EARTHQUAKE RESISTANT DESIGN
FOR
QUAYWALLS AND PIERS IN JAPAN

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1. INTRODUCTION

In this paper, the following topics located at an intersection of the earthquake engineering and the harbour engineering will be presented; namely they are the damage to quaywalls by earthquakes, the current procedures of earthquake resistant design for quaywalls and piers, and the research activities on the earthquake resistant design of quaywalls.

The latest earthquake which affected port structures was the 1983 Nipponkai-chubu earthquake of magnitude 7.7. The earthquake hit the northwestern district of Japan and the damage to harbour facilities and shore protection facilities was estimated as about 8.1 billion yen.

According to a revision of the Port and Harbour Law in 1973, the important points of the design procedures on the harbour structures were officially legislated. In 1979 the Technical Standard of Port and Harbour Facilities and its Commentary was compiled under the supervision of the Bureau of the Port and Harbours, Ministry of Transport, and was revised in 1989[1].

Activities in the earthquake engineering research in the four years were considerable and major research papers in the years are introduced in chapter 4.

2. DAMAGE TO QUAYWALLS BY EARTHQUAKES

Quaywalls in Japan suffered a severe damage by earthquakes, especially by the Niigata Earthquake in 1964. Some of quaywalls sank completely under the water. Foundation soil is sand down to EL. -40 m which is underlain by a silty clay layer. Approximate correlation between the earthquake damage and the compactness of sand which is represented by the N value of sand, was found. The looser the sand was, the severer the damage was. A characteristic damage of quaywalls on loose sand was a large settlement. The structural feature of most of the quaywalls in the port area was so complicated, that the remedial reinforcement had to be done against an intense and extensive ground subsidence.

At the time of the Tokachi-oki Earthquake in 1968, accelerations of ground motions were perfectly recorded at various ports. Against such casualities the present design method for earthquake resistance on quaywalls has been reexamined in the light of the damage to the structures and observed ground motion records.

Steel sheetpile quaywalls in Ishinomaki port were damaged by the 1978 Miyagi-ken-oki earthquake. Steel sheepile quaywalls and gravity type quaywall in Akita port were also severely damaged by the 1983 Nipponkai-chubu earthquake. It was concluded by the field investigation that liquefaction of the backfilling sand mainly caused damage in both ports.

2.1 DAMAGE TO GRAVITY TYPE WALLS

Nankai Earthquake in 1946

A slight earthquake damage was observed in gravity type walls in Uno Port. Foundation soil is clay to sandy clay down to EL. −23 m. The surface soil was replaced by sand of 8 m thick, and the concrete blocks and caissons were supported by piles. Cross sections of the wall before and after the earthquake are shown in Fig. 1. The wall slid toward the sea by 35 cm at maximum. No appreciable settlement of the wall itself was observed, however; an overall subsidence of 30 cm at maximum took place in the filled area. An acceleration of the earthquake was estimated to be between 0.1 g and 0.15 g in this district.

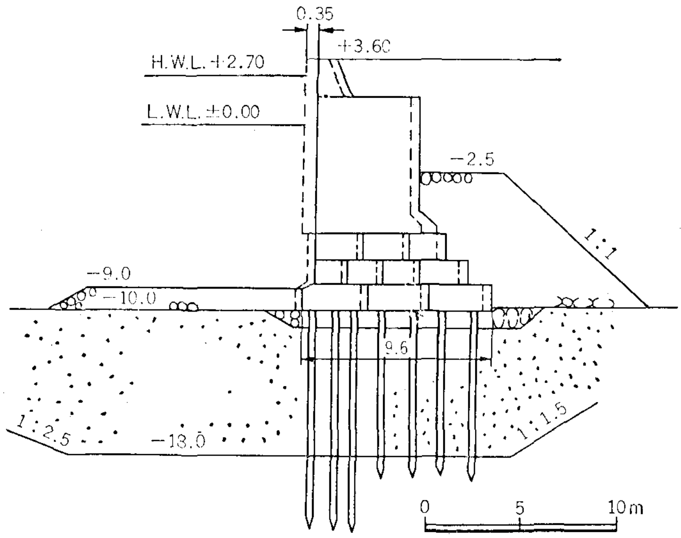


Fig. 1 Cross Section of Gravity Type Wall in Uno Port

Tokachi-oki Earthquake in 1952

The concrete caisson type walls in Kushiro Port were damaged by the earthquake. Foundation soil is sand and concrete caissons were placed on a rubble mound. As seen in Fig. 2, there occurred a settlement and sliding-out of walls. The settlement might have resulted from the local failure due to high toe pressure, or from the sliding along the shallow slip surface. Many cracks were observed on the surface of backfill.

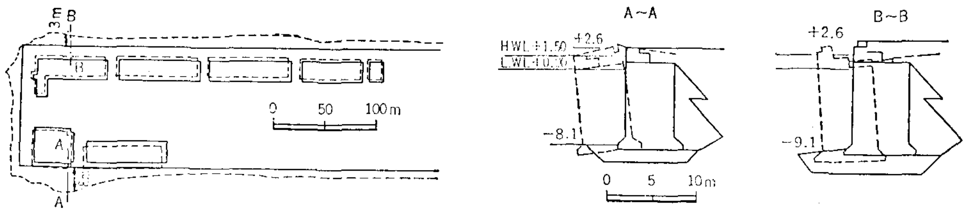


Fig. 2 Cross Section of Gravity Type Walls in Kushiro Port

Niigata Earthquake in 1964

Fig. 3 shows a cross section of the quaywall in the Central Pier in Niigata Port. Originally it was of concrete block type. In the remedial reinforcement against the ground subsidence steel pipe piles were driven down and prepacked concrete was put in front of the quaywall. The damage of quaywall itself was a swelling of the face line and sselltement of the fill. It is noted that the damage of warehouses behind the quaywall was severe.

The most drastic damage was observed in Rinko disttict. Soil investigation before the earthquake showed that sand in this district was very loose, the mean N value above EL. -10 m being less than five. Fig. 4 shows the cross section of the concrete block quaywall in the A berth. As seen in this figure, a large settlement was the major damage. The concrete wells in the D berth slid and inclined toward the sea as shown in Fig. 5. The well was of rectangular cross section and a back fill was confined inside a double wall of steel sheetpiles.

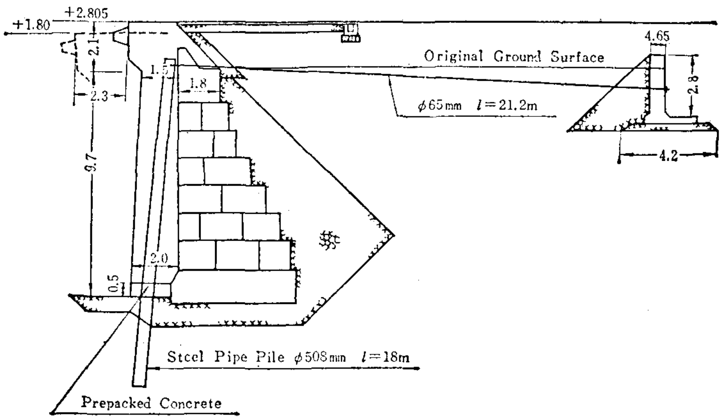


Fig. 3 Cross Section of Quaywall in Central Pier in Niigata Port

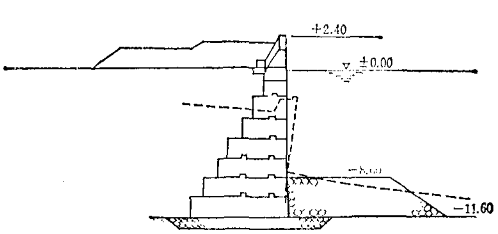


Fig. 4 Cross Section of Quaywall in A Berth in Niigata Port

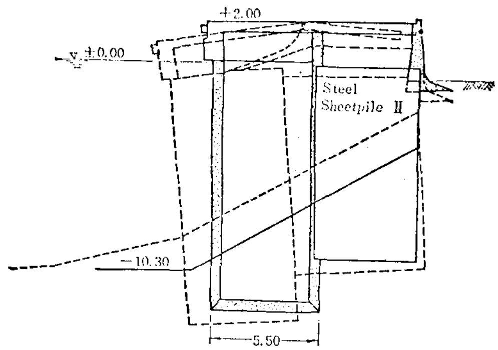


Fig. 5 Cross Section of Quaywall in D Berth in Niigata Port

Tokacni-oki Earthquake in 1968

As seen in Fig. 6 the concrete block quaywalls in Noheji Port were damaged by the earthquake. A typical feature of the damage was swelling of face line and settlement of blocks and wharf surface, in an order of several tens of centimetets. An acceleration of the earthquake was observed to be 0.23 g in this district.

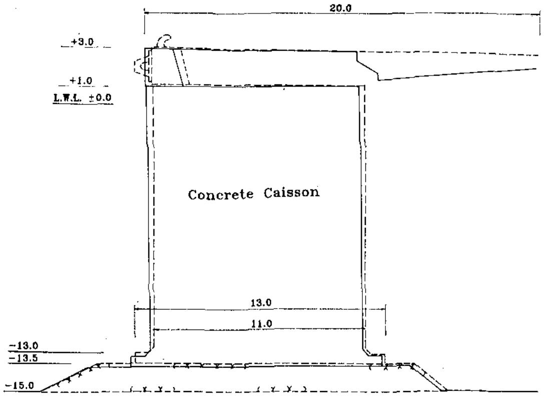


Fig. 6 Cross Section of Quaywall at Gaiko District in Akita Port

Nipponkai-chubu Earthquake in 1983

Fig. 6 shows a cross section of quaywall at Gaiko district in Akita port. A typical feature of the damage was a large settlement of apron in an order of 1.0 to 1.5 m, and the caisson wall inclined toward the sea by 1.6 degrees. An acceletation of the earthquake was observed to be 0.22 g in Akita port.

2.2 DAMAGE TO TRESTLE TYPE QUAYWALLS

Tonankai Earthquake in 1944, Mikawa Earthquake in 1945 and Nankai Earthquake in 1946

The quaywalls in Yokkaichi Port were severely damaged by three successive earthquakes. Most of the damage was caused by the first one, Tonankai Earthquake. Cross sections of the quaywall in No. 1 pier, before Tonankai Earthquake and after Nankai Earthquake are compared with in Fig. 7 The top layer of the foundation soil is soft clay down to EL. -18 m which is underlain by alternating layers of gravel and clay. The feature of the damage would imply the occurrence of slip failure. In Fig. 8 is shown a criss section of the repaired quaywall after the earthquakes.

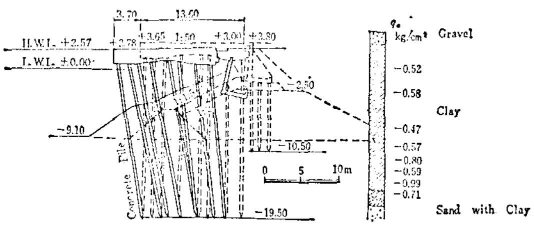


Fig. 7 Cross Section of Trestle Type Pier in Yokkaichi Port

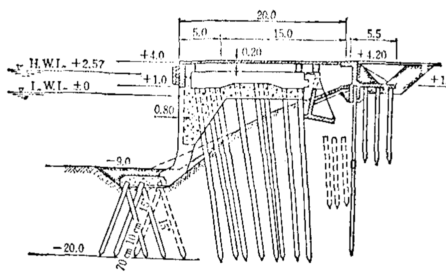


Fig. 8 Cross Section of Repaired Quaywall in Yokkaichi Port

Niigata Earthquake in 1964

The drastic damage was observed on the trestle type quaywalls in Rinko district in Niigata Port. A typical feature of damage in this area was a large settlement because the

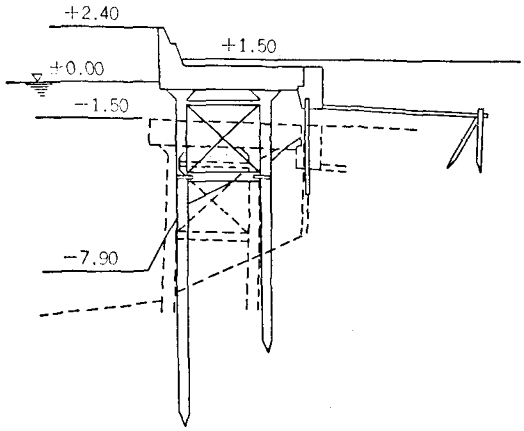


Fig. 9 Cross Section of Trestle Type Pier in B Berth in Niigata Port

ground is consisting of very loose sandy alluvial layer, as aforementioned in **2.1.** Quaywall in Fig. 9 sank completely under the water.

2.3 DAMAGE TO SHEETPILE BULKHEADS

Niigata Earthquake in 1946

The majority of quaywalls in Niigata Port were sheetpile bulkheads. A typical damage of the sheetpile bulkheads was their swelling and tilting toward the sea. This type of damage was observed mostly in bulkheads with poor anchor resistance. In such cases, the swelling of bulkheads was accompanied by a horizontal shear at a joint of the top concrete and the upper end of sheetpiles.

