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SEISMIC DESIGN OF PORT STRUCTURES

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ABSTRACT: As a result of the Kobe disaster in 1995, new analysis methods including the effective-stress analysis for quay walls and pushover analysis for pile-supported wharves are introduced in the technical standards for port and harbour facilities in Japan to assess seismic performance of structures beyond the limit of force balance. In this article, design practice of port structures in Japan is described, with focuses on these new analysis methods. Some of the problems to be solved are also briefly described.

Key Words: The 1995 Hyogo-ken Nanbu Earthquake, quay wall, pile-supported wharf, effective-stress analysis, pushover analysis, performance-based design

INTRODUCTION

The 1995 Hyogo-ken Nanbu (Kobe) Earthquake brought great damage to structures in the Port of Kobe, which is one of the primary ports in Japan (Photo 1). Since then, an enthusiastic discussion has been going on with respect to every aspect of the design of port structures. The discussion extended to the evaluation of design ground motions, performance requirements and analysis methods for evaluating seismic performance of port structures.

One of the central issue was the incorporation of so-called "the level II design ground motion." According to the Japan Society of Civil Engineers (2000), it is a safety-level design ground motion and it is defined as the most intense ground motion from physically possible and reasonable earthquake scenarios. It is site-dependent and it may result from inland active faults or from subduction earthquakes depending on the location of the port. In general, the level II design ground motion thus defined is so intense that conventional design methods including the pseudo-static design method for quay walls and the allowable stress design method for pile supported wharves does not work for the level II design ground motion. A need was recognized for a performance-based design method (International Navigation Association, 2001), in which seismic performance of a structure beyond the limit of force-balance is evaluated instead of requiring that the limit equilibrium not be exceeded. Although the authors understand that there are several definitions for performance-based design method, when port engineers in Japan say "performance-based design method", they are often referring to a design method that considers the performance of a structure beyond the limit of force-balance. For the purpose of implementing performance-based design method, several new analysis methods were developed including the effective-stress analysis for quay walls and pushover analysis for pile-supported wharves. These new analysis methods are incorporated in the latest version of the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999).

The authors, however, consider that the 1999 version of the seismic code is a tentative one because many important issues raised by the Kobe disaster are still not sufficiently addressed in the 1999 version. Many aspects of the 1999 version may be subject to a fundamental revision in the near future as a result of the discussion among Japanese port experts. In this article, therefore, the authors would like to restrict themselves to aspects of the seismic code that are not likely to be subject to a major revision in the near future. Aspects of the seismic code such as the evaluation of design ground motions and the performance requirements are not referred to in this article because these aspects are the central issues of recent discussions and may subject to major revision.

Photo 1 Damage to a quay wall at the Port of Kobe during the 1995 Hyogo-ken Nanbu Earthquake

CAISSON QUAY WALL

Pseudo-static analysis

The pseudo-static analysis has a long history and it is still useful for serviceability-level design ground motions. In the pseudo-static approach, the stability of caisson quay walls is evaluated with respect to sliding, overturning, and loss of bearing capacity. In the limit state under strong shaking, the instability with respect to overturning is much more serious than that for sliding because tilting of the wall, if excessive, will lead to collapse. Thus a higher safety factor is assigned for overturning than for sliding and loss of bearing capacity (Ministry of Transport, Japan, 1999). Loss of bearing capacity is evaluated based on the simplified Bishop method (Kobayashi *et al*., 1987), which is a kind of circular slip analysis. The design parameters, including the internal friction angle of foundation soil, for use with this simplified analysis can be found in the seismic code (e.g. Ministry of Transport, Japan, 1999).

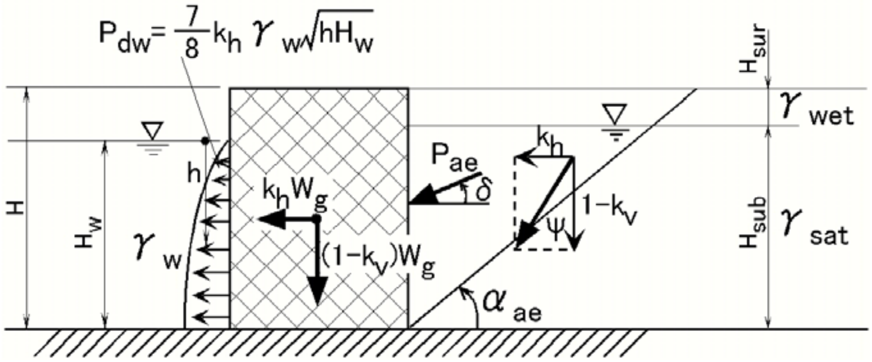


Fig. 1 Seismic actions on caisson quay walls for pseudo-static analysis

Active earth pressure

In the pseudo-static approach, the earth pressure due to a sandy backfill is estimated using the Mononobe-Okabe equation (Mononobe, 1924; Okabe, 1924). This equation is derived by modifying Coulomb's classical earth pressure theory (Coulomb, 1776; Kramer, 1996) to account for inertia forces. In the uniform field of horizontal and (downward) vertical accelerations, $k_h g$ and $k_v g$, the body force vector, originally pointing downward due to gravity, is rotated by the seismic inertia angle, ψ, defined by (see Fig. 1)

$$\psi = \tan^{-1}\left[\frac{k_h}{1-k_v}\right] \tag{1}$$

The Mononobe-Okabe equation is, thus, obtained by rotating the geometry of Coulomb's classical solution through the seismic inertia angle, ψ, and scaling the magnitude of the body force to fit the resultant of the gravity and the inertia forces (Mononobe, 1924; Whitman and Christian, 1990). For a vertical wall having a friction angle, δ, between the backfill and the wall, and retaining a horizontal backfill with an angle of internal friction, ϕ, the dynamic active earth pressure coefficient, K_{ae}, is given by

$$K_{ae} = \frac{\cos^2(\phi-\psi)}{\cos\psi\cos(\psi+\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\psi)}{\cos(\delta+\psi)}}\right]^2} \tag{2}$$

