CHAPTER 271

Seabed and Foundation Response to Wave Loading

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Abstract

The interactions between wave loading and soil response in both open waters and around the foundations of coastal structures are observed experimentally. Experimental results for wave action over sand beds are compared to analytical predictions based on both Sleath's porous bed model and on Biot's consolidation theory. Experiments on wave action in the vicinity of a rigid caisson, in conjunction with Mei's boundary layer theory, allow for the development of guidelines for interpreting the relative importance of drainage on the effective stress response of foundations to wave action.

Introduction

The geotechnical response of seabeds and foundations to wave loading can be a critical factor in evaluating the stability of proposed coastal and offshore structures. This paper reviews some of the techniques available for analysis of seabed response to wave action and presents a new heuristic approach for evaluating the relative importance of soil drainage in the response of coastal foundations.

Bea and Aurora (1981) and Wright and Dunham (1972) proposed total stress analysis for seabeds under wave loading. These techniques view the seabed loading as a harmonic tractive stress on the upper boundary, total stress analysis is used to calculate seabed response. Such analysis can use elastic, elasto-plastic or visco-elastic constitutive models and is commonly undertaken within a finite element framework. These approaches are particularly suitable to conditions where the soil is relatively soft and impermeable. These techniques have been successfully applied in analyzing the behaviour of soft soils such as Mississippi Delta muds in the Gulf of Mexico.

The pore pressure response within a seabed was originally explored by Putnam (1949) and later expanded by Sleath (1970). This analysis assumes that pore fluid flows are independent of soil stresses. This assumption is generally valid for stiff, permeable beds such as coarse sands and gravels. Yamamoto (1978) and Madsen (1978) independently proposed the use of 'poro-elastic' analysis for seabed response to wave action using Biot's (1941) linearized equations of consolidation. This allows the treatment of the coupled response to wave-seabed interactions, that is the porous

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media response coupled with the elastic soil response. This provides a technique suitable for a wide range of soil and wave conditions ranging from soft, impermeable silts and clays through to stiff sands. Since the initial work by Madsen and Yamamoto, a wide range of papers has been written proposing expansions and adaptations of the work, e.g. Finn et al (1983) and Okusa (1985). The development of full finite element solutions of time-dependent effective stress problems has provided another analysis tool. Models such as those of Shen et al (1986) provide a finite element framework within which the coupled effective stress response of soils can be investigated.

In experimental studies of soil-wave-structure interaction, measurements of soil stress or strain are difficult. Pore pressure response is, however, readily measured and is thereby often used as the main indicator of soil behaviour. Other techniques tend to be highly intrusive and fail to provide a time history of soil response (e.g. cone penetration testing). One promising technique for measurement of soil response is measurement of the acoustic emissions from the soil mass (AE). This technique was successfully employed in this research program and was described in Davies, et al (1990). The present paper focuses on the use of pore pressure response as a measure of soil behaviour.

Pore pressure in seabeds

In a porous seabed under harmonic wave loading (wave frequency, ω) the pore pressure, U of a fluid with unit weight, γ, has three components:

$$U = \gamma z + \gamma P(\cos \omega t) + u_r \tag{1}$$

Where γz is the hydrostatic component of the pore pressure at depth z below the free surface, P is the wave-induced component of the pore pressure fluctuation (expressed as a piezometric head), and u_r is the residual component of the pore pressure (due to consolidation and or shearing action). It is important to draw the distinction between the wave-induced fluctuation in the pore pressure, P and u_r, the residual pore pressure in excess of hydrostatic. The wave-induced pore pressure fluctuation is an indicator of the steady-state harmonic fluctuations in the effective stress state of the soil caused by wave loading. The residual pore pressure, u_r is a more gradual change in the pore pressure (non-harmonic) possibly caused by shearing action of the soil which results in soil volume change and consequent changes in the residual pore pressure.