A cross section of the sheetpile bulkhead in Yamanoshita Revetment is shown in Fig. 10.

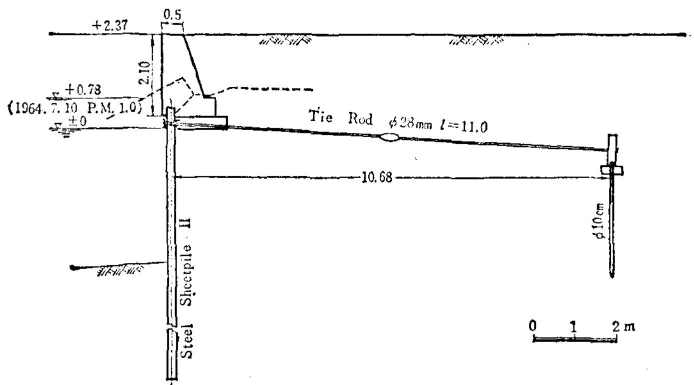


Fig. 10 Cross Section of Sheetpile Bulkhead in Yamanoshita Revetment in Niigata Port

A characteristic feature of the damage was an overall settlement. A face line of the walls swelled more or less toward the sea and some of the top concrete blocks sank completely under the water.

Fig. 11 shows a cross section of the sheetpile bulkhead in Yamanoshita Wharf. Construction of the wharf was completed about one year before the earthquake. The current

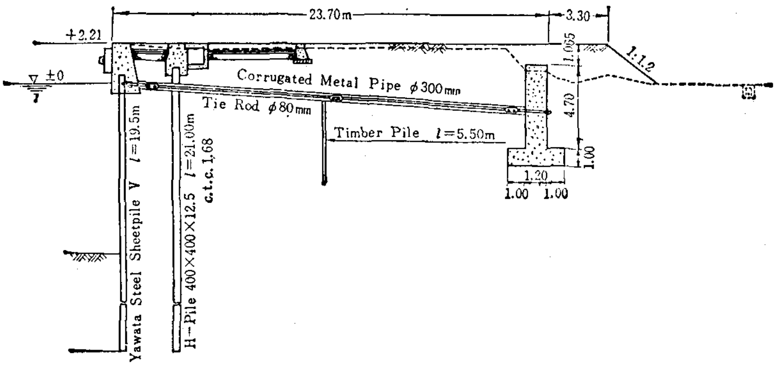


Fig. 11 **Cross Section of Sheetpile Bulkhead in Yamanoshita Wharf in Niigata Port**

earthquake resistant design was made by assuming a horizontal component of seismic coefficient of 0.12. As seen in the figure, no appreciable damage was observed, except for a local sinking of the fill behind the anchor plate.

Tokachi-oki Earthquake in 1968

As shown in Fig. 12, the Konakano No. 1 Quaywall in Hachinohe Port was heavily damaged by the earthquake. The walls tilted 5 degrees and swelled toward the sea by 60 centimeters at maximum due to insufficient anchor resistance. Tension cracks in the direction parallel to the face line and settlement in an order of several ten centimeters

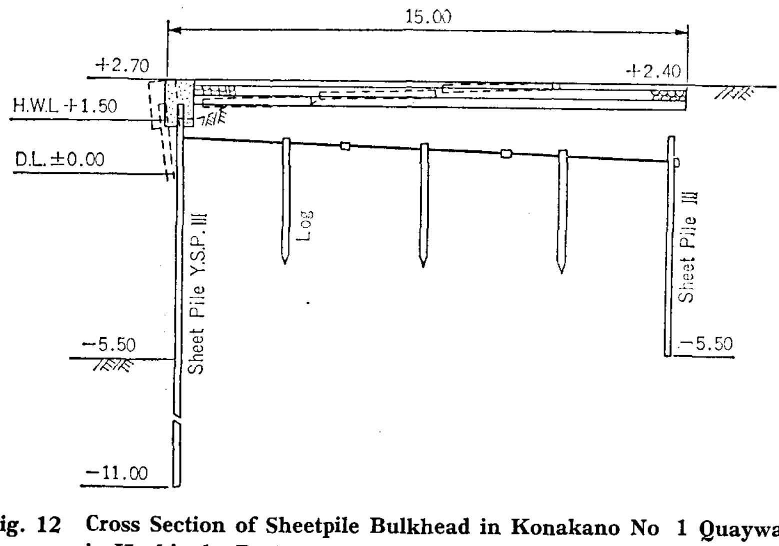


Fig. 12 **Cross Section of Sheetpile Bulkhead in Konakano No 1 Quaywall in Hachinohe Port**

occured in the backfill surface. An acceleration of the earthquake was observed to be 0.26 g in this district.

The sheetpile bulkhead with batter anchor piles had been introduced in practice not so manys before the earthquake, hence it had not met with any strong earthquake before. As shown in Fig. 13, the quaywall of Kitahama Pier in Hakodate Port was damaged by the earthquake. The fixation point of sheetpiles and anchorpiles was broken and the face line of quaywall swelled toward the sea by 59 centimetets at maximum.

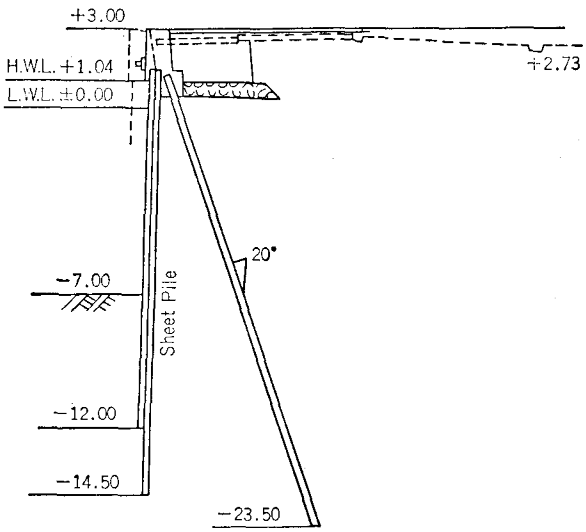


Fig. 13 Cross Section of Sheetpile Bulkhead in Kitahama Pier in Hakodate Port

Nemuro-hanto-oki Earthquake in 1973

As shown in Fig. 14 the sheetpile bulkhead was severely damaged by the earthquake. According to the investigation the tie rods were not cut and the damage was caused by the decrease of anchoring capacity due to the seismic effect.

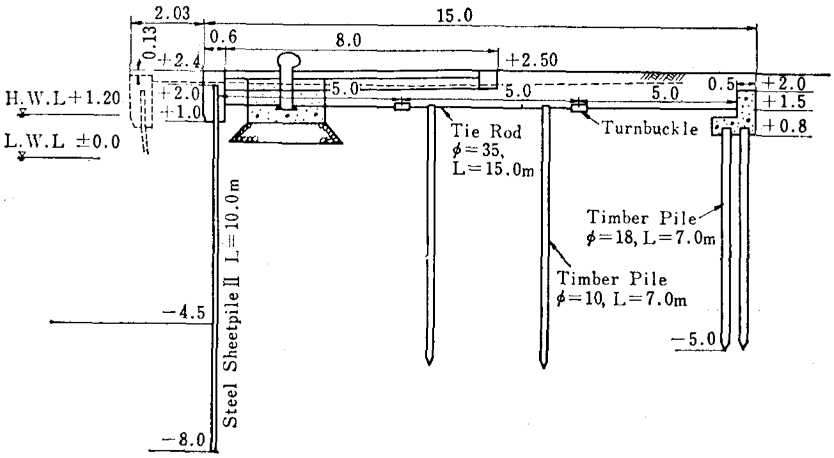


Fig. 14 Cross Section of Sheetpile Bulkhead in Hanasaki Port

Miyagi-ken-oki Earthquake in 1978

In Ishinomaki port both of the steel sheetpile walls in Nakajima wharf with -10 m of water depth and Hiyori wharf with -9 m slid toward sea by 57 cm in maximum and a wall in Shiomi wharf slid 119 cm in maximum. As shown in Fig. 15 the type of anchor of these quaywalls was steel sheetpile. At the back of these quaywalls sands from the cracks and the joints of the pavement were observed.

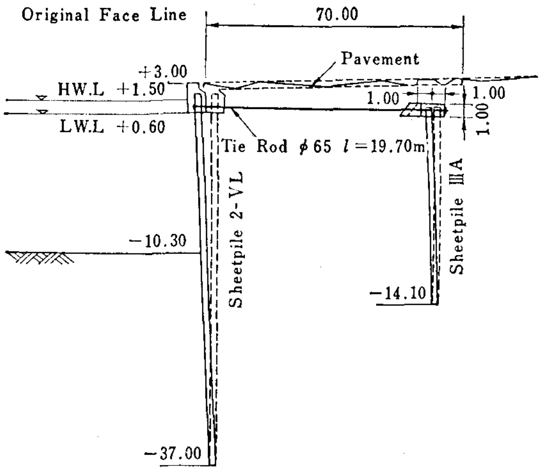


Fig. 15 Cross Section of Sheetpile Bulkhead in Nakajima Pier in Ishinomaki Port

Nipponkai-chubu Earthquake in 1983

Most of quaywalls in Akita port were sheetpile quaywalls. The drastic damage occured on sheetpile quaywalls at Ohama No. 2 wharf with -10 m. A typical features of damage in the quaywalls were a large settlement of the apron and a tilting of the coping. According to the investigation the sheetpile under the water were snapped as shown in Fig. 16. These damage were mainly caused by liquefaction of the backfilling sand.

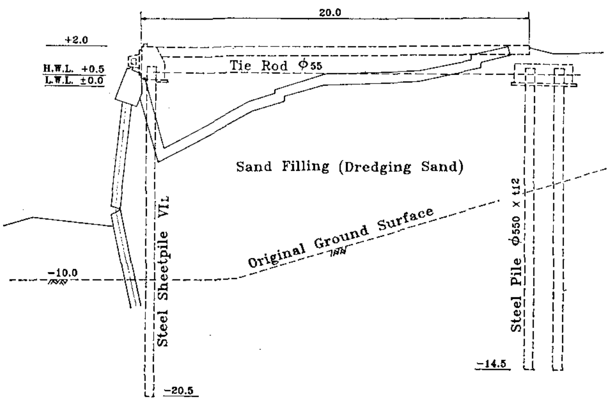


Fig. 16 Cross Section of Sheetpile Bulkhead at Ohama No. 2 Pier in Akita Port

3. CURRENT PROCEDURES OF EARTHQUAKE RESISTANT DESIGN FOR QUAYWALLS AND PIERS[1]

In 1973 the Port and Harbour Law was revised. In 1975 the engineering requirement was established as the Ordinance of the Ministry of Transport and it was prescribed in the ordinance that the facilities in ports and harbours must be safe against earthquakes as well as dead load, wave force, impact due to ships and so on. In 1975 the details on earthquake resistant design, such as design procedures, factor of safety and allowable stresses, were specified as the order of the Director General of Bureau of Ports and Harbours. In 1979 the Technical Standard of Port and Harbour Facilities and its Commentary was compiled under the supervision of the Bureau of the Ports and Harbours, Ministry of Transport, and was revised in 1989. It has been widely used as a design handbook by the engineers.

3.1 EARTHQUAKE LOAD

(1) Calculation of Earthquake Load

(A) The earthquake load acting on the breakwater and the pier should be calculated by the following formula which places them severer condition.

i) Earthquake load = Dead load × Seismic coefficient

ii) Earthquake load = (Dead load + Surcharge) × Seismic coefficient

where, the seismic coefficient should be determined by the method described in 3.1 (2).

(B) The earthquake load acting on the other kinds of structures should be determined by the instructions described in 3.13.

(2) Determination of Seismic Coefficient

(A) The seismic coefficient should be determined by the following formula, taking regional probability of occurrence of earthquake, condition of foundation soil and importance of the structure into consideration.

Seismic coefficient = Regional seismic coefficient × Factor for subsoil condition × Importance factor

As a general rule, it is considered that the earthquake load acts horizontally on the center of gravity of the structure. The earthquake load in vertical direction is not taken into consideration with the exception of the special structure which is remarkably influenced by the load. Seismic coefficient should be calculated down to two places of decimals and the last larger than or equal to 0.08 is counted as 0.1, that between 0.07 and 0.03 as 0.05 and cut away the rest.

(B) Regional seismic coefficient

Standard value of regional seismic coefficient should be decided from Fig. 17. The appropriate seismic coefficient in the region which is not described here should be determined by the seismicity of the region and the coefficients in the neighboring regions in Fig. 17.

