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港湾の技術基準とEurocodesの比較研究

森屋陽一・松本英雄・田邊俊郎・山本修司

国土交通省 国土技術政策総合研究所

National Institute for Land and Infrastructure Management
Ministry of Land, Infrastructure and Transport, Japan

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Yoichi MORIYA*
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Synopsis

Current Technical Standards for Port Structures in JAPAN are based on either the allowable stress design method or the safety factor method. On the other hand, International (ISO) standards and Eurocodes are based on the partial factor method in the limit states. Then the Vienna agreement provides that it simultaneously becomes a draft ISO standard.

WTO/TBT Agreements have required to make each technical standards consist with ISO standards. In Japan, relevant organizations have been working on revising technical standards, including those for port structures.

From the above-mentioned background, we have to perform the study for revising current technical standards in JAPAN and prepare for the correspondence to ISO standards. Therefore, to clarify the difference between the design method based on the technical standards in JAPAN and one based on Eurocodes, we carried out comparative designs of example structures to study the following design issues: (1) slope stability, (2) bearing capacity of the pile foundation, (3) bearing capacity of the spread foundation, (4) sliding of the gravity quaywall, (5) stability of the sheet pile quaywall, and (6) estimation of the design seismic coefficient. We studied items (1) to (3) under normal conditions and (4) and (5) under earthquake conditions.

The quantitive difference of the degree of safety and the designed structural size between the design method based on the technical standards in JAPAN and one based on Eurocodes was clarified. The some knowledge for revising current technical standards in JAPAN and for the correspondence to ISO standards were obtained.

Key Words: technical standard, Eurocodes, comparative design, internationalization

* Researcher of Port Facilities Division, Port and Harbor Department
** Senior Researcher of Port and Harbor Department
*** Head of Port Facilities Division, Port and Harbor Department
**** Director of Port and Harbor Department
3-1-1 Nagase, Yokosuka, 239-0826 Japan
National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure and Transport
Phone : +81-468-445029 Fax : +81-468-445029 e-mail:moriya-y92y2@ysk.nilim.go.jp

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港湾の技術基準とEurocodesの比較研究

森屋　陽一*・松本　英雄**・田邊　俊郎***・山本　修司****

要　旨

現行の我が国の港湾の技術基準は，許容応力度法または安全率による設計法を基本としている．一方，国際規格における構造物の設計法は，我が国の技術基準とは異なり，部分係数法を用いた限界状態設計法を基本としている．

WTO/TBT協定により，各国の技術基準はISOと矛盾していないことが求められており，現在，日本では各種基準とISOとの整合性を考慮した技術基準の改定作業が始まっている．一方，ヨーロッパでは，構造物を設計するための統一規格として，Eurocodesを策定中である．Eurocodesはウィーン協定によりISO規格原案となることが決まっている．

上記の背景から，本研究では，ISOへの対応および港湾の技術基準の改定に資することを目的として，港湾の技術基準とEurocodesに従った設計による安全性や設計断面に及ぼす違いを定量的に明らかにした．検討項目としては，①斜面の安定，②杭基礎の支持力，③直接基礎の支持力，④重力式岸壁の滑動，⑤矢板式岸壁の安全性，⑥設計震度の算定法の6項目である．

上記6項目について，港湾の技術基準とEurocodesに従った設計による安全性や設計断面に及ぼす違いを定量的に明らかにし，ISOへの対応および港湾の技術基準の改定に資するに値する基礎的知見を得た．

キーワード：技術基準，Eurocodes，比較設計，国際化

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* 港湾研究部港湾施設研究室
** 港湾研究部主任研究官
*** 港湾研究部港湾施設研究室長
**** 港湾研究部長
〒239-0826　横須賀市長瀬3-1-1　国土交通省国土技術政策総合研究所
電話：0468-44-5029　Fax：0468-44-5029　e-mail:moriya-y92y2@ysk.nilim.go.jp

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1. Introduction

CEN (Committee for European Standardization) is preparing Eurocodes as unified standards for designing structures. These Eurocodes are now in the prEN stage of final draft for voting by the EU countries. A prEN approved by vote becomes a EN that is a local standard in each EU country. Then the Vienna Agreement provides that it simultaneously becomes a draft ISO standard.

As ISO 2394 exemplifies, ISO's structural design method aims at "performance-based design based on the reliability of structures". The design philosophy of Eurocodes is generally in accordance with that of ISO.

WTO/TBT Agreements have demanded to make each technical standards consist with ISO standards. In Japan, relevant organizations have been working on revising technical standards, including those for port structures, to make them consistent with ISO and other international standards.

We have to perform the study for revising current technical standards in JAPAN and prepare for the correspondence to ISO standards. Therefore, to clarify the difference between the design method based on the technical standards in JAPAN and one based on Eurocodes, we developed comparative designs of example structures.

[Difference between Current Technical Standards for Port Structures in JAPAN and Eurocodes]

Current Technical Standards for Port Structures in JAPAN (OCDI, 2001, "Technical Standards and Commentaries for Port and Harbour Facilities in Japan " (in press), hereinafter abbreviated T.S.P.H.) are based on either the allowable stress design method or the safety factor method. These design methods have track records of actual use for more than one hundred years and are used as technical standards in every country in the world. On the other hand, Eurocodes and ISO standards are based on the partial factor method in the limit states.

The Annex to Eurocode 7, part 1, specifies the partial load factors and partial material factors in standard use. Since they are closely related to the calculation formulas (e.g., formulas for bearing capacity and earth pressure) and to the characteristics of the target ground, the application of the partial factors proposed in the Eurocodes to locations in Japan may cause problems.

To clarify the difference between the design method based on the T.S.P.H. and one based on Eurocodes, we carried out comparative designs of example structures to study the following design issues: (1) slope stability, (2) bearing capacity of the pile foundation, (3) bearing capacity of the spread foundation, (4) sliding of the gravity quaywall, (5) stability of the sheet pile quaywall, and (6) estimation of the design seismic coefficient. We studied items (1) to (3) under normal conditions and (4) and (5) under earthquake conditions.

We referred to the following Eurocodes: Eurocode 1, part 1 (now prEN 1990), which describes the basic design principle; Eurocode 3, part5 which describes the design of steel structures, especially about pile. Eurocode 7, part 1 which describes the geotechnical design; and Eurocode 8, part 1 and 5, which describe the design of structures for earthquake resistance. Eurocode 8, part 5 describes the seismic design of the foundations, retaining structures, and geotechnical aspects.

Eurocode 7 (ENV) specifies that the following cases should be studied: Case A, which addresses the overall safety system such as floating of the structure; Case B, which describes uncertainly of the actions; and Case C, which deals with uncertainty of the material characteristics. More specifically, Case A is important when water pressure is the major load, Case B is important for structural design of foundations and retaining walls, and Case C is important in determining the size of elements in the ground, such as the size of the foundation and the embedded depth of the retaining wall. For problems in which the strengths of the soil is significant in providing resistance, such as slope stability, only Case C is important.

Table 1.1 shows the values of the partial factors listed for each case in the explanatory book of Eurocode 7 (ENV) (Orr and Farrell, 1999). The table also shows Cases C2 and C3 as substitutes for Cases B and C, respectively. The partial factors used in the resistance factor method and the resistance model factors are introduced in C2 and C3, respectively. Since our study aimed to clarify the difference between a design based on the T.S.P.H. and one based on the Eurocodes (partial factor method), and because our examples did not include problems associated with the overall safety system such as floating of the structure, we only focused on Cases B and C.

In the latest version of Eurocode 7, part 1 (prEN 1997-1), the definition of the ultimate limit state and the design approach are slightly modified, and the values of partial factors that are different from those in **Table 1.1** are listed.

[Assumption for Comparative Design]

(1) We referred to actual design cases in Japan and the examples shown in the explanatory book of Eurocode 7 (ENV) (Orr and Farrell, 1999) to define our examples. We also referred to Orr and Farrell (1999) to determine the calculation methods used for the Eurocode-based design.