Hydrodynamic pressure

During seismic shaking, the free water in front of the structure exerts a cyclic dynamic loading on the wall; the critical mode occurs during the phase when suction pressure is applied on the wall. The resultant load can be approximated by (Westergaard, 1933):

$$P_{dw} = \frac{7}{12}k_h\gamma_w H_w^2 \tag{3}$$

where γ_w : unit weight of seawater
H_w : water depth

The point of application of this force lies 0.4 H_w above mudline.

Equivalent seismic coefficient

In the current version of the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999), the horizontal seismic coefficient for design is given as a product of the regional seismic coefficient, the soil-type coefficient and the importance coefficient. For simplicity, the vertical seismic coefficient is neglected (Ministry of Transport, Japan, 1999). The regional seismic coefficient takes a value between 0.08 and 0.15, depending on the regional seismicity. The soil-type coefficient takes a value between 0.8 and 1.2. The importance coefficient takes a value between 0.8 and 1.5. As a consequent, the horizontal seismic coefficient for design ranges from 0.05 to 0.27. The values may be subject to a revision in the next version of the seismic code as a result of the on-going probabilistic seismic hazard analysis by the National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure and Transport, Japan.

The design seismic coefficient is not necessarily equal to the design level PGA/g due to the transient nature of the earthquake motions. The ratio of equivalent seismic coefficient k_e to PGA/g has been studied in the Port and Harbour Research Institute based on case histories (e.g., Noda *et al.*, 1975; Nozu *et al.,* 1997). In these studies, the threshold seismic coefficients obtained by back-analyses of a damaged and non-damaged structure at sites of non-liquefiable soils provide a lower and upper bound estimate for an equivalent seismic coefficient. Fig. 2 shows a summary of the lower and upper bound estimates based on the case histories of 129 gravity-type quay walls during 12 earthquakes (Noda *et al*, 1975). The roots of the arrows in the figure, rather than the points, show the exact values. The arrows pointing up indicate lower bound estimates; those pointing down indicate upper bound estimates. The equation for an upper bound envelope was given by Noda *et al.* (1975) as

$$
\begin{aligned}
k_e &= \frac{a_{\max}}{g} && a_{\max} < 0.2g \\
k_e &= \frac{1}{3}\left(\frac{a_{\max}}{g}\right)^{\frac{1}{3}} && a_{\max} \geq 0.2g
\end{aligned}
\tag{4}
$$

As shown in Fig. 2, the relation in Eq. (4) is an envelope. An average relationship between the effective seismic coefficient and the peak ground acceleration may be obtained as

$$
k_e = 0.6\left(\frac{a_{\max}}{g}\right) \tag{5}
$$

These relations have been providing important information in specifying the regional seismic coefficient in conjunction with the results of probabilistic seismic hazard analysis.

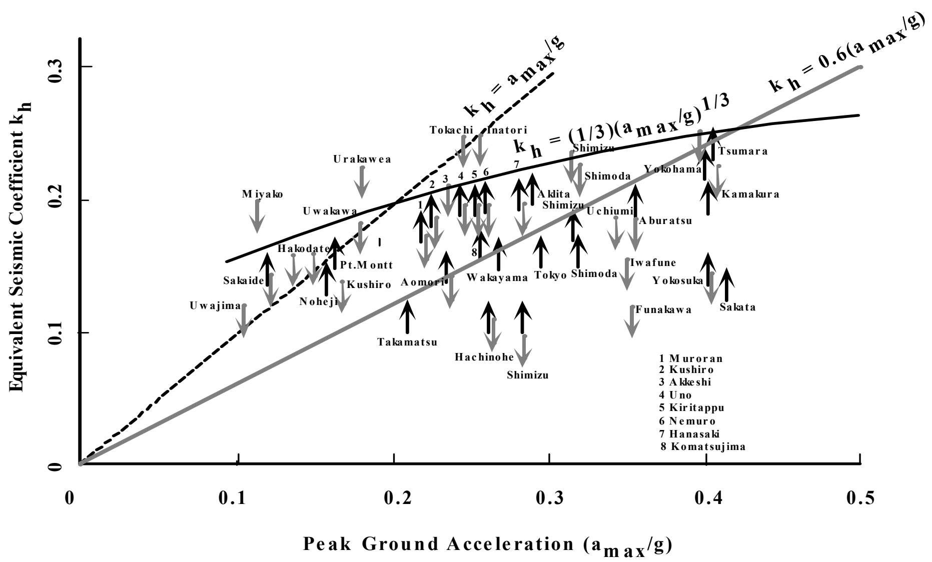


Fig. 2 Equivalent seismic coefficient k_e for gravity-type quay walls (Noda, *et al.*, 1975)

Effective-stress analysis

After the 1995 Hyogo-ken Nanbu (Kobe) Earthquake, it has been recognized that, although the pseudo-static analysis for caisson quay walls is effective for the serviceability-level design ground motion, it is not applicable for the level II design ground motion, which is a safety-level design ground motion, because of its intensity. Thus a need was recognized for a more sophisticated analysis, in which seismic performance of a quay wall beyond the limit of force-balance can be assessed. For this reason, the effective-stress analysis for quay walls was incorporated in the latest version of the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999).

