types of structures, with pile foundations often penetrating through weak soil layers. Liquefaction, lateral spreading, and slope stability are all special items that are primary considerations for the analysis and design of piers and wharves. The proposed standard also codifies the displacement-based design methods that have become widely used in the industry and are supported by a number of industry-sponsored testing programs.

This paper will focus on the detailing of pile-to-deck connections and presents an overview of test data for the connections that are considered in the proposed standard, along with permitted strategies for reducing earthquake damage at pile-to-deck connections.

OVERVIEW

Pier and wharf construction throughout the United States typically consists of cast-in-place or precast deck superstructures supported by precast, prestressed concrete or steel pipe piles connected to the deck by methods that depend on location and project requirements. The goal of the proposed standard is to provide mandatory provisions and commentary for all commonly used connections that have been proved by testing to be ductile and suitable for seismic applications.

The proposed standard recognizes that selection of the pile-to-deck connections should be based on the relationship among a number of parameters such as the type of structure (pier or wharf), seismic demand, length of pile, type of pile, soil stiffness, magnitude of other lateral loads such as waves and mooring, post-earthquake reparability, corrosion, and life cycle cost. Therefore, connections may have full or partial moment capacity. These connections are listed according to pile type, e.g., steel pipe piles or prestressed concrete piles. Committee members reviewed the literature and gathered test data for all the connections discussed in the proposed standard (see Table 1). Other connections are permitted provided their performance is verified by testing and/or appropriate analysis.

TERMINOLOGY

A brief description of several key pile-to-deck connection terms used in the proposed standard follows (see Figure 1).

Capacity Protected. Elements designed to elastically resist forces and moments imposed by adjacent yielding element(s).

Interface. The construction joint between the top of the pile and the deck.

Joint Region. The portion of the deck above the pile which transfers forces and moments from the pile to the deck.

Plastic Hinge Zone. The region of the pile and/or connection where inelastic action occurs in an earthquake.

Strain Penetration Length. The distance of dowel debonding on each side of the interface.

Full Moment Connections. Connections which develop the moment capacity of the pile. The connections may or may not fix the pile head against rotation.

Partial Moment Connections. These connections are under-reinforced at the interface with respect to the body of the pile. As a result, the inelastic behavior and damage is confined primarily to a short length of the dowels just above and below the connection interface, i.e., the strain penetration length.

Table 1. **Pile-to-Deck Connections Considered in the Proposed Standard**

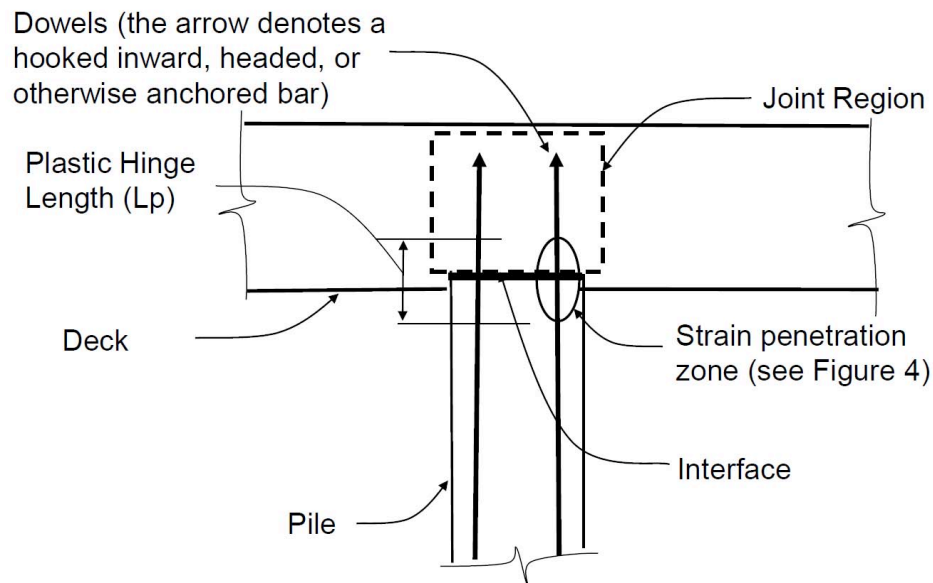| Connection Type	Full Moment	Partial Moment
Pipe Pile Connections		
Embedded Pile	X	
Pile Welded to Embed Plate	X	
Dowelled with Embedded Shell		X
Dowelled with Isolated Shell		X
Welded Dowels		
Prestressed Concrete Pile Connections		
Embedded Pile	X	
Pile Build-Up	X	
Extended Strand	X	X
Dowelled	X	X
Hollow Dowelled		X
Isolated Interface		X

