pedrito was built in 1970 and two more in 1981 and in 1988. Land was also reclaimed from the lagoon to build a road and a railway during the early seventies, dividing the lagoon into two parts. The newest facility is a Container Terminal that was commissioned in 1992 (see fig 3).

Liquefaction in San Pedrito Harbor

The liquefaction potential of the sandy fills was estimated as part of the geotechnical studies performed before the construction of the Container Terminal, in 1990 and 1991. The analysis was carried out using the Seed and Idriss (1971) simplified approach with the results of standard penetration tests which showed that the fills would liquefy if maximum ground surface accelerations exceeded 0.15 g, the recommended value for designing the facilities at the Container Terminal, according to a seismological study performed in early 1985 (Jica, 1985). The maximum ground acceleration for design purposes was raised to 0.2 g as a result of this study, but the fills were left uncompacted.

From the results of a soil exploration program performed after the earthquake, which included SPT and CPT soundings as well as seismic cone tests, it was concluded that liquefaction could occur again in future earthquakes and that even relatively dense materials would suffer it during earthquakes similar to the Octorber 1995 event. These studies were complemented with a laboratory investigation into the undrained behaviour of the fills and are reported elsewhere (Ovando *et al*, 1999).

Structural characteristics of the docks

Structurally, the docks follow a common design for this kind of facilities: reinforced concrete slabs supported by vertical and battered piles. The Container Terminal has one dock divided into five modules, 50 x 21.6 m, each having an independent reinforced plane concrete slab, 45 cm thick, with a separation of 2 cm between the slabs. The piles, shown in fig 4, are also reinforced concrete elements and their cross section is square, 50 x 50 cm.