Experimental Set-up—Flat Bed Testing

A series of flume tests was conducted on a sand bed consisting of 15 m^3 of a fine Ottawa sand (D_{50}=0.07mm). The sand was hydraulically placed using a positive displacement slurry pump. The resulting berm had a uniform crest height of 0.91m over a 10 m width. This uniform section was flanked by 1:10 side slopes. A vertical array of pore pressure transducers had been installed in the flume prior to construction of the berm. This consisted of 14 Druck PDCR81 pore pressure transducers (7 kPa capacity) at 10 cm vertical spacing. A capacitance-type wave gauge was located directly above the pressure transducers.

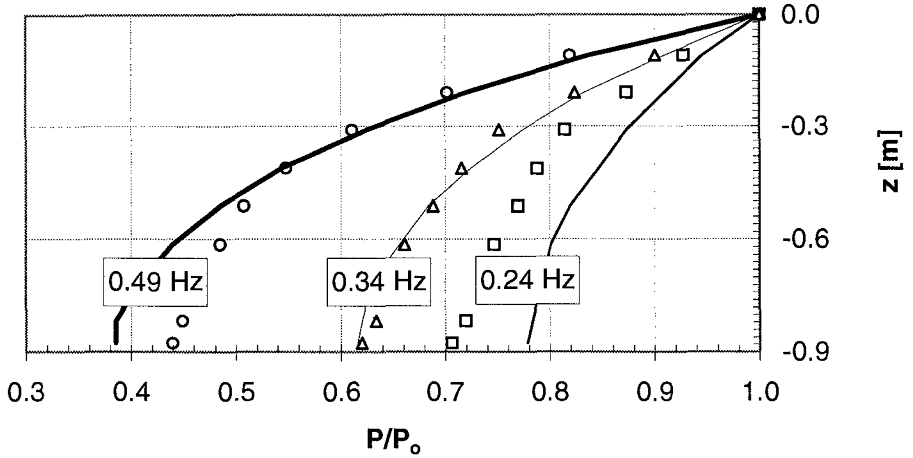

Figure 1 Vertical distribution of pore pressure as per Sleath (1970) for k_x=1.8k_z

Sand densities were determined using a 12 cm diameter brass sampling tube. Soil relative density was determined to be 47% (soil unit weight was 15.5 kN/m^3). Constant head permeability tests yielded a coefficient of permeability of k=5.0 x 10^{-5} m/s. The experimental setup is described in detail in Davies (1992).

Steady-state harmonic response

The pore pressure response measured in these experiments was compared to Sleath's rigid, porous bed model and to that predicted by poro-elastic theory as per Yamamoto, and Finn.

In general, the Sleath formulation was seen to provide a good estimate of the vertical distribution of the magnitude of the pore pressure fluctuation if the effects of anisotropic permeability are considered (see Figure 1). Here the amplitude of the harmonic pore pressure, P is normalized by P_o, the pressure at the top of the seabed. Since Sleath's model assumes a porous media flow within a rigid bed, this solution provides no information about the stress response of the soil. Poro-elastic analysis is required to examine the coupled response of the soil and the pore fluid.

In comparing the pore pressure response to that predicted by poro-elastic theory, initial comparisons were made to the model of Yamamoto, 1978. This formulation does not consider the effect of anisotropic permeability and it was not possible to obtain good agreement between the model and theory. To match the observed vertical attenuation of the pressure magnitude, the predicted phase lags were far too large – on the order of 30°. Measured phase lags in the model were not seen to exceed 15°. Consequently, Finn's Stabmax routine was employed since it allows for anisotropic permeability. Using Stabmax, good agreement could be obtained for both the amplitude and phase of the pore pressure response. This agreement is shown in Figures 2 and 3. It should be noted here that the poro-elastic models were seen to be extremely sensitive to the volumetric degree of soil saturation, S_r. Sensitivity tests showed that varying the soil saturation ratio from 98% to 99.8%

resulted in predicted phase lags at the bottom of the bed varying from 28° down to 3°, the trend being that phase lag reduces with reducing air content. These findings are described in more detail in Davies (1992).