Figure 1. **Pile-to-deck connection terms.**

PIPE PILE CONNECTIONS

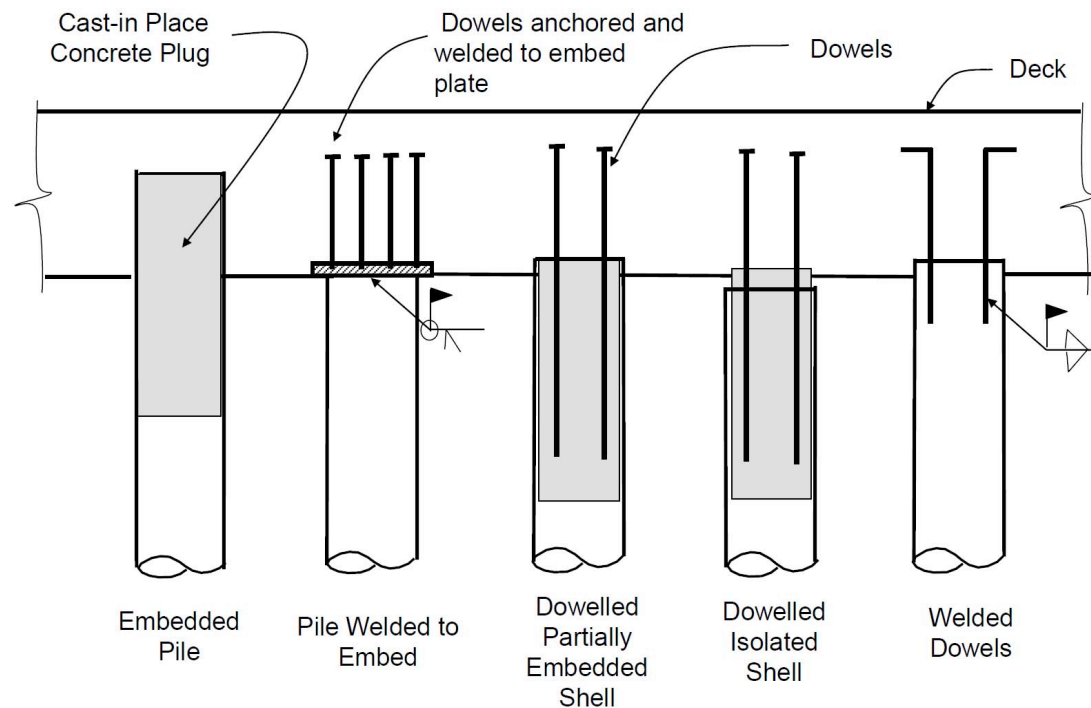Figure 2 illustrates the pipe pile connections listed in Table 1. A summary of the significant test data follows.

Cast-in Place Concrete Plug

Dowels anchored and welded to embed plate

Dowels

Deck

Embedded Pile

Pile Welded to Embed

Dowelled Partially Embedded Shell

Dowelled Isolated Shell

Welded Dowels

Figure 2.   Pipe pile connections.

Embedded Pile. Tests by Stephens and McKittrick (2005) for piles embedded one diameter and loaded in the long direction of a bridge bent cap indicate it is possible to develop the plastic moment in a pipe pile and effectively fix the pile head against rotation.

Pile Welded to Embed Plate. Tests by Steunenburg et al. (1998) indicate the plastic moment capacity was developed by welding the pipes to thick plates anchored to the precast pile caps with deformed bar anchors stud-welded to the plate. However, a significant number of the deformed bars fractured at the weld. Therefore, the proposed standard will have criteria to ensure the plastic hinge forms in the pile and the anchorage is capacity protected.