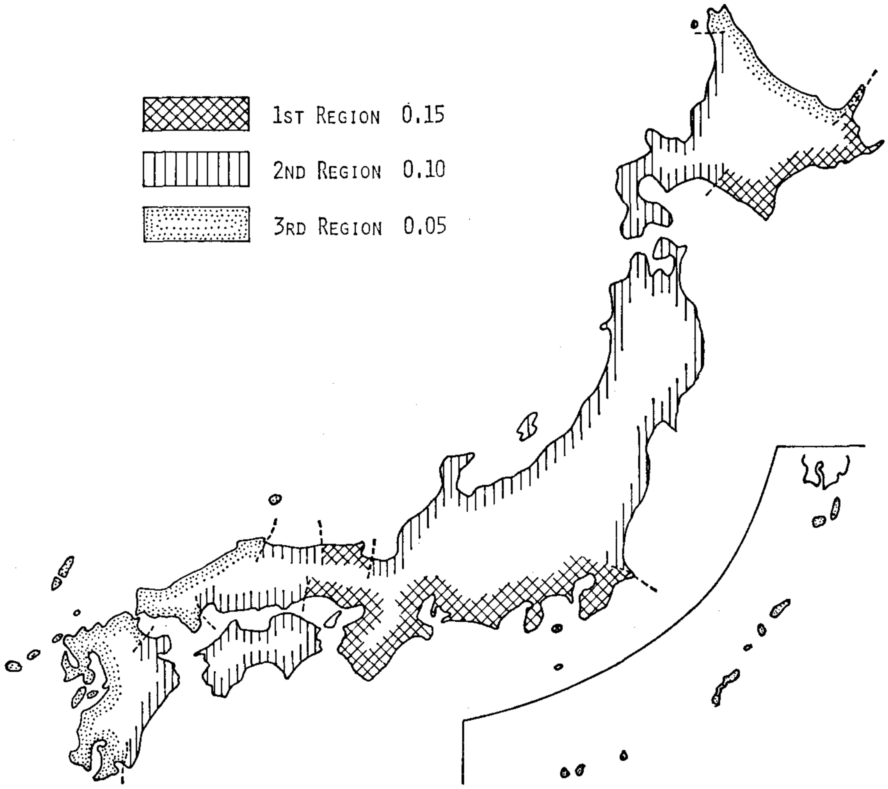


Fig. 17 Seismic zoning map for harbour structures

(C) Factor for subsoil condition

The standard value of factor for subsoil condition should be determined as shown in Table 1.

Table 1 Factors for subsoil condition

Classification	1st kind	2nd kind	3rd kind
Factor	0.8	1.0	1.2

Table 2 Classification of subsoil

Thickness of quaternary deposit	Gravel	Sand or clay	Soft groud
Less than 5 m	1st kind	1st kind	2nd kind
5~25 m	1st kind	2nd kind	3rd kind
More than 25 m	2nd kind	3rd kind	3rd kind

The classification of the subsoil condition should be assigned as shown in Table 2, considering the thickness of the quaternary deposit and the kinds of subsoil condition.

(D) Factor depending on importance of structure

The standard value of the importance factor of the structure should be determined by Table 3.

Table 3 Factors depending on importance of structures

Classification of structure	Characteristics of structure	Importance factor
Special class	The structure has significant characteristics described by items of (1)~(3) in A class.	1.5
A class	(1) If the structure is damaged by an earthquake, a large number of human life and property will possibly be lost. (2) The structure will perform an important role on the reconstruction work of the region after an earthquake. (3) The structure handles a hazardous or a dangerous object, and it is feared that the damage on the structure will cause a great loss of human life or property. (4) If the structure is damaged, economical and social activity of the region will be severely suffered. (5) If the structure is damaged, it is supposed that the repair work of it is considerably difficult.	1.2
B class	The structure is other than Special, A and C classes.	1.0
C class	The structure is small and easy for repairment, exceping that in Special and A class.	0.5

(3) Apparent Seismic Coefficient

In calculation of the earthpressure below the groundwater table, when the submerged unit weight of soil is used, the apparent seismic coefficient given by the following equation will be used.

$$k' = \frac{\gamma}{\gamma - 1} k \qquad \cdots\cdots(1)$$

where γ: unit weight of saturated soil

k: seismic coefficient above groundwater table

k': apparent seismic coefficient.

Since Eq. (1) is based on the simplified assumption that relative movement of water and soil particles during earthquake is prevented by frictional resistance of soil particles, the actual value of apparent seismic coefficient k' might be between the values given by Eq. (1) and by the equation in which γ is replaced by G_s, which is the specific gravity of soil particle. In the latter case dynamical water pressure acting on a wall should be considered in addition to lateral earthpressure in earthquakes.

3.2 LATERAL EARTHPRESSURE AND DYNAMIC WATER PRESSURE IN EARTHQUAKES

(1) Lateral earthpressure in earthquakes

Lateral earthpressure of sandy soil in earthquakes is computed by using the Mononobe-Okabe Formula[2),3)] which is derived from Coulomb's formula by statically applying a seismic force to the mass in question. For horizontal ground surface, the formula is given by the following expression (Fig. 18).

$$p=(w+\Sigma\gamma\cdot h)K \qquad \ldots.(2)$$

$$K=\frac{\cos^2(\varphi\pm\psi-\theta)}{\cos\theta\cdot\cos^2\psi\cdot\cos(\delta+\psi\pm\theta)\left[1\pm\sqrt{\frac{\sin(\varphi\pm\delta)\sin(\varphi-\theta)}{\cos(\delta+\psi\pm\theta)\cos\psi}}\right]^2} \qquad \ldots.(3)$$

$$\cot\zeta=\mp\tan(\varphi\pm\delta\pm\psi)+\sec(\varphi\pm\delta\pm\psi)\sqrt{\frac{\cos(\psi+\delta\pm\theta)\sin(\varphi\pm\delta)}{\cos\psi\sin(\varphi-\theta)}} \qquad \ldots.(4)$$

where, p: intensity of lateral earthpressure in earthquakes (t/m^2)

w: intensity of uniform load on the ground surface (t/m^2)

φ: angle of internal friction of sandy soil (°)

for general case ....30°

for particularly good backfill....40°

γ: unit weight of soil (t/m^3); buoyed unit weight should be used below water level and the followings are the standards:

above water table in backfill....1.8 t/m^3

below water table in backfill....1.0 t/m^3

h: depth from the ground surface (m)

K: coefficient of lateral earthpressure

ψ: angle between wall surface and the vertical (°)

δ: angle of friction between soil and wall (°); usually $|\delta|<15°$

θ: angle given by the following equations; $\theta=\tan^{-1}k$ or $\theta=\tan^{-1}k'$

ζ: angle between failure surface and horizon (°)

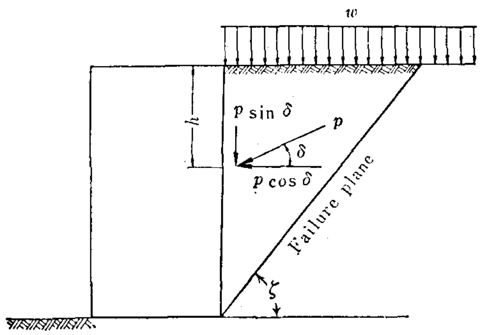


Fig. 18 Earthpressure acting on a vertical wall

In Eqs. (3) and (4), upper signs are for active case and lower signs for passive case. Coefficients of active and passive earthpressure and angles between failure surface and horizon ζ for typical values of φ and δ are shown in Fig. 19 and Fig. 20 respectively.

Correction should be made for the lateral earthpressure below water level as follows:

(i) Lateral earthpressure at the water table in the backfill is computed by employing seismic coefficient in the air.

(ii) Lateral earthpressures below the water table are computed by employing apparent seismic coefficient at the boundaries of layers in the backfill.

(iii) Distribution of lateral earthpressures under water is represented by the straight lines connecting these values.

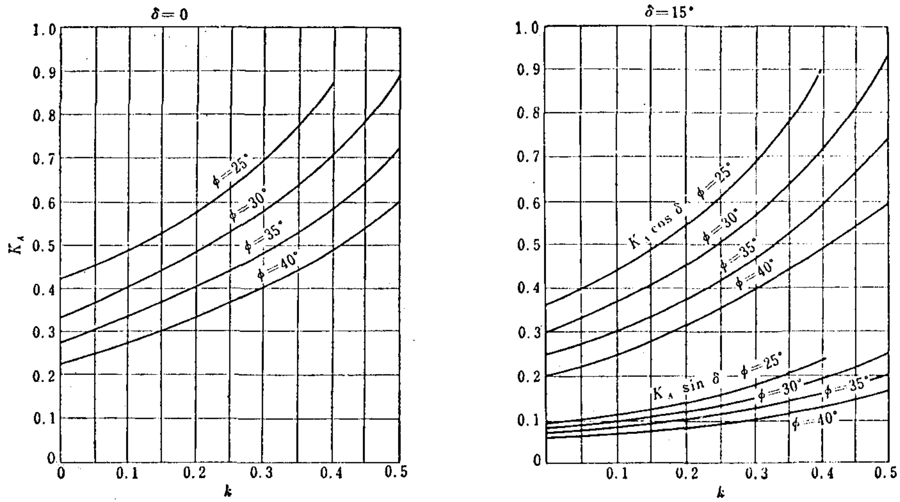


Fig. 19 (a) Coefficients of active earthpressure

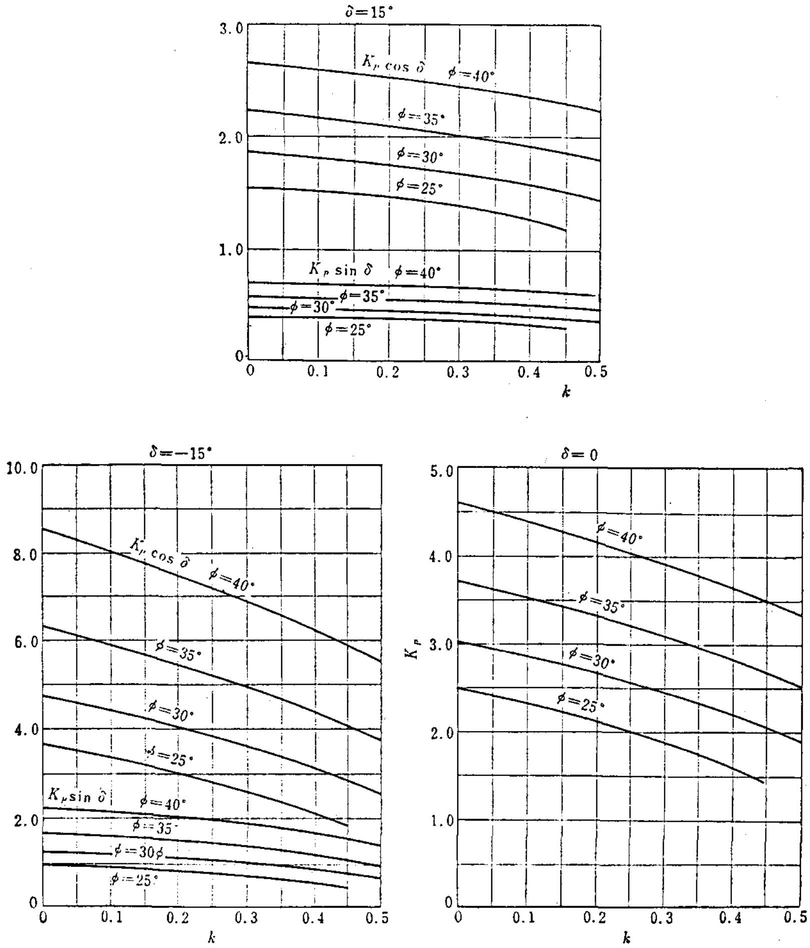


Fig. 19 (b) Coefficients of passive earthpressure

(2) Dynamic water pressure in earthquakes

Dynamic pressure of water in backfill is not taken into consideration in current design procedure, because dynamic water pressure in earthquakes is to be included in lateral earth-pressure in earthquakes, when the latter is computed by employing the apparent seismic coefficient given in Eq. (1) which is based on the assumption of combined movement of water and soil mass. Dynamic pressure of water in front of a wall is not taken into consideration, because it is recognized that the effect of dynamic water pressure in front of the wall is compensated by the other factors in the whole course of design calculation. As to the water in or between structures of a quaywall, mass force of the water due to earthquake should be considered instead of dynamic water pressure.