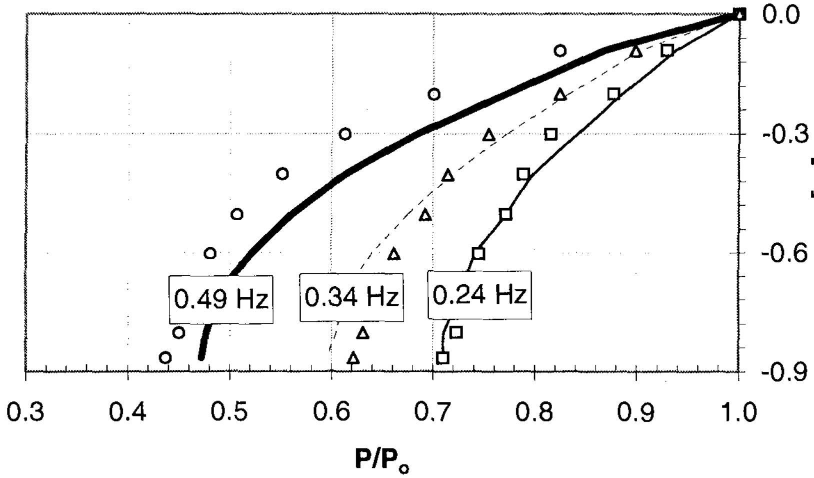

Figure 2 Measured pore pressure response vs poro-elastic theory (Stabmax, Finn, 1982)

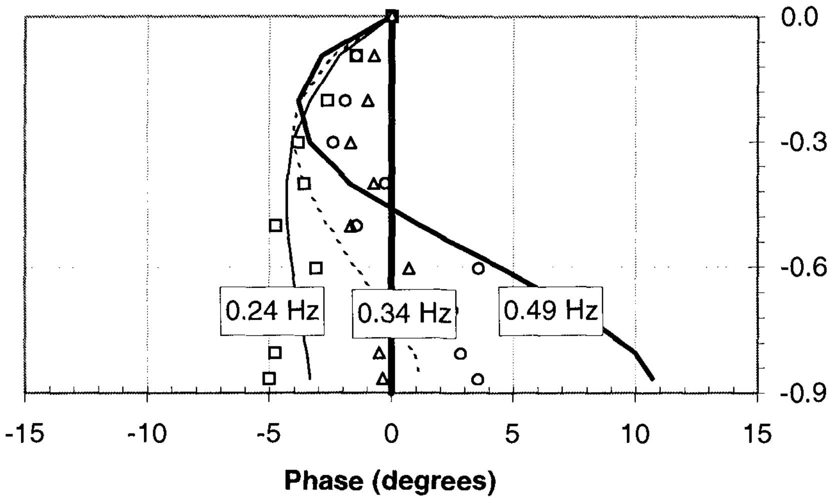

Figure 3 Phase lag of pore pressure response vs poro-elastic theory (Stabmax, Finn, 1982).

Experimental Set-up—Caisson Testing

The second experimental test program in this study was designed to examine the interactions between seabeds and rigid coastal structures under the influence of wave action. Large-scale flume tests were undertaken to study the effective stress response of seabeds beneath caissons, and to examine the combined effective stress, pore pressure and scour response of the toes of caisson-type coastal structures. This experimental study was also used to explore the validity of the Tsai et al (1990) analytical poro-elastic approach for the prediction of foundation response.

The Coastal Wave Basin of the National Research Council in Ottawa was used for the caisson testing. This basin is 85m long and 14m wide. The maximum water depth available for testing is 1.2m (allowing 0.3m of freeboard for waves). Two parallel 2.4m wide channels were built within the basin. This layout enabled the simultaneous testing of two different caisson foundations. Berms were built in each of these channels to serve as foundations for two identical steel caissons (see Figure 4). The berms were composed of two different sands; a coarse sand with D_{50}=0.38mm, and a fine sand, with D_{50}=0.10mm. Due to the large amount of fill required to build the foundations for the caisson (roughly 25 tonnes of sand in each flume) it was not possible to rebuild the berm between tests. The test sequence employed gradually exposed the caissons to irregular waves of increasing amplitude and period in a manner simulating the building of a storm. For all tests, the hydrodynamics acting on the structure and the response of both the soil and the caisson were measured in detail.