Dowelled with Partially Embedded or Isolated Shell. These connections as shown are usually considered partial moment connections, as it takes a large amount of reinforcement to develop a plastic hinge in the pile. Both connections have been shown to achieve remarkable ductility. Tests by Park et al. (1983) on concrete filled and partially embedded pipe piles connected to the deck with reinforcing bars extending from the concrete into the deck exhibited extremely large ductility with dependable moment capacity, and the connections exhibited strengths well in excess of theoretical predictions due to the large lever arm created by bearing of the pile

shell on the deck. Tests by Silva et al. (2001) used pipe piles with discontinuous casing at the interface to minimize the overstrength moment.

Welded Dowels. This connection, which uses reinforcing field welded to the pile shell. No test results were found for this connection. Therefore, the connection is not permitted if inelastic performance of the dowels is desired, as the presence of the weld creates the potential for brittle fracture of the dowel.

PRESTRESSED CONCRETE PILE CONNECTIONS

Figure 3 illustrates the prestressed concrete pile connections listed in Table 1. A description of the significant test results follows.
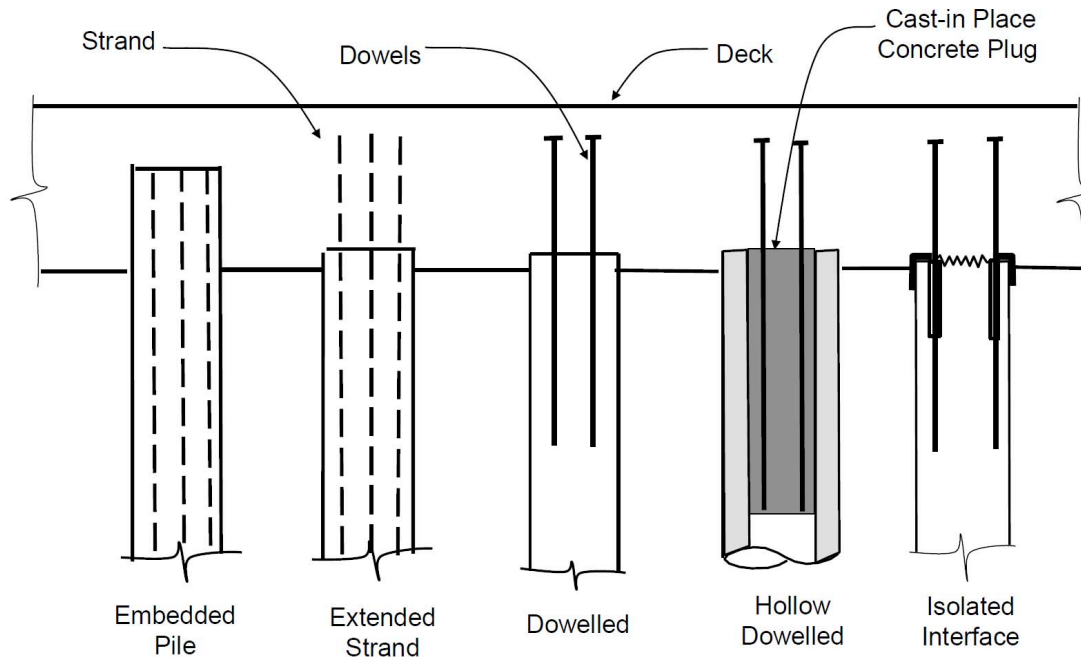

Figure 3.   Prestressed concrete pile connections.

Embedded Pile. Tests of this connection by Harris and Petrou (2001) under constant axial loading determined that with adequate embedment length, the connection would develop the moment capacity of the pile. The proposed standard sets the minimum embedment at one pile diameter or 50 strand diameters.

Extended Strand. Joen and Park (1990) tested six pile-cap connections. All of the connection details developed the moment capacity of the pile while exhibiting satisfactory ductile behavior. The proposed standard allows extended strand connections, provided the strands are anchored into the deck and undamaged.

Dowelled. This is the most commonly used connection for pier and wharf construction and has been the subject of significant research. The performance of prestressed concrete piles with dowelled connections varies depending on the amount of reinforcement and the axial load on the pile. This is illustrated in tests by Restrepo et al. (2007) on 24-inch prestressed concrete piles with typical Port of Los Angeles

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(POLA) connections using either eight #10 or four #9 bars. The eight #10 bar connection develops the moment capacity of the pile and is used for short piles at the bulkhead with large seismic moments (referred to as seismic piles by POLA). The four #9 bar connection is used for longer piles with smaller seismic moments referred to by POLA as a secondary-seismic pile. In tests of both connections, a large crack developed at the pile-deck interface while a few hairline cracks developed in the piles. The seismic pile performed adequately but exhibited a significant reduction in capacity when the cover spalled at a ductility demand of about 3. Other tests by Roeder et al. (2001), Priestley and Sritharan (2004), Blandon (2007), and Jellin (2008) indicate similar performance for dowelled connections.