Effective-stress analysis, generally using a finite element technique, involves coupled soil-structure interaction wherein the response of the foundation and backfill soils is incorporated in the computation of the structural response. While the stress-strain behavior of the soil is idealized with an effective-stress constitutive model (e.g., Iai *et al*., 1992), modeling of a caisson itself is generally accomplished using a linear model. Fairly comprehensive results can be obtained from the effective-stress analysis, including failure modes of the soil-structure systems and extent of residual deformation of the system. The computer code FLIP (Iai *et al*, 1992) is one of the most widely used in Japanese practice.

Applicability of the code has been tested against case histories of seismic performance of caisson quay walls. Fig.3 shows a typical damage to a caisson quay wall at the Port of Kobe during the 1995 Hyogo-ken Nanbu earthquake. The quay wall was constructed on a loose saturated foundation of decomposed granite, which was used for replacing the soft clayey deposit in the Port of Kobe to attain required bearing capacity. Subjected to a strong earthquake motion having an approximate peak acceleration of 0.5g, the caisson wall displaced an average of 3 m toward the sea, settled 1 to 2 m and tilted about 4 degrees toward the sea.

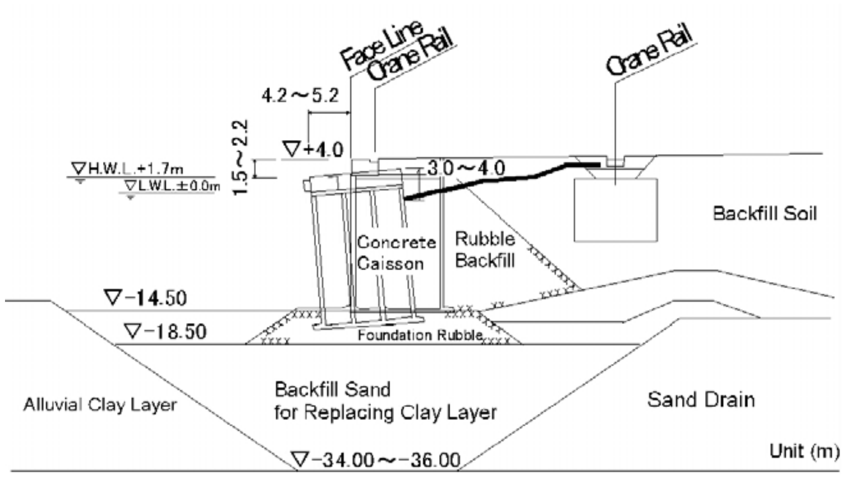


Fig. 3 Damage to a caisson quay wall at the Port of Kobe (Iai *et al.,* 1998)

An effective-stress analysis was conducted (Ichii *et al*, 1997; Iai *et al.,* 1998) to simulate the damage using the computer code FLIP. The model parameters were evaluated based on in-situ velocity logging, the blow counts of Standard Penetration Tests (SPT N-values) and the results of cyclic triaxial tests. The specimens used for the cyclic triaxial tests were undisturbed samples obtained by an in-situ freezing technique. The input earthquake motions were those recorded at the Port Island site about 2km from the quay wall. The spatial domain used for the finite element analysis covered a cross-sectional area of about 220 m by 40 m in the horizontal and vertical directions. The effective-stress analysis resulted in the residual deformation shown in Fig. 4. As shown in this figure, the mode of deformation of the caisson wall was to tilt into and push out the foundation soil beneath the caisson. This was consistent with the observed deformation mode of the rubble foundation shown in Fig. 5, which was investigated by divers (Inagaki *et al.,* 1996). The order of wall displacements was also comparable to that observed and shown in Fig. 3.

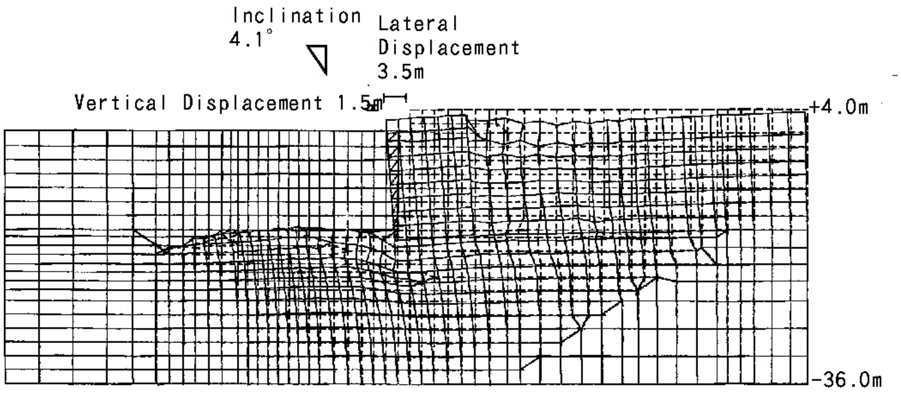


Fig. 4 Computed deformation of a caisson quay wall at the Port of Kobe (Iai *et al.,* 1998)