(2) In **Table 1.1**, the material factor for the unit weight of soil is 1.0 in all cases. The weight of the soil, however, acts as the load while it contributes to the

Table 1.1 Partial Factor in Ultimate Limit State (Orr and Farrell, 1999)

Parameter	Factor	Case A	Case B	Case C	Case C2	Case C3
Partial Load Factors (γ_F)						
Permanent Unfavorable Action	γ_G	**1.00**	**1.35**	**1.00**	*1.35*	*1.00*
Variable Unfavorable Action	γ_Q	**1.50**	**1.50**	**1.30**	*1.50*	*1.20*
Permanent Favorable Action	γ_G	**0.95** *(0.90)*	**1.00**	**1.00**	*1.00*	*1.00*
Variable Favorable Action	γ_Q	**0**	**0**	**0**	*0*	*0*
Accidental Action	γ_A	**1.00**	**1.00**	**1.00**	*1.00*	*1.00*
Partial Material Factors (γ_m)						
$\tan\phi'$	$\gamma_{\tan\phi'}$	**1.10**	**1.00**	**1.25**	*1.00*	*1.20*
Effective Cohesion c'	$\gamma_{c'}$	**1.30**	**1.00**	**1.60** *(1.25)*	*1.00*	*1.20*
Undrained Shear Strength c_u	γ_{cu}	**1.20**	**1.00**	**1.40**	*1.00*	*1.40*
Compressive Strength q_u	γ_{qu}	**1.20**	**1.00**	**1.40**	*1.00*	*1.40*
Pressuremeter Limit Pressure p_{lim}	γ_{plim}	*1.40*	*1.00*	*1.40*	*1.00*	*1.40*
CPT Resistance	γ_{CPT}	*1.40*	*1.00*	*1.40*	*1.00*	*1.40*
Unit Weight of Ground γ	γ_g	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
Partial Resistance Factors (γ_R)						
Bearing Resistance	γ_{Rv}	-*	**1.00**	**1.00**	*1.40*	*1.00*
Slide Resistance	γ_{Rs}	-*	**1.00**	**1.00**	*1.10*	*1.00*
Earth Resistance	γ_{Re}	-*	**1.00**	**1.00**	*1.40*	*1.00*
Pile Base Resistance	γ_b	-*	**1.00**	**1.30**	*1.30*	*1.00*
Pile Shaft Resistance	γ_s	-*	**1.00**	**1.30**	*1.30*	*1.00*
Total Pile Resistance	γ_t	-*	**1.00**	**1.30**	*1.30*	*1.00*
Pile Pull-out Resistance	γ_{st}	*1.40*	**1.00**	**1.60**	*1.40*	*1.00*
Anchor Pull-out Resistance	γ_A	*1.30*	**1.00**	**1.50**	*1.20*	*1.00*
Partial Aaction Effect and Resistance Model Factors (γE, γsd, γrd)						
Action Effects and Resistances	γ_{sd}, γ_{rd}	*1.00*	*1.00*	*1.00*	*1.00*	*1.40*

Values in **bold** are partial factors either given or implied in the ENV version of Eurocode 7.
Values in *italic* are proposed partial factors not in the ENV version that may be in the EN version.
* Partial factors that are not relevant for Case A.

resistance. In calculating the earth pressure in Case B, we therefore multiplied the unit weight of the soil by the load factor. This corresponds to assuming that the load contributing to the failure is the difference between the load and the resistance, which is then multiplied by the load factor.

(3) Variables currently used to design port structures based on the T.S.P.H. are not only determined by the results of a soil survey and soil tests, but the designer's judgment. As a result, the probability characteristics of the variables are unclear. It is also unclear whether the values used for design are average values, characteristic values, or design values of the variables. We hence assumed that variables, such as the geotechnical parameters, that are used for design based on the T.S.P.H. are characteristic values and compared them to those based on the Eurocodes.

(4) The Eurocodes specify that the drained condition (permeable ground) and the undrained condition (impermeable ground) should be treated separately. The designer should first judge whether the drained or the undrained condition should be used. In our study, we applied the drained condition to sandy ground, and the undrained condition to cohesive ground.

(5) The Eurocodes specify two limit states: the ultimate limit states and the serviceability limit states. We focused on the ultimate limit state in this study.

(6) We defined the degree of safety as the ratio of the action to the resistance and distinguished it from the safety factor specified in the T.S.P.H..

2. Slope Stability

(1) Calculation Method

The T.S.P.H. uses the modified Fellenius method to evaluate the slope stability. Eurocode 7, part 1, however, does not specify the calculation method for slope stability. Orr and Farrell (1999) mentioned that they used a program named SLOPE. Since we do not know the calculation method of SLOPE, we used the modified Fellenius method for the Eurocode-based design.

The modified Fellenius method is given by the following expression:

$$F_s = \frac{R\sum\left(c_u l + W'\cos\alpha\tan\phi\right)}{\sum Wx + \sum Ha} = \frac{\sum\left(c_u b + W'\cos^2\alpha\tan\phi\right)\sec\alpha}{\sum W\sin\alpha + \frac{1}{R}\sum Ha} \tag{2.1}$$

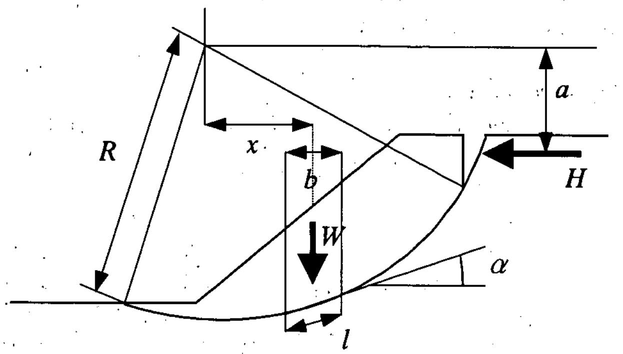

Figure 2.1 Circular Slip Analysis by Modified Fellenius Method

where

F_s : Degree of safety against a circular slip failure
R : Radius of the slip circle (m)
c_u : Undrained shear strength of the cohesive soil ; apparent cohesion of the sandy soil in the undrained condition (kN/m^2)
l : Base length of a slice (m)
W' : Effective weight of a slice per unit length (kN/m)
α : Angle of the base of a slice with respect to the horizontal plane (°)
ϕ : 0 for cohesive soil ; internal friction angle under drained conditions for sandy soil (°)

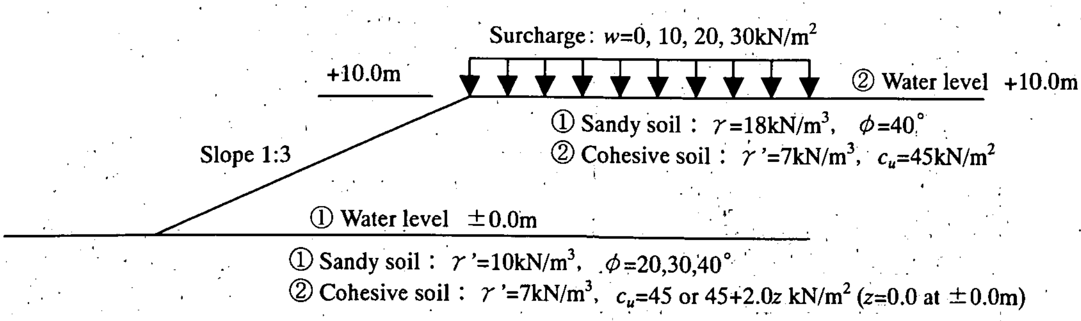

Figure 2.2 Shape of the Slope and Ground Conditions

W : Total weight per unit length of a slice (kN/m)
x : Horizontal distance between the center of gravity of a slice and the center of the slip circle (m)
H : Horizontal external force acting on the soil mass in the slip circle (kN/m)
a : Arm length of the action point of the horizontal external force, H, from the center of the slip circle (m)
b : Width of a slice (m)

(2) Objective

The objective is to compare the degree of safety of slope stability F_s (the ratio of resisting moment to the driving moment) and to clarify the relationship between the safety factor used in the T.S.P.H. and the partial factors proposed in Eurocode 7.

(3) Assumptions

The study was made for ① sandy and ② cohesive soils with the slope and ground conditions shown in **Figure 2.2.** The safety factor specified in the T.S.P.H. is equal to or greater than 1.1 ～ 1.2 for sandy soil and is equal to or greater than 1.3 for cohesive soil.

Note:
① The water level of the sandy soil was assumed to be at the existing ground level surface.
② The water level of the cohesive soil was assumed to be at the surface of the fill.

(4) Results

Tables 2.1 and **2.2** show the results of the study on sandy and cohesive soils, respectively.

The Eurocode-based calculation shows that, for both the cohesive and sandy soils, the degree of safety in Case C is less than that in Case B. Thus, the slope stability is determined by Case C.