Hollow Dowelled. Dowels are commonly extended from a concrete plug due to the difficulty of extending reinforcement from the shell into the deck. The committee was unable to obtain relevant test data for this connection. However, calculations indicate that if the connection is too strong, longitudinal cracks may develop in the shell due to splitting forces caused by horizontal reactions of the plug bearing on the shell. Therefore, it is recommended that this connection be designed as a partial moment connection.

Isolated Interface. Tests by Jellin (2008) indicate the seismic performance of a prestressed concrete extended dowel connection is significantly enhanced by isolating the pile-to-deck interface, and sleeving the dowels to minimize spalling of the deck and reduce the strain in the dowels. Compared to the conventional connection, the isolated interface connection permits larger pile rotation before damage, and provides larger hysteretic energies, although the tests indicate the moment capacity of the connection is reduced compared to the dowelled connection due to the smaller lever arm. The isolation material should be of closed cell foam or other material suitable for the marine environment.

PILE-TO-DECK CONNECTION KINEMATICS

Tests on dowelled connections often find rotation at the pile-to-deck connection, concentrated at the interface between the pile and the deck and attributable to the type of prying action shown in Figure 4. During the cyclical earthquake loading, the gap opens and closes repeatedly, resulting in these events—generally in this order.

1. Tension and compression yielding causes the deformations to grind an annulus in the concrete adjacent to the bar. This debonding is also known as strain penetration.
2. Deck spalling occurs on the tension side of the connection due to prying.
3. Deck spalling takes place on the compression side of the connection due to bearing.
4. Cover spalling occurs on the compression side of the pile due to bearing.
5. Yielding of the spiral is present, which results in loss of confinement of the concrete core and reduction in lever arm and shear strength.
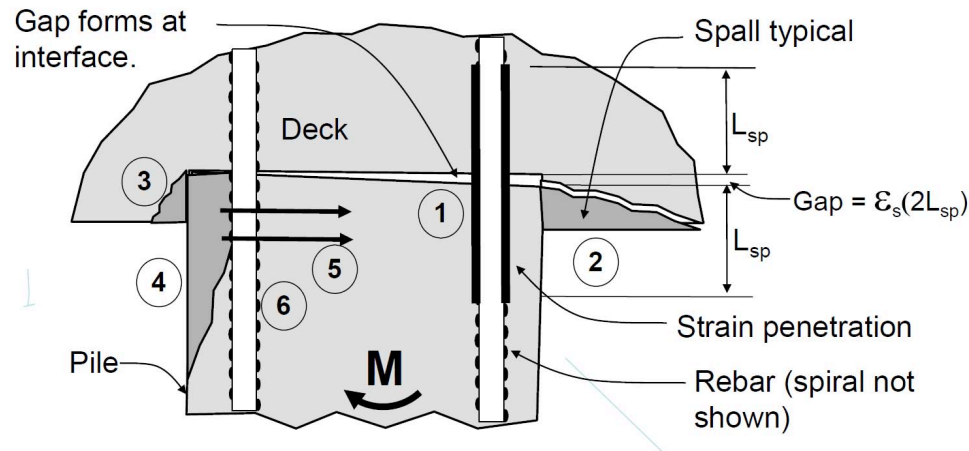6. The dowels buckle and fracture.

Figure 4.   Pile-to-deck connection kinematics.

The strain penetration length, L_{sp}, is commonly defined by the following expression (Priestley and Park 1984).

$L_{sp} = 0.15 \ f_{ye} \ d_b; \ L_{sp} = 0.022 \ f_{ye} \ d_b \ (f_{ye}$ in ksi, MPa) (Equation 1)

where

L_{sp} is the strain penetration length (inches, mm), taken as equal in the pile and deck

f_{ye} is the expected yield stress (ksi, MPa) and d_b is the bar diameter (inches, mm)

In the proposed standard, the plastic hinge length, L_p, will be defined as twice the strain penetration length for all dowelled connections.