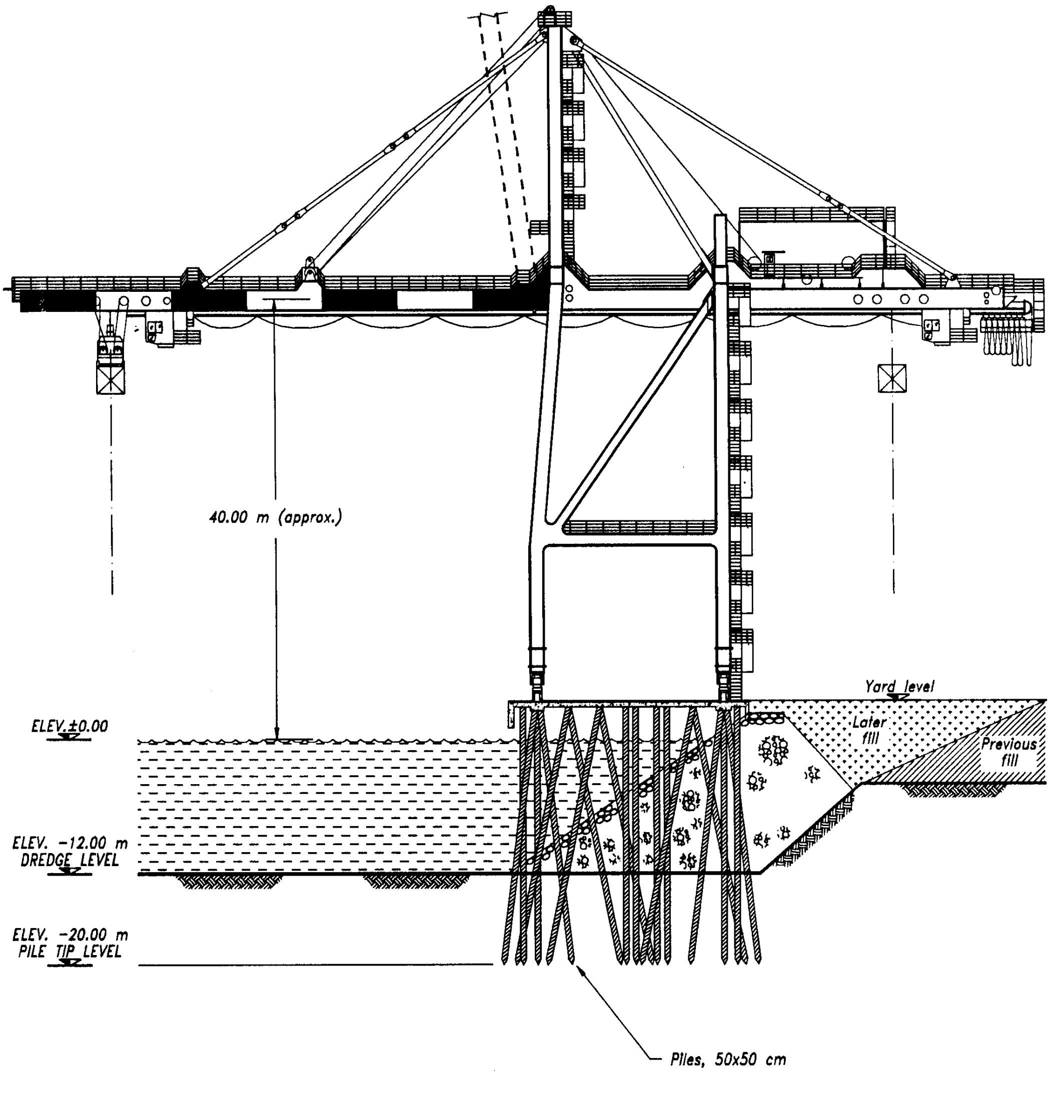

Fig 4. Cross section of the wharf in the Containers Terminal in San Pedrito

Constructive method

The docks were constructed by first driving the piles, with their tips located some 8 m below the seabed. After removing the seafloor mud, coarse gravel was then dumped between the piles to form an embankment that was finished with rockfill faces. Three auxiliary embankments perpendicular to the dock were built to facilitate the construction of the main one. The seaward toe of this embankment was aligned with the outer line of piles, as seen in fig 4. Silty sands dredged from the lagoon were placed behind the embankment with the hydraulic fill method. These materials now constitute the subsoil that underlies the Container Terminal yards.

DAMAGE IN THE CONTAINER TERMINAL DOCK

A detailed inspection was carried out in the weeks following the earthquake including an underwater survey to examine the full length of the piles (Rodríguez, 1996). Damaged piles were divided into two categories: a) cracked piles, i.e. piles that underwent structural damage that reduced considerably their seismic resistance; b) fissured piles, that is, piles without significant structural damage.

Most cracked piles were located in the landward side of the dock and had been provided with additions cast after the installation of the piles; 90 % of them were battered, with inclinations towards the seaward toe. Furthermore, the shearing patterns observed at their connection with the platform slab suggest that during the earthquake these piles were subjected to strong thrusts from the embankment as it displaced horizontally. Inadequate structural behaviour at the joints precluded the transmission of stress to the piles inclined towards the inland side of the dock. There were 29 piles damaged in this way, from a total of 1,170 piles, i.e. 2.5%. The authors conluded that these piles were either unproperly driven, or moved during the construction of the embankment, or both. Hence their inadequate behaviour during the earthquake.

Fissured piles that did not undergo structural damage were distributed more or less randomly. There were 164 of these which represent 14 % of the total number of them.

Damages were also observed in the slabs, at the connections with the battered piles. These resulted in the punching failure of the structural elements at these joints brought about by high shearing forces and bending moments produced by horizonatal displacements. This mode of failure is also consistent with the hypothesis that the embankment settled and displaced horizontally during the earthquake. The number of failed joints was 53, 4.5% of their total number.

Assessment and interpretation of the observed behavior

During the post earthquake inspection it became apparent that pile heads in many vertical or inclined piles were displaced from their projected position. Pile movements during construction induce additional stresses that are seldom accounted for in design and will thereby increase their seismic vulnerability. This results from flaws in supervision and control during construction. On the other hand, even under the best conditions possible, piles inclined towards the seaside will naturally tend to be displaced and even vertical piles driven are prone to move on account of thrusts produced as the embankments is being formed.

