Construction with Prefabricated Caissons vs. Open-piled quay wall in Marine Works

Case Study - Expansion of Terminal XXI quay wall

(extended abstract)

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Abstract: The PSA Sines proceeded in late 2009 to the second phase of expansion of Terminal XXI quay wall to duplicate the berth pier to 730 meters. The solution provided to the expansion was completely identical to the existing one in prefabricated caissons, however an open-piled quay wall was constructed.

Port facilities have an economic importance increasingly relevant, including the Port of Sines. On the other hand port structures have been damaged in several earthquakes over the past decades, involving high repair costs and causing significant economic impacts in the region. Having said that evaluating port structures seismic resistance assumes then a special importance.

Hence the purpose of this dissertation is to develop a study of the two solutions proposed to the expansion. Open-piled quay walls have clear advantages in terms of hydrodynamic behavior, construction schedule and cost. However from the seismic performance of the Caisson quay walls are expected minor damages after strong ground movements.

Keywords: Caisson quay wall, Open-piled quay wall, Seismic design, Terminal XXI

1 Introduction

Portuguese Company for Harbor Works – CPTP won the quay expansion contract with a structural design variant to the base project. The solution provided for the expansion was identical to the existing one in prefabricated caissons. However the solution adopted consisted of a beamed slab of reinforced concrete fully cast in situ using a sliding formwork system supported on reinforced concrete piles.

The occurrence of an earthquake can cause such structural and non-structural damage that it jeopardizes not only the safety of people and equipment, as well as the operation of the port, involving high repair costs and causing significant economic impacts in the region. Evaluating port structures seismic resistance assumes then special importance.

For developing this study a contextualization of the Terminal XXI was done with the evolution of the port, the existing structure and the natural conditions of his surroundings that enable the expansion of it. Then an introduction to the caisson and open-piled quay wall modes of earthquake damage, followed for guidelines regarding seismic design. This allowed choosing the more accurate methodology to check their seismic resistance, and then perform a finite element dynamic analysis of both solutions. Finally studying on construction schedule and cost, structural and hydrodynamic behavior of each solution is made, factors which may influence the type of port structure to build.

2 Terminal XXI

The Port of Sines has the main advantage of having excellent maritime accessibility, with sufficient deep water where no maintenance dredging is needed and is devoted to the service of large vessels
due to the lack of restrictions on navigation channel that leads to a quick responsiveness to vessels. Also there’s a possibility of expansion by creating embankment areas adjacent to the quay (Figure 1).

All these features make the Terminal XXI able to service large container ships, as it is demonstrated by his current pace of development (Figure 2).

This leads to the development plan for the expansion of Terminal XXI having as main objective to serve the megacarriers and super megacarriers containerships. The expansion of the quay to 730 meters will allow simultaneous berthing and operation of two post-panamax vessels, stimulating the movements in the terminal and increasing the competitiveness of the port across borders.

Regarding the existing quay Terminal XXI’s infrastructure comprises a gravity wall in caissons with a backfill, which is a common solution used in the Port of Sines. The caissons are founded at EL -16.5 m and its total height, including base slab, is 18.5 m (31.75×19×18.5 m), reaching EL +2.0 m (mean water level) after placement, and at the top a beam from EL +4.0 to +6.1 cast in situ. The total height of the wall is 22 meters. The area intended for the expansion of the quay develops over several meters among EL -15.0 and -17.0 m which makes possible futures expansions (Figure 3).

Soil conditions have a great influence on the choice of the infrastructure, and, for example, if the soil is loose and has a low bearing capacity it would serve no useful purpose to build a gravity quay wall type

Figure 1 – General layout of the Port of Sines and Terminal XXI highlighted, from Google Earth

Figure 2 – Evolution and growth of TEU handling at Terminal XXI, adapted from [1]

Figure 3 – Terminal XXI aerial view after construction of the 1st phase [1] and in 2nd construction phase
of structure. It would be better to consider an open type founded on piles driven down to the rock. Therefore reliable and complete soil investigations must be carried out at the site of a future new berth structure. The SPT tests conducted mention the existence of soil with adequate strength for mobilization by the foundation elements.