To illustrate the magnitude of the deformations at the interface, consider that for a connection with #10 bars at f_{ye} = 66 ksi (442 MPa), the calculated strain penetration length is 12.3 inches (312mm) and the plastic hinge length is 24.6 inches (625 mm). The bar elongation, i.e., the gap at the interface, is obtained by multiplying by the strain in the bar by the plastic hinge length. At the OLE and CLE strain limits of 0.01 and 0.05, the gap is therefore 0.23 inches (6 mm) and 1.1 inches (28 mm), respectively.

DAMAGE REDUCTION STRATEGIES

As shown in Figure 4, the damage to a prestressed pile connection is progressive and results in connection failure. All of the behaviors are avoidable except for strain penetration. There is no strain penetration if the connection remains elastic. Figure 5 illustrates three strategies to reduce damage.

External Confinement. The first strategy is similar to that used for retrofit of concrete columns to produce a highly ductile connection, as illustrated by the

performance of dowelled pipe pile connections.   Unfortunately, this strategy introduces an additional element, increases complexity and cost, and does not prevent deck spalling.

Reduced Moment Capacity.   The second strategy, used by POLA for secondary seismic piles, uses an under-reinforced connection to reduce the moment demand and minimize spalling of the pile cover, but does not prevent deck spalling. The reduced demand keeps the joint region uncracked, thereby allowing the dowels to be anchored below the top mat of deck reinforcement to simplify construction (Restrepo et al. 2007).

Interface Isolation.   The third strategy uses interface isolation on the top to prevent concrete-to-concrete contact at low to moderate rotations, thereby minimizing pile damage (Jellin 2008). Figure 6, (Jellin 2008) shows the effectiveness of the isolated connection versus the dowelled connection as the damage to the dowelled connection is substantially less than the dowelled connection.
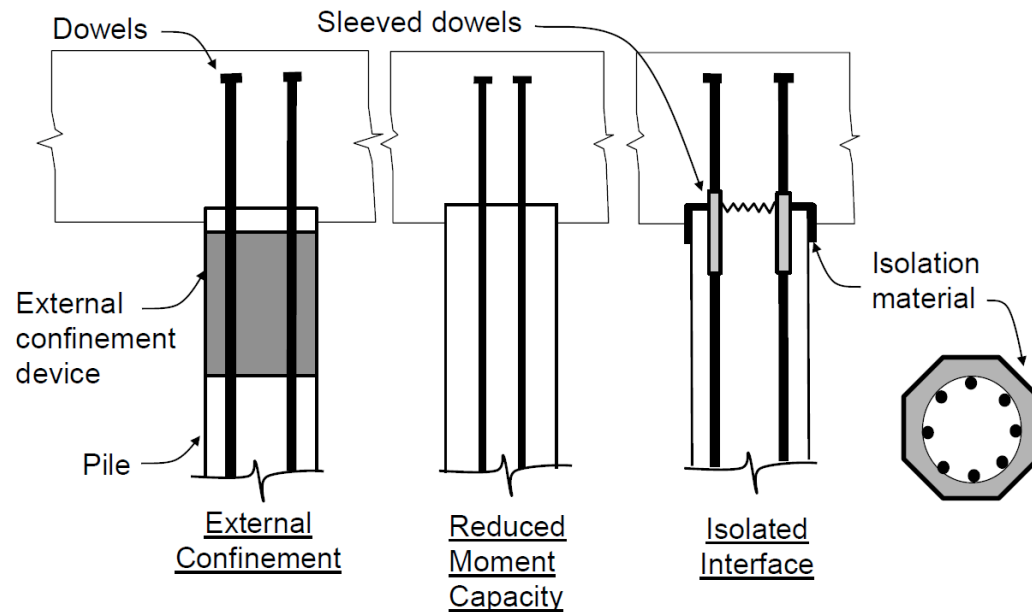