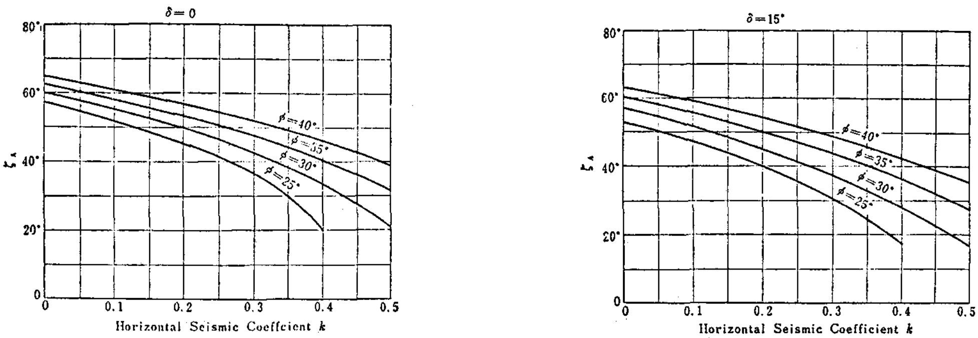


Fig. 20 (a) Angles between failure surface and horizon (Active state)

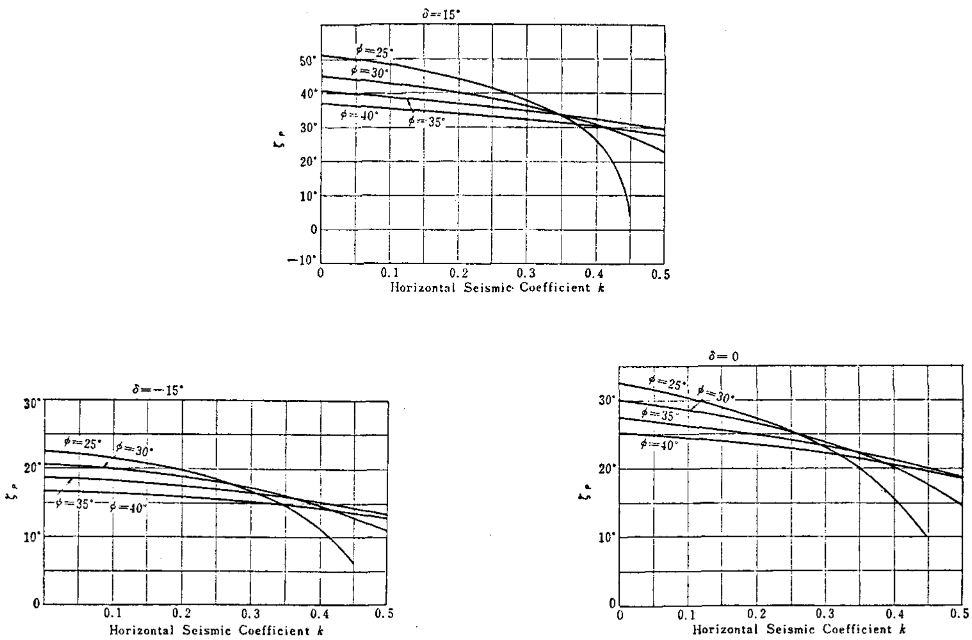


Fig. 20 (b) Angles between failure surface and horizon (Passive state)

3.3 BEARING GAPACITY IN EARTHQUAKES

Current procedure of computing bearing capacity in earthquakes is similar to that for static state and no consideration is made on the effects of dynamic load and of dynamical properties of soil.

The bearing capacity of a shallow foundation against an eccentric and inclined load should be calculated by the circular slip analysis based on the simplified Bishop method.

The safety factor should be as indicated in Table 4.

Table 4 Safety Factor for Bishop Method

	Wharf or revetment	Breakwater
Ordinary condition	More than 1.2	—
During an earthquake	More than 1.0	—
For wave pressure	—	More than 1.0

The inclined load means a load having the load inclination factor of more than 0.1.

As shear strength constants, the values obtained from triaxial compression tests should be used. However, if no results of triaxial tests are available, $C=2$ tf/m^2 and $\phi=35°$ may be used for ordinary rubble. For sand, the standard values are $\phi=40°$ for N-value of less than 10 and $\phi=45°$ for N-value of more than 10.

The starting point of the sliding surface is, as shown in Fig. 20, the point symmetrical to the end of foundation closer to the point of application of the load. The vertical load acting to the bottom surface is converted into the uniformly distributed load acting on the portion between the starting point of the sliding surface and the front of the bottom surface of wall body as shown in Fig. 21 (b) and (c). Horizontal force is applied to the bottom surface of the wall body. However, for calculations during earthquakes, no seismic force should be applied to the mound and ground. The safety factor can be expressed by the ratio between resistance moment due to shear resisting force and sliding moment due to external force and weight of earth.

When the Subgrade Reaction has Trapezoidal Distribution : $q=\frac{(p_1+p_1)}{4b'}B$

When the Subgrade Reaction has Triangular Distribution : $q=\frac{p_1 b}{4b'}$

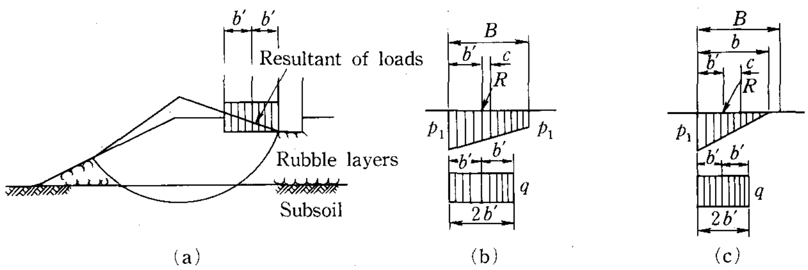


Fig. 21 Loading Conditions by Bishop Method

3.4 LATERAL RESISTANCE OF PILES

(1) Lateral resistance of vertical piles

In the design of a laterally loaded piles, three values, namely, deflection at the pile head, bending moment induced in the pile, and necessary length of pile embedment, should be estimated in advance. At present so-called Chang's method[4], in which elastic behaviours of pile and soil are assumed, is most widely used because of its relative simplicity and reliability.

The fundamental equations of deflection for a vertical pile are as follows:

$$\begin{aligned} &\text{above the ground surface:} \quad EI\frac{d^4y_1}{dx^4}=0 \qquad (0\geqq x\geqq -h) \\ &\text{below the ground surface:} \quad EI\frac{d^4y_2}{dx^4}+E_s\cdot y_2=0 \quad (x\geqq 0) \end{aligned} \qquad \cdots\cdots(5)$$

where, EI: flexural rigidity of pile

x: depth from the ground surface

y: deflection of pile at depth x

B: width of pile

E_s: elastic modulus of soil

h: height of the point of load application

In Chang's method, Eqs. are solved with an assumption that $E_s=\text{constant}=k_hB$. The solutions can give necessary data for the design of a laterally loaded pile.

The value of k_h, which is called "coefficient of horizontal subgrade reaction", can be esti-

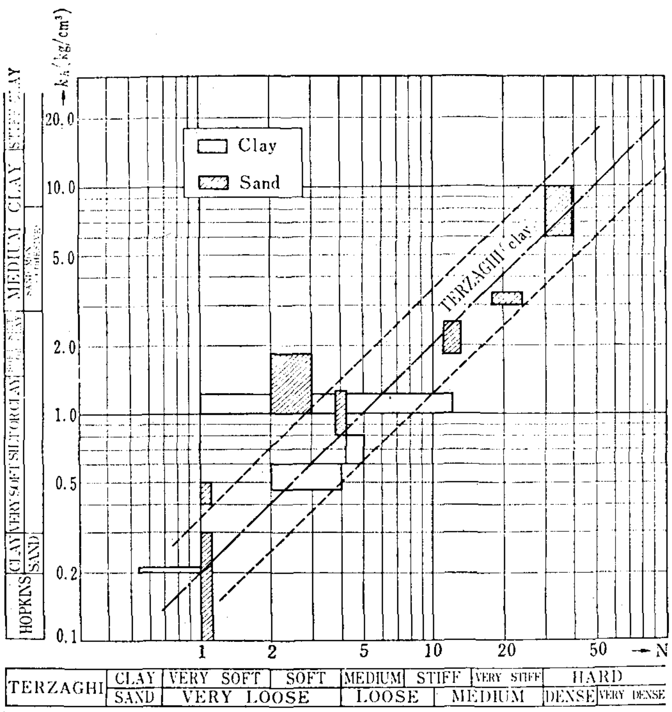


Fig. 22 Relation between kh and N (Yokohama)

mated from the standard penetration value N of the site. The relationship between k_h and N based on field test data is shown in Fig. 22.

A reliable method of lateral resistance estimation was proposed by Shinohara and Kubo[5),6),7)], in which all of the necessary computations can readily be made by means of computation charts.

The method is based on the following findings as the results of lateral load tests on a number of steel model piles embedded in saturated sand layer.

(i) A new expression $p=kxy^{0.5}$ for the relationship between soil reaction p and pile deflection y can far better explain the actual behaviours of piles than any other expression so far proposed.

(ii) Soil resistance per unit area of pile surface decreases with increasing width of pile, but it becomes almost constant when pile width is larger than 20 cm.

(iii) Effective length of embedment for a laterally loaded pile is considered to be 1.5 l_{m_1}, in which l_{m_1} is the depth of the first zero point of the moment distribution curve for an infinitely long pile.

(iv) Conversion factors are obtained by introducing the expression $p=kxy^{0.5}$ into the law of similarity. Also standard curves of the pile-top deflection, maximum bending moment, and effective length are established on the basis of the model test results. Conversion of the standard curve by means of the conversion factots gives the estimation of behaviours of a prototype pile. The necessary computation for the above mentioned conversion can easily be made with the aid of computation charts given in the reference document.

(v) Many field test data are collected and analyzed to show that an unique relationship exists between soil constant k and standard penetration value N both in sandy soil and clayey soil.

(2) Lateral resistance of coupled piles

Coupled piles show a far larger lateral resistance than the same number of vertical piles, since the major part of a lateral load is supported by axial resistance of the piles. Therefore, ultimate lateral resistance of coupled piles can be computed from the two axial forces each of which may take the ultimate bearing capacity of a pile.

3.5 STABILITY OF SLOPES IN EARTHQUAKES

Analysis of slope stability often becomes very important in the design of quaywalls, especially in the case of trestle type pier with small retaining wall or gravity type quaywalls. Stability of slopes in earthquakes is analyzed by the circular slip surface method, the horizontal seismic force being taken into consideration. The lower limits of safety factor for the stability of slopes under the static and seismic conditions are to be 1.3 and 1.0 respectively when a permanent structure is proposed. Usually, in sandy soil or gravelly soil, base failure need not be considered. However, possibility of toe failure, or sliding along plane slip surface, should be carefully examined.

3.6 STABILITY AGAINST LIQUEFACTION

Saturated loose sandy deposits tend to liquefy during earthquakes, causing damage to structures. Liquefaction, if a relevant factor, should be taken into consideration in design and construction of structures.

Liquefaction potential should be assessed by the following procedure[8].

1) Classify the soils under consideration by comparing the grain size accumulation curves with the ranges shown in Fig. 23;

   Soils within the range (A):

   Very easily liquefiable soil.

   Soils within the ranges (B_f) or (B_c):

   Easily liquefiable soil but less liquefiable than the soils within the range (A).

   Assume that no liquefaction will take place in the soils that do not belong to either the ranges (A), (B_f) or (B_c).

2) Obtain the equivalent N-Values from SPT N-values (i.e. Standard Penetration Tests blow counts) with correction for the effective confining pressure of 0.66 kgf/cm^2 and for the fines content. Conduct an earthquake response analysis of the ground by the equivalent linear method and obtain the equivalent acceleration from the computed maximum shear stress ratio (the ratio of maximum shear stress over-vertical effective stress) by multiplying the factor of 0.7 g as a conversion factor.

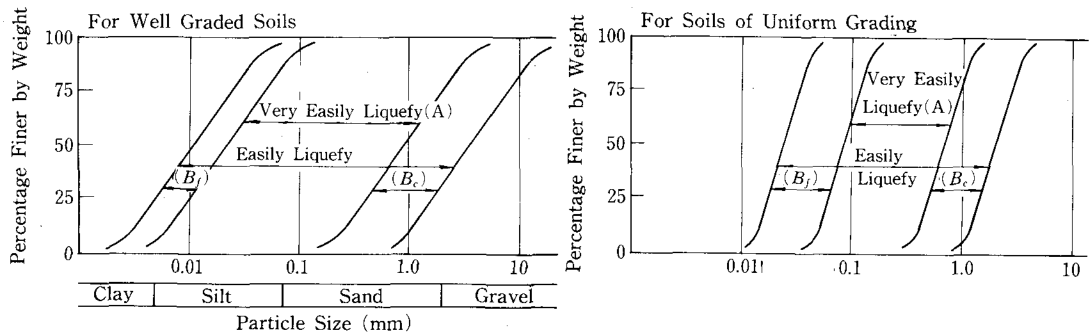
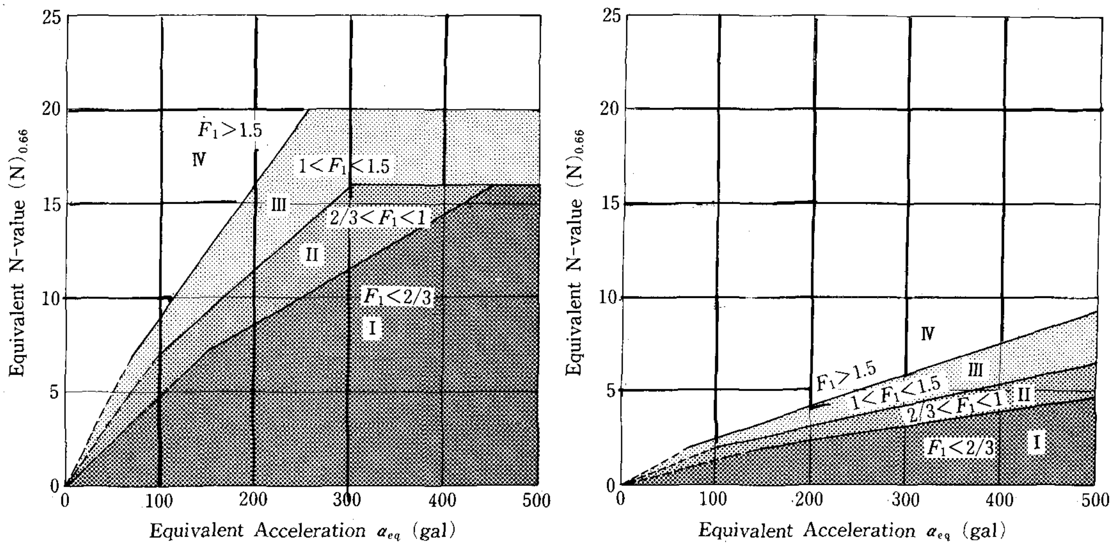


Fig. 23 Ranges of Grain Size Accumulation curves for Liquefiable Soils

3) Use the design charts shown in Fig. 24 in accordance with the ranges for grain size accumulation curves. Assess liquefaction potential in accordance with the regions to which the equivalent N-value and the equivalent acceleration belong:

   Region I: Conclude that liquefaction will take place.