Table 2.1 Calculation Results for Sandy Soil

ϕ (°)	Surcharge (kN/m^2)	Degree of Safety① T.S.P.H.	Degree of Safety② CaseB	Degree of Safety② CaseC	①/② CaseB	①/② CaseC
20.0	0.0	1.278	1.278	1.022	1.000	1.250
	10.0	1.271	1.270	1.013	1.001	1.255
	20.0	1.259	1.256	1.001	1.002	1.258
	30.0	1.242	1.236	0.981	1.005	1.266
30.0	0.0	1.882	1.882	1.506	1.000	1.250
	10.0	1.872	1.871	1.494	1.001	1.253
	20.0	1.846	1.840	1.463	1.003	1.262
	30.0	1.818	1.809	1.435	1.005	1.267
40.0	0.0	2.518	2.518	2.014	1.000	1.250
	10.0	2.501	2.499	1.995	1.001	1.254
	20.0	2.470	2.462	1.958	1.003	1.261
	30.0	2.434	2.422	1.922	1.005	1.266

Table 2.2 Calculation Results for Cohesive Soil

Cohesion (kN/m^2)	Surcharge (kN/m^2)	Degree of Safety① T.S.P.H.	Degree of Safety② CaseB	Degree of Safety② CaseC	①/② CaseB	①/② CaseC
C_u=45.0	0.0	1.466	1.086	1.047	1.350	1.400
	10.0	1.383	1.018	0.971	1.359	1.424
	20.0	1.309	0.958	0.906	1.366	1.445
	30.0	1.242	0.905	0.848	1.372	1.465
C_u=45.0+2.0z z=0 at ±0.0m	0.0	2.040	1.511	1.453	1.350	1.404
	10.0	1.920	1.413	1.344	1.359	1.429
	20.0	1.813	1.327	1.250	1.366	1.450
	30.0	1.718	1.251	1.168	1.373	1.471

In the case of sandy soil with a no surcharge condition, the ratio of the degrees of safety for the design based on the T.S.P.H. to that based on the Eurocodes is 1.00 in Case B and 1.25 in Case C. The reason why the ratio of the degree of safety is 1.00 in Case B is that the dead weight of the soil (W and W') affects the calculation of both the resisting moment in the numerator and the driving moment in the denominator of Equation (2.1), which cancels the effect of the load factors. The ratio of the degree of safety is 1.25 in Case C because 1.25 is the material factor corresponding to $\tan\phi$ in that case. The largest value of ①/② in Case C shown in **Table 2.1** is 1.267 under surcharged conditions, demonstrating that the effect of the surcharge is very small.In the case of sandy soil, the safety factor specified in the T.S.P.H. is at least 1.1 ～ 1.2. The Eurocode-based design thus gives somewhat safe-side results regardless of the presence or absence of the surcharge.

In the case of cohesive soil under the no surcharge condition, the ratio of the degree of safety of the design based on the T.S.P.H. to that based on the Eurocodes was 1.35 in Case B, and 1.40 in Case C. This is because 1.35 is the load factor in Case B, and 1.4 is the material factor related to cohesion in Case C. The largest value of ①/② in Case C shown in **Table 2.2** is 1.471 under surcharged conditions, showing that the effect of the surcharge is very small.

In the case of cohesive soil, the safety factor specified in the T.S.P.H. is at least 1.3. The Eurocode-based design thus gives somewhat safe-side results regardless of the presence or absence of the surcharge.

3. Bearing Capacity of Pile Foundation

(1) Calculation Method

a) Calculation Method for Bearing Capacity of Pile Foundation Based on T.S.P.H.

① Sandy-Soil Ground (the Case of Pile Driving by Hammer)

・Ultimate Bearing Capacity

$$R_u = 300NA_p + 2\overline{N}A_s \tag{3.1}$$

・Maximum Pulling Resistance

$$R_u = 2\overline{N}A_s \tag{3.2}$$

where

R_u: Ultimate bearing capacity of the pile (kN)
A_p: Area of the pile base (m^2)
A_s: Total circumferential surface area of the pile (m^2)
N: N value of the subsoil at the pile base
$\overline{N}$: Mean N value for the total embedded length of the pile

N is calculated from the following expression:

N: $(N_1+N_2)/2$
N_1: N value at the pile base
N_2: Mean N value in the range from the pile base to the point of $4B$ (B is the diameter or the width of pile) above

② Cohesive-Soil Ground (the Case of Pile Driving by Hammer)

: Ultimate Bearing Capacity

$$R_u = 8c_pA_p + c_aA_s \tag{3.3}$$

・Maximum Pulling Resistance

$$R_u = c_aA_s \tag{3.4}$$

where

R_u: Ultimate bearing capacity of a pile (kN)
c_p: Cohesion at the pile base (kN/m^2)
c_a: Average adhesion for the total embedded length (kN/m^2)
for $c \leqq 100$kN/m^2, $c_a = c$
for $c > 100$kN/m^2, $c_a = 100$kN/m^2
c: Average cohesion for the total embedded length (kN/m^2)

b) Design Method of Pile Foundation Adopted by Orr and Farrell (1999) Based on Eurocode 7

① Study of Push-In Case

・Calculation of Design Load

$$F_c = \gamma_G G_k + \gamma_Q Q_k \tag{3.5}$$

where

F_c: Design load (kN)
γ_G: Partial factor for the permanent load (Case B: 1.35, Case C: 1.0)
γ_Q: Partial factor for the variable load (Case B: 1.5, Case C: 1.3)
G_k: Characteristic value of the permanent load (kN)
Q_k: Characteristic value of the variable load (kN)

・Calculation of Bearing Capacity of Pile Per Unit Area and Characteristic Value of Shaft Resistance

(Sandy soil)

$$q_{bk} = q' N_q / \xi \tag{3.6}$$

$$q_{sk} = K_S \sigma'_{V0} \tan\delta / \xi \tag{3.7}$$

(Cohesive soil)

$$q_{bk} = (9c_u + \sigma_V) / \xi \tag{3.8}$$

$$q_{sk} = \alpha c_a / \xi \tag{3.9}$$

where

q_{bk}: Characteristic value of the bearing capacity of the pile base (kN/m^2)
q': Effective overburden pressure (kN/m^2)
N_q: Bearing capacity factor of the soil at the pile base. We determined the value after consulting "Pile

Design and Construction Practice."

ξ: Correlation factor of the bearing capacity (This depends on the number of load carrying tests of the pile. Here the number of the test was 1, and the value was 1.5.)
q_{sk}: Characteristic value of the shaft resistance (kN/m^2)
K_s: Lateral earth pressure coefficient on the pile shaft (see below)
δ : Angle of shearing resistance between the soil and the pile shaft (°) (the material coefficient is not considered, see below)
σ_{V0} ': Effective surcharge pressure (kN/m^2)
c_u: Cohesion of the ground at the pile base (kN/m^2) (the material factor is not considered)
σ_V: Total vertical stress at the pile base (kN/m^2)
α: Adhesion factor (see below)
c_a: Average adhesion for the total embedment length (kN/m^2) (the material factor is not considered)

・Calculation of Bearing Capacity

$$R_{bk} = A_b q_{bk} \tag{3.10}$$

$$R_{sk} = \sum A_s q_{sk} \tag{3.11}$$

$$R_c = R_{bk} / \gamma_b + R_{sk} / \gamma_s \tag{3.12}$$

where
R_{bk}: Bearing capacity of the pile base (kN)
A_b: Surface area of the pile base (m^2)
R_{sk}: Shaft resistance (kN)
A_s: Circumferential surface area of the pile (m^2)
R_c: Bearing capacity of the pile in the ultimate limit state (kN)
γ_b: Partial factor of the pile base resistance (Case B: 1.0, Case C: 1.3)
γ_s: Partial factor of the pile shaft resistance (Case B: 1.0, Case C: 1.3)

The structure is therefore safe in the ultimate limit state if $R_c > F_c$.

② Study of Pull-Out Case

・Calculation of Design Load

$$F_t = \gamma_G \cdot G_k \tag{3.13}$$

where
F_t: Design pull-out load (kN)
γ_G: Partial factor for the pull-out load
G_k: Characteristic value of the pull-out load (kN)

・Calculation of Characteristic Value of Bearing Capacity
(Sandy soil)

$$q_{tk} = K_S \sigma'_{V0} \tan\delta / \xi \tag{3.14}$$

(Cohesive soil)

$$q_{tk} = \alpha c_u / \xi \tag{3.15}$$

where
q_{tk}: Characteristic value of the pull-out resistance (kN/m^2)

・Calculation of Bearing Capacity

$$R_{tk} = \sum A_s \cdot q_{tk} \tag{3.16}$$

$$R_t = R_{tk} / \gamma_{st} \tag{3.17}$$

where
R_{tk}: Characteristic value of the pull-out resistance force (kN)
R_t: Pull-out resistance force of the pile in the ultimate limit state (kN)
γ_{st}: Pull-out resistance factor of the pile (Case B: 1.0, Case C: 1.6)

The structure is therefore safe in the ultimate limit state if $R_t > F_t$.

Orr and Farrell (1999) did not use the geotechnical parameter rebated by the material factor but used the non-rebated geotechnical parameter to estimate the characteristic value of the bearing capacity. They rebated the characteristic value of the bearing capacity by the resistance factor to estimate the bearing capacity.

(2) Objective

The objective is to calculate the bearing capacity of the pile foundation, and, for the push-in and pull-out cases, to compare the necessary embedment length and quantitatively estimate the difference between the calculation formula for bearing capacity used in the T.S.P.H. and that adopted by Orr and Farrell (1999). We also clarify the relationship between the safety factor adopted in the T.S.P.H. and the partial factor proposed in the Eurocodes.

(3) Assumptions

We studied sandy and cohesive soils under the ground conditions shown in **Figure 3.1**.

We applied the following load for both the sandy soil and cohesive soil cases:

Push-in force (Vertical load) : 2,000kN (Permanent load of 1,000kN + Variable load of 1,000kN)
Pull-out force (Vertical load) : 1,000kN (Permanent load of 1,00kN)

The pile setting was as follows:
① The diameter of the concrete pile: 4 different diameters of 800, 1,000, 1,200, and 1,500mm
② The blockage ratio of the pile base: 100%
③ The unit weight (in air) of the pile: 24.0kN/m^3

The safety factor specified in the T.S.P.H. to determine the embedment length of the pile is equal to or greater than 2.5 and 3.0 for the push-in and pull-out forces, respectively.