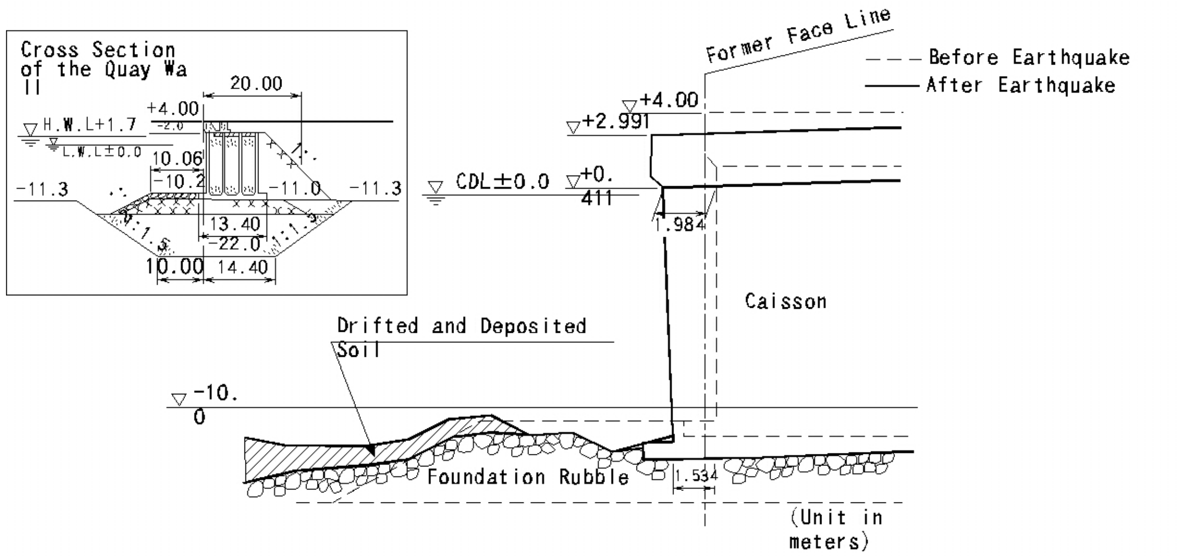


Fig. 5 Deformation of rubble foundation of a quay wall investigated by divers (Inagaki *et al.,* 1996)

ANCHORED SHEET-PILE QUAY WALL

Pseudo-static analysis

In the pseudo-static approach, stability of anchored sheet-pile walls is evaluated with respect to gross stability and stresses induced in structural components. In particular, gross stability is evaluated for a sheet-pile wall to determine the embedment length of the sheet piles into competent foundation soils. Stability is also considered for an anchor of the sheet-pile wall to determine the embedment length and the distance from the wall. Stresses are evaluated for the sheet-pile, anchor, and tie-rod. In the ultimate state, the rupture of tie-rods results in catastrophic failure of the wall and, therefore, this mode of failure must be avoided. Thus, it is common practice to assign a large safety factor for tie-rods (Ministry of Transport, Japan, 1999). Although excessive displacement of the anchor is undesirable (Gazetas *et al*, 1990), it is recognized that balanced movement of the anchor reduces the tension in the tie-rods (Kitajima and Uwabe, 1979). The ratio of equivalent seismic coefficient k_e to PGA/g for anchored steel-sheet pile quay walls was studied using data from 110 case histories (Kitajima and Uwabe, 1979). It was concluded that the relation for an upper bound envelope (Noda *et al.,* 1975; equation 4 of this article) is also applicable for the anchored sheet pile quay walls.

Effective-stress analysis

Applicability of the effective-stress analysis for anchored sheet-pile walls has been tested against case histories in Japan. During the 1983 Nihonkai-Chubu earthquake in Japan, many anchored steel sheet pile walls suffered damage at the Port of Akita. Most of the walls were constructed by backfilling clean sand dredged from the nearby seabed with SPT N-values ranging from 20 to 50. During the earthquake, these walls were shaken by an earthquake motion with an approximate peak acceleration of 0.25g. Fig. 6 shows a typical damage to an anchored sheet pile wall. The backfill liquefied and the sheet pile wall moved 1.1 to 1.8 m towards to the sea as shown by the broken lines. The sheet pile yielded and a crack opened about 6 m below sea level.

An effective-stress analysis (Iai and Kameoka, 1993) using the computer code FLIP resulted in the deformation shown in Fig. 7. Increases in earth pressures and bending moments were also computed as shown in Fig. 8. In this analysis, the model parameters were evaluated based on the SPT N-values and cyclic triaxial test results. The wall and anchors were modeled using linear beam elements and, thus, there was a limitation in simulating the seismic response of the wall after yield of the sheet pile. It is to be noted, however, that the displacement computed at the top of the wall was 1.3 m, consistent with that observed. The maximum bending moment computed in the sheet pile exceeded the yield level, also consistent with that observed. Now the computer code has been improved to include ductile response of structural components beyond elastic limit.

