

New Seismic Design of Port and Harbor Steel Structures

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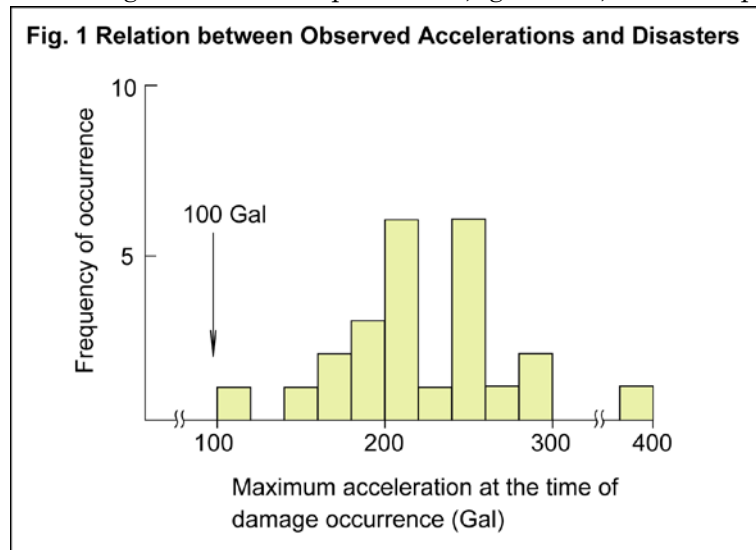
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1. Disaster Classification for Port and Harbor Facilities

Japan as well as Indonesia is an earthquake country with a history of numerous major earthquakes and disastrous damages. Specifications for the seismic resistance of port and harbor facilities in Japan are based on the circumstances of such previously recorded earthquakes. Relying on an analysis of representative disasters to port and harbor facilities caused by seventeen major earthquake disasters spanning the period from the Great Kanto Earthquake of 1923 to the Miyagi-oki Earthquake of 1978, a classification by disaster level was compiled and published in the bulletin of the Port and Harbor Research Institute (No. 409, 1982). Particular analysis was made of the disastrous damages of the Tokachi-oki Earthquake of 1968 and the Nemuro-hanto Earthquake of 1973. Subsequent analyses were made of the damages of the Sea of Japan Chubu Earthquake of 1983 and the Hyugo-Nanbu Earthquake of 1995. HYUGO-Nanbu earthquake brought to new design concept such as earthquake level and calculation method considering of rupture of the structures.

Fig. 1 shows the relation between observed accelerations and damages. The damages are classified into the following levels: 0 to IV. In levels 0 and I, port and harbor facilities remain available for continued use; the displacement and inclination of gravity-type quays and crane foundations at these two levels were specified. Further, additional assessments were made to determine the relationship between usability and extent of damages to port and harbor facilities and to determining the cost of restoration in levels 0 and I. The analyses thus far made on damage severity constitute

the basis for the design criteria (displacement, gradient, etc.) adopted in current



performance-based design work for port and harbor facilities.