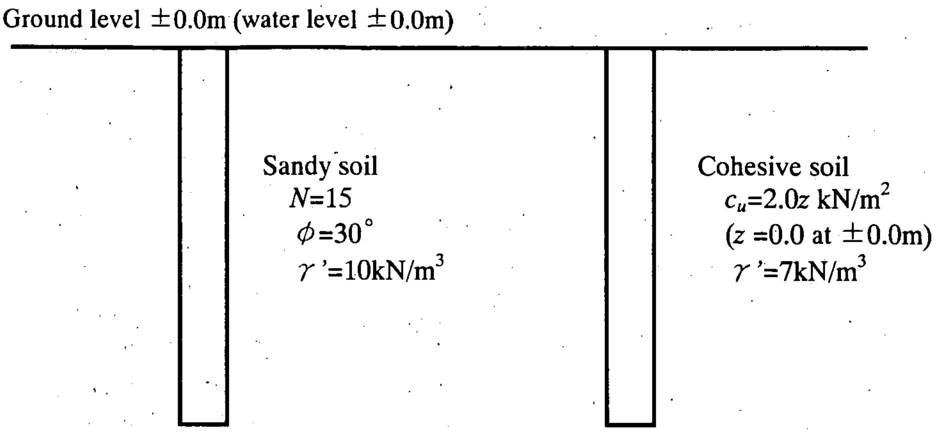

Figure 3.1 Overview of Pile Foundation and Study Conditions

Referring to the "Design of Pile Foundations" published by ASCE, we used the following values for the lateral earth pressure coefficient K_s and the angle of shearing resistance δ for concrete piles in sandy soil: $K_s = 1.5$, $\delta = 0.9\phi$. We set the adhesion factor α to 0.8 consulting Tomlinson's "Pile Design and Construction Practice."

(4) Results

Figures 3.2 to **3.5** show the results of the study on sandy and cohesive soils.

In the Eurocode-based design, the necessary embedment length is determined by Case C except for the 1.0, 1.2, and 1.5m diameter piles in cohesive soil. The difference in the embedment length between Cases B and C is very small.

The necessary embedment length against the push-in force for sandy soil calculated from the T.S.P.H. is shorter for larger pile diameters and longer for smaller pile diameters than that calculated from the Eurocodes. This comes from differences in the formulas for the bearing capacity. The bearing capacity formula used in the T.S.P.H. [Expression (2.1)] calculates both the bearing capacity of the pile base ($300NA_p$) and the shaft resistance ($2\overline{N}A_s$) from the N value. If the N value is constant, the bearing capacity of the pile base becomes constant regardless of the embedment length, and the shaft resistance is proportional to the total surface area A_s of the pile; i.e. it is proportional to the embedment length. In the Eurocode-based design, the bearing capacity formula considers the overburden pressure in the calculation of the bearing capacity of the pile base and the shaft resistance. If the geotechnical parameters are constant, both the bearing capacity of the pile base and the shaft resistance are proportional to the embedment length.

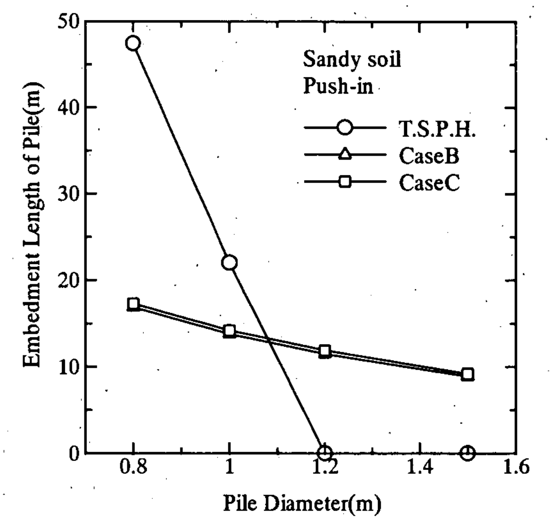

Figure 3.2 Necessary Embedment Length of Pile in Sandy Soil against Push-In Force

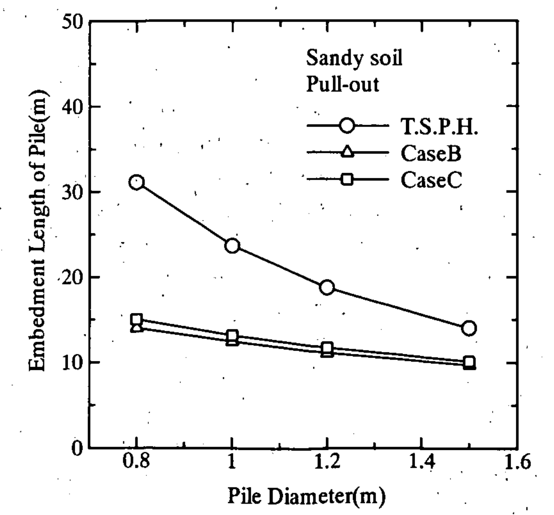

Figure 3.3 Necessary Embedment Length of Pile in Sandy Soil against Pull-Out Force

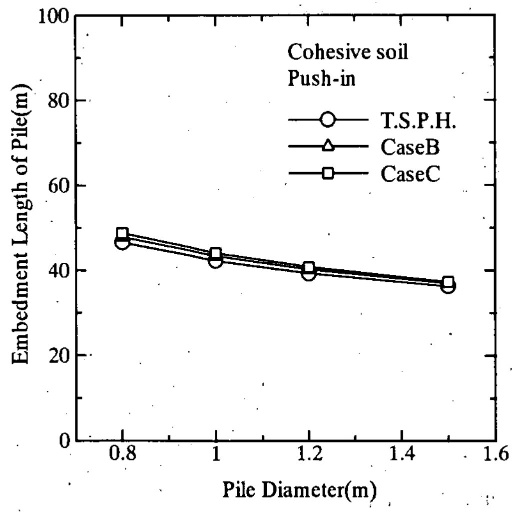

Figure 3.4 Necessary Embedment Length of Pile in Cohesive Soil against Push-in Force

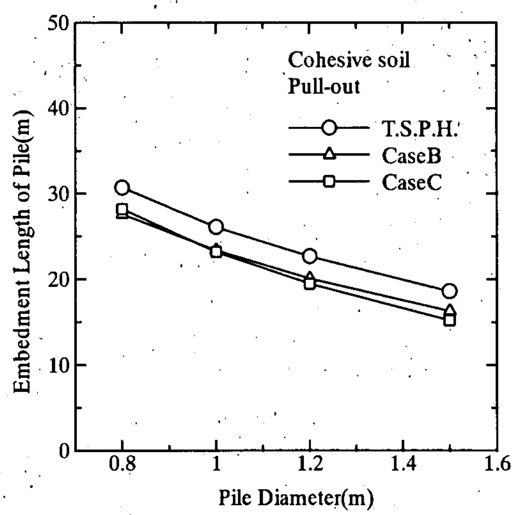

Figure 3.5 Necessary Embedment Length of Pile in Cohesive Soil against Pull-out Force

For the pull-out case in cohesive soil, the necessary embedment length calculated from the T.S.P.H. becomes longer than that calculated from the Eurocodes because of the different shaft resistance formulas. In the pull-out case, however, the difference in the embedment lengths calculated by the T.S.P.H. and the Eurocodes is less than the difference in the push-in case.

In the case of sandy soils, the difference in the embedment lengths partially comes from differences in the safety factors and partial factors, but mainly comes from the difference in the bearing capacity formulas. This difference in the bearing capacity formulas greatly affects the design.

In cohesive soils, designs based on the T.S.P.H. and the Eurocodes give approximately the same values of the necessary embedment length.

In the push-in case for cohesive soil, the T.S.P.H. gives a bearing capacity factor for the pile base of 8, whereas Orr and Farrell (1999) give 9, which also considers the overburden pressure. The Eurocode-based design method therefore gives a larger value for the bearing capacity of the pile base. On the other hand, the shaft adhesion given by Orr and Farrell (1999) is less than that calculated from the T.S.P.H.. This is because we multiplied the shaft adhesion by the adhesion factor α (=0.8). In designs based on the T.S.P.H., buoyancy is generally subtracted from the dead weight of the pile. The concept of Orr and Farrell (1999) is different in that they do not subtract the buoyancy (uplift water pressure force on the base) from the dead weight of the pile for cases of cohesive soils (undrained conditions).

For the pull-out case in cohesive soil, a different consideration is that Orr and Farrell (1999) use the cohesion factor α (=0.8) and do not subtract buoyancy from the dead weight of the pile.

In the case of cohesive soil, there are some differences, as described above. However, each effect is balanced, including the safety factor and the partial factors. We do not see any significant difference between a design based on the T.S.P.H. and one based on the Eurocodes, such as seen in the case of sandy soil.