3 Seismic response of port structures

The damage observed on the occurrence of an earthquake depends not only on the local magnitude of the earthquake but also from the structural features of the quay. Evaluating the performances of each structural system in the event of an earthquake can introduce improvements and effective reduction of risk.

3.1 Gravity quay wall

A gravity quay wall is made of a caisson or other rigid wall put on the seabed, and maintains its stability to the pressures induced from the landfill through friction at the bottom of the wall. Caissons may generally only be used where the seabed is good and the risk for settlement is low.

Typical failure modes during earthquakes involve seaward displacement, settlement, and tilt. For a quay wall constructed on a firm foundation, an increase in earth pressure from the backfill plus the effect of an inertia force on the body of the wall result in the seaward movement of the wall, as show in Figure 4 a). When the subsoil is loose and excess pore water pressure increases in the subsoil the movement of the wall is associated with significant deformation in the foundation soil, resulting in a large seaward movement involving tilt and settlement, as show in Figure 4 b) [2].

Case histories for gravity quay walls with foundations of the same nature subjected to the same seismic action presented different behaviors, and caissons with lower width-to-height ratio displayed larger displacements towards sea [2]. If the width-to-height ratio of the wall is small, less than 0.75, tilt may also be involved and be the predominant mode of deformation over horizontal displacements.

Horizontal displacement and uniform vertical settlement may be generally acceptable from a structural point of view as it may not significantly reduce the residual state of stability, and can be easily repaired. Concerning the tilt of the wall, it reduces the residual stability and leads to unacceptable structural stability situations plus to infeasible repairs given the large size of the caissons.

3.2 Open-piled quay wall

A pile-supported wharf is composed of a deck supported by a substructure of piles and a slope. It is most widely used in areas with large water depth or if the seabed is too weak to carry a massive gravity quay wall plus the ground condition below being suitable for bearing piles.

The seismic response of pile-supported wharves is influenced mainly by complex soil-structure interaction during ground shaking. This structural system resists the earthquake-induced lateral loads.
by bending of the piles and associated moment resistance [3].

Three causes of failure may be identified for a pile-supported quay [2]. When constructed on a firm foundation having a rigid and stable slope, the seismic inertia force on the deck will be the main cause of failure, as shown in Figure 5 a). If there is an excessively large displacement at the top of the slope, the deck will be pushed seaward, as shown in Figure 5 b), and when founded on a loose subsoil failure due to liquefaction or caused by lateral displacement of loose subsoil, as shown in Figure 5 c).

Most pile failures are associated with liquefaction of soil which can result in buckling of the pile, loss of pile friction capacity, or development of pile cracking and hinging.

![Figure 5 - Deformation modes of pile supported-wharf, [2]](image)

The most widespread source of earthquake-induced damage to port and harbor facilities has been liquefaction of the loose, saturated, sandy soils, that occurs even under moderate levels of seismic action. The strength and stiffness of the soil is reduced due to generation of interstitial pressure during ground shaking and the water pressure in the pores of saturated soil deposits increases to a level were particles momentarily float apart. Annex B of Eurocode 8 - Part 5 [4] presents empirical diagrams for a simplified evaluation of liquefaction potential based on SPT tests. Solutions for the prevention of liquefaction can be divided into two categories, soil improvement for reducing the probability of liquefaction and structural solutions to minimize damages.

### 4 Seismic performance evaluation and analysis

The evaluation of resistance to earthquakes should be wisely studied in order to have a seismic behavior appropriate to the socio-economic importance of the port. The most commonly method of structural design is based on the assurance of absence of collapse of the structure and the safeguarding of human life for a given seismic loading.