Photo.1 Damage of pile-type wharf

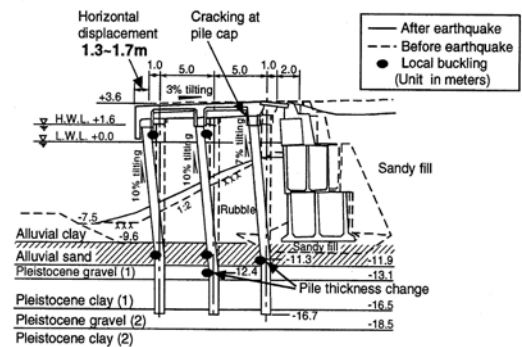


Fig.2 Movement of structure

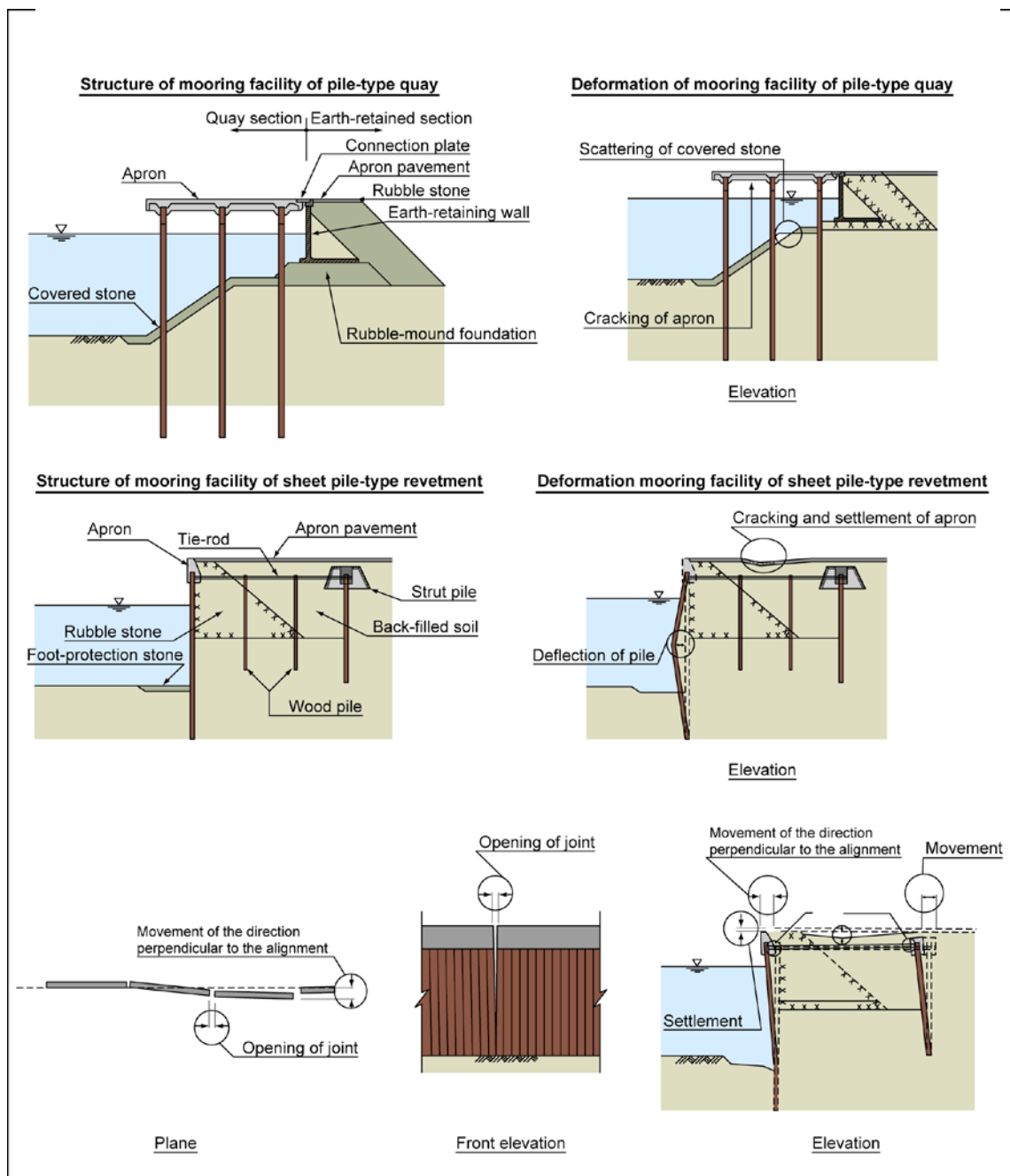
Even in port facilities where unloading was very difficult, emergency unloading was frequently carried out, and thus there are cases in which the severity of a damage does not directly relate to the practical serviceability of port and harbor facilities. The following article introduces steel pipe pile-type quays and steel sheet pile-type revetments that are representative of port and harbor facilities constructed of steel.

Photo 1 and Fig. 2 shows movement of a pile-type quay and a sheet pile revetment caused by Hyougoken-Nanbu earthquake at Kobe port in Japan.

