Technical Note

DESIGN AND CONSTRUCTION OF A CAISSON QUAY WALL IN THE SOUTH PARS PORT PROJECT

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ABSTRACT

The paper presents a new quay wall design applied in the South Pars Port Project. The quay is constructed of 7 metre diameter circular caissons made of reinforced concrete ring elements stacked upon each other. The method was developed in order to avoid construction of a dry dock needed for casting of conventional caissons. The design conditions, design loads and the stability calculations are explained as well as the from the construction of the quay.

Keywords: quay, caisson, seismic design, caisson stability

1. INTRODUCTION

The South Pars harbour at Assaluyeh on the coast of the Persian Gulf is under construction. At this stage, ten quay walls with different water depths are planned. Figure 1 shows the layout of the port and the location of the caisson quay discussed in this paper.

Originally a steel sheet pile quay wall for 5000 DWT vessels and water depth up to 11 m was planned. However, the soil conditions for the sheet pile was questionable as was the long-term durability of the structure. Therefore, a proposal was submitted for a concrete caisson structure involving a 65% saving in foreign currency. Conventionally, caissons are cast in a permanent (if available) or a provisional dry dock, or cast floating after launching the bottom part cast in the dry. Various cost optimisation studies showed that circular caissons, if made of precast relatively small ring elements that could be placed in-situ by a small land based crane, would be an economical solution, provided that the structure would have sufficient stability against loadings including earthquake impact.

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2. DESIGN CONDITIONS

British Standard, Code of Practice for Maritime Structures, Parts 5, 6 and 7 (1991), Technical standards and Commentaries for Port and Harbour Facilities in Japan (1999), and the PIANC Seismic Design Guidelines for Port Structures (2001), were used as basis for the design calculations. The quay is designed for accommodation of a 15,000 DWT cargo vessel with the following dimensions:

- Overall length 153 m
- Moulded breadth 22.3 m
- Moulded depth 12.5 m
- Full load draft 9.3 m

For cargo ships:
- GT is 0.541 DWT

Where
- GT: gross tonnage
- DWT: deadweight tonnage

The values of the GT and the international GT can differ from one another (see ref.4).

Figure 1. Layout of the port project
The bollard load is 600 KN, the berthing velocity is 0.3 m/s perpendicular to the quay front, and the berthing angle is 15° degrees. Fender loadings from a moored vessel will be smaller than loadings from berthing. Maximum berthing energy is 24.5 t.m. The astronomical tidal range is 1.9 m. Water depth at the quay is 11 m relative to CD (Chart Datum).

Normal, extreme and temporary (construction) loading conditions were analysed. Soil conditions were investigated on the basis of five borings drilled approximately five metres into the seabed (see Table 1).

Table 1: Geological features 5 meters drilled into the seabed

<table>
<thead>
<tr>
<th>From seabed surface to 0.5 m depth:</th>
<th>sand, algae and cobble (to be dredged)</th>
</tr>
</thead>
<tbody>
<tr>
<td>From 0.5 m to 2 m depth:</td>
<td>Weathered coral reef</td>
</tr>
<tr>
<td>From 2 m to 3.6 m depth:</td>
<td>Moderately weathered coral reef</td>
</tr>
<tr>
<td>From 3.6 m to end of boreholes</td>
<td>Fresh coral reef</td>
</tr>
</tbody>
</table>

A simplified method for earthquake stability analysis was used in which equivalent static loads are applied. Seismic zone 1 (peak ground surface acceleration 0.05-0.15 g for 475 years return period), subsoil class 1, and importance category A for the structure was assumed.

Design seismic coefficient = seismic zone coefficient x subsoil condition factor x importance factor:

\[ K = 0.15 \times 0.8 \times 1.2 = 0.144 \]

Apparent seismic coefficient for soil pressure when submerged

\[ K' = \frac{\gamma}{\gamma - 1} K = 0.304, \quad \gamma = 1.9(tf/m^3) \]

Being the unit weight of water – saturated soil in air.

Equivalent seismic force = (dead weight + surcharge) x design seismic coefficient. Internal water table was set 1 m above the water level at the quay. Applied surcharges were: uniform load 50 KN/m², and crane pad load 500 KN acting in one square metre.
3. QUAY STRUCTURE

Figure 2 shows the cross section of the quay wall. Figure 3 shows a caisson ring element.

Figure 3. Reinforced concrete caisson ring element
4. STRUCTURE ELEMENTS AND STABILITY ANALYSES

Considering the natural soil conditions after removal of the soft seabed top layer, and the structure materials, it was found that liquefaction will not be a problem. Consequently liquefaction is not included in the overall stability calculations.

4.1. Overall stability of the quay wall

4.1.1. Foundation bearing capacity
The caissons for the quay walls are subjected to earth pressure from behind the caissons, surface loads from behind the caissons, dynamic water pressure during earthquake and to bollard forces acting on top of the caissons [6]. A difference in the water level from behind the caisson to the front of the caisson of 1.0 m has been assumed. The gravity force for one caisson is calculated to be about 835 tons. With the vertical friction forces the total vertical base plate load on the seabed surface will be about 943 tons. To ensure foundation bearing capacity the internal friction angle of the rubble and the soil beneath the caissons has to be at least 33°. This requirement is fulfilled.