#### 4.1 Current seismic provision for port structures

Among other existing codes and publications globally used in various ports exists Eurocode 8. The methodology of it describes in general a dual-level approach, with two performance levels [4]:

- “No collapse” requirement: Retain structural integrity and a residual bearing capacity.
- “Damage limitation” requirement: No damage and no limitation of use, whose costs would be extremely high compared with the cost of the structure itself.

For piles, it is stated that they must be designed to remain elastic and when this is not feasible, guidance is given for the design of potential plastic hinging and the region it will cover.

For retaining structures, the seismic horizontal \(k_h\) and vertical \(k_v\) coefficients affecting all masses should be [4]:
\[ k_h = \alpha \frac{s}{r} \]  
\[ k_v = \pm 0.5k_h \text{ if } a_{eg}/a_g \text{ higher than } 0.6 \]  
\[ k_v = \pm 0.33k_h \text{ in other cases} \]

Where: \( \alpha = \frac{a_g}{g} \) is the ratio between the design ground acceleration on type A ground \( a_g \), and the acceleration due to gravity \( g \); \( s \) factor depends on the subsoil class; \( a_{eg}/a_g = 0.75 \) from Table NA-3.4 National Annex to EC8 [5]; \( r = 2; 1.5; 1 \) depending on free gravity walls acceptable displacements.

For a linear analysis design of pile-supported structures are defined design spectra. The \( a_g \) value corresponds to a return period of 475 years.

### 4.2 Analysis methods for quay walls

The analysis method to consider depends on the type of port structure, and its complexity may vary depending on the class of importance, study phase or local seismicity.

**Simplified analysis:** For retaining structures is based on the limit equilibrium method. Caissons can be idealized as rigid blocks of soil and structural masses. Earthquake motions are represented by a seismic coefficient for use in conventional pseudostatic design procedures (Figure 6). Caisson capacity to resist seismic force is evaluated in terms of a threshold seismic coefficient, beyond which the caisson begins to move.

Analysis for pile-supported structures is typically done modeling the pile/deck as linear single degree-of-freedom system.

**Dynamic analysis:** For retaining structures as for the pile/deck system is based on soil-structure interaction using Finite Element Model (Figure 7). Seismic loading is represented by accelerograms or response spectrums. Soil is idealized either by equivalent linear or by an effective stress model.

### 5 Caissons quay wall

Caissons predicted for the new solution are very much similar to the existing quay infrastructure described, having the same height and placed at the same level, however not as large in plant (19.0×20.0 m). The sea side rail is founded on this infrastructure while the landside rail is based on a beam founded on piles, and therefore independent from the caisson structure.

#### Analysis to seismic action

Assessment of caissons quay wall resistance to seismic action was done using a simplified analysis by a pseudo-static method [2], based on a conventional approach of balance of forces as the weight of the structure, overloads in the backfill, static pressures, hydrodynamic pressure, and the resulting effects of seismic action as the acceleration of the structure’s mass and the active pressures.

This analysis is appropriate for evaluating the level of stability to sliding, overturning and bearing capacity.
Seismic action is defined according to section 7.3.2.2(4) from Eurocode 8 – part 5 [4], previous equations 1, 2 and 3 of this text. For the seismic action type 1, conditioning in a pseudo-static analysis and for the values of National Annex to Eurocode 8 [5] we have \( k_h = 0.198 \) and \( k_v = \pm 0.099 \).

Forces acting on the retaining structure on the side field, according to Eurocode 8 – Part 5 [4] are,

\[
E_d = \frac{1}{2} \gamma^* (1 \pm k_v) K H^2 + E_{ws} + E_{wd}
\]

(5.1);

where: \( H \) is the height of the wall; \( E_{ws} \) static pressure of water; \( E_{wd} \) hydrodynamic pressure of water; \( \gamma^* \) specific weight of soil; \( K \) earth pressure coefficient (static + dynamic);