The damages by of seismic vibration and ground liquefaction inflicted on port and harbor facilities by the Great Hanshin-Awaji Earthquake of 1995 were remarkable. As shown in Photo 1, the apron of the pile-type quay moved forward and the pile showed

buckling and other collapse phenomena. In the sheet pile revetment, collapse extended over the embedded section of the pile, the pile, tie rods and struts. In the pile-type quay, there were three major cases of collapse as shown in Fig.3. There was collapse due to vibration. There was collapse of the pile below ground surface and of the pile head, due to ground movement in the area of the pile foundation and to breakage of the slope beneath the floor slab. And, there was collapse of the pile head due to seaward movement of the apron along with horizontal movement of retained earth at the rear of quay. In order to create a rational and economical seismic design, it is necessary to

Fig.3 Damage pattern of pile structures



fully understand the behavior of port and harbor facilities during an earthquake and the type of disasters sustained by them.

In the Hyougoken-Nanbu Earthquake, the container crane on the revetment suffered great damages, as shown in Photo 2. The crane travels on both seaside and landside rails. When subjected to an earthquake, cranes show two large movements: locking due to the force of inertia and leg buckling due to ground deformation. Generally, the seaside rail is installed on a gravity-type caisson and the landside rail on a pile foundation or on improved ground. Under installation conditions such as this, when the pile moves significantly seaward due to ground liquefaction, the seaside rail pulls widely away from the landside rail and relatively uneven settlement occurs. When the settlement is large, this can cause the legs of the crane to buckle as shown in Fig.4.

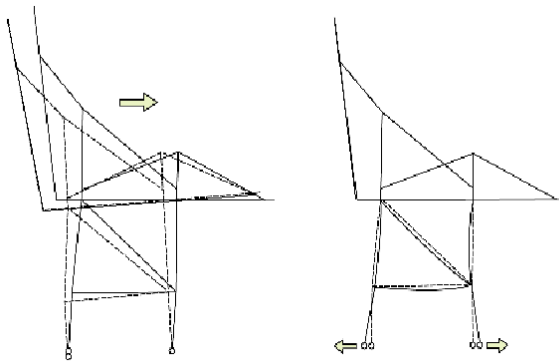


Fig.4 Collapse of crane



Photo 2 Damage on the wharf

2. Introduction of Seismic Design Concept

In structural design that takes seismic resistance into account (seismic design), seismic resistance of structures is prescribed by appropriately setting an allowable damage level according to the level of seismic motion. Because port and harbor facilities have diverse structural types and functions, the allowable damages level is set by taking into account the character and function of the target facility. For example, in the case of a container wharf equipped with a gantry crane, it is necessary in the design to examine even the function of the crane. In setting the allowable damages level, it is necessary to consider the effect of reduced functionality of the facilities and the relative difficulty of completing a full-scale restoration (cost and time).

The seismic resistance is determined by taking into account the

characteristic features of the target facility, with the aim of "retaining the soundness of the facility" against level 1 seismic motion and "retaining anticipated function" against level 2 seismic motion. This is explained using the conceptual drawing on seismic resistance shown in Fig. 5. In the figure, the ordinate designates the level of seismic motion (level 1 and 2 seismic motions are indicated), and the abscissa shows the allowable damages level. Classified as shown in Table 1, there are four assumed grades of seismic resistance: XS, XA, XB and XC. Facilities of the enhanced seismic-resistant type and highway facilities are designed to retain a seismic resistance of around XS, while temporary structures are to possess an XC rating. As shown in Table 2, the allowable damage level of pile-type quays is set according to the degree of reduced facility function and the relative difficulty of full-scale restoration (cost and time). The required functions are assumed from the facility's post-quake serviceability by examining previous earthquake damages. It is particularly important whether or not ships can moor and whether or not vehicles can operate on the apron. Regarding inclination, rain water etc. do not pool up and stagnate in cases of a seaward inclination, but when the inclination is landward, rain puddles form that make vehicle transportation difficult. Because of this, the utmost effort is made to establish a disaster mechanism that avoids the occurrence of rain puddles. In cases of a level I disaster, facilities are determined to be serviceable as is, or with slight repair. In the case of level II disasters, continued use is basically impossible and this is determined by a diagnostic survey. In the case of level III and IV disasters, facilities are on the verge of collapse or are already in a state of collapse and can be renewed only by means of large-scale repairs or by dismantling.