4. Bearing Capacity of Spread Foundation

(1) Calculation Method

a) Calculation Method of Bearing Capacity of Spread Foundation Based on T.S.P.H.

① Sandy Soil

$$q_a = \frac{1}{F_s}\left(\beta\gamma_1 BN_r + \gamma_2 DN_q\right) + \gamma_2 D \tag{4.1}$$

where

q_a: Allowable bearing capacity of the foundation considering buoyancy of the submerged part (kN/m^2)

F_s: Degree of safety for bearing capacity of the sandy soil

β: Shape factor of the foundation (0.4 for a square)

γ_1: Unit weight of the soil under the bottom of the foundation (the submerged unit weight below the water surface) (kN/m^3)

B: Minimum width of the foundation (m)

N_r, N_q: Bearing capacity factors (the T.S.P.H. specifies that N_r =7.0 and N_q =8.5 for an internal friction angle of 30°)

γ_2: Unit weight of the soil over the bottom of the foundation (the submerged unit weight below the water surface) (kN/m^3)

D : Embedment length of the foundation (m)

② Cohesive Soil

$$q_a = N_{c0}\left(1+n\frac{B}{L}\right)\frac{c_u}{F_s}+\gamma_2 D \qquad (4.2)$$

where

N_{c0} : Bearing capacity factor for the strip foundation (where the rate of increase of the strength of the clay is 0, N_{c0} =5.14)

n : Shape factor of the foundation (where the rate of increase of the strength of the clay is 0, n=0.2)

L : Length of the foundation (m)

c_u : Undrained shear strength of the cohesive soil at the base of the foundation (kN/m^2)

F_s : Degree of safety of the bearing capacity of the cohesive soil

b) Design Method of Pile Foundation Adopted by Orr and Farrell (1999) Based on Eurocode 7

① Total Action F_d

$$F_d = \gamma_G G_k + \gamma_Q Q_k \qquad (4.3)$$

where

γ_G : Partial factor for the permanent load (Case B: 1.35, Case C: 1.0)

γ_Q : Partial factor for the variable load (Case B: 1.5, Case C: 1.3)

G_k : Characteristic value of the permanent action

Q_k : Characteristic value of the variable action

② Sandy Soil

$$R_d/A' = q'N_q s_q i_q + 0.5\gamma' B' N_\gamma s_\gamma i_\gamma \qquad (4.4)$$

where

R_d : Bearing capacity of the foundation base resistance (kN)

A' : Effective area of the foundation (m^2)

q' : Design effective overburden pressure at the foundation level base (kN/m2)

γ' : Unit weight of the soil (the submerged unit weight below the water surface) (kN/m^3)

B' : Effective width of the foundation (m)

The bearing capacity factor:

$$N_q = \exp(\pi \tan\phi')\ \tan^2(45+\phi'/2)$$

$$N_\gamma = 2(N_q-1)\tan\phi' \quad \text{when } \delta > \phi'/2 \text{ (rough base)}$$

$$\phi' = \tan^{-1}(\tan\phi'_k / \gamma_{\tan\phi'})$$

where

ϕ'_k : Characteristic values of the internal friction angle of the soil (°)

δ : Friction angle on the foundation base (°)

$\gamma_{\tan\phi}$: Partial factor of tan ϕ'

The shape factor of the foundation (for a square):

$$s_q = 1+\sin\phi'$$

$$s_\gamma = 0.7$$

The eccentric inclination factor of the load:

The eccentric inclination factors i_q and i_γ were set to 1.0 because we did not study eccentric inclinations.

③ Cohesive Soil

$$R_d/A' = (2+\pi)c_u s_c i_c + q = 5.14 c_u s_c i_c + q \qquad (4.5)$$

$$c_u = c_{uk}/\gamma_{cu}$$

where

c_{uk} : Characteristic value of undrained shear strength of the cohesive soil at the base of the foundation (kN/m^2)

q : Design total overburden pressure at the base of the foundation (kN/m^2)

A' : Effective area of the foundation (m^2)

γ_{cu}: Partial factor of c_u

The shape factor of the foundation (for the square):

$$s_c = 1.2$$

The eccentric inclination factor of the load:

The eccentric inclination factor i_c was set to be 1.0 because we did not study eccentric inclinations.

In the case of sandy soils, assuming a partial shear failure, Orr and Farrell (1999) calculated the bearing capacity factor based on Prandtl's solution. On the other hand, the T.S.P.H. calculate the bearing capacity factor considering the change of the shear failure type (full shear failure and partial shear failure).

In the case of cohesive soil, the formula for the bearing capacity used by Orr and Farrell (1999) is the same as that used in the T.S.P.H..

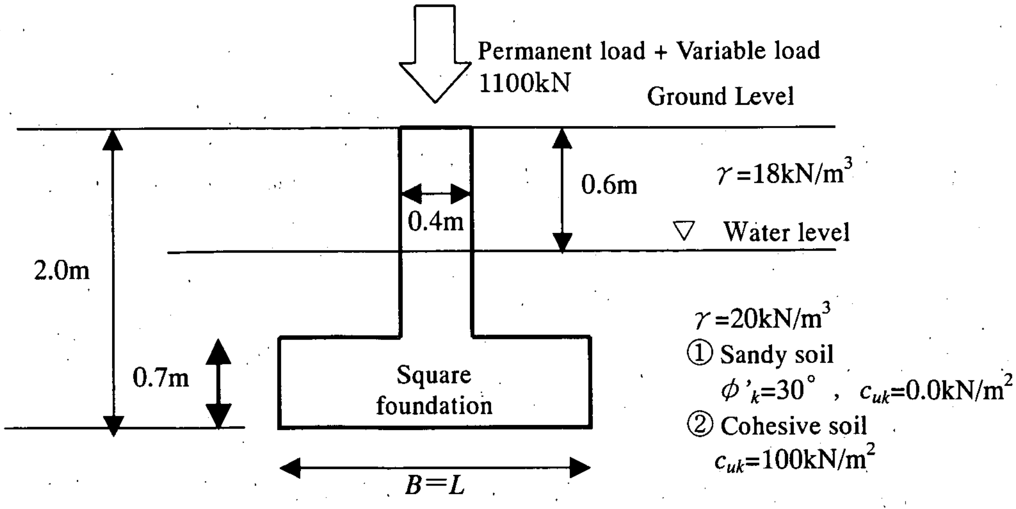

Figure 4.1 Overview of Spread Foundation and Study Conditions

(2) Objective

The objective is to calculate the stable width of the spread foundation on which the vertical load is imposed, study the effect of the ratio of the permanent load to the variable load of the vertical force, compare the ratio (degree of safety) of the bearing capacity to the action (the vertical force considering the partial factor), and quantitatively estimate the difference between the calculation formulas for bearing capacity used in the T.S.P.H. and in Orr and Farrell (1999). We also clarify the relationship between the safety factor specified in the T.S.P.H. and the partial factor proposed in the Eurocodes.

(3) Assumptions

Figure 4.1 shows an overview of the spread foundation and the study conditions.

The T.S.P.H. specifies that the safety factor is equal to or greater than 2.5 and 1.5 for sandy and cohesive soils, respectively.

(4) Results

We first determined the width of the foundation based on the Eurocodes under a permanent load of 800kN and a variable load of 300kN. The width of the foundation was 1.75m for the sandy soil, and 1.65m for the cohesive soil. We then calculated the degree of safety, that is the ratio of the bearing capacity to the action (vertical force considering the partial factor), using both the T.S.P.H. (Equations (4.5) and (4.6)) and the Eurocode. **Figures 4.2** and **4.3** show the calculated degree of safety.

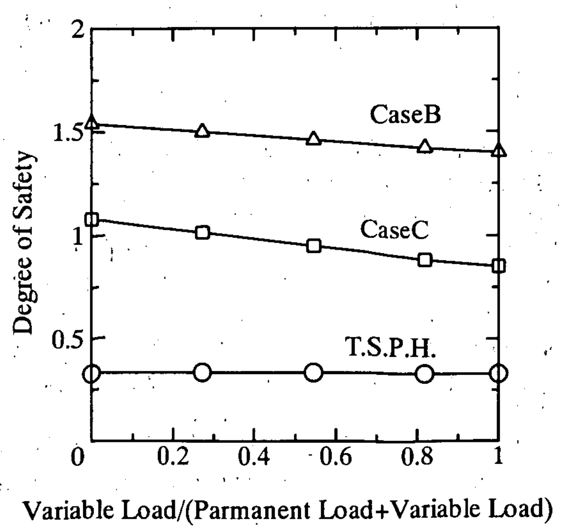

Figure 4.2 Degree of Safety with Respect to Ratio of Permanent Load to Variable Load (Sandy Soil)

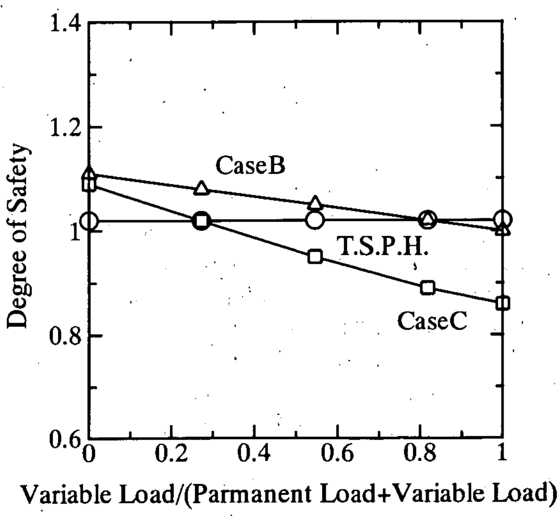

Figure 4.3 Degree of Safety with Respect to Ratio of Permanent Load to Variable Load (Cohesive Soil)

The Eurocode-based design shows that the degree of safety in Case C are smaller for both cohesive and sandy soils than that in Case B, which means that the width of the foundation is determined by Case C.