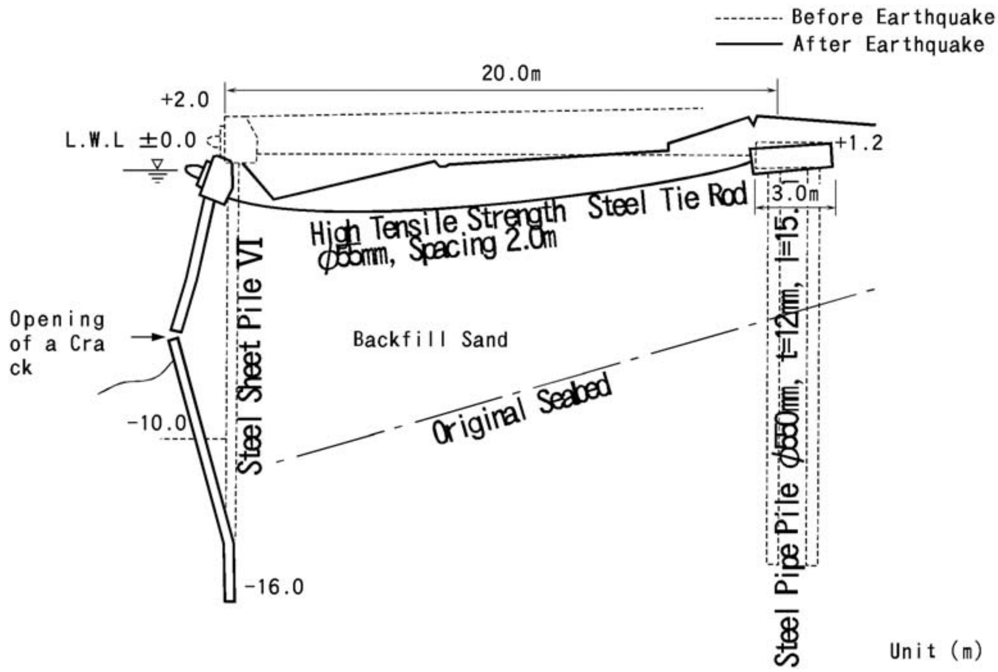


Fig. 6 Damage to a sheet-pile quay wall at the Port of Akita (Iai and Kameoka, 1993)

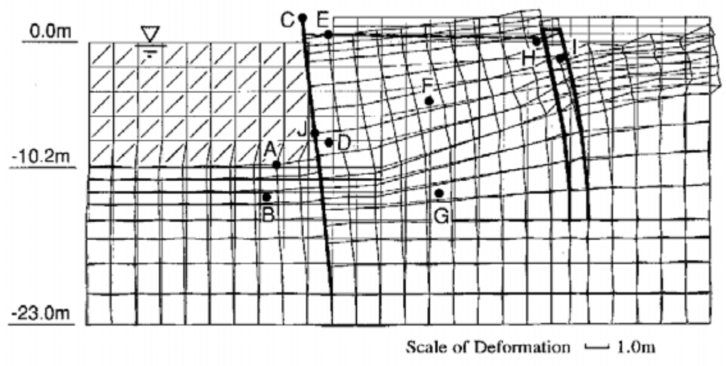


Fig. 7 Computed deformation of a sheet-pile quay wall at the Port of Akita (Iai and Kameoka, 1993)

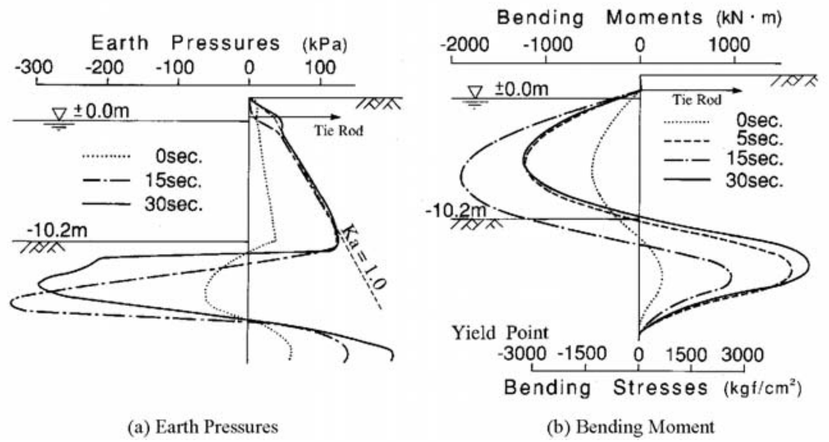


Fig. 8 Computed earth pressures and bending moments in a sheet-pile quay wall at the Port of Akita (Iai and Kameoka, 1993)

PILE SUPPORTED WHARF

Pushover analysis

The pushover analysis (Yokota *et al.,* 1999) was incorporated in the latest version of the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999) to assess seismic performance of pile-supported wharves beyond the elastic limit.

In the pushover analysis of pile-supported wharves, seismic inertia force is applied to the deck, where the mass of the pile-deck system is concentrated. Stresses in the pile-deck system computed in the analysis are compared with the limit stresses at initiation of yield and at plastic hinge formation in the piles.

The embedded portion of the piles are typically idealized as a beam on a Winkler foundation (Fig. 9) described by

$$EI\frac{d^4\Delta}{d\xi^4} = -P = -pD_p \tag{6}$$

where

EI: Flexural rigidity (kNm^2)

ξ : depth from the ground surface (m)

Δ : lateral displacement of a pile at the depth ξ (m)

P: Subgrade reaction per unit length of a pile at the depth ξ (kN/m)

p: $(=P/D_p)$ Subgrade reaction per unit area of a pile at the depth ξ (kN/m^2)
D_p: Pile diameter (or equivalent pile width) (m)