Fig. 5 Damages and Seismic Level

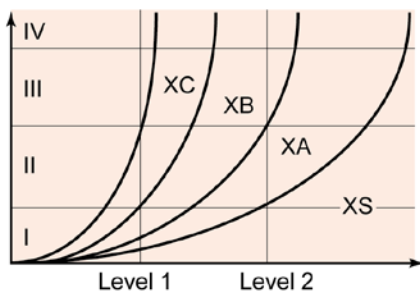


Table 1 Required Damage Levels

Grade	Required damage level
XS	Remain at the damage level I against the level 2 seismic motion
XA	Remain at the damage level I against the level 2 seismic motion, and the damage level against the level 2 seismic motion
XB	Remain at the damage level I against the level 2 seismic motion, and the damage level against the level 2 seismic motion
XC	Remain at the damage level II against the level 1 seismic motion, and the damage level against the level 2 seismic motion

Table 2 Functions Required for Pile-type Quays

Damage level		I	II	III	IV
Residual displacement	Relative settlement amount of apron and ground surface	0.1 ~ 0.3 m and under	—	—	—
	Inclining to the seaward side	2~3 degrees and under	—	—	—
Maximum response	Pile (Designed in that the occurrence of bending fracture precedes to that of shear fracture)	Elastic response within the range of no occurrence of residual movement	Inelastic response with the range in which the structure can be reinforced and occurrence of residual movement	Response similar to fracture (plastic hinges occur with one or more piles)	More than the damage level III

Note: The table is applied only to the pile and apron

3. Current Seismic Design of Port and Harbor Structures

Seismic design of port and harbor structures is based on the "Notification of the Director of Port and Harbor Bureau that Prescribes Interpretation and Operation of the Ministerial Ordinance to Regulate the Technical Standards for Port and Harbor Facilities (Ordinance of the Ministry of Transport, No. 30, 1974)" (April 1, 1999). A detailed description is available in the "Technical Standards for Port and Harbor Facilities and Their Commentary" (2007 Edition) published by the Japan Port and Harbor Association. Reliability method is widely applied to port facilities. In the performance based design, objective of structures is primary determined. And then performance requirement is described. Performance assesment and evaluation method are not indicated. Designers can adopt appropriate structure type and materials.

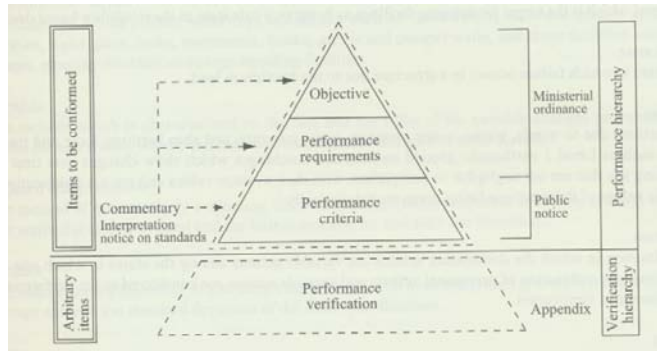


Fig.6 Hierarchy of performance based design

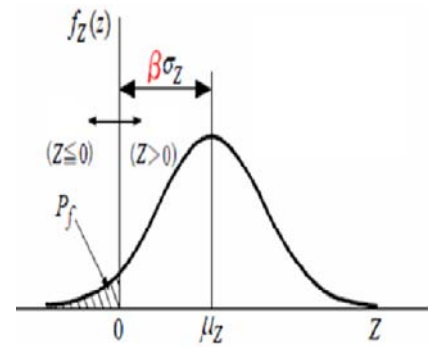


Fig.7 Failure probability and safety index

Reliability method is based on next equation.

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \quad (1)$$

This equation shows safety index(β) is calculated from mean values(μ) and standard deviations(σ) of both force and resist. Allowable safety index(β_a) is determined by

failure probability(P_f). Performance function is adopted for evaluation to obtain safety index. Then obtained safety index is compared to allowable safety index such as $\beta > \beta_a$. Allowable safety index depends on importance of facilities as shown in Table 3. For port facilities, three levels on seismic resistance are classified, namely, specially seismic reinforced facility, seismic reinforced facility and ordinary facility as shown in Table 4.