One of the results of the test program was a series of measurements of the pore pressure response within the foundations beneath the leading edge of the caissons. The following section gives a review of some of these pore pressure measurements.

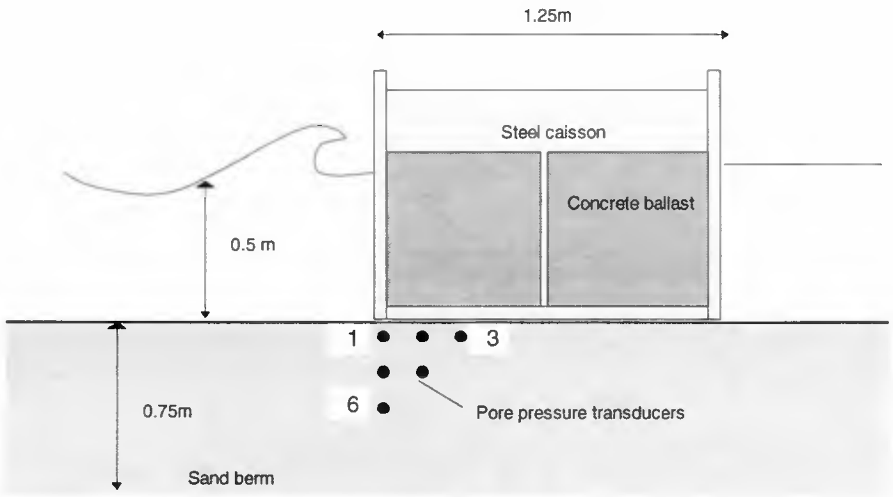

Figure 4 Schematic cross-section of caisson tests.

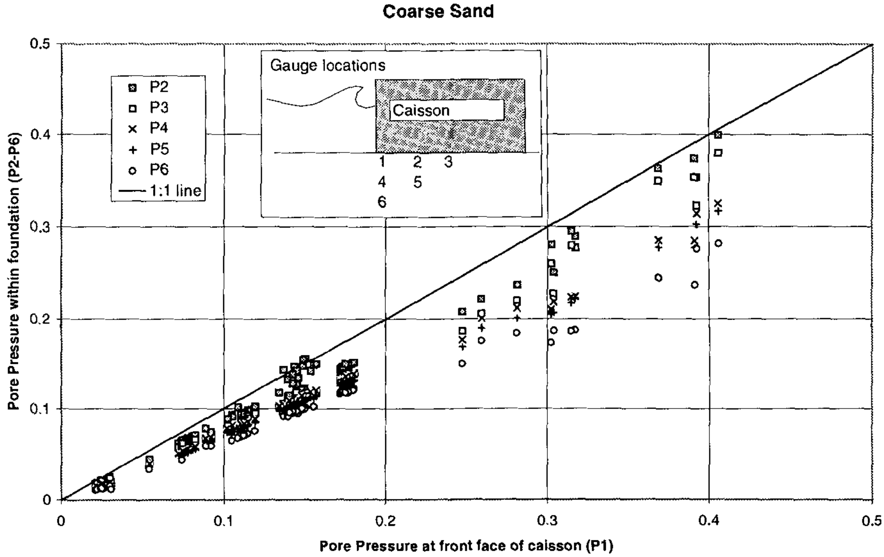

Figure 5 Pore pressure response - coarse sand.

Pore pressure response within coastal foundations

Figure 5 shows how the pore pressure response measured beneath the caisson compared to that at the outside front corner of the caisson (at the mudline) for the coarse sand foundation. This plot shows the amplitude of the pressure fluctuations measured by each of six pore pressure transducers installed beneath the caisson. These transducers are denoted as P1 through P6 and their locations are shown schematically in the inset in Figure 5. The measured pore pressure at each of gauges P2 through P6 is plotted against the pore pressure measured at the front corner (P1). There is a general trend that the pore pressure diminishes with depth into the seabed and with distance beneath the caisson, i.e. P3 and P6 are significantly lower than P1.