   Region II: Conduct such a laboratory study as undrained triaxial cyclic loading test of soils if desirable. Otherwise, conclude that liquefaction will take place.



(a) For Soils in the Range (A) (b) For Soils in the Ranges (B_f) or (B_c)

Fig. 24 Design Chart for Assessing Liquefaction Potential

Region III: Conduct such a laboratory study as undrained triaxial cyclic loading test of soils if high reliability is required in the stability of soil deposit. Otherwise, conclude that liquefaction will not take place.

Region IV: Conclude that liquefaction will not take place.

To maintain the function of the structures, appropriate measures, if necessary, should be taken against the possible consequence of soil liquefaction. Relevant studies can be referred to on designing the area of ground compaction[9] and on designing the spacing of gravel drains[10].

3.7 GRAVITY TYPE QUAYWALLS

Gravity type quaywalls, which are very popular in our country, are relatively durable and can well withstand the strong impact of ships. However, the safety factor for bearing capacity decreases when great depth of water is required, because rapid increase of lateral earthpressure brings about the great increase of dead weight and toe pressure of the quaywall. This tendency is particularly conspicuous in earthquakes.

The fundamental idea of earthquake resistant design of a gravity type quaywall is to prevent irreparable damage due to earthquakes, although some amount of wall movement may be allowed.

Main external forces which should be considered in earthquake resistant design are lateral earthpressure (cf. section 3.2), water pressure (cf. section 3.2) and mass force of the wall itself. Stability analysis of a gravity type quaywall is done on three items as shown below.

(1) Sliding of the wall along its base

The ratio between horizontal and vertical components of the total external force should be smaller than the coefficient of friction between soil and base, which usually is 0.6.

The safety factor is to be greater than 1.2 for static condition and can be reduced to 1.0 for seismic condition.

(2) Bearing capacity at the base

Computation of the allowable bearing capacity is done according to the method shown in section 3.3.

(3) Sliding in the foundation soil

Sliding in the foundation soil is analyzed by the following procedure given in section 3.5.

In addition to the above mentioned analyses, it is desirable to take the following measures to prevent the damage at the corner and the approach of the quaywall and also at the joint of different structures; namely (i) increasing seismic coefficient by some 20%, (ii) providing stays, or (iii) connecting the walls in the direction of quaywall.

3.8 SHEETPILE BULKHEADS

Sheetpile bulkheads are well constructed in our country, because they can easily and rapidly be built up in the field at relatively low cost. Furthermore, availability of large size steel sheetpile and improvement of the cathodic protection technique have made sheetpile bulkheads more popular in recent years.

Sheetpile bulkheads with relieving platform are constructed, when excessively large lateral earthpressure is expected to act on the wall due to the large height of wall and heavy surcharge and the ordinary sheetpile bulkhead can not support it.

The fundamental principles of the current design method of sheetpile bulkheads are based on the extensive studies by G. P. Tschebotarioff[11], P. W. Rowe[12] and others on static state, and a special consideration is given to the seismic condition by utilizing field experiences in this country.

The earthquake resistant design of a sheetpile bulkheads is carried out on the following steps;

(1) Computation of lateral earthpressure and residual water pressure

The computation is done, following the procedures given in section 3.2.

(2) Estimation of necessary length of sheetpile embedment

Length of sheetpile embedment is computed by free-earth support method. The safety factors against the failure of embedment are 1.5 and 1.2 for static and seismic conditions respectively in sandy stratum. In cohesive soil stratum, the safety factor is 1.2 for both static and seismic conditions.

(3) Design of tie-rod

In the case of sheetpile bulkhead constructed in sandy ground, tie-rod tension is computed on the assumption that the bulkhead is a simple beam which is supported at the sea bottom and the position of tie-rod connection and is carrying the load of lateral earthpressure and residual water pressure. In the case of cohesive soil, tie-rod tension is computed by the "Fixed-Earth Support Method (Elastic Line Method)." Allowable stress of tie-rods is to be 40% and 60% of the proved yield strength of steel for the static and the seismic conditions repectively. These relatively low values of allowable stress are adopted to have taken account of bending moment in the tie-rod due to surcharge and the concentration of lateral earth pressure near the position of tie-rod connection.

(4) Design of sheetpile section

In the case of sheetpile bulkhead constructed in sandy ground, the maximum bending moment is computed for the simple beam mentioned in (3). This value of maximum moment, which is about 40~50% of that computed by free-earth support method, corresponds to the value computed by fully taking into account the moment reduction due to flexibility of sheetpile. In the case of cohesive soil, fixed-earth support method is used in the computation of the maximum bending moment. The allowable stresses of sheetpile for static and seismic conditions are 60% and 90% of the proved yield strength of steel materials respectively.

(5) Design of anchor plate

Usually anchor plates are provided to support tie-rod tension, but in the case of a sheetpile bulkhead with relieving platform no anchor plate is used, because the horizontal force is supported by the platform and piles. It is desirable to use anchor piles when the soil in front of the anchor easily liquefies due to vibration.

Lateral resistance of an anchor plate should be 2.5 times of the tie-rod tension for static and seismic conditions respectively. Anchor plates should be placed behind the active failure wedge starting from the sea bottom.

(6) Design charts

These computations for sheetpile bulkheads are illustrated in the design charts, and sheetpile section, tie-rod diameter, length of embedment etc., can readily be known from the charts when the design conditions such as soil condition, water depth, level of residual water, position of tie-rod, surcharge and seismic coefficient are given. As an example, charts for the condition, H=9 m and w=3.0 t/m², are shown in Fig. 33.

Symbols appearing in the charts have the following meanings.

H: water depth in front of quaywall (m)
H': top level of quaywall (m)
k: seismic coefficient to be used in the design
w: surcharge (t/m²)
$\phi=25°$: angle of internal friction of soil=25°, etc.
φ_{100}: diameter of tie-rod=100 mm, etc.
M_{max}: maximum bending moment of sheetpile (t-m/m)
A_p: tension of tie-rod (t/m)
D: embedded length of sheetpile (m)
$\sigma_s=900$: allowable stress of steel=900 kg/cm², etc.

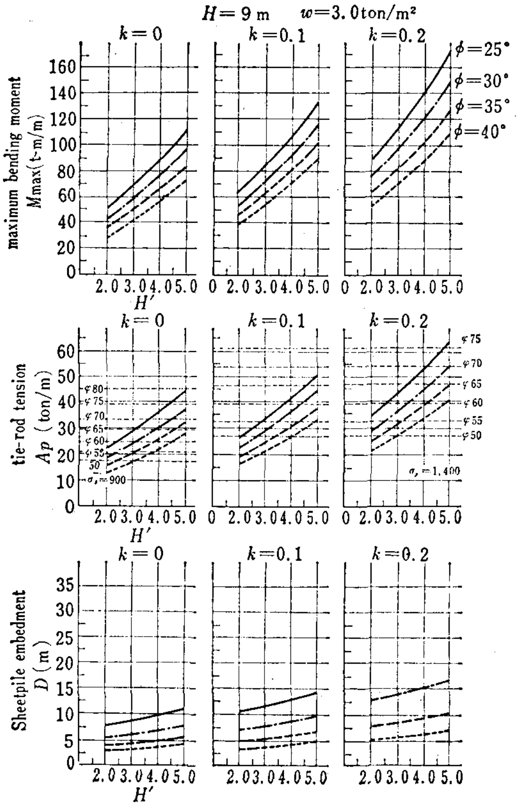


Fig. 25 Design charts for sheetpile bulkhead

As previously mentioned, the horizontal force acting on a sheetpile bulkhead with relieving platform is supported by platform and piles. The maximum bending moment in sheetpile decreases as the height of platform increases, but at the same time the horizontal mass force of platform and fill due to earthquakes increase considerably, causing large reaction in piles. Accordingly, the height of platform is determined from the balance between the strength of sheetpile and the bearing capacity of piles.

3.9 CELLULAR BULKHEADS

Sheetpile cellular bulkhead are constructed by driving straight-web sheetpiles to form cells and then filling them with soil (Fig. 26). This type of quaywall is becoming

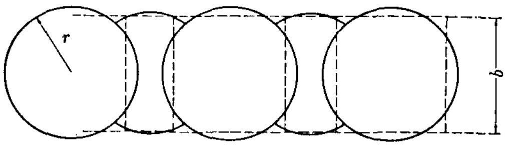


Fig. 26 Plan of cellular bulkhead

very popular because of simplicity and rapidity of construction, and low cost for greater depth quaywalls.

In earthquake resistant design of a cellular bulkhead, both of the stability of the whole structure and the stability of bulkhead itself should be examined. The stability of the whole structure is examined by supposing it is a gravity type quaywall. The stability of bulkhead itself is considered to be depending on lock tension of sheetpiles and internal shearing resistance of bulkhead against shear failure. Accordingly stability analysis should be performed on these items.

The width of bulkhead is so determined that deforming moment due to external forces doesn't exceed resisting moment of the bulkhead against shear failure[18]. The resisting moment (M_r) consists of two components, namely, that of fill material (M_{rf}) and that of sheet-piling (M_{rs}).

$$M_r = M_{rf} + M_{rs} \qquad \ldots.(6)$$

Computation of the ultimate resisting moment is done by Kitajimals formula[16] based on the model experiment.

$$\left.\begin{array}{ll} M_{rf} = \dfrac{1}{6}\gamma h^3 \cdot \dfrac{2}{3}\nu^2(3-\nu\cos\phi)\tan\phi\sin\phi & \ldots.\ \text{for static condition} \\ M_{rf} = \dfrac{1}{6}\gamma h^3 \cdot \nu^2(3-\nu\cos\phi)\sin\phi & \ldots.\ \text{for seismic condition} \\ M_{rs} = \dfrac{1}{6}\gamma h^3 \cdot \dfrac{3}{2}\nu f\tan\phi & \end{array}\right\} \qquad \ldots.(7)$$

Symbols

M_r: ultimate resisting moment of bulkhead
M_{rf}: ultimate resisting moment of fill material
M_{rs}: ultimate resisting moment of sheet-piling
γ: unit weight of fill material
h: height of bulkhead from the sea bottom
b: width of bulkhead
ν: b/h
ϕ: angle of internal friction of fill material
f: coefficient of friction between locks of sheetpiles ($f=0.3$)

Safety factor against shear failure should be 1.2 for static condition and seismic conditions.

The lock tension T given in the following equation should be smaller than the allowable value, which is 150 t/m for the straight-web sheetpile produced in Japan.

$$T = K_i \gamma h \cdot r \qquad \ldots.(13)$$

where, r: radius of cell
K_i: coefficient of earthpressure to be used in the computation of lock tension. ($K_i=0.6$)

3.10 TRESTLE TYPE PIER

Trestle type piers are usually very stable in earthquakes because they are relatively light structures and are subjected to no lateral earthpressure.

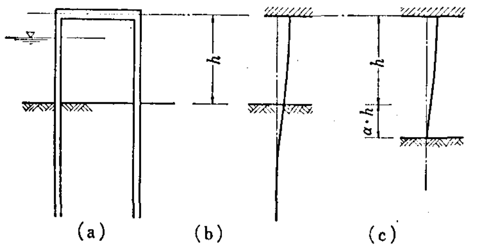


Fig. 27 Piled pier

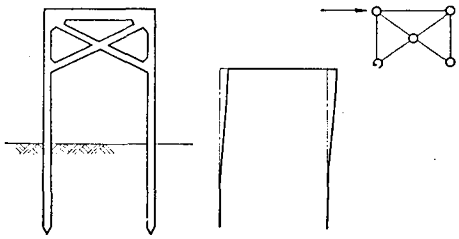


Fig. 28 Piled pier with bracings

The most simple form of trestle type pier is piled pier. Piles supporting super structure are designed as "fixed-head" (cf. 3.4 (1)) subjected to lateral forces at the pile head in accordance with the procedure shown in section 3.4 (1) [Fig. 27 (a), (b)]. Main lateral forces to be considered in the design are ship impact, pulling force of moored ships, and earthquake force acting on the pier. When piles are braced, pile heads is assumed to be fixed at the lower connecting point with the bracings (Fig. 28).