In Japanese practice, linear or non-linear relationships outlined in Ministry of Transport, Japan (1999) have been widely adopted. The linear relationship, widely known as Chang's method in Japan, is defined by (Chang, 1937):

$$P = E_s \Delta = k_{h-sub} D_p \Delta \tag{7}$$

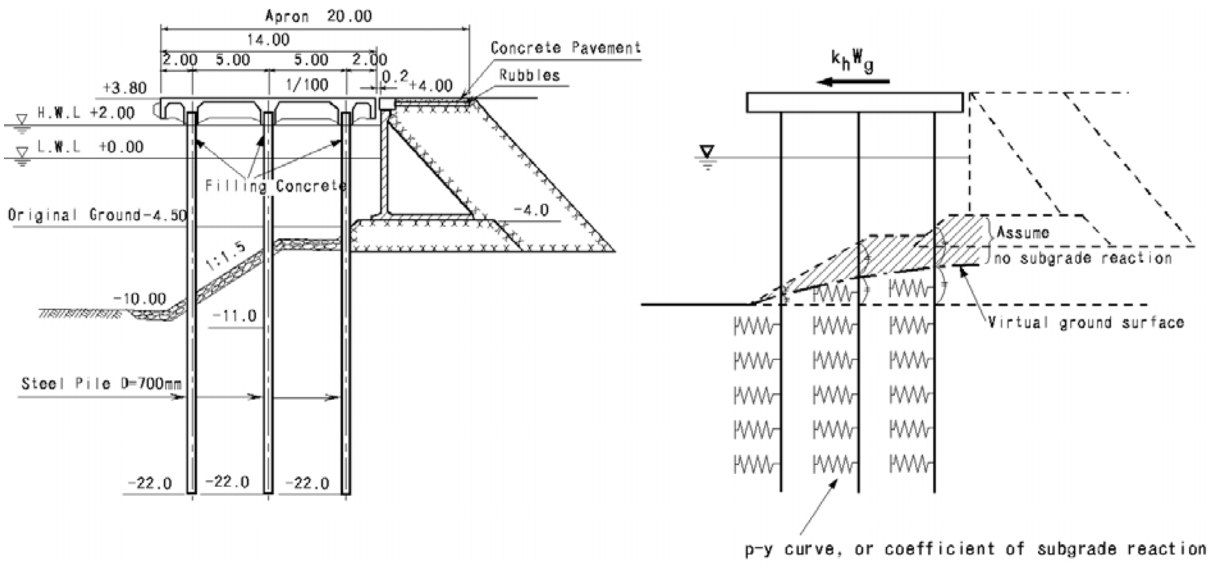
where $E_s(=k_{h-sub}D_p)$: equivalent subgrade elastic modulus (kN/m^2) is assumed constant.
The non-linear relationships evolved from Kubo (1962) are defined as

$$p = k_{s-type} \xi \Delta^{0.5} \text{ (for S-type subsoil)} \tag{8}$$

$$p = k_{c-type} \Delta^{0.5} \text{ (for C-type subsoil)} \tag{9}$$

where S-type subsoil refers to that with linearly increasing SPT N-values with depth and C-type subsoil refers to that with constant SPT N-values with depth. The coefficients k_{h-sub}, k_{s-type} and k_{c-type} are correlated with SPT N-values (Ministry of Transport, Japan, 1999).

It is quite common to have pile-supported structures that are founded on sloping embankments. In order to take into account the effect of sloping embankments, a virtual (or a hypothetical) embankment surface, which is set below the actual embankment surface, may be assumed for evaluating the lateral resistance of piles embedded in an embankment as shown in Fig. 9(b) (Ministry of Transport, Japan, 1999). As shown in Fig.9(b), no subgrade reaction is assumed above the vertual embankment surface.



(a) Pile-supported wharf

(b) Modelling by a frame structure supported by Winkler foundation

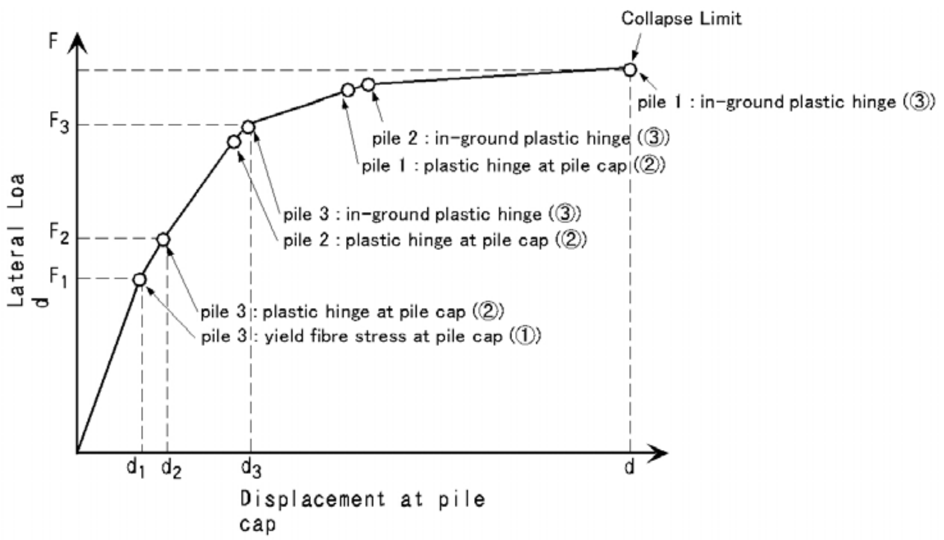
Fig. 9 Modeling of pile-deck system of a pile-supported wharf

The pushover analysis is accomplished by performing a multi-stage pseudo-static analyses with an increasing level of pseudo-static force applied on the deck. With an increasing level of external load, the sequence of yield in the structures and a transition from elastic response to the ultimate state of failure will be identified. The yield generally begins from the pile heads most landward to those seaward and then moves down to the embedded portion of the piles. Fig. 10 shows an example of the results of a pushover analysis.