Table 3 Relationship between facility level and safety index

Facility level	failure probability : P_f	safety index: β_a
Temporary	10^{-2} - 10^{-3}	about 2
Ordinary	10^{-3} - 10^{-5}	2-4
Important	10^{-5} - 10^{-6}	$4 \geq$

Table 4 Target safety index for port facilities

Structural type	Specially seismic-reinforced facility	Seismic-reinforced facility	Ordinary facility
Pipe pile-type pier	3.65	3.2	2.7
Sheet pipe-type pier	3.6	3.2	2.7

Table 5 Design condition and verification approach

Design condition	Recommended performance verification approach
Permanent condition	Reliability design method
Variable condition	<ul style="list-style-type: none"> • Non-linear seismic response analysis that takes into account the dynamic interaction of earth and structure Reliability design method <ul style="list-style-type: none"> • Quasi-static method
Accidental condition	Numerical analysis method (specific assessment of deformations and damage level)

Design condition is divided into three categories such as permanent condition, variable condition and accidental condition as shown in Table 5. For evaluation method, new calculation method can be adopted. Non linear dynamic response analysis can be used

in seismic design. Seismic condition is located at both variable condition and accidental condition.

In seismic design, studies are made primarily of the following:

Seismic conditions near the location of port and harbor structures (existence of active faults, past earthquake history, etc.)

Geological conditions at the location of port and harbor structures (topography, stratum, strength, kind of subsoil, etc.)

Importance of structures to local social life and economy

Based on the above, the overall stability of the structures, stability of the foundations, ground liquefaction, soundness of the structural members, effect on adjoining structures and other related factors are examined.

Because social infrastructure suffered great damages in the area of Osaka, Kobe and Awaji due to the Hyogoken-Nanbu Earthquake, the Society of Civil Engineers proposed that seismic design takes a two-levels approach. A two-levels seismic design method was then presented for port and harbor facilities as shown in Table 6.

Conventional port and harbor improvements promote revetment projects of the high seismic resistant type. A major conceptual method of this type of improvement is as follows. Revetments with high seismic resistance (designed for a design seismic intensity of 0.25) are positioned at several locations in their important ports. Even in cases when an earthquake occurs with seismic motion that far surpasses the assumed level of seismic motion, these ports can function at the minimum level required for the loading/unloading of emergency commodities and for the conduct of rescue operations in the immediate damage of a quake. In essence, it can be said that this design concept coincides with the concept proposed by the Society of Civil Engineers for level 2 seismic motion.

Table 6 Seismic Motions in Design

Seismic motion level	Seismic motion taken into account in design	Target facility	Seismic resistance
Level 1	Assumed seismic motion of an earthquake with 75-years return period years	All facilities (excluding those prescribed by other codes)	No damage to sound functions of facility
Level 2	Assumed seismic motion of an earthquake occurring several hundred years-year return period, seismic motion inter- plates or seismic motion at the plates	Facility of the enhanced seismic resistance type (quay of the enhanced seismic resistance type, revetment requiring high seismic resistance among emergency response bases and other facilities; the facility that takes into account the level 2 seismic motion among bridges, immersed tunnels and other port/harbor facilities	Maintain expected function

In the Hyougoken-Nanbu Earthquake, a revetment of the high seismic resistant type installed on the Maya Wharf remained functional after the earthquake. However, in cases when only the revetment is reinforced for high seismic resistance but the apron

and portside roads are collapsed, the port can no longer retain its required functionality. As a result, it is necessary to assess total seismic resistance of facilities of the high seismic resistant type, including those located at hinterland.