When a block of trestle type piers rotates in a horizontal plane subjected to lateral eccentric loads, lateral loads distributed into individual piles are calculated by following equation.

$$H_i=\frac{K_{Hi}}{\Sigma K_{Hi}}H+\frac{K_{Hi}X_i}{\Sigma K_{Hi}X_i^2}eH \qquad \cdots\cdots(8)$$

where, $K_{Hi}=\dfrac{12EI_i}{\left(h_i+\dfrac{1}{\beta_i}\right)}$ (t/m): lateral stiffness constant

H_i: lateral loads distributed into individual piles (t)

h_i: heignt between pile top and sea bottom (m)

$\beta=\sqrt[4]{\dfrac{kD}{4EI}}$ (m^{-1})

EI: flexural rigidity of pile (t-m^2)

H: applied lateral force to a block of trestle type pier

e: distance between centre of block and the point of load application (m)

X_i: distance between the axis of symmetry of a block and individual piles (m)

k: coefficient of horizontal subgrade reaction (t/m^3)

($k=150\,N$, N: N value by standard penetration test)

D: width of pile (m)

Axial forces of piles due to lateral loads are calculated assuming that the distributions

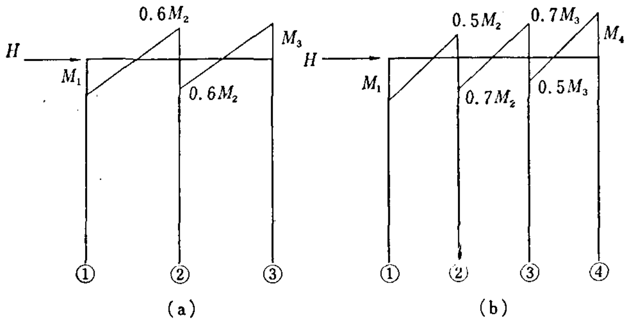


Fig. 29 Distribution of bending moment

of bending moment are as shown in Fig. 29 (a), (b). In Fig. 29 (a), (b), M_i denotes the end moment of pile head of the ith pile.

For the sake of simplicity, computations of piles are sometimes carried out with an assumption that piles are vertical cantilever beams fixed at a certain depth below the ground level. The depth of the point of fixity is assumed to be αh, where h is the height of the pile from the sea bottom and α, is a coefficient (Fig. 27 (c)). Yokohama[14] recommended the following formula for α, in which both of the characteristics of soil and pile are taken into consideration

$$\alpha=\frac{1}{\beta h} \qquad \ldots.(9)$$

In the case of a pier which has trestles of coupled piles the design of trestles is performed following the procedure shown in section 3.4 (2).

It is a common practice to connect trestles rigidly with floor frames, but it is desirable to divide a pier into several blocks to prevent the damage due to differential settlement. In such case a dynamical analysis can be applied to each block, because it is considered to be a relatively simple system of vibration.

3.11 TRESTLE TYPE PIERS WITH SMALL RETAINING WALL

Trestle type piers with small retaining wall are built on a slope and are favourable where soft foundation prohibits the construction of a gravity type or sheetpile type quaywall because of the insufficiency of bearing capacity or that of passive resistance.

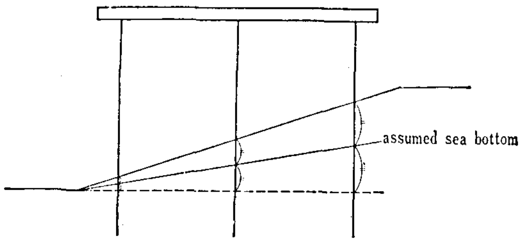


Fig. 30 Trestle type pier on slope

Trestles are designed by the method shown in section 3.10 assuming that the sea bottom as shown in Fig. 30, and retaining wall is designed in accordance with the design procedure for a similar type quaywall.

In addition to the separate analysis of the each part, the stability of the whole structure including slope and retaining wall should carefully be studied with the procedure shown in section 3.5. It is desirable not to connect rigidly the main part and the retaining wall taking the possibility of differential settlement into account.

3.12 ALLOWABLE STRESSES OF MATERIALS

(1) Steel

Allowable stresses of steel material are determined as shown in Table 5 through 8. Allowable stress of tie rods must be equal to or below 40 percents of yielding stress certified.

Table 5 Allowable stresses of structural steel

(unit: kg/cm²)

Kind of stress	SS41, SM41, SMA41	SM50	SM50Y, SM53, SMA50
Axial tensile stress for net section	1,400	1,900	2,100
Axial compressive stress for gross section	1,400	1,900	2,100
Bending tensile stress for net section	1,400	1,900	2,100
Bending compressive stress for gross section	1,400	1,900	2,100
Shearing stress for net section	800	1,100	1,200
Bearing stress between steel plates	2,100	2,800	3,100

(2) Concrete

Allowable stresses of concrete should not exceed the values shown in Table 9.

(3) Increase of allowable stresses

Allowable stresses of steel and concrete for short period loads such as earthquake loads could be same as 1.5 times of the allowable stresses in normal condition.

Table 6 Allowable stresses of steel pile materials

(unit: kg/cm^2)

Kind of stress	SKK41, SHK41, SHK41M, SKY41	SKK51, SHK50M, SKY50
Axial tensile stress for net section	1,400	1,900
Axial compressive stress for gross section	(a) $(l/r) \leq 20$, 1,400 (b) $20 < (l/r) < 93$, $1,400-8.4((l/r)-20)$ (c) $93 \leq (l/r)$, $\frac{1,2000,000}{6,700+(l/r)^2}$	(a) $(l/r) \leq 15$, 1,900 (b) $15 < (l/r) < 80$, $1,900-13((l/r)-15)$ (c) $80 \leq (l/r)$, $\frac{12,000,000}{5,000+(l/r)^2}$
Bending tensile stress for net section	1,400	1,900
Bending compressive stress for gross section	1,400	1,900
Combination of axial compressive force and bending moment	(a) For axial tensile force $\sigma_t+\sigma_{bt} \leqq \sigma_{ta}$ and $-\sigma_t+\sigma_{bc} \leq \sigma_{ba}$ (b) For axial compressive force $(\sigma_c/\sigma_{ca})+(\sigma_{bc}/\sigma_{ba}) \leq 1.0$	
Shearing stress for gross section	800	1,100

Notation l: effective backling length (cm)
r: radius of gyration for gross section (cm)
σ_t, σ_c: tensile stress due to axial tensile force and compressive stress due to axial compressive force (kg/cm^2)
σ_{bt}, σ_{bc}: maximum tensile stress and maximum compressive stress due to bending moment (kg/cm^2)
σ_{ta}, σ_{ca}: allowable tensile stress and allowable axial compressive stress (kg/cm^2)
σ_{ba}: allowable bending compressive stress (kg/cm^2)

Table 7 Allowable stresses of steel sheetpile materials

(unit: kg/cm^2)

	SY30	SY40
Bending tensile stress for net section	1,800	2,400
Bending compressive stress for gross section	1,800	2,400
Shearing stress for gross section	1,000	1,300

Table 8 Allowable tensile stress of reinforcing bars

(unit: kg/cm^2)

Kind of reinforcing bars	SR24	SR30	SD30A	SD35	SD40
(a) Tensile stress in normal condition	1,400	1,600	1,800	2,000	2,100
(b) Tensile stress due to yield point	1,400	1,800	1,800	2,000	2,200
(c) Tensile stress for fatigue loading	1,400	1,600	1,600	1,800	1,800

Notation 1) Diameter of bars is less than 32 mm.
2) Tensile stresses in (b) are adopted to determination of development length and lap splice length of reinforcing bars.

Table 9 Allowable stresses of concrete

(unit: kg/cm^2)

Kind of stress	Items	Reinforced concrete $\sigma_{ck}=240$	Plain concrete
Axial compressive stress	—	90	$(\sigma_{ck}/4)\leq 55$
Shearing stress	τ_{a1} { beam slab	4.5 9	—
	τ_{a2}, only shear force	20	
Bond stress	round bars deformed bars	8 16	—
Axial tensile stress	—	0	$(\sigma'_{ck}/7)\leq 3$
Bearing stress	—	$0.3\sigma_{ck}$	$0.3\sigma_{ck}\leq 60$

Notation σ_{ck}: design standard compressive strength.
σ'_{ck}: design standard tensile strength.
τ_{a1}: allowable shearing stress without calculation of diagonal tension bars.
τ_{a2}: allowable shearing stress with calculation of diagonal tension bars.

3.13 OTHER REGULATIONS AND RECOMMENDATION

When the seismic coefficient is determined under the consideration of survey on seismic activity of the region, characteristics of earthquake motion and response characteristics of the ground against earthquake, the regulations described in 3.1 may not be applied for design.

When the earthquake resistant design is confirmed by the consideration of the dynamic characteristics of the structure and the investigation on the response analysis for the structure against earthquake, the regulations described in 3.1 and 3.2 may also not be applied.

It is further advisable in case of necessity that the earthquake resistant design is confirmed by the consideration of the dynamic characteristics of the structure and the investigation on the response analysis against earthquake.

4. RESEARCH ACTIVITIES ON THE EARTHQUAKE RESISTANT DESIGN OF QUAYWALLS

Research activities on the earthquake resistant design of quaywalls in our country will be classified into two categories. One is the adequate application of the advanced knowledge of soil mechanics to the earthquake resistant design of quaywalls which are essentially earth-retaining structures. The other is the study of earthquake resistant design in consideration of the structural feature of quaywalls. The former category is considered to be a part of the general problem of soil engineering, therefore, the content of the present paper shall be limited to the researches closely connected to harbour structures.

4.1 SEISMIC COEFFICENT FOR EARTHQUAKE RESISTANT DESIGN

In 1975 S. Noda and T. Uwabe[15] proposed the relation between the seismic coefficient and the maximum ground acceleration for gravity type quaywall. According to the present design standard for port and harbour structures, they carried out the stability analysis of 129 gravity type quaywalls at 49 ports in 12 earthquakes and obtained the seismic coefficients corresponding to the severity of ground motions during the past earthquakes. The maximum ground accelerations in the ports were also estimated by calculating the ground response during the earthquakes with reference to the attenuation curves of the base rock acceleration. The waves had been derived from accelerograms in port area.

S. Kitajima and T. Uwabe[16] also analyzed the seismic damage in anchored sheetpile bulkheads of 110 in 22 ports. They concluded that the relation between the seismic coefficient and the maximum ground acceleration for gravity quaywalls is applied correspondingly to the sheetpile bulkheads.

In 1976 T. Uwabe, S. Noda and E. Kurata[17] studied characteristics of vertical components of 574 strong-motion accelerograms and they made clear the effects of vertical ground motion on dynamic stability of gravity structures. They concluded that the ratios of the maximum accelerations of vertical component to those of horizontal component were less than 1/2, and the vertical component has little effect on the safety factor of the gravity quaywalls.

In 1984 S. Kitazawa, T. Uwabe and N. Higaki[18] studied expected value of maximum base rock accelerations along coasts of Japan. They used the earthquake data in the vicinity of Japan in and after 1885. They calculated expected maximum accelerations at 190 points using Noda and Uwabe's formula which gives the maximum base rock accelerations by magnitude of earthquake and distance, and the extreme value distribution.

In 1987 T. Uwabe[19] presented an estimation method to give the quantity of the damage cost using the ratio of the corresponding seismic coefficient of the ground acceleration to the seismic coefficient which gives the safety factor of one in the stability analysis of the design standard. And he studied an optimum seismic coefficient from an economic viewpoint for the rational seismic design, on the basis of the qnantitative estimation method of the cost of damage.

4.2 BEHAVIOUR OF SOIL LAYER DURING VIBRATION

Judging from the engineering point of view, the dynamic behaviour of soil layers is very important.

H. Arai and Y. Umehara[20),21] examined the dynamic behaviour of saturated sand layers and sand layers made in the box on the shaking table with 5 m long, 1.5 m wide and 1.6 m high.

Values of rigidity are found to be strongly dependent on the effective overburden pressure. In addition, it is found that the vibrational characteristics of saturated sand layers is approximately equivalent to that of dry sand layers in a small deformation range, assuming and grains to move completely together with water and the rigidity corresponding to the effective overburden pressure to be used. While in a large deformation range, the rigidity reduces rapidly due to the effect of pore water pressure.

The relation between the shear modulus of sand layers and confining pressure as well as shear strain, and the relation between the damping constant of sand layers and shear strain were compared with the results by the tests of small sand samples. From the comparison it was made clear that the results by both methods did not show serious contradiction.