Despite the failures at the pile heads or at structural joints, the dock was not in danger of collapsing after the earthquake but its seismic resistance reduced substantially and its ability to survive future large earthquakes was judged to be doubtful. The observed damage pattern suggests that the embankment settled and displaced horizontally, hence large dynamic thrusts were exerted on the piles, probably influenced by the liquefaction of the loose granular fills retained by the embankment.

Analyses and expected behavior

Rehabilitation of the dock after the earthquake required immediate action and an emergency plan to carry out the necessary work. The repair and retrofitting project was formulated on the basis of an elastic 3-D finite element analyses of the dock in which the seismic excitations were defined by means of the design spectrum discussed previously. A cross section of the model used in the analyses is shown in fig 5. Runs were made using a commercially available computer program. A critical aspect for modeling the behavior of the dock was the ability to simulate the interaction of the piles with the underlying soils and with the embankment. In view of the urgent need to start the rehabilitation of the Container Terminal, the interaction effects were simulated conservatively by means of horizontal subgrade reaction coefficients that were converted into equivalent spring constants. A detailed discussion of the results is provided by others (Rodríguez et al, 1996).

A reassessment of the seismic safety is presently being undertaken in which dynamic P-Y curves are being incorporated in the analyses, following a formulation that accounts for the strain dependency of the relevant soil moduli and that can include radiation and hysteretical damping (Romo and Ovando, 1999).

LESSONS LEARNED

The seismic response of San Pedrito port facilities added valuable information regarding the behavior of batter pile systems and upon how rock embankments used for slope stability and soil-slope-erosion protection endanger the integrity of the pile-system foundation when shaken by an earthquake.

Marginal wharves supported on batter pile systems have been widely used in many locations around the world because they generally provide the deck area needed to carry out loading and unloading operations of container facilities. This type of foundation systems, which include both batter and vertical piles, are a preferred structural foundation element for marine facilities because they allow foundation construction in dry conditions, as opposed to gravity-tie wharves structures. However, there exists an increasing bulk of information which shows that batter piles do not fare well under earthquake loading. In general, batter piles in ports have been particularly susceptible to damage because of their large lateral stiffness which mobilizes large lateral seismic forces at pile cap and decking connections. This usually leads to severe cracking and fracture piles, piles cap and decking. These damages could be prevented if the piles are provided with adequate strength and the connections have sufficient ductility to resist lateral displacements, bending moments and shear forces caused by seismic forces. Fig 6 shows earthquake-induced damage to batter piles in San Pedrito wharf. It may be seen that failure occurred both in compression and shear due to low ductility as has been observed in many ports like that of Los Angeles, Oakland, Kobe, etc.