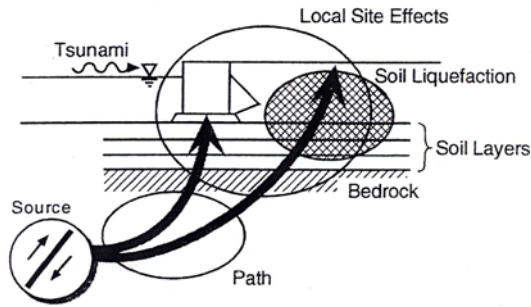


Fig.8 Schematic figure of propagation of seismic waves

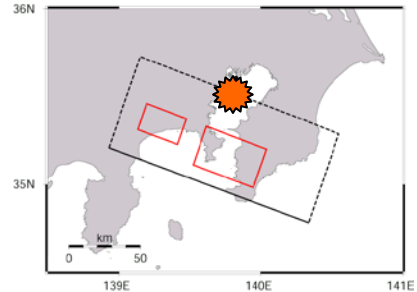


Fig 9 Fault at Tokyo

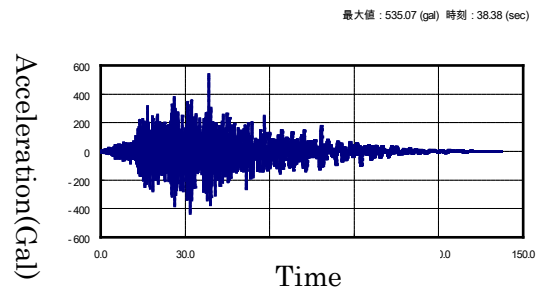


Fig.10 Artificial earthquake wave

The regional seismic coefficient is prescribed in terms of the strength of seismic movements with a 75-year return period, based on observations recorded by nationwide networks that observe strong earthquakes in port and harbor areas. The seismic coefficient is obtained by dynamic response analysis by one dimensional analysis at construction site. The seismic coefficient method is only applied for Level 1 earthquake motion. For level 2 earthquake motions, dynamic response analysis is carried out to estimate ultimate strength and allowable displacement for important port facilities. Fig 8 shows wave propagation from the fault to the facility. In put data: acceleration time history data for dynamic response analysis is obtained by artificial simulation calculation considering of fault movement, site soil condition, past acceleration records, etc. Fig.9 shows the active fault at Tokyo bay. Its width is 130km and its length is 70km. Average displacement of the fault is 2.1m. Fig 10 shows artificial acceleration record obtained from the fault to use input data for earthquake level 2. Allowable horizontal displacement is determined to be about 10cm for level 1 earthquake motion and about 1m for level 2 earthquake motion.

4. Design of Pile-type Quays

The pile-type quay is a mooring facility that consists of the quay, which is structured using steel pipe piles and a reinforced concrete apron, and an earth-retaining revetment constructed in the rear area of the quay. The earth-retaining revetment consists of a slope and a commonly adopted earth-retaining wall that is built to resist earth pressure from the rear subsoil of the quay and to prevent any part of the slope from sliding. In the case of pile-type quays, the dominant load for in the design stage covers the mooring force and the crane load in addition to the seismic force. For the structural section determined by primary design work, the seismic resistance is assessed in the following procedures.

In contrast to gravity-type and sheet pile-type revetments, pile-type quays adopt the modified seismic coefficient method so that the dynamic response characteristics of the quay can be reflected, the seismic resistance even for level 1 seismic motion can be specified and seismic resistance checking can be carried out. For steel pipe piles, a method is adopted that takes into account energy absorption by means of the plastic deformation capacity to check the load-carrying capacity.

Regarding the level of damages, the moment where steel pipe piles show total plasticization underground and plastic hinges or buckling occur is set as the parameter for the ultimate state. This is because, in the case of underground damage, it is difficult to find the damage and to assess its severity; it is also difficult to repair and reinforce the damaged section. The following cases are cited in the assessment of the maximum displacement: it is feared that the excess horizontal force works on the quay apron via the connection plate in the case of the vibration mode where the distance between the quay and the revetment becomes narrow and that the connection plate drops in the case of the vibration mode where the distance become wide. Allowable residual displacement is determined from the scale and type of the moored ships and from the system of loading/unloading.

The seismic coefficient method used for checking seismic resistance is determined by assuming the quay to be a one mass type structure installed on the ground and by employing the linear acceleration response spectrum. The seismic coefficient method used for checking level 1 seismic motion is set by finding the acceleration response at an assumed fixed point in the quay piles by means of dynamic response calculations using base accelerations given in the area classification (A~E) for the structures, preparing the response spectrum, and dividing the response acceleration conforming to the natural period of the quay by the gravity acceleration. The natural period of the quay is calculated by using the following two factors: the horizontal-direction spring constant (as the rigid-frame structure with a position $1/\beta$ below the surface of the

seafloor as the assumed fixed point) and the total sum of the quay's dead load and live load during an earthquake etc.