Figure 6 shows a similar plot for the fine sand test. The general pattern of diminishing pressure amplitude with depth and with distance from the front edge of the caisson still exists with one exception: for test conditions generating a pore pressure at the front face of the caisson (gauge P1) of around 0.1 m, the pressures along the underside of the caisson were significantly increased. This trend was not observed for the coarse sand caisson when exposed to the same wave conditions with the same caisson geometry and caisson ballast. This suggests that, under this combination of wave conditions, the finer sand caisson was experiencing complex wave-soil-structure interactions which were not observed with the coarser sand. The following sections describe a boundary layer approach to provide some insight to these interactions.

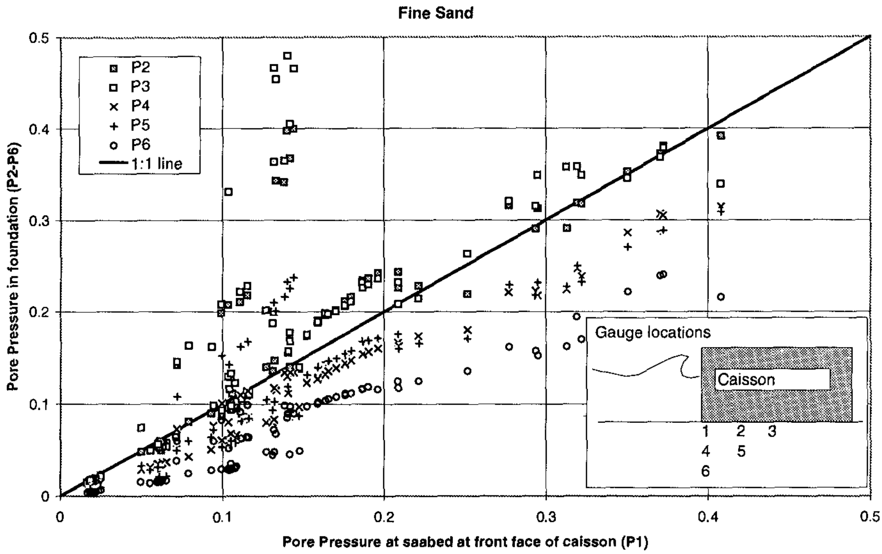

Figure 6 Pore pressure response - fine sand

The Boundary Layer Approach

The boundary layer approach of Mei and Foda (1981) shows that, in the treatment of a poro-elastic solid using Biot's linearized theory of consolidation, for the frequencies of typical ocean waves, a Stokes' type of boundary layer exists near the top of the bed. The soil and fluid move together as a single phase in the region below this boundary layer. Within the boundary layer, the effects of the free draining upper boundary are evident. This allows a greatly simplified analysis where the outer region (far below the mudline) can be treated as a single phase elastic solid for which analytical solutions to the soil's behaviour are readily found. Mei and Foda show that in the boundary layer, a one-dimensional analysis using Terzaghi's theory of consolidation can be applied (provided that the boundary layer thickness is small relative to wavelength). Solutions to a variety of poro-elastic problems using this boundary layer technique have been presented in the literature (see Mei and Foda, 1981 and Mei, 1982)

Intuitively, one would expect that for a soil of permeability, k, which is small relative to the frequency of loading, $\omega=2\pi f$, there is little fluid motion relative to the soil skeleton. For such a case the soil and fluid move together and the medium can be treated as a single phase as described by the equations of elastic dynamics. Near the seabed there will be a free-draining region extending to some depth, δ. For $\delta/L \ll 1$, where L is the wavelength, then the upper boundary region becomes essentially one-dimensional. Mei and Foda show that the first order approximation of the boundary layer is the Terzaghi equation for one-dimensional consolidation.