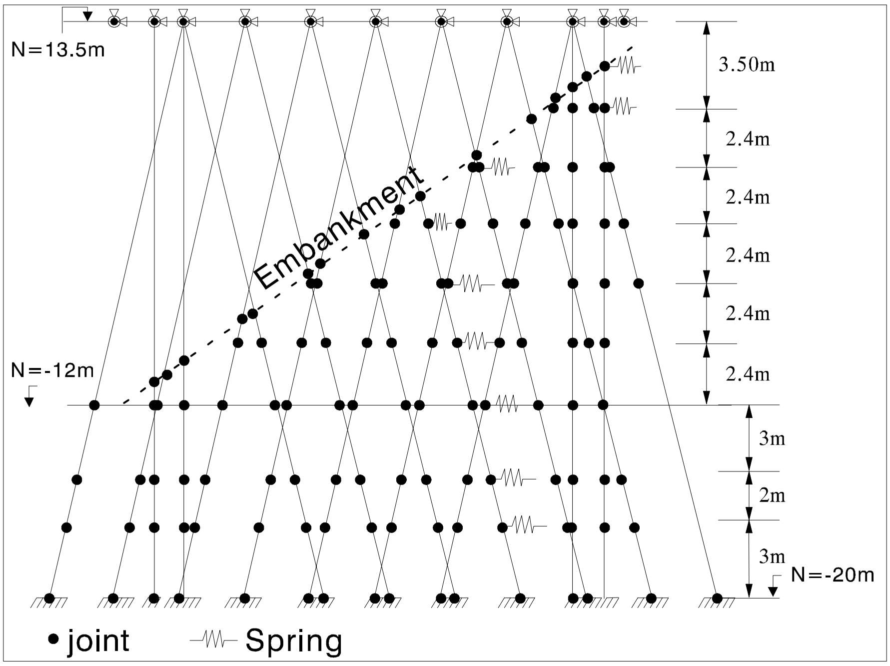

Fig 5. Finite element mesh for the structural analysis of the dock at the Container Terminal (after Rodríguez et al, 1996)

Vertical piles perform well when they have been designed to work with the pile cap and the overlying deck structure as a ductile moment frame capable of accomodaitng deck and ground lateral displacements. In San Pedrito Port vertical piles did not undergo appreciable damage, but it is important to note that vertical piles in liquefable soils have to be designed with sufficient ductility and anchorage to withstand large concentrated curvatures caused by the reduced lateral stiffness of a sand layer when it liquefies. Lateral pile displacements bring about secondary or P-delta effects on the element shear forces, axial forces and bending moments that must be included in the analysis of pile systems.

Fig 6. Earthquake induced damage to batter-piles in San Pedrito wharf

The multi-lift rack embankment that underlies the deck of the wharf at San Pedrito Port reduces the free length of the piles landward, making variable the lateral stiffness of the pile group. This, undoubtly leads to a chaotic response of the pile-deck system in the inland-seaward direction, inducing additional dynamic loading concentrations that contributed to pile cracking. Furthermore, the seismic excitation caused slumping, densification and lateral displacements of the rock dike that, in turn, produced more lateral forces to the pile foundation. From the observed behavior of this port it may be concluded that rock-dike structures are likely to cause large lateral forces on the piles. Since the dike slopes towards the sea, these forces are applied at different pile heights, an aspect that must be taken into account when designing the pile foundation.

From the discussion above it becomes apparent that wharf-problem modeling for rational design purposes is still an elusive task. Although there are many numerical tools like finite element, boundary element and finite difference procedures that can handle non linear soil-structure interaction problems, the main problem in designing wharf systems is that these structures and their boundary conditions involve so many variables that their modeling is an uncertain task. To better understand the behavior of these type of structures it is neccesary to have more information about the quantitative response of wharves to seismic excitations. To fullfill this need it is important that existing and future wharves structures be instrumented to monitor their response to seismic loading. The instruments that should be installed would at least include strong motion accelerometers deployed in the free field and on the deck, pore water pressure meters and ground deformation transducers installed within key geotechnical structures of the port, pressure cells in the piles, inclinometers along the piles and dike embankments.

CONCLUSIONS

The Manzanillo earthquake of 9 October, 1995, damaged a non negligible amount of piles and structural joints in the docks at the Container Terminal at the San Pedrito Harbour. As a result, seismic resistance of these docks was reduced. Seismological data suggest that an even larger event may occur within the next few decades in the same region.

The project for rehabilitating the docks in San Pedrito was the result of an emergency plan which must be revised in the future, since seismic risk in the region is high. The reassessment of the seismic safety of the docks must incorporate the lessons learned from the observation of the damages found in the docks after the earthquake. Among these, it was found that the constructive process had a detrimental effect on the behaviour of the piles that support the dock in the Container Terminal. Battered piles inclined towards the seaward toe of the retaining embankment may have been damaged during construction and, consequently, underwent severe structural damage during the earthquake. Damage of these piles was produced by horizontal displacements of the embankment enhanced by the liquefaction of the loose granular fills retained by the embankment.

On the other hand, rock-dike structures are likely to produce large lateral forces on piles. Even under optimum conditions battered-pile systems for supporting decks have not fared well during strong earthquakes and instrumental observations are needed for understanding their behavior.

Future analyses must include an improved means for modelling the dynamic interaction between the pile and the soil or the piles and the embankment. It is also necessary to re-examine the criteria followed to specify the design spectrum for the wharves and docks in Manzanillo.

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