A new soil improvement method called Deep Mixing Method was developed and has been widely applied to construction work on soft clay ground. T. Inatomi and others[22] analyzed the vibration characteristics of the improved ground by using the acceleration records observed and they compared the observed records with the seismic response computed by finite element method using the equivalent linear method. It was found that the improved ground behaves like a rigid body, the maximum acceleration of the top of the improved ground are smaller than that of the surface of soft clay layer far from the improved ground, and the maximum acceleration of the improved ground and of the clay layer do not occur at the same time.

T. Inatomi and others[23] also carried out the shaking table tests on the models of wall type improved ground. Major results are as follows;

(1) The wall type improved ground behaves like a rigid body.

(2) As the table acceleration increases, dynamic earth pressure of clay layer acting on reclamation side of the improved ground increases and that acting on sea side of the improved ground decreases.

(3) It was found that the proposed earthquake resistant design method is able to evaluate the stability of the improved ground.

Inatomi and others[24] were also analyzed the results of earthquake observation, which has been carried out at Daikoku island in Yokohama port. The dimension of improved ground is 57 m wide and 37 m deep. The base ground of improved ground is soft clay from −49 m to −73 m. As the bottom of improved ground does not touch to rigid layer, this is called as Floating type. The results of observation are summarized as follows;

1) When the acceleration of improved ground indicated maximum value, vibration mode of original soft ground was the second or third mode. Its phenomenon affected to restrain vibration of improved ground.

2) Dynamic behavior of the floating type improved ground was the same as that of original ground in low frequency range, but in high frequency range the behavior of improved ground was different from that of original ground.

4.3 LATERAL EARTH PRESSURE AND DYNAMIC WATER PRESSURE IN EARTHQUAKES

N. Mononobe[2] and S. Okabe[3] expanded Coulomb's and Rankine's formulae for the lateral earth pressure during earthquake by supplementing mass force $k_h \cdot W$ to the weight of the sliding wedge W. (k_h: Horizontal component of seismic coefficient).

Y. Ishii, H. Arai and H. Tsuchida[25] have performed extensive studies on the lateral earth pressure in a vibration by using a box on a large vibration table since 1953, mainly trying to find the relation between the lateral earth pressure and motion of gravity type wall in consideration of soil characteristics.

From these observations, it would be concluded that the earth pressure against the gravity type wall in earthquakes should be safely taken as the earth pressure calculated by the Mononobe-Okabe formula and increased by 10%. On the other hand, there is a field evidence that, if a gravity type wall is designed by considering the earth pressure by the Mononobe-Okabe formula and the coefficient of base friction of 0.6 is assumed, the wall has a sufficient safety against the horizontal sliding.

Results of the above mentioned researches have given the basis for the recommendation for a use of the Mononobe-Okabe formula as stated in section 3.3.

Y. Ishii and H. Tsuchida[26] performed an experimental research on the dynamic earth pressure and dynamic pore water pressure in saturated sand. They used a box of length of 262 cm, breadth of 50 cm and height of 100 cm, which was fixed on a vibration table. Saturated sand was filled in the box in thickness of 70 cm and dynamic pressures against a fixed and a movable walls respectively were measured. The following conclusions were obtained from results of the experiment.

(1) Saturated sand liquefies easily by application of vibration. As sand liquefies a remarkable decrease in voids ratio takes place and an excess pore water pressure sets up. The lower the permeability is and the larger the initial voids ratio is, the more readily a sand liquefies. In an extreme case, the sand liquefies under the vibration of acceleration as low as 50 gals. In any particular sand, there is a unique relationship between an acceleration of the applied vibration and the voids ratio attained through the vibration. Therefore, the possibility of liquefaction of sand at certain acceleration can be predicted from this relationship.

(2) When the saturated sand is subjected to vibration, a pore water pressure sets up. In the case of highly permeable sand, the measured pore water pressures are approximately equal to those calculated by the simplified Westergaad's formula. If the permeability of sand is low, however, the measured pore water pressures are found to be smaller than the calculated pressures.

H. Arai[27] performed the stress analysis in the semi-infinite dry sand stratum subjected to the horizontal force which relates to the seismic coefficient and derived the following facts.

The mean normal stresses increase in the active state and decrease in the passive state with the increase of the seismic coefficient. Both stresses converge to one value, when the seismic coeffcient equals to the friction coefficient of the sand. These results of analysis was compared with those obtained by the method using the vibration table.

T. Uwabe and others[28] studied the aseismicity of the large composite breakwater by means of the shaking table model tests and the earthquake response calculation by finite element method. The model tests show that the hydrodynamic pressures acting on a caisson in resonant condition estimated by the formula of West regard using the water depth and the seismic coefficient of the top of the mound.

T. Uwabe, H. Tsuchida and E. Kurata[29] analyzed the strong-motion accelerograms of the Ofunato composite breakwater in the water depth of −38 m. They also compared the observed accelerations with those by the equivalent linear method on the coupled hydrodynamic response. The comparison showed that a relatively good agreement, and the estimated strain-depended characteristics of the mound's material were almost equal to the cyclic triaxial test results of the gravel.

In 1989 T. Uwabe, K. Kudo and M. Osada[30] studied the hydrodynamic pressure to act on caissons and rubble mound slope of composite breakwater by a shaking table model test.

In 1990 T. Uwabe and M. Osada[31] investigated characteristics of hydrodynamic pressure acting on a double cylindrical caisson, which is composed of a permeable outer cylinder whose upper part has a number of openings and a impermeable inner cylinder by means of shaking table model test.

4.4 BEARING CAPACITY OF SOIL IN EARTHQUAKES

During an earthquake, foundations must be subjected to eccentric and inclined loads. However, the effect of the horizontal component of the loads upon the bearing capacity has not been taken into consideration in the conventional design of gravity walls.

T. Tateishi[32] computed the bearing capacity of sandy soil for eccentric and inclined loads by using the following assumptions.

(1) The sliding surface has a shape of circular arc.

(2) A sliding surface starts from one of three points i.e., both end points of the base slab and intermediate point of the base slab where the distribution of sand reaction greatly changes.

(3) The bearing capacity is computed by friction circle method.

(4) The smallest of the minimum loads, which are computed for the three possible sliding surfaces, will be the ultimate bearing capacity.

And also he performed the experiments by using an experimental tank (200×60×20 cm) with a glass side wall. The load was instantaneously applied by releasing an oil jack which supports the loading disk carrying a certain amount of weights. By this experiment he found that the ultimate bearing capacity for sandy soil can be computed with sufficient accuracy by circular sliding surface method.

4.5 STABILITY OF SLOPE IN EARTHQUAKES

Y. Ishii, S. Hayashi and H. Arai[33] investigated the behaviours of failure of the sand slope during vibration of different durations and different rates of increase of acceleration. In this study a large vibration table was used.

Generally speaking the similarity is hardly satisfied between prototype of soil structure and small scale models. In order to avoid this complicated problem, H. Tsuchida, S. Noda and E. Kurata[34] carried out a series of the vibration tests on the large scale models of the sea embankments. They constructed a large shaking table which consisted of a reinforced concrete box with 11.5 m long, 6 m wide and 3 m high, and eight vertical steel piles, and was vibrated by 100 tons vibrator in the frequency range of 2 Hz to 11 Hz. The maximum height of the models which were made of sand and silty sand were 3.5 m and the models were sinusoidally vibrated under the maximum table acceleration of 200 gals. From the tests it was made clear that the dynamic response of the models were remarkable and the collapse along the sliding surface might occure in sand model, but any sliding and severe settlement were not observed in silty sand models.

In 1985 T. Uwabe, S. Kitazawa and N. Higaki[35] studied the seismic stability of an embankment resting on saturated sand layers by carrying out shaking table tests using large model. The test results were in good agreement with results calculated by a method of circular arc analysis which takes into account the effects of the seismic forces and the excess pore pressures.

4.6 LATERAL RESISTANCE OF PILES

Extensive model study on lateral resistance of piles was carried out by T. Shinohara and K. Kubo[5]. The main conclusions obtained from their study have already been explained in section 3.4 (1) (i)～(v).

Y. Ishii, S. Hayashi and others[36] carried out a field experiment on the lateral resistance of H piles, in an attempt to investigate an influence of the ground conditions on the charcteristics of lateral resistance of piles. In this experiment, piles were subjected to not only the statical load, but also the alternating and the dynamic load. The result of the experiment under the statical load is summarized as follows.

(1) Relationship between the soil reaction p and the deflexion of pile y is better expressed in the form, $p=k\cdot y^{0.5}$.

(2) An analog computer can be used to solve the equation of equilibrium $EI\dfrac{d^4y}{dx^4}+p=0$, to obtain the quantities for expressing the behaviour of piles, by substituting the expression $p=k\cdot x\cdot y^{0.5}$ or $p=k\cdot y^{0.5}$. Since it is not convenient to use an electronic computer for every case, the standard curves similar to those by Shinohara and Kubo are to be used in practical calculations. The standard curve which satisfies the relationship $p=k\cdot y^{0.5}$, has been prepared by the use of electronic computer.

As a continuation of the above-mentioned field test on H piles, a few more field tests of piles under the alternating and the dynamic loads were carried out.

S. Hayashi and others[37] obtained the following conclusions from the analyses of the test results.

(1) Relationship between the soil reaction p and the pile deflexion y under the alternating load can be estimated from the relationship in the statically loaded piles. In this case, p is not a unique function of y, but the relationship is represented in the types of hysteresis curve.

(2) By substituting the above mentioned relationship between p and y into the equation of equilibrium, $EI\dfrac{d^4y}{dx^4}+p=0$, the relationship between the load F and the displacement at pile top y_{top} is obtained. This relationship between F and y_{top}, in general, is also represented in the type of hysteresis curve.

(3) A hysteretic relation for one degree-of-freedom system is proposed. Steadystate response of the system is obtained analytically together with the equivalent spring constant and damping ratio. Examining the accuracy of the results, it is found that the analytical results can be used for the system with relatively strong non-linearity.

K. Kubo[11] collected the informations of study on lateral resistance of piles, and classified the ground into two types. One is called the S-type ground, in which the relationship $p=k\cdot x\cdot y^{0.5}$ holds, and the other the C-type ground where the relationship $p=x\cdot y^{0.5}$ holds. He proposed a new method of calculating the lateral resistance of piles, which can be applied to both types of ground, and compiled the design graphs.

I. Yamashita and others[38] presented the new standard curves both for S-type and C-type soil. The curves were computed for the new standard pile with round number dimensions by numerically integrating the equilibrium equation of a non-dimensional laterally loaded pile and largely enriched their contents by being added many new cases so as to increase the applicability of the PHRI method to various real life problems.

4.7 LIQUEFACTION OF SUBSOIL

By the Niigata earthquake of June 1964 the extensive damage due to the liquefaction of subsoil was caused in the area of new alluvial sand deposits and the engineer began to consider the liquefaction as one of the important problems in earthquake resistant design. Before the Niigata earthquake Y. Ishii et al had observed liquefaction of saturated sand layers in vibrations in the laboratory experiments with a vibration table. H. Tsuchida extended the experiments with the sand which liquefied in the Niigata earthquake and the sands of different grain size distribution. The results together with the field experiences on the liquefaction in the past earthquakes were interpreted into the critical N-value for liquefaction and the grain size distribution of soils which have danger to liquefy during an earthquake[39].

Y. Umehara and others[40] conducted the undrained cyclic loading triaxial tests on saturated sands for investigating the dynamic properties of sands including liquefaction. They concluded that the susceptibility of liquefaction differs according to types of sand except the influence of factors such as relative density and mean grain size.

Y. Umehara and others[41] performed the dynamic triaxial tests under different conditions of drainage, with open and closed drainage system as well as with retarded drainage. They found that the strength ratio defined by the ratio of liquefaction resistances under a partial drainage condition to that under a perfectly undrained condition can be well represented by the relative density and the non-dimentional parameter defined by $\bar{\alpha}=k/fL$, where k is the coefficient of permeability, L is the drainage distance and f is is frequency. They proposed the prediction method of liquefaction on the basis of the results described above.

S. Iai and others[42] improved the aforementioned procedure for assessing the liquefaction potential of soil proposed by H. Tsuchida in 1970. This is adopted in the current design standard as shown in section 3.6.

Compaction of a loosely deposited sandy ground is one of the remedial measures against liquefaction. S. Iai and others[43] conducted the shaking table tests to partly loose and partly dense saturated sands in a container of 5 m long, 1.5 m high and wide. The followings are major results;

1) When the ground is compacted enough, the ground dose not liquefy due to the liquefaction of uncompacted ground. The standard value of the compaction is about 80% of relative density and 16 of equivalent N-value (corresponding to 0.66 kgf/cm^2 of effective vertical pressure) against usual design seismic conditions.

2) Even if the ground is compacted well, the excess pore water pressure is raised in the adjacent uncompacted ground due to the seepage flow. Therefore, it must be considered that the specific part at the boundary of the compacted ground is unstable part.

3) At the boundary, the hydraulic pressures act. These are the static pressure with 1.0 of coefficient of earthpressure, and the dynamic pressure expected by the Westergaad's formula.