The earth pressures coefficients are estimated using the Mononobe-Okabe (see Figure 6) equation present in Eurocode 8 – Part 5 [4]:

\[
K = \frac{\sin^2(\psi + \phi_d' - \theta)}{\cos \theta \sin^2 \psi \sin(\theta - \delta_d)} \left( 1 + \frac{\sin(\phi_d' + \delta_d) \sin(\phi_d' + \theta)}{\sin(\phi_d' - \delta_d) \sin(\theta - \delta_d)} \right)^2 \quad (5.2);
\]

\[
K = \frac{\sin^2(\psi + \phi_d' - \theta)}{\cos \theta \sin^2 \psi \sin(\theta - \delta_d)} \quad (5.3);
\]

where: \( \phi_d' \) design value of internal friction angle of the soil and \( \phi_d' = \tan^{-1} \left( \frac{\tan \phi'}{\tan \psi} \right) \); \( \delta_d \) design value of friction angle between soil and wall \( \delta_d = \tan^{-1} \left( \frac{\tan \delta}{\tan \psi} \right) \); \( \theta \) angle of inertia force due to seismic action defined by:

\[
\theta = \tan^{-1} \frac{k_h}{k_v} \quad \text{(above water level)} \quad (5.4);
\]

\[
\theta = \tan^{-1} \frac{\gamma_{sat}}{\gamma_\psi} \frac{k_h}{k_v} \quad \text{(below water level)} \quad (5.5);
\]

Water pressure from the free water in front of the structure during seismic shaking is set by the Westergaard equation [7]:

\[
E_{wd} = \frac{7}{12} k_h \gamma_w h_d^2
\]

(5.6);

Once all acting forces are estimated to attest the conditions of stability of the caissons under seismic actions the following combination was considered [7]: Deadweight + Static active earth pressure from backfill + 50% Overload + Seismic action.

**Stability to sliding:** To ensure caissons stability to sliding the following condition needs to be satisfied:

\[
H_{d,dstb} \leq R_{d,stb}
\]

(5.7);

\( H_{d,dstb} \) is the design value for total horizontal destabilizing forces and \( R_{d,stb} \) for total stabilizing ones.
For an earthquake acting upwards it was verified a coefficient (0.6) lower than unit. This happens when a negative vertical seismic coefficient is used which “reduces” the influence of the caisson’s weight. According to [8] in areas of high seismic activity gravity, quay walls with a sliding factor greater than the unit are uneconomical. This publication recommends accepting permanent displacements provided they are not excessive and don’t compromise the stability of the structure.

![Figure 9 – Simplified analysis for seismic analysis of caissons](image)

**Stability to overturning:** For stability to overturning the following criteria must be satisfied:

\[
M_{d,destb} \leq M_{d,stb}
\]  

(5.8);

where: \(M_{d,destb}\) is the sum value of the destabilizing moments; \(M_{d,stb}\) is the sum value of the stabilizing moments (both calculated to the edge that would be the axis of rotation). The worst situation occurs when the vertical component of the earthquake acts upward, which gives us a security factor of 1.1.

**Bearing capacity:** To satisfy the condition of loading resistance of the foundation the following criteria must be satisfied:

\[
V_d \leq R_d
\]  

(5.9);

where: \(V_d\) is the tension design value transmitted to the ground surface; \(R_d\) is the tensile strength of the foundation surface. The worst case scenario occurs when the vertical component of the earthquake acts downwards however stress is below the maximum permissible stress (560,0 kN/m² < 900 kN/m²). The entire foundation slab is subject to compression.

**Damage limitation**

To evaluate the displacement of the caisson under seismic action a dynamic analysis was performed using FEM. Effects of earthquake motion were represented by an accelerogram generated from the elastic spectra of Eurocode 8 [9] using software GOSCA [10] (Figure 10).

![Figure 10 – Accelerogram generated from type 1 earthquake (return period of 95 years)](image)

![Figure 11 – Deformation under seismic action of computational modeling (PLAXIS) of caissons (displacements zoomed 100x)](image)

The horizontal and vertical displacements at the caisson’s top, as show in Figure 11, confirms that it
will slide towards the sea, however only 40 and 15 mm respectively. This flexion/rotation should not cause instability to the caisson and therefore will be acceptable.