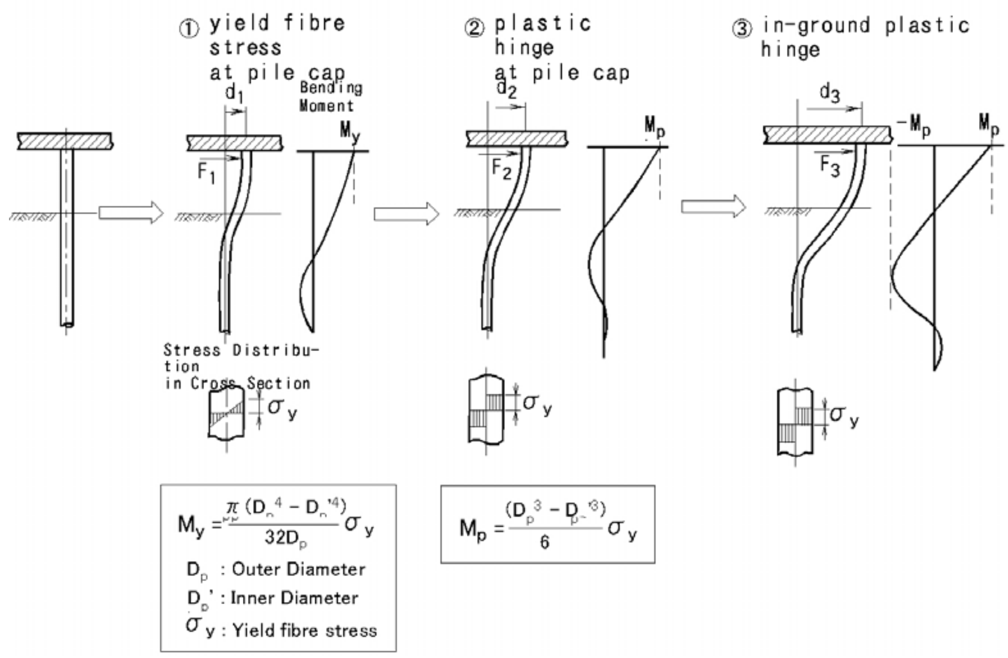
In Japanese practice, the strain ductility limit for use with the pushover analysis is given by

$$\varepsilon_{\max} = 0.44\frac{t_p}{D_p} \tag{10}$$

where t_p: thickness of steel pipe pile.
Repairable limit state is exceeded when the embedded portion of a pile reach at the strain limit specified by Eq.(10).



(a) Load - displacement relationship



(b) Sequential change in bending moment in piles

Fig. 10 Pushover analysis of a pile-supported wharf

PROBLEMS TO BE SOLVED

Although several new analysis methods were incorporated in the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999), many important issues raised by the Kobe disaster are still not sufficiently addressed in the 1999 version. In this section the authors would like to describe some of the problems to be solved.

Importance of site-specific design ground motions

Evaluation of design ground motion is one of the issues that have been enthusiastically discussed since the 1995 Hyogo-ken Nanbu Earthquake. It has been recognized that predominant periods of earthquake ground motions are site-dependent. Examples from recent observation in Japanese ports are shown in Fig.11. In the Port of Hachinohe, both of the Fourier spectra from the 1968 Tokachi-oki earthquake (M_J=7.9) and the 1994 Sanriku Haruka-oki earthquake (M_J=7.5) are characterized by a peak at 2.5 seconds. The former record is famous as the *Hachinohe wave* and has been widely used for the design of port structures in Japan. On the other hand, in the Port of Kushiro, both of the Fourier spectra from the 1993 Kushiro-oki earthquake (M_J=7.9) and the 2003 Tokachi-oki earthquake (M_J=8.0) are characterized by peaks from 1.0-2.0 seconds. The difference of predominant periods can be attributed to the difference of deep subsurface structure at each observation site. It is obvious from these observations that a structure designed for a *Hachinohe wave* is not necessarily suitable for ports with different subsurface structures. It is important to introduce design ground motions that reflect site characteristics appropriately.

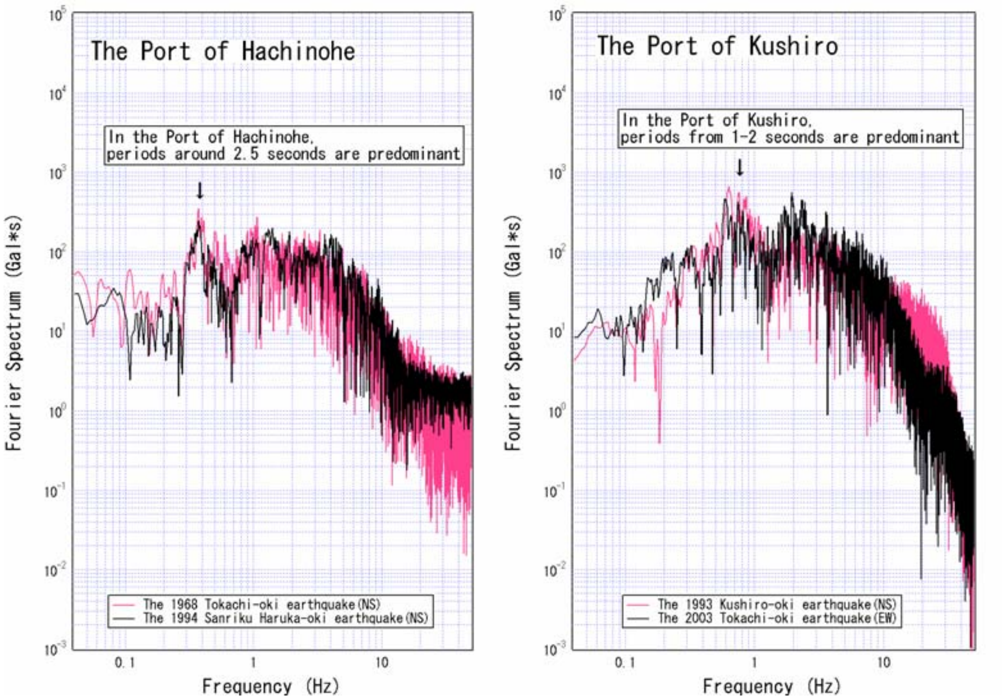


Fig. 11 Site dependent predominant periods of earthquake ground motions in Japanese ports