4.8 GRAVITY TYPE QUAYWALLS

Kazama and others[44),45] obtained the fundamental information on the seismic stability of caisson type quaywall by the shaking table tests which were conducted both in ordinary 1G field and in centrifugal field. In 1G field test a 80 cm high caisson model was used, on the other hand in centrifugal field test a 20 cm high caisson model was used at a different centrifugal acceleration 4G, 30G and 40G. Examples are the effects of caisson mass and of backfill ground resonance on dynamic earth pressure, the phase difference between dynamic earth pressure and inertia force of caisson, etc. They focused their attention on the ratio of dynamic earth pressure to inertia force, and of bottom friction force to inertia force, for understanding the dynamic behavior related to caisson dynamic stability. It was found that dynamic earth pressure was not a critical force to make caisson sliding in lower frequency range than the natural frequency of backfill ground.

4.9 SHEETPILE BULKHEADS

Main problems in earthquake resistant design of sheetpile bulkheads consist in the dynamical characteristics of lateral earth pressure in earthquakes. In former days, several extensive studies on a flexible wall had been made, however, they dealt with the problem under the statical conditions exclusively.

Y. Ishii[46], taking these circumstances into consideration, suggested a tentative method of the earthquake resistant design of sheetpile bulkheads. In his paper, after giving a historical review of various design methods and introducing the main findings of contemporary studies by Tschebotarioff, Rowe and others, he pointed out the following fundamental concepts on the behaviour of sheetpile bulkheads.

(1) At the time of embedment failure, the lateral earth pressure distribution approaches

to that assumed in free-earth support method.

(2) In the case of sufficient embedment, the passive pressure distribution differs very much from the triangular distribution and the resultant of earth pressure acts near the sea bottom causing the reduction of the maximum moment. Amount of this reduction depends on the flexibility of sheetpiles and the compressibility of soil.

(3) Embedded length slightly greater than that computed by free-earth support method is sufficient for the stability of the bulkhead.

(4) Tie rods are likely to be subjected to a severe bending due to the settlement of backfill and surcharge on them.

On the basis of the above mentioned concepts as well as experiences on the behaviour of quaywalls in earthquakes, Ishii presented a principle of the earthquake resistant design (cf. section 3.8).

Later on S. Kurata, H. Arai and T. Yokoi[47] performed a series of model experiments on anchored sheetpile walls under vibration. Dry sand was used as a foundation soil and a backfill. A tie-rod was fixed to an end of the steel box which contained whole the model of sheetpile wall. Examination of the similatity showed that the model similarity was satisfied for most part, except for the soil which involved much uncertainty. In the experiment, an acceleration of the vibration table, a bending strain and deflexion of the model sheetpile wall, anchor load and an earth pressure respectively were measured over three stages of vibration, i.e. before, under and after the vibration.

(1) Influence of the dynamic active earth pressure on the maximum bending moment, resultant of the earth pressure and the anchor load respectively is investigated in connection with the current design method. Comparing the results of the measurement with the values calculated by the current Mononobe-Okabe formula, it is found that the current design method gives an adequate value for the resultant of earth pressure, but it overestimates the maximum bending moment and underestimates the anchor load. However, these situations are counterbalanced by the assumptions about an allowable strength of material (cf. section 3.8). In all events, the relevance of emphasizing a reinforcement of the anchor plate and the tie-rod in the current design method is confirmed by this experiment.

(2) In the other series of test, three types of the anchor, namely, no anchor yield, rigid plate type anchor and flexible wall type anchor were used, and the influence of the degree of fixity below the dredge level on the behaviour of the model anchored sheetpile walls were examined. It is seen that the distinct resonances occur in the vibration of models. And as the intensity of driving vibrations increase, both the resonance frequencies and the acceleration response ratios of models go down, The first resonance frequency of the no anchor yield type model is nearly equal to that of the sand layer. Those of the anchor yield type models are lower than that of the no anchor yield type model. The measured values of the tie rod tension of the anchor yield type models are almost equal to that of the no anchor yield type model. The displacements of anchor caused by the vibrations are much greater than that to the same load in the static horizontal load test.

This fact represents that the displacement of anchor is affected by the reduction of the soil stiffness due to the vibration. And the bending moment distribution of flexible wall type anchor caused by the vibrations represents the decreasing tendency of the coefficient of subgrade reaction.

The above-mentioned experiment has some limitations as the study of sheetpile walls. Namely it discarded the influence of water which was inevitable in the waterfront structure. The experiment is to be continued and it is hoped that the further findings are helpful in clarifying the dynamic behaviours of sheetpile walls.

In order to simplify the design for anchored sheet pile wall, S. Kitajima and others[48] proposed new calculation method.

Assuming that clayey foundations around the embedment are elastic supports and the their coefficients of lateral subgrade reaction are constant, simple equations for computing the stress is derived. Comparing calculated bending moment diagrams with directly measured ones at construction sites, S. Kitajima and others find fairly good accordance between them.

4.10 CELLULAR BULKHEADS

The cellular bulkhead type quaywalls have been built in our country, and no failure by earthquake has been reported yet.

Establishment of rational method of earthquake resistant design for cellular bulkheads is important. However, some of behaviours of cellular bulkhead are not clear even in the static condition, for example, bursting or shear deformation of a cell due to the lateral load.

S. Kitajima[13] carried out a series of model experiments on the circular cell and bouble wall under the lateral load. From the result of the experiment together with a theoretical analysis of the shearing resistance of fill material, he proposed a new method of designing cellular bulkheads on rock foundation (cf. section 3.9).

Making use of the theory of mechanics of granular materials, shape of the boundary surfaces of slip zones and stresses on the surfaces were obtained. The results of theoretical analysis compared favourably with those observed in the model experiment, in which a process of shear failure in a fill material and the corresponding lateral loads were recorded.

The fill pressure against the fore wall was also measured in the experiment. And it was found that the fill pressure was maximum at the end of filling, decreased as the model deformed, and finally the minimum value was reached at the instant of shear failure. According to the results of measurement, the coefficient of earth pressure at the end of filling is nearly equal to $\tan\phi$ and little larger than $\frac{1}{2}\tan\phi$ at the instant of shear failure.

S. Noda and others[49] carried out the shaking table tests in order to clarify the earthquake resistance of steel plate cellular bulkheads with embedment. Major results are as follows,

(1) Under the strong ground motion, the cells behave like rigid body, and their predominant modes are rocking. The installation of embedment significantly affects on the stability of cellular bulkheads.

(2) When the ground acceleration increases, the mass of cell fill which contributes to

the inertia force decreases apparently. The experimental equations on the least upper bound of the effective mass of cell fill are obtained;

$$\xi=1.0-k_H \qquad k_H\leqq 0.2$$
$$\xi=0.8 \qquad k_H>0.2$$

where, ξ is effective mass coefficient as the ratio of effective mass to real mass, k_H is seismic coefficient, respectively.

(3) The analytical method in which the effective mass coefficient is considered and the ground is replaced into springs gives good agreement with the experimental results.

Measurements of the dynamic behavior of the actual steel plate cellular bulkhead with 19.5 m in diameter and 20 m in height under earthquake conditions have been performed[50]. 52 earthquakes were observed, and the validity of the analytical method mentioned-above was verified.

S. Noda and others[51] have carried out the shaking table tests to make clear the difference of dynamic behaviors between sheet pile cells and steel plate cells. The tests were mainly intended to evaluate the extent of influence by the joints. Major results are as follows,

(1) Under the strong ground motions, the existence of joints gives no significant difference on the deformation of cells.

(2) Although larger strains possibly occurred in fills of sheet pile cells, the failure due to shear did not happen in any models.

(3) Judging from the model tests, a current design method for sheet pile cellular bulkheads based on Kitajima's study gives rather conservative results, and this tendency increases with increase of embedment.

4.11 TRESTLE TYPE PIERS

Generally speaking, the trestle type pier is considered to be a relatively simple structure from the dynamical point of view. In order to obtain the fundamental data for the dynamical method of earthquake resistant design, a number of vibration tests of prototype trestle type piers have been performed.

S. Hayashi and N. Miyajima[52] carried out a vibration test in the No. 7 Pier in Kobe Port. The pier is a trestle type pier with caissons. Items of the test were; (i) free vibration by the shock load which was applied by cutting a pulling rope, (ii) forced vibration by means of the vibrator with rotating eccentric masses, (iii) forced vibration induced by the explosion at the sea bottom at a distance of 400~1,600 m from the pier and (iv) the vibration by actual earthquakes.

In the analysis of the test results, a simplified model was assumed to simulate the prototype. The assumed model was a system of particles corresponding to the system of shear vibration of continuous body. Then the distribution of acceleration and the response of the pier to an earthquake motion of the ground were computed.

R. Yamamoto and others[53] proposed very rationalized method, which is based upon experimental study with proto structure, for evaluation of seismic stability of trestle type pier. The characteristics of a pier both in its elastic and plastic ranges were investigated experimentally and the results are summarized into following two items: 1. dynamic properties of a pier can be described by that of a one degree-of-freedom system, 2. restoring force can be approximated by the family of smooth hysteretic curves proposed by P. C. Jennings. Then the earthquake responses of the above-mentioned model are computed for three earthquakes: El Centro (1940), Shimizu (1965) and Kushiro (1965). Final results are arranged into one chart by which the earthquake response of a pier can be easily evaluated. Finally, the ductility factor of a pier is suggested as a safety factor in the proposed method. The new method is actually applied to nine working piers, and the new method is compared with the seismic coefficient method.

The open type steel piled wharves with large container cranes have recently been constructed at many ports in the country, However, there is a small knowledge on their vibration characteristics upon which their earthquake proof design should be stand. Therefore, T. Inatomi, S. Hayashi and I. Yamashita[54] have carried out a series of the vibration tests of the prototype wharf. In the tests the natural frequencies, and the damping factors of the wharf and the container crane were observed.

S. Ueda and S. Shiraishi[55] obtained the earthquake response records of a coupled pile offshore platform in Kashima seaberth. They compared the results of observation with the results of computation by use of one degree of freedom model and the multi degree of freedom model. They made clear the values of added mass coefficient of pile in water and damping factor of the platform, and earthquake response of this type of structure is exactly computed by use of the multi degree of freedom models.

4.12 OBSERVATION OF STRONG MOTION EARTHQUAKES

The strong motion earthquake observation in port areas in Japan was started in 1962 by the Port and Harbour Research Institute in cooperation with the several organizations related to the harbour construction work[56]. At the present the network consists of 81 strong motion accelerograph stations located at 54 ports. The 61 accelerographs were on the ground, the 6 accelerographs were in the ground and the rest on the structures. Two kinds of accelerograph are being used in the network; the SMAC-B2 accelerograph and the ERS accelerograph which was developed by S. Hayashi et al.[57] By the end of 1991 about 4216 accelerograms were obtained in the network and collected at the Geotechnical Earthquake Engineering Laboratory. They were published as the annual reports every year[58], in which the digitized records and their computer plots, the response spectra and the Fourier spectra of accelerograms with large acceleration amplitude were included. Vertical ground accelerations of 28 records, their response spectra, and their Fourier spectra were presented coresponding to the data of horizontal ground accelerations having been published since 1963[59]. The records of the 1968 Tokachi-oki earthquake[60], the 1978 Izu-Oshima-Kinkai earthquake[61], the 1978 Miyagi-ken-oki earthquake[62], the 1982 Urakawa-oki earthquake[63] and the 1983 Nipponkai-chubu earthquake[64] were published separately, but in the same format[58]. The site conditions of the strong motion earthquake stations were published in details as the separate reports[65].

In 1972 H. Tsuchida and T. Uwabe[66] studied the characteristics of the base-rock motions during earthquakes based on 37 accelerograms at 10 observation stations. Incident waves traveling through the base rock were calculated from the accelerograms by the multiple reflection theory and the undamped acceleration ratio response spectra of them were obtained. The spectra of the incident waves were compared with those of the ground motions recorded at outcrop of rock.

S. Iai and others[67] proposed the procedures for base line corrections, instrument corrections and integration of acceleration time histories. These procedures were applied to the data processing for annual reports on strong-motion earthquake records in Japanese ports since 1976.

PHRI has also been operating the following three types of dense instrument array systems consisting of several accelerometers installed on and in grounds[68]. Especially in 1988 the large scale system has newly been established along the No. A runway at the extended area of Haneda airport. The array consists of 16 sets of accelerometers on the ground along the runway of 2500 m, and 24 sets of accelerometers and 24 sets of pore water pressure gauges in the ground.

5. CONCLUSION

Very intensive efforts have been made in the earthquake engineering research in connection with the harbour structures and also interpreting the research results into the appropriate form for design practices. The advance in those areas in the last decade was really considerable. However, the harbour structures are continuously increasing their importances in the society because of the concentration of population in the cities and also the concentration of industries in the coastal areas. In some places, the harbour structures are protecting cities located in low lands from flooding of sea water. Immediately after an intensive earthquake the harbour structures must function properly to transport materials in an emergency. Under the circumstances more and more reliability on the structures is demanded and continuous efforts in those fields must be made in the future.

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