6 Open-piled quay wall

The open-piled quay wall designed for the extension of the quay consists of a beamed slab deck of reinforced concrete fully cast in situ with 0.45 m thick, 36.55 m width, 12 m spans and a top elevation at +6.10 m, supported on Ø1.30 m concrete piles with 25 m long spacing 4.0+9.5+9.5+8.75 m, and driven 4 m in the bedrock (Figure 12). It’s a sequence of four similar sections with 78+101+90+78 m for a total 347 m deck. To reduce the effects from variations in temperature and concrete shrinkage, the deck has transverse joints with cross beams at the ends of the sections.

![Figure 12 – Cross section of the open-pile quay wall, from [11]](image)

The horizontal loads due to seismic action are transmitted to the foundations level, developing horizontal loads and concentrated moments supported by the side reaction of the soil generating interaction between structure and soil. Thus to analyze the earthquake action a FEM (SAP2000) was used via a linear dynamic analysis according to Eurocode 8 [9]. This model is a fairly plausible simulation of the structure and leads to satisfactory results, ranging from the vibration modes, extensions, displacements, strains and stresses.

Analysis to seismic action

The most precise modeling of this foundation involves inelastic Winkler springs [12] along the piles as, shown in Figure 7, characterized by a proportionality constant (linear elastic behavior):

\[ K_s = n_h \cdot Z = K_{mola,i} = n_h \cdot Z_i \]  \hspace{1cm} (6.1);

where \( n_h \) is a horizontal reaction module (5000 kN/m³) from [12] and \( Z_i \) the node’s depth and so near the soil ground is where the reaction module has the most influence.

Earthquake action was quantified through a modal analysis that enables the dynamic performance of the wharf, demonstrating the various vibration modes and deformations (Figure 14). Three significant vibration modes (of 20) were identified: two translations (in two principal directions) and a torsional. Total mass was mobilized at the 4th mode.
Seismic action was defined using Eurocode 8 [9] response spectra. The definition of these spectra takes into consideration five parameters: the type of earthquake; seismic zone; structure location, class of importance, foundation soil type and the damping coefficient considered (Figure 13).

Effects of simultaneous seismic excitation in orthogonal horizontal directions were considered,

\[ E_{Ex} = E_{Edx} + 0.30 \times E_{Edy} \]  \hspace{1cm} (6.2); \hspace{1cm} \[ E_{Ey} = 0.30 \times E_{Edx} + E_{Edy} \]  \hspace{1cm} (6.3);

being \( E_{Edx} \) and \( E_{Edy} \) the earthquake loads in the principal directions \( x \) and \( y \), respectively.

![Figure 13 – Response spectra for seismic action](image)

![Figure 14 – First three modes of vibration (1 to 3) from FEM (SAP2000)](image)

**Evaluation of the seismic resistance of the structure**

According to Eurocode 8 – part 2 [13] wharf should be checked for the following two seismic load combinations: Dead load + 50% Design live load + Seismic action (\( E_{Ex} \) or \( E_{Ey} \)).

The maximum bending moment occurs at the row of pile heads most landward, as these piles have the shortest unsupported length. It's the seismic combination in the X direction that determines the maximum stresses at the edge of the wharf, except for A alignment. Alignments D and E require almost twice the longitudinal reinforcement of the remaining.