4.1.2. Sliding failure of caisson
The caisson has been examined for a possible sliding failure. The sliding situation has been calculated both for a normal load situation and for an earthquake situation. According to the Japanese standard a friction coefficient of 0.7 can be used for the sliding between concrete and gravel. Calculations showed that for the normal load situation the necessary friction coefficient is 0.3, and for an earthquake situation the necessary coefficient is 0.5. Consequently a sliding failure will not be a problem.

4.1.3. Overall slip failure
The total stability for caisson and tie rod anchor plate was analysed. In the actual situation only friction soils are to be considered. The surface load was placed only where it contributes to the driving moment. The slip failure was examined for an anchor length of 24 m. For the normal load situations a safety factor of more than 10 was found. For the earthquake situation a safety factor of 2.1 was found. When the caissons are built on soft ground conditions, an extended rubble foundation is used to counteract slip failure of the ground [7].

4.2 Caisson ring elements
The caisson ring design is based on filling with sand or quarry run. The reinforcement in the caisson rings has been checked for the following combination of the loads:
- Pressure from the soil inside the caisson. No outside pressure from soil behind the caisson is applied. The combination of the loads corresponds to the situation, where the caissons have been filled with soil but not backfilled. For this load case a difference of 1 m in water level inside and outside the caisson has been assumed.
- Inside pressure from the soil together with a berthing load from a 15000 DWT ship.
The ring elements exposed to reaction forces from the fenders was filled with concrete in order not to increase the ring reinforcement. The rings were connected by stainless steel bolts in order to avoid rings moving apart from each other, as it is important that the structure acts as a monolith. The connections will be subjected to shear forces and bending moments. In the completed structure the maximum shear force will appear in the top and in the bottom of the caisson while the maximum bending moment will be close to the middle of the caisson. To be on the safe side it was decided that the ring connections should take all the shear force although there are tongue and groove joints between the elements, Figure 3. The reason for this is that even small tolerances for the joints would allow the rings to slide a bit and as a consequence the connection bolts will get large deformations before the concrete tongue and groove joints will be effective. It was found that the connection bolts Ø32 CK-45 are able to carry the shear forces. For the bending moment there was no problems. The number of bolts varies with respect to the elevation of the caisson rings.

The static behaviour of the structure as described above was changed as the client asked for filling the caisson with mass concrete. If concrete fill had been initially considered as an economic solution a somewhat cheaper type of ring element caisson could have been used.

4.3. Tie rods
The tie rod anchoring of one Ø 100 mm diameter CK-45 steel rod per caisson ensure the overall stability as the rather small diameter of seven metre of the caissons is insufficient for gravity based stability. A safety factor of 1.7 for extreme load conditions was applied for the design of the tie rod. Corrosion of the steel, placed in soil above sea level in the Persian Gulf, is assumed to be 0.02 mm per year.

4.4. Deadman anchorage
The deadman anchorage of the tie rod consists of a continuous reinforced concrete wall, 1.5 m high and 2.0 m thick. A safety factor of 2 was applied in calculating the needed capacity of the anchorage.

5. CONSTRUCTION ASPECTS

The construction procedure was as follows:

5.1. Procedure and the equipment
Figure 4 shows the gantry crane used for the land base construction. The original seabed was dredged and a horizontal flat gravel base was placed using simple equipment designed on the site. The bottom ring element was placed on the gravel, and tremie concrete was pumped inside it, making penetration of the concrete into the base. All the underwater work, e.g.. bolt fixing was done by diving.
5.2 Gantry crane
The Gantry crane used for placing of ring elements has a capacity of 1250 T-m. The total weight is 300 ton. The movement of the crane is controlled by joystick, fully electronic and remotely operated. The crane is equipped with an underwater video camera. The design and construction of the crane was done by Iranian people in Perlite Company. Figure 5 shows the construction activities, before the completion of the quay.
5.3. Time needed for construction

The facilities on the site can produce ring elements needed for one complete caisson structure in one-day. The time for placing, filling and other activities needed for completing the work for each caisson is in average three days. Figure 6 shows the construction activities of the fender panels before installation of fenders. In total for one kilometre of quay wall a one-year construction period was sufficient in one working shift. However, the production could be increased to three kilometres in one year. This is due to the gained experience and available equipment as well as possibility of using more working hours.

![Figure 6. Photograph of the caisson quay under construction at Assaluye site](image)

6. INNOVATIVE USE OF RING ELEMENT CAISSONS FOR BREAKWATERS

The easy and fast production of ring elements together with the easy construction procedure, which is not sensitive to moderate wave action, makes it possible to use ring elements for breakwater caissons in which case mass concrete should be used as fill. Circular caissons have been successfully used at Hanstholm, Denmark and at Brighton, U.K. (see Ref. [8]).

7. CONCLUSIONS

A new quay wall design on circular caissons built up of reinforced concrete elements is presented. Design conditions and stability investigation procedure are also included. It is believed that the structure is economic providing the environmental and material supply
conditions on the Iranian coast of the Persian Gulf.

REFERENCES