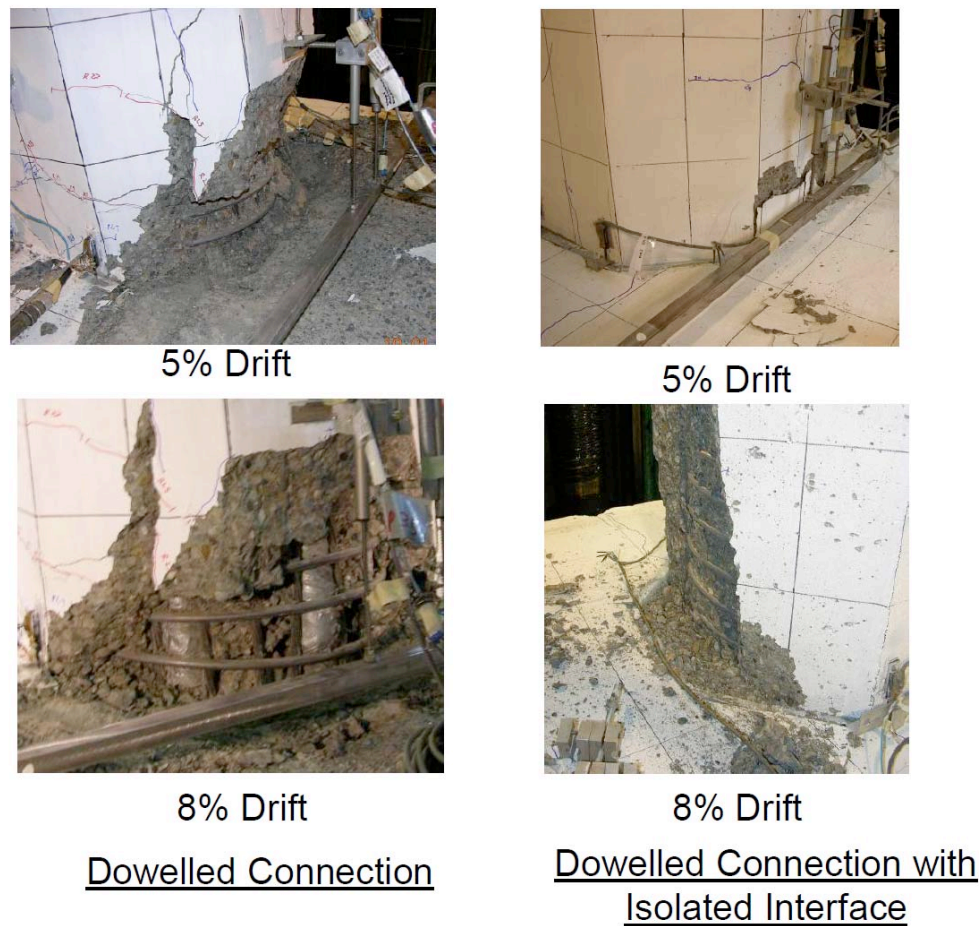Figure 5.   Damage reduction strategies.

Figure 6. **Performance of dowelled versus isolated interface connections.**

SUMMARY

The proposed standard codifies displacement-based design methods that have become widely used in the industry over a period of several years and are supported by a number of industry-sponsored testing programs. The paper described detailing of pile-to-deck connections and presented an overview of the test data for the connections considered along with strategies for reducing earthquake damage at the pile-to-deck connections.

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Seismic Design of Marine Caisson Waterfront Structures

C. Brodbaek, P.E. [1], P. Laursen, P.E., Ph.D.[2]

[1] Regional Manager, Ben C. Gerwick, Inc., 200 West Mercer Street, Ste. E506, Seattle, WA 98119; cxb@gerwick.com

[2]Assistant Professor, California Polytechnic State University, Bldg. 21-221B, San Luis Obispo, CA 93407; plaursen@calpoly.edu

ABSTRACT

The objective of this paper is to describe state-of-the-art procedures for seismic design of concrete caisson structures in a marine environment. The proposed caissons are cellular self-floating reinforced concrete structures that are prefabricated, towed to the project site and submerged to their final position. They are often used as waterfront retaining structures or as breakwaters for new ports, and LNG terminal developments. This paper investigates caissons of representative dimensions located at a typical water depth.

The paper is concerned with the seismic design using the finite element method in a combined model for both structural and geotechnical design of the caissons and the foundation. The structural design is driven by seismic soil loads from fill behind the caissons and inside the caisson cells. The soil-structure interaction is modeled through non-linear time-series. Seismic foundation design of waterfront structures is often performance based with maximum allowable permanent displacements of the caisson as the design criteria. Structural design of the cellular walls is based on effective soil pressures. Results, in terms of maximum displacement, caisson tilt and soil pressures are presented and compared to conventional design methods available in the literature.