![Figure 15 – Stress values for seismic combinations](image)

<table>
<thead>
<tr>
<th>stress</th>
<th>section</th>
<th>Edge of the wharf's deck (x=3,0 e 87,0 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td>( N_{Ed} ) (kN)</td>
<td>base</td>
<td>-2774</td>
</tr>
<tr>
<td></td>
<td>top</td>
<td>-2213</td>
</tr>
<tr>
<td></td>
<td>shaft</td>
<td>-2467</td>
</tr>
<tr>
<td>( M_{Ed} ) (kN.m)</td>
<td>top</td>
<td>1439</td>
</tr>
<tr>
<td></td>
<td>shaft</td>
<td>482</td>
</tr>
</tbody>
</table>

**Table 2 – Maximum stress values for seismic combinations**

The following charts depict the combined flexure and axial load interaction diagram (spreadsheets from mpa – The Concrete Center) where's achieved the resistance to seismic action according to Eurocode 2 (Figure 15).
Figure 15 – M-N interaction diagram for circular columns [14] for the top (16 bars) and shaft (14 bars) of the extreme piles (x=3 e x=87 m)

It’s on the top section of the piles where stresses are more critical to D and E alignments. It would take an reinforcement equivalent to 16Ø40 to check the Ultimate Limit State.

**Damage limitation**

To evaluate the displacement of the piles under seismic action a dynamic analysis was performed using FEM. The seismic action for damage limitation was quantified by reducing the elastic spectra by a reduction coefficient (0,4) as predicted in the National Annex to Eurocode 8 [5].

Figure 16 and 17 – Displacements of wharf’s computer modeling (extreme piles x = 3 and 87 m)

Minor vertical displacements along the deck show that it will tilt towards the sea but won’t cause instability to the quay. Small horizontal displacements are also observed however piles are sufficiently flexible to accommodate them and are considered to be acceptable (Figure 16 and 17).

**7 Evaluation Caissons vs. Open-Piled quay wall**

Different factors affect the choice of port structures. Here are some of the most important:

**Soil conditions:** Terminal XXI natural soil conditions are suitable for the two studied solutions. Direct foundations would require local dredging, which is expensive and involves mobilization of equipment.

**Hydrodynamic behavior:** Some energy of the wave action against the berth structure is dissipated, and some is reflected, which generates new waves in the front of the structure. The wave reflection is sometimes a problem due to the additional agitation created in the harbor basin. Open-piled quay wall
for not being a vertical obstacle presents itself more favorable as the wave energy is dissipated by the slope armor stone.

**Construction schedule:** Since it's an expansion of a running terminal, and assuming that construction works may hinder the operations during the construction period, a constructive solution that saves time is probably advantageous even if its construction cost is higher.

Within the gravity wall structures a production of 3-5 linear meters per week is expected [3]. For the construction of the open-piled quay wall CPTP achieves a production of 12 linear meters per week. This will save 4 months in the work plan of the open-piled quay wall construction against caissons.

**Construction costs:** Generally open berth structures are relatively cheaper than solid structures the greater is the water depth at the front [3].

The corrosion of reinforcement is usually the most serious problem related to the durability of concrete structures in marine environments. Caissons are completely submerged but open-piled quay could be more vulnerable due to the splash zone below the deck. Maintenance costs with open-piled quay can be higher.

**Structural behavior:** Generally solid berth structures are considered more resistant to loadings than the open berth structures. Since the deadweight of caissons constitutes a greater part of the total structure weight than the deadweight of pile-supported, caissons are less sensitive to overloading [3].

Pile-supported quay provides a better solution for the gantry crane rail since it's a monolithic solution rejecting the risk of differential settlement between land and sea side rails.

**Final Considerations**

Pile-supported solution is the most economically advantageous, dispenses underwater work, previous dredging, less equipment and therefore lower costs with mobilization and preparatory work. Also brings advantages to the construction schedule, and dissipates the wave energy on the harbor basin. Caissons have a high internal stiffness and thus a greater ability to resist vertical and horizontal loads. However the pile-supported quay provides a better solution for the gantry crane rail in order to be serviceable after an earthquake.

From the seismic performance of the caisson quay wall there are expected minor damages after strong ground movements. Insufficient seismic resistance of pile-supported solution would result in total destruction of the pier, while seismic failure in caissons might be repaired. Seismic analysis performed at caissons obtained minor displacements and rotations although with minor significance. The use of finite element identical software would have been desirable, as well as the same analysis method.

**References**