Importance of the deformation of soil for a pile-supported wharf

During the 1995 Hyogoken Nambu earthquake, a pile-supported wharf suffered damage at Takahama Wharf in the Port of Kobe. The horizontal residual displacement of the wharf ranged from 1.3 to 1.7m, with a typical example of the cross-section and deformation of the pile-supported wharf shown in Fig. 12. As shown in this figure, the wharf was constructed on a firm foundation deposit consisting of alternating layers of Pleistocene clay and sandy gravel. The SPT N-values ranged from 10 to 25 for the clay and 30 to 50 or higher for the sandy gravel. The firm deposit was overlain by an alluvial sand layer with SPT N-values of about 15, the thickness of which was variable, of about two meters on average. Behind the retaining wall made of concrete cellular blocks was a backfill of decomposed granite with SPT N-values of about 10. The deck of the quay was made of reinforced concrete slabs and beams supported by steel pipe piles having a diameter of 700 mm.

The steel piles buckled at the pile heads, except for the piles located most landward. A crack was found at the pile cap – concrete beam connection located most landward. The piles, pulled out after the earthquake for investigation also showed buckling below the mudline at the depths shown in Fig. 12. As shown in this figure, some of the buckling was located close to the boundary between the layers of alluvial sand and Pleistocene gravel. Displacements of the rubble dike, measured by divers at five locations 5 m apart, were about the same as those of the deck. The backfill behind the retaining structure settled about 1 m. These measurements indicate a somewhat uniform movement of the dike and the retaining wall toward the sea. Effective-stress analysis (Iai, 1998) using the computer code FLIP resulted in a significant deformation in the alluvial sand layer, which is consistent with the observed deformation of the pile-supported wharf. The computed bending moment at the pile heads located most landward was not large, contrary to the results obtained by conventional design practice including pushover analysis taking into account only the effect of the inertia force on the deck. These observations and computations indicate the importance of the deformation of soil, which is not necessarily sufficiently addressed in the current version of the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999).

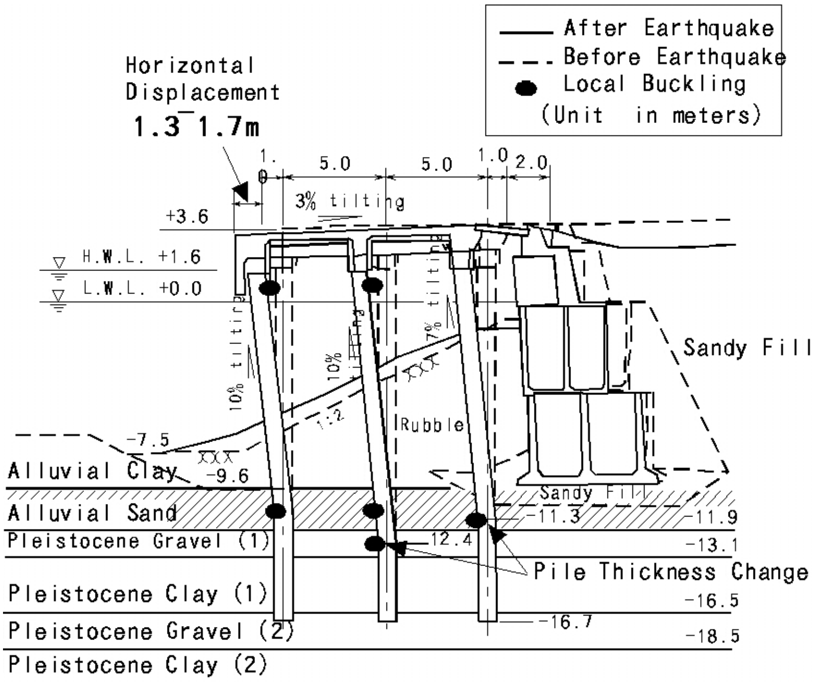


Fig. 12 Damage to a pile-supported wharf at the Port of Kobe

CONCLUSIONS

As a result of the Kobe disaster in 1995, new analysis methods including the effective-stress analysis for quay walls and pushover analysis for pile-supported wharves are introduced in the technical standards for port and harbour facilities in Japan (Ministry of Transport, Japan, 1999) to assess seismic performance of structures beyond the limit of force balance. Many important issues raised by the Kobe disaster are, however, still not sufficiently addressed in the 1999 version as stated in the previous section. The authors believe that recent discussion among Japanese port experts will lead to an establishment of a more reasonable seismic design code in the near future.

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The authors would like to thank members of the working group PIANC/MarCom/WG34 formed by the International Navigation Association (PIANC; formerly the Permanent International Association for Navigation Congresses) and chaired by Prof. Susumu Iai for fruitful discussions. The authors would like to thank all of the port experts in Japan who have been participating in the enthusiastic discussion related to seismic design of port structures.

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