Assuming that the result of the nonlinear elasto-plastic response will be equivalent to that of the linear response, the load carrying capacity is found employing Newmark's equal-energy principle. Checking is made by the following equation.

$$R_a \geq k_h W \quad (1)$$

Where R_a : load-carrying capacity of quay (kN), K_h : seismic coefficient for checking seismic resistance found by linear response. W : total sum of vertical loads of quay (kN)

In the above equation, the load-carrying capacity during an earthquake is calculated by the following equation

$$R_a = \sqrt{2\mu - 1 + \theta(\mu - 1)^2} P_y \quad (2)$$

Where μ : ductility factor, θ : ratio of secondary gradient to primary gradient in load-horizontal displacement relation, P_y : horizontal force conforming to the elastic limit

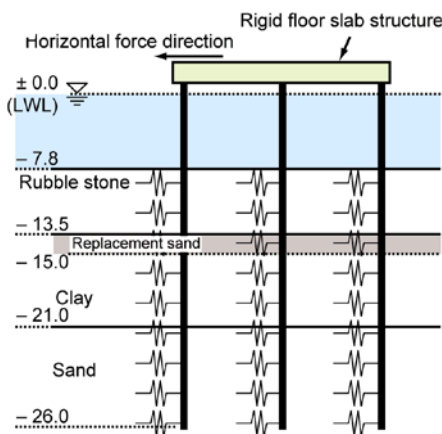


Fig.11 Modeled pile-type pier

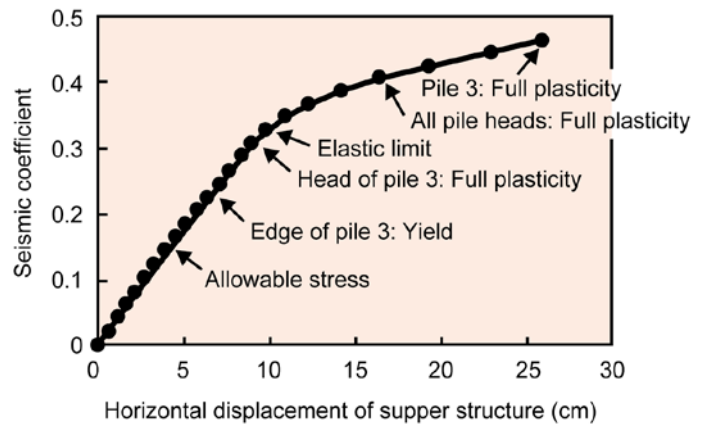


Fig.12 Push over analysis

The ductility factor μ is defined using the ratio of the ultimate-state horizontal displacement of the quay apron and the horizontal displacement at the point of elastic limit. The point of elastic limit is generally set as the point when the pile head moment of more than half the piles in the pile row of the cross section perpendicular to the quay slope reaches the total plastic moment. It is possible to trace the seismic resistance of a pile-type quay until its ultimate state by making a push-over analysis using the beam model of the finite element method. In this analysis, shown in Fig. 12, material nonlinearity is taken into account and collapse behavior of a pile-type quay is traced by

applying the force of inertia to the apron while increasing the force of inertia in the horizontal direction. Of course, it is possible to trace the collapse behavior by use of dynamic response analysis.

To estimate horizontal displacement, dynamic response analysis considering of material non-linearity is effective calculation tool. Program name FLIP is widely adopted in Japan to estimate these design allowable displacements. Soil property is modeled by effective stress and strain-stress non-linearity of members are also treated. Validity of this method is confirmed through comparison about deformations between calculation results and observed damage.

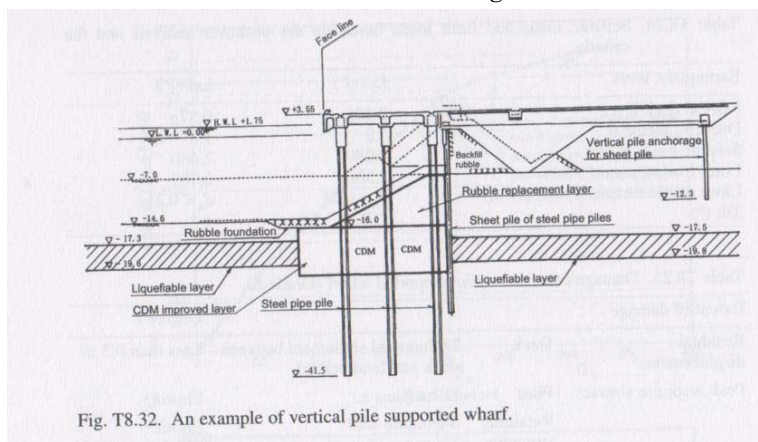


Fig. T8.32. An example of vertical pile supported wharf.