In the boundary layer $\partial/\partial z \gg \partial/\partial x$. The thickness of the boundary layer, δ is given by

$$\delta = \sqrt{\frac{c'_v}{\omega}} \tag{2}$$

where c'_v is the soil consolidation coefficient

$$c'_v = \frac{k\gamma}{m'_v} \tag{3}$$

Here k is the soil's coefficient of permeability, γ is the unit weight of water, and m_v' is the effective modulus of volume compressibility for soil with a partially saturated pore fluid.

$$m'_v = n\beta' + \frac{1-2\nu}{2(1-\nu)G} \tag{4}$$

Here the stiffness of the soil matrix is characterized by the bulk shear modulus, G, and Poisson's ratio, ν. The soil porosity is n and the stiffness of a partially saturated fluid is β' (as per Verruijt, 1969),

$$\beta' = \beta + \frac{1-S_r}{P_{tot}}. \tag{5}$$

Here β is the compressibility of pure water (β=4.3 x 10^{-10} Pa^{-1}), and S_r is the volumetric degree of saturation of the soil. P_{tot} is the absolute pressure at the point of interest.

The exact total solution for dynamic stresses and pore pressures is determined through the solution of the dynamic elastic equations governing the outer region and subsequent application of a boundary layer correction:

$$(\ )_{exact} = (\ )^{o} + (\ )^{b} \tag{6}$$

where the superscripts o and b denote outer region and boundary layer region terms, respectively.

The boundary layer formulation provides an accessible and heuristically appealing treatment of the soil-wave interaction problem. The boundary layer thickness, δ quantifies the relative depth of influence of wave action in a seabed.

Foundations - Drained vs Undrained

The problem of wave action in the vicinity of coastal structures can be considered in terms of the permeability and stiffness of the soil mass relative to the size of the structure. Wave loading on a rigid coastal structure such as a caisson causes wave stresses to be transferred from the caisson to the seabed. At the same time wave-induced pressure fluctuations at the seabed cause uplift pressures along the underside of the structure. The complex interactions between a rigid structure and the seabed under wave loading have been examined in the literature by Lee and Focht (1985), Lindenberg et al, (1982) and Tsai et al (1990). Analytic solutions of the wave-soil-structure interaction problem have been formulated by Mei (1982) as well as by Tsai et al (1990). These analytic solutions use elastic solutions for a rigid block resting on a flexible base to solve the total stress state in the foundation. A boundary layer correction is then applied to take account of the presence of the free-draining upper boundary. These solutions rely on an assumption that the boundary layer thickness, δ is small relative to the wavelength, L and furthermore that the boundary layer thickness is small relative to the caisson size, C (for some solutions the half-width of the caisson is used as the representative caisson dimension).

These calculations are quite complex but yield some interesting insight into the behaviour of wave-soil-structure interactions. For practical problems, a finite element effective stress model is more adaptable (e.g. Shen, 1990).

One of the insights provided by boundary layer theory is the concept of relative boundary layer thickness. In assessing the appropriate analysis to be undertaken for a specific structure, simple calculations of boundary layer thickness relative to wavelength and relative to caisson geometry can allow the classification of the problem in terms of the influence of pore fluid flow on soil response.

There are two bounding conditions often considered in analysing foundation response. These are **undrained** and **drained** behaviour:

1) For **undrained** analysis, it is assumed that the rate of application of loading is rapid relative to the soil's permeability. Consequently the pore fluid and soil matrix move together. At a lower bound, for example, the case of a structure resting on clay, the role of pore pressures in generating uplift forces is negligible. For undrained analysis the total stress state is used, and the concept of effective stresses is not applicable.

2) For **drained** analysis, it is assumed that the rate of application of loading is slow relative to the soil's permeability. Consequently, the pore fluid is free to move relative to the soil matrix. At an upper bound, the Shore Protection Manual (1984) and Goda's models of wave-induced uplift pressures are reasonable, (a triangular pressure distribution acting along the underside of the structure). For drained analysis, an effective stress approach is employed.

In reality, these limiting cases rarely exist. The true soil response lies somewhere between fully undrained and fully drained. What is needed is a set of practical guidelines to evaluate the relative importance of drainage to the structure.

Foundation response guidelines

When the boundary layer thickness, δ is small, free-drainage occurs only close to the mudline. When δ is large the free-drainage zone extends further into the seabed.