In the case of sandy soil, the T.S.P.H. and the Eurocodes have different concepts regarding calculation of the bearing capacity factor. Since the values of the bearing capacity factor used in the T.S.P.H. are smaller, the degree of safety of the T.S.P.H. are less than those of the Eurocodes. Under a permanent load of 800kN and variable load of 300kN, the Eurocode-based design gives the width of the foundation as 1.75m, whereas the design based on the T.S.P.H. gives 2.0m and 4.1m for the safety factors of 1.0 and 2.5, respectively. In the case of cohesive soil, the formula for the bearing capacity used by Orr and Farrell (1999) is identical to that used in the T.S.P.H.. We therefore compared the partial factor that contributes to the determination of the width of the foundation in Case C to the safety factor calculated from the T.S.P.H..

In Case C, the variable load is multiplied by a load factor of 1.3. For example, a permanent load of 800kN and a variable load of 300kN results in a design load of $800+1.3\times300=1190$kN. This indicates that a load factor of $1190/1100=1.08$ is imposed on the total load. Since the shear resistance is rebated by a material factor of 1.4, the total safety factor becomes 1.51, which is close to the safety factor of 1.5 specified by the T.S.P.H..

An effect of the ratio of the permanent load to the variable load in the Eurocode-based method is that the degree of safety decreases as the variable load increases. This is because the variable load multiplied by the load factor increases.

5. Sliding of Gravity Quaywall

(1) Calculation Method

The following equation is used in both the T.S.P.H. and the Eurocodes to evaluate the safety of a gravity quaywall against sliding:

$$F_s = \frac{fW}{P} \tag{5.1}$$

where

F_s: Degree of safety of the wall body against sliding

f: Coefficient of friction

W: Total vertical force acting on the wall body (kN/m)

P: Total horizontal force acting on the wall body (kN/m)

The T.S.P.H. assume that soil particles and water behave as a single body against seismic forces because the action time of the seismic force is very short compared with the drainage time. They also use an apparent seismic coefficient to calculate the submerged earth pressure, thus considering dynamic water pressure from the land side. The total vertical force W and the total horizontal force P acting on the wall body in Equation (5.1) are thus given by the following equations:

$$W = W_s - W_b + P_{AV} \tag{5.2}$$

$$P = P_{AH} + E_{ws} - E_{ws}' + E_{wd}' + W_k \tag{5.3}$$

where

W_s: Weight of the wall body in the air (kN/m)

W_b: Buoyancy of the wall body (kN/m)

P_{AH}: Resultant force of the horizontal component of the earth pressure during an earthquake (kN/m)

P_{AV}: Resultant force of the vertical component of the earth pressure during an earthquake (kN/m)

E_{ws}: Resultant force of the static water pressure from the land side (kN/m)

E_{ws}': Resultant force of the static water pressure from the sea side (kN/m)

E_{wd}': Resultant force of the dynamic water pressure from the sea side (kN/m)

W_k: Horizontal seismic force acting on the wall body (kN/m); $W_k = k_h\ W_s$ where k_h is the horizontal seismic coefficient.

The formula for calculating the resultant force of the static water pressure is expressed as follows:

$$E_{Ws} = \frac{1}{2}\gamma_w H_w^{\ 2} \tag{5.4}$$

The formula for calculating the resultant force of the dynamic water pressure is expressed as follows:

$$E_{wd} = \frac{7}{12}k_h\gamma_w H_w^{\ 2} \tag{5.5}$$

where H_w is the water level, and γ_w is the unit weight of the water.

The resultant force of the horizontal component of the earth pressure during an earthquake P_{AH} is expressed as:

$$P_{AH} = \sum\left(\frac{p_{ai-1}+p_{ai}}{2}\frac{h_i}{\cos\psi}\right)\cos(\psi+\delta) \tag{5.6}$$

The resultant force of the vertical component of the earth pressure during an earthquake P_{AV} is expressed as:

$$P_{AV} = \sum\left(\frac{p_{ai-1}+p_{ai}}{2}\frac{h_i}{\cos\psi}\right)\sin(\psi+\delta) \qquad (5.7)$$

where

p_{ai}: Active earth pressure acting on the wall surface of the bottom of soil layer i (kN/m^2); the formula for calculating p_{ai} is given below.

ψ: Angle of the wall surface with respect to the vertical plane (°)

δ : Friction angle on the wall surface (°)

h_i: Thickness of soil layer i (m)

Eurocode 8, part 5 requires that the designer evaluate the drainage condition of the target ground and distinguish the drained condition from the undrained condition in the design. It also specifies that the strength of the vertical seismic coefficient k_v should be 1/2 of the horizontal seismic coefficient k_h. Both the upward and downward directions of the action should be evaluated. When the ground moves downward, i.e. when it is accelerated downward, the inertia force acts upward on the wall body.

The total vertical force W and the total horizontal force P acting on the wall body under drained conditions are hence expressed as follows:

$$W=(1-k_v)\cdot(W_s-W_b)+P_{AV} \qquad (5.8)$$

$$P=P_{AH}+E_{ws}+E_{wd}-E_{ws}'+E_{wd}'+W_k \qquad (5.9)$$

where E_{wd} is the dynamic water pressure from the land side. The forces for the undrained condition are expressed as follows:

$$W=(1-k_v)\cdot(W_s-W_b)+P_{AV} \qquad (5.10)$$

$$P=P_{AH}+E_{ws}-E_{ws}'+E_{wd}'+W_k \qquad (5.11)$$

Both the T.S.P.H. and the Eurocodes adopt the Mononobe/Okabe Formulas for calculating the earth pressure during an earthquake. The formulas for determining the active earth pressure given by Mononobe/Okabe Formulas are expressed as follows:

$$p_{ai} = K_{ai}\left[\sum\gamma_i h_i + \frac{w\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (5.12)$$

$$\cot(\zeta_i-\beta) = -\tan(\phi_i+\delta+\psi-\beta) + \sec(\phi_i+\delta+\psi-\beta)\sqrt{\frac{\cos(\psi+\delta+\theta)\sin(\phi_i+\delta)}{\cos(\psi-\beta)\sin(\phi_i-\beta-\theta)}} \qquad (5.13)$$

$$K_{ai} = \frac{\cos^2(\phi_i-\psi-\theta)}{\cos\theta\cos^2\psi\cos(\delta+\psi+\theta)\left[1+\sqrt{\frac{\sin(\phi_i+\delta)\sin(\phi_i-\beta-\theta)}{\cos(\delta+\psi+\theta)\cos(\psi-\beta)}}\right]^2} \qquad (5.14)$$

where

p_{ai}: Active earth pressure acting on the wall surface of the bottom of soil layer i (kN/m^2)

ϕ_i: Internal friction angle of the soil in soil layer i (°)

γ_i: Unit weight of the soil in soil layer i (kN/m^3)

h_i: Thickness of soil layer i (m)

K_{ai}: Coefficient of the active earth pressure for soil layer i

ψ : Angle of the wall surface with respect to the vertical plane (°)

β: Angle of the ground surface with respect to the horizontal plane (°)

δ : Friction angle on the wall surface (°)

ζ_i: Angle of the failure plane of soil layer i with respect to the horizontal plane (°)

w: Surcharge per unit area of the ground surface (kN/m^2)

In the above equations, θ is the composite seismic angle, for which the T.S.P.H. gives the following:

$$\theta=\tan^{-1}k_h \qquad (5.15)$$

for the soil above the residual water level, and

$$\theta=\tan^{-1}k' \qquad (5.16)$$

for the soil below the residual water level, where k_h is the horizontal seismic coefficient and k' is the apparent seismic coefficient. The T.S.P.H. give the apparent seismic coefficient k' as follows:

$$k' = \frac{2\left(\sum\gamma_t h_i + \sum\gamma h_j + w\right)+\gamma h}{2\left\{\sum\gamma_t h_i + \sum(\gamma-10)h_j + w\right\}+(\gamma-10)h}k_h \qquad (5.17)$$

where

k': Apparent seismic coefficient

γ_t: Unit weight of the soil above the residual water level (kN/m^3)

h_i: Thickness of soil layer i above the residual water level (m)

γ: Unit weight of the water saturated soil in air (kN/m^3)

h_j: Thickness of soil layer j below the residual water level lying over the soil layer used for calculating the earth pressure (m)

w : Surcharge per unit area of the ground surface (kN/m^2)
h : Thickness of the soil layer below the residual water level used for calculating the earth pressure (m)
k_h : Horizontal seismic coefficient

Eurocode 8, part5 gives the following formulas:

$$\theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \tag{5.18}$$

for the soil above the residual water level , and

$$\theta = \tan^{-1}\left(\frac{\gamma_t}{\gamma_t - \gamma_w}\frac{k_h}{1-k_v}\right) \tag{5.19}$$

for permeable soil below the residual water level, and

$$\theta = \tan^{-1}\left(\frac{\gamma_d}{\gamma - \gamma_w}\frac{k_h}{1-k_v}\right) \tag{5.20}$$

for impermeable soil below the residual water level, where γ_d is the unit weight of the dry soil and k_v is the vertical seismic coefficient.