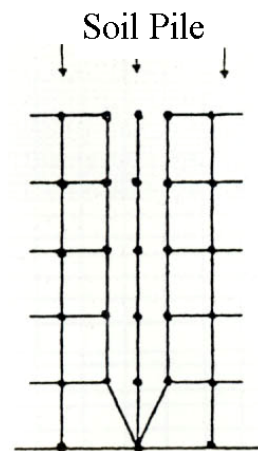


Fig.13 Calculation model for dynamic response analysis Fig.14 Model of interface between pile and soil

A calculation example about a pile supported wharf at water depth of -14.6m is shown in Fig.13. Surface layer consists of sandy grave, sandy clay, silty clay and sand. The sand layer is a liquefiable and a part of under the wharf is improved by a cement deep mixing stabilization method(CDM). The cross section of the pile -supported wharf is idealized into finite elements. Piles and CDM elements are not connected directly but connected by plane elements to allow free response of CDM portions and Joints elements are used to simulate the friction at the soil-structure interface as shown in Fig.14. Fig.15 shows total calculation model for FLIP.

Computed deformation resulted in the differential of 0.5m between the deck and the land behind as shown in Fig.16. This differential settlement is considered not acceptable based on the damage criteria. Liquefiable layer is completely liquefied.

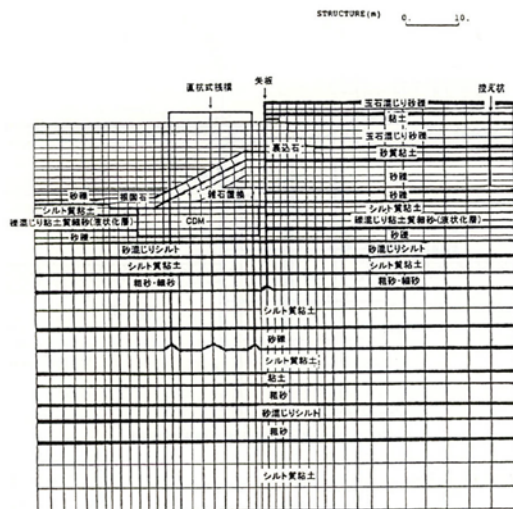


Fig.15 Calculation model for FLIP

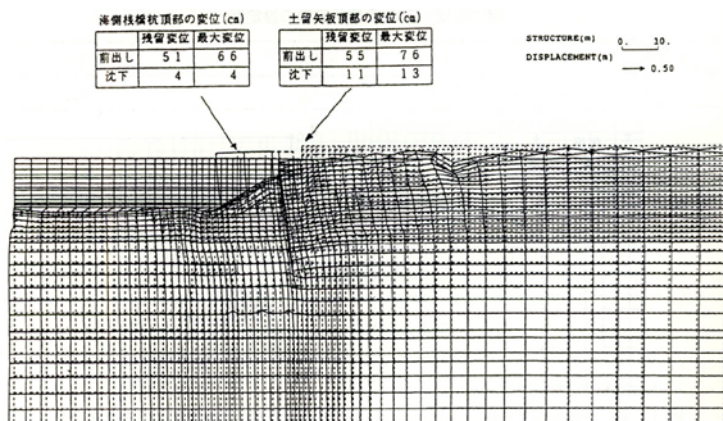


Fig 16 Deformation of the structure

Reference

- (1) International Navigation Association: Seismic Design Guidelines for Port Facilities, A.A.BALKEMA PUBLISHERS, 2001
- (2) The Overseas Coastal Area Development Institute of Japan: Technical Standards and Commentaries for Port and Harbor Facilities in Japan, 2009