The pore pressure response within the seabed in the vicinity of a caisson will be related to two dimensionless parameters:

1. f/f_n - describes the frequency response of the system and how close the loading frequency, f is to the natural frequency of the caisson, f_n. For a single degree of freedom oscillator (such as the caisson response in pure pitch motion), the natural frequency of the caisson-soil system is related

to the foundation stiffness and the polar mass moment of inertia of the caisson as follows:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{E'B^3}{12\,I_p}} \qquad (7)$$

Here, E' is the equivalent foundation stiffness, and B is the caisson width.

2. δ/C - the thickness of the boundary layer, δ relative to the caisson size gives a measure of how large the free-draining boundary is relative to the size of the caisson.

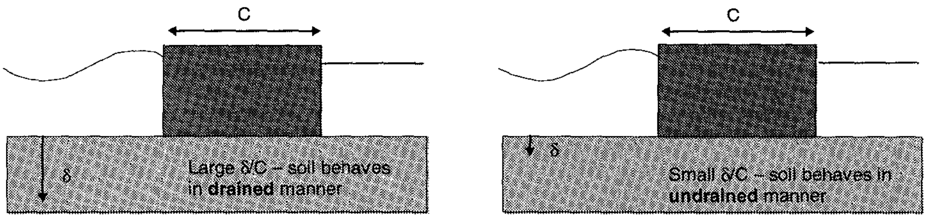

Figure 7 Illustration of boundary layer thickness concept for caissons

For a given caisson geometry (and hence given f/f_n), the pore pressure response could be expressed as a function of the ratio of boundary layer thickness to caisson width alone:

$$P/P_o = \phi\left(\delta/C\right) \qquad (8)$$

Three conditions can then exist:

1) If $\delta \ll C$ then the boundary layer effect will be small, uplift pressures will only exist very close to the outer edge of the caisson and undrained analysis will be sufficient.

2) If $\delta \gg C$ then the entire caisson will lie within the boundary layer, uplift pressures will be large and drained analysis will be appropriate. Here solution of the pore pressure response (and uplift) can be 'de-coupled' from the soil stress response.

3) Where δ is of the order of C, the response will be partially drained – the interactions between the pore fluid and soil matrix must be considered. Figure 7 provides a schematic of these concepts.

Consider the behaviour of the two caisson test series shown in Figure 5 and Figure 6. The major differences between these two caissons were the soil's coefficients of consolidation. This is reflected in different boundary layer thicknesses for the two caisson datasets. Figure 8 shows a plot of the relative amplitude of the pore pressure response beneath the caisson (normalized by P1) against the ratio of boundary layer thickness to caisson size, δ/C. This figure shows that for test conditions corresponding to δ/C around 0.1, amplification of the pore pressure response beneath the caisson is observed (e.g. P3 in caisson 1). This trend is not observed for sensors mounted deep in the bed at the front face of the caisson (vis P6). For test conditions of δ/C much greater than 0.5, the amplitude of the pore pressure response starts to approach the linear distribution suggested by Goda and others for caissons resting on very coarse rubble beds.