INTRODUCTION

In a marine environment float-in concrete caissons are used as waterfront retaining structures and breakwaters. They are towed to the project site from a casting facility and placed on a pre-prepared foundation, such as a rubble mound or level bedrock. The caissons are hereafter filled with ballast material, such as sand, to obtain adequate stability. Soil-structure-interaction plays a key-role in the seismic design of this type of structures. Advanced finite element modeling tools allow for a consistent design approach, including both structural and geotechnical analysis to determine overall seismic caisson performance and soil pressures on the caisson walls. This paper considers waterfront retaining structures, but the methodologies herein are also applicable for concrete caisson breakwaters.

GEOMETRY AND MATERIAL PROPERTIES

The geometry of the caissons and the surrounding soils retained for this assessment are defined on Figure 1. It consists of a 20m tall concrete caisson with four cells in the transverse direction. The width of the caisson varies from 16m, 20m and 24m, which is equivalent to width/height (B/H) ratios of 0.8, 1.0 and 1.2, respectively. The cell dimension in the longitudinal direction is 5m.
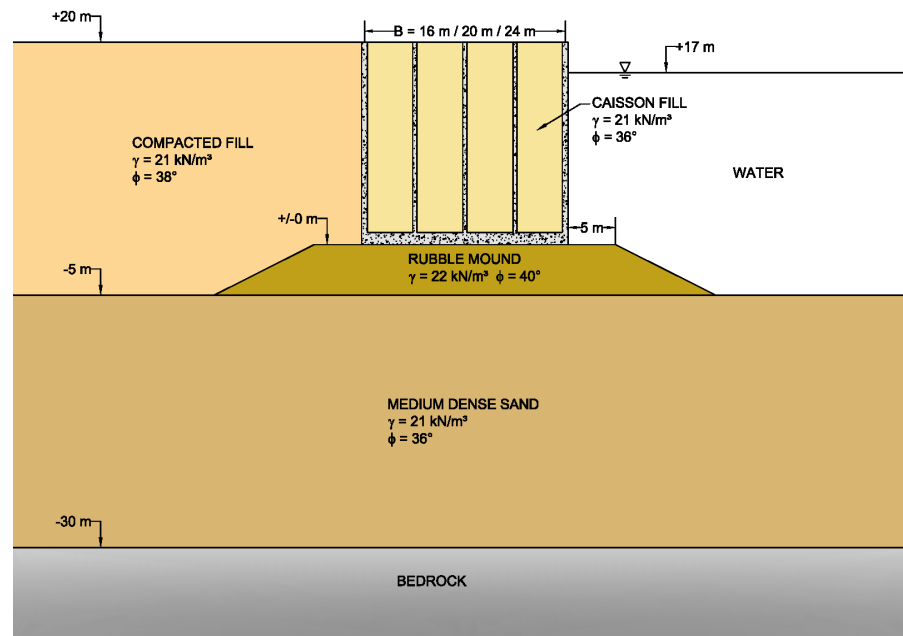

Figure 1: Definition sketch of caisson.

For simplicity, The caisson walls and bottom slab thicknesses are kept the same for the three B/H ratios: Base slab thickness, t_b = 1.2m, Exterior wall thickness, t_e = 0.6m, Interior wall thickness, t_i = 0.4m.

For each B/H ratio, a SAP 2000 (2002) finite element (FE) shell model was developed to determine the caisson displacement response due to lateral pressure applied to the exterior wall against the soil. Figure 2(a) shows the shell model representing a 5m wide slice of the caisson with B/H = 1.0. Equivalent SAP 2000 beam/truss models (Figure 2(b)) were developed to emulate the lateral caisson displacement response of the exterior and internal walls as determined with the shell models. The vertical wall and base slab beam element dimensions were taken directly from the caisson geometry while the brace and tie truss element dimensions were proportioned to accurately represent the diaphragm stiffness of the central wall in the transverse direction, such that the overall lateral displacement response of the beam/truss model closely emulated that of the shell model. The specific beam and truss mechanical properties from the beam/truss models were used in the finite element soil-structure interaction analyses carried out in PLAXIS 2D v8.6, which is a