The methods for calculating the design seismic coefficient between the Eurocodes and the T.S.P.H. have the following differences:

① The method for calculating the composite seismic angle (Eurocode 8, part 5 considers a vertical seismic coefficient)
② The dynamic water pressure from the land side is treated differently (Eurocode 8, part 5 considers it for permeable ground)

(2) Objective

The Eurocode-based seismic design considers the vertical seismic coefficient and the permeability of the ground, which are not considered in the T.S.P.H..

Regarding sliding of the gravity quaywall during an earthquake, we estimated the stable width of the wall body by the design methods based on the T.S.P.H. and the Eurocodes (especially, Eurocode8, part5), and evaluated the effects of the vertical seismic coefficient and ground permeability.

(3) Assumptions

Figure 5.1 shows an overview of the gravity quaywall and the study conditions. From the figure, ψ and β in the Mononobe/Okabe Formulas [Equations (5.12) to (5.14)] become 0.

In the T.S.P.H., the safety factor against sliding during earthquakes is equal to or greater than 1.0. We used a surcharge of $10kN/m^2$ for both the T.S.P.H. and the Eurocodes designs. Values for the unit weight of the wall body were set as: γ =$21kN/m^3$, and γ '=$11kN/m^3$.

The Eurocodes and the T.S.P.H. use different formulas for calculating the design seismic coefficient from acceleration of the bedrock. However, we used the same value for the design coefficient in both cases to study the effects of the vertical seismic coefficient and ground permeability. The values of the design horizontal seismic coefficient we studied were 0.18, 0.10, and 0.05.

The value of the load factor used for seismic design in the Eurocodes is specified as 1.0, although the material factor must be determined adequately. Eurocode 8, part 5 specifies that the recommended value of the material factor for c_u is 1.4, and that for tan ϕ is 1.25. In our study, however, we used a material factor of 1.0 to serve the above objective.

The T.S.P.H. define the angle of wall friction as 15°, and the coefficient of friction as 0.6. Eurocode 7 specifies that the design friction angle on the wall δ_d should be set in response to the condition, and that the coefficient of friction should be estimated from tan(δ_d) rebated by the material factor. In order to examine the objectives mentioned above, we used an angle of wall friction of 15° and a coefficient of friction of 0.6.

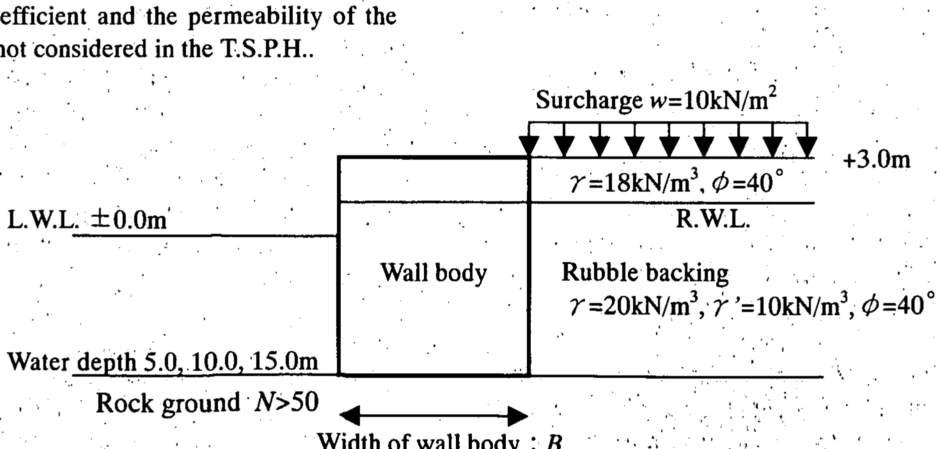

Figure 5.1 Overview of Gravity Quaywall and Study Conditions

(4) Results

Figures 5.2 and **5.3** show the results of the study.

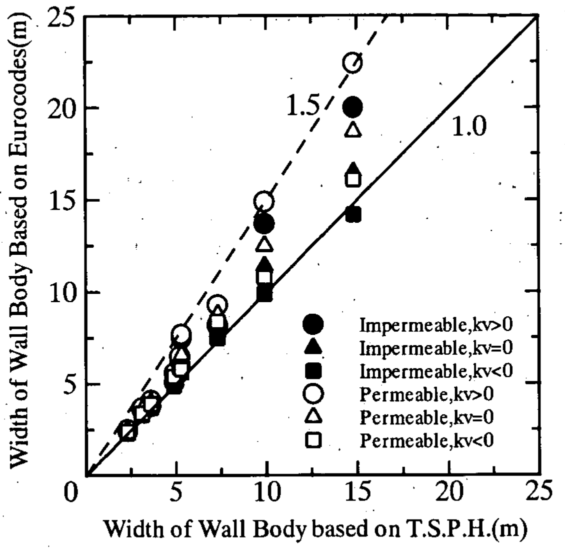

Figure 5.2 Widths of Wall Body Based on T.S.P.H. and Eurocodes

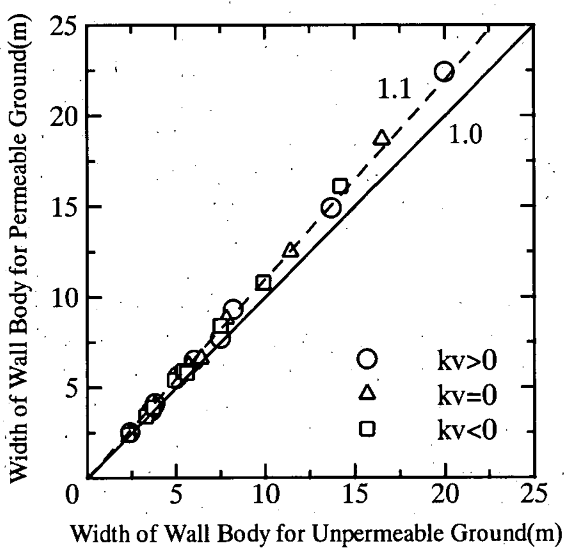

Figure 5.3 Comparison of Widths of Wall Body for Permeable and Impermeable Ground

The solid line in **Figure 5.2** shows the case that the width of the wall body calculated from the Eurocodes is the same as that calculated from the T.S.P.H., whereas the dotted line shows the case that the width of the wall body calculated from the Eurocodes is 1.5 times wider than that calculated from the T.S.P.H..

The Eurocode-based design that ignores the vertical seismic coefficient and the design based on the T.S.P.H. give different values for the necessary width of the wall body. Their results are different even though neither design considers the vertical seismic coefficient. This is because the calculation methods used for the apparent seismic coefficient are different; i.e., the T.S.P.H. consider the effect of the surcharge and the overburden pressure in the calculation of the apparent seismic coefficient given by Equation (5.17).For the effect of the vertical seismic coefficient, in the case of k_v>0, when an earthquake causes a downward acceleration to the ground, the required width of the wall body becomes 1.4 to 1.5 times wider than that calculated from the method based on the T.S.P.H..

The solid line in **Figure 5.3** shows the case that the width of the wall body for the permeable ground calculated from the Eurocodes is the same as that for the impermeable ground, whereas the dotted line shows the case that the width of the wall body for the permeable ground calculated from the Eurocodes is 1.1 times wider than that for the impermeable ground.

The T.S.P.H. assume that the seismic force acts instantaneously, and the ground behaves impermeably. For the Eurocode-based design, we compared the widths of the wall body both considering and not considering the permeability. The required width of the wall body considering permeability was about 1.1 times larger than that for the latter case.

6. Stability of Sheet-pile Quaywall

(1) Calculation Method

The T.S.P.H. provide two methods for evaluating a sheet-pile quaywall: ① a method using together the free-earth support method to evaluate the embedment length of the sheet pile and the equivalent beam method to determine the section size. ② Rowe's method, which determines the embedment length and section size taking the rigidity of the sheet pile section into consideration to simultaneously.

We made a comparison study of the embedment length and the section size.