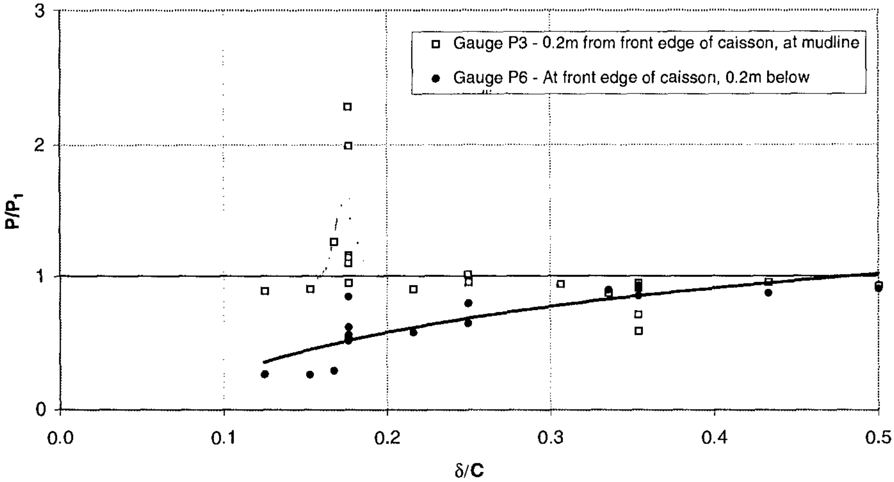

Figure 8 Pore pressure response vs drainage ratio, δ/C (combined results for both coarse and fine sands).

Table 1 shows typical ranges of foundation conditions and wave conditions for some typical coastal structure geometries

Wave period, T= 15 s			ω=2π/T= 0.42	
Influence of foundation type	c_v '(m^2/s)	δ [m]	C [m]	δ/C
Rubble fill	1000	48.86	15	3.26
Hard sand	100	15.45	15	1.03
Soft sand	10	4.89	15	0.33
Stiff clay	0.1	0.49	15	0.03
Soft clay	0.001	0.05	15	0.00
Influence of caisson size	c_v '(m^2/s)	δ [m]	C [m]	δ/C
Rubble fill	1000	48.86	*3*	16.287
Rubble fill	1000	48.86	*10*	4.886
Rubble fill	1000	48.86	*100*	0.489
Soft clay	0.001	0.05	*3*	0.016
Soft clay	0.001	0.05	*10*	0.005
Soft clay	0.001	0.05	*100*	0.000

Table 1 Relative boundary layer thickness for typical field conditions

For cases where the foundation material is stiff and permeable (such as rubble fill), Table 1 shows that, except in the case of exceptionally wide structures, the ratio δ/C

is large and the free-draining condition can be expected to be observed along the entire underside of the structure. This is in accordance with the design techniques presented by Goda and the Shore Protection Manual where, for caissons resting on rubble foundations, a triangular uplift pressure distribution is assumed along the underside. For cases where either the foundation material is soft and impermeable (soft clay) or where the structure is very large (large C), the ratio δ/C can be seen to be quite small. In these cases, the uplift pressures due to wave action will be negligible and the undrained analysis is most appropriate (e.g. as described by Lee and Focht). In cases such as this, skirts are sometimes installed around the base of the structure to act as cut-off walls to further reduce the likelihood of uplift pressures existing. The structures are then designed for a no-uplift condition. The ratio δ/C serves to delineate how foundations in intermediate conditions may behave. For example if a structure rests on soft sand or stiff clay, the ratio δ/C becomes useful in combining the effects of wave period, consolidation coefficient and structure size to give a sense of the relative size of the drainage path, δ.

Conclusions

The pore pressure response of a seabed exposed to wave loading can be reasonably well described by the rigid, porous bed solution of Sleath (1970). To obtain information about the effective stress state in the soil, however, it is better to use a poro-elastic seabed response model such as that of Finn (1982).

Often the response of a coastal structure is neither fully drained nor fully undrained but somewhere in between. For engineering analysis, practical guidelines are needed to delineate the extent of influence of the free-draining boundary. Through boundary layer theory, it is possible to interpret this influence through δ, the boundary layer thickness. Preliminary analysis of test results indicates that the ratio, δ/C might be useful here. For small values of δ/C (less than 0.05, say), the soil can be treated as fully undrained and a total stress analysis can be used. As δ/C becomes large (greater than 0.3, say), soil behaviour starts to become fully drained and analysis techniques such as those proposed by Goda and the Shore Protection Manual become more appropriate. Soil conditions in the intermediate range ($0.05 < \delta/C < .3$) require particularly close attention since the response of the soil and pore fluid is closely coupled.

The idea of using the relative boundary layer thickness, δ/C as a predictor of seabed response is hypothetical at this stage. Further verification and extension of this idea requires examination of a wider range of experimental conditions. Variation of the ratio of f/f_n (the frequency of wave loading relative to the natural frequency of the system) should also be further explored.

Acknowledgements

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