We first used the free-earth support method to determine the embedment length of the sheet-pile quaywall based on the T.S.P.H., and then checked it using Rowe's method. Eurocode 8 does not describe the design method, but Orr and Farrell (1999) used the free-earth support method to study the stability of the sheet pile under normal conditions. We hence used the free-earth support method for our study.

The calculation using the free-earth support method is shown below:

$$F_s = \frac{M_P}{M_A} \tag{6.1}$$

where

F_s: Degree of safety

M_P: Moment with respect to the installation point of the tie rod caused by passive earth pressure
M_A: Moment with respect to the installation point of the tie rod caused by active earth pressure and residual water pressure

We calculated the active earth pressure using Equation (5.12). The passive earth pressure is calculated as follows:

$$p_{pi} = K_{pi}\left[\sum \gamma_i h_i + \frac{w\cos\psi}{\cos(\psi - \beta)}\right]\cos\psi \tag{6.2}$$

$$\cot(\zeta_i - \beta) = \tan(\phi_i - \delta - \psi + \beta) + \sec(\phi_i - \delta - \psi + \beta)\sqrt{\frac{\cos(\psi + \delta - \theta)\sin(\phi_i - \delta)}{\cos(\psi - \beta)\sin(\phi_i + \beta - \theta)}} \tag{6.3}$$

$$K_{pi} = \frac{\cos^2(\phi_i + \psi - \theta)}{\cos\theta\cos^2\psi\cos(\delta + \psi - \theta)\left[1 - \sqrt{\dfrac{\sin(\phi_i - \delta)\sin(\phi_i + \beta - \theta)}{\cos(\delta + \psi - \theta)\cos(\psi - \beta)}}\right]^2} \tag{6.4}$$

where
p_{pi}: Passive earth pressure acting on the wall surface of the bottom of soil layer i (kN/m^2)
ϕ_i: Internal friction angle of the soil of soil layer i (°)
γ_i: Unit weight of the soil in soil layer i (kN/m^3)
h_i: Thickness of soil layer i (m)
K_{pi}: Coefficient of the passive earth pressure of soil layer i
ψ: Angle of the wall surface with respect to the vertical plane (°)
β: Angle of the ground surface with respect to the horizontal plane (°)
δ: Friction angle on the wall surface (°)
ζ_i: Angle of the failure plane of soil layer i with respect to the horizontal plane (°)
w: Surcharge per unit area of the ground surface (kN/m^2)

In these equations, θ is the resultant angle during an earthquake. Refer to Equations (5.15) to (5.17) for calculating the resultant angle during an earthquake.

To calculate the embedment length D_F from Rowe's method, we must satisfy the following equation:

$$\delta_s = \frac{D_F}{H_T} \geq 5.0916\omega^{-0.2} - 0.2591 \tag{6.5}$$

where
δ_S: Ratio of the embedment length of the sheet pile to the height from the seabed to the installation point of the tie rod
D_F: Embedment length of the sheet pile (m)
H_T: Height from the seabed to the installation point of the tie rod (m)
ω: Similarity number (= $\rho \cdot l_h$)
ρ: Flexibility number (= $H_T{}^4 / E\,I$) (m^3/N)
E: Young's modulus of the sheet pile (N/m^2)
I: Moment of inertia of the section per unit width of the sheet pile (m^4/m)
l_h: Coefficient of subgrade reaction of the sheet-pile wall (N/m^3)

To evaluate section properties required for the sheet pile based on the T.S.P.H., we generally use the equivalent beam method and Rowe's method as models and check that the maximum bending tensile/compressive stress falls within the allowable unit stress.Since the model for evaluating the section properties of the sheet pile is not defined in Eurocode 8, we used the equivalent beam method. Then we checked that the bending moment was equal to or less than the fully plastic bending moment of resistance based on the Eurocode 3, part 5.

(2) Objective

Regarding the stability of a sheet-pile quaywall during an earthquake, we used the design methods based on the T.S.P.H. and the Eurocodes to estimate the stable embedment length of the sheet pile and then evaluated the effect of the vertical seismic coefficient and ground permeability as we did for the gravity quaywall.
We also examined the effect of differences in the methods for evaluating the section properties of the sheet pile.

(3) Assumptions

Figure 6.1 shows an overview of the sheet-pile quaywall and the study conditions. In the study of the embedment length of the sheet pile, we calculated the earth pressure and set the load factor, the material factor, and the vertical seismic coefficient in the same manner used for the case of the gravity quaywall.

The T.S.P.H. specify that the safety factor for the embedment length of the sheet pile during an earthquake calculated from the free-earth support method is greater than or equal to 1.2.

The horizontal seismic coefficient for the study condition was 0.15. The T.S.P.H. define that the angle of wall friction is 15° on the active side and -15° on the passive side, whereas Eurocode 8, part 5 specifies that it is less than or equal to 2/3 ϕ on the active side and 0° on the passive side. In our study based on the Eurocodes, we used an angle of wall friction of 2/3 ϕ on the active side and 0° on the passive side. The types of the sheet pile used in the Rowe's method calculations were Ⅱ$_W$-type, V$_L$-type, and ϕ1100×12t-type, comprised of steel sheet

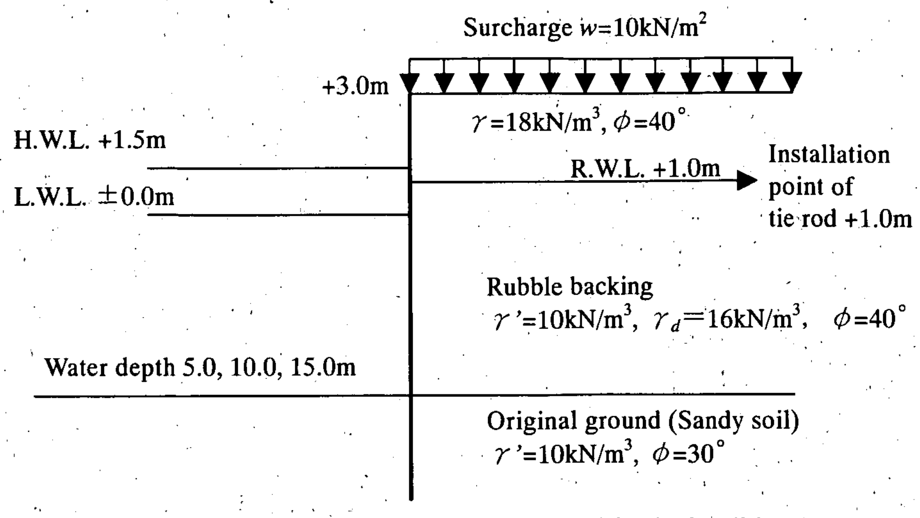

Figure 6.1 Overview of Pile-Sheet Quaywall and Study Conditions

piles for water depths of 5.0m, 10.0m, and 15.0m, respectively.

We set the allowable stress under normal conditions at approximately 60% of the bending tensile/compressive yield stress σ_y and multiplied it by 1.5 for earthquake conditions.

The fully plastic bending moment of resistance is calculated by using the bending tensile/compressive yield stress σ_y as the characteristic value for the strength of the steel.

For the plastic section modulus Z_p of the steel sheet pile, we assumed that the section was structurally equivalent to a box section and calculated the modulus from the shape factor f (here assuming the value of f=1.14) and section modulus Z_y as follows:

$$Z_p = f \cdot Z_y \tag{6.6}$$

For the section modulus Z_y and the plastic section modulus Z_p of the steel-pipe sheet pile, we ignored the shape of the joint and assumed that the section was equivalent to a continuous pipe section with diameter d and thickness t. The plastic section modulus Z_p was calculated from the following equation:

$$Z_p = \frac{1}{6} d^3 \left\{ 1 - \left(1 - \frac{2t}{d} \right)^3 \right\} \tag{6.7}$$

Using the plastic section modulus, the characteristic value σ_y of the strength of the steel, and the material factor γ_m (=1.1), we calculated the plastic resisting moment M_{pd} from the following equation:

$$M_{pd} = \frac{\sigma_y}{\gamma_m} Z_p \tag{6.8}$$

(4) Results

Table 6.1 and **Figures 6.2** and **6.3** show the results for the embedment length of the sheet pile.

For this example by the T.S.P.H., the embedment lengths for water depths of 5.0m and 15.0m be determined by Rowe's method. The embedment length determined by the free-earth method greatly differs from that determined by Rowe's method, particularly when a steel-pipe sheet pile is used for the 15m water depth.

The embedment length for permeable ground is generally longer than that for impermeable ground, although the difference is negligible.

The embedment length is longest when the vertical seismic force is downward (k_v<0, the weight of the soil appears the heaviest).

The embedment length determined by the Eurocodes is longer than that determined by the T.S.P.H. In our study, however, we set the material factor of tan ϕ used for the design based on the Eurocodes at 1.0. When considering the material factor of 1.25 for tan ϕ, the difference in the embedment lengths between the T.S.P.H. and the Eurocodes becomes larger.

In our study conditions, the maximum bending moment of the sheet pile calculated from the Eurocodes takes the largest value when a downward inertia force acts on a soil mass in permeable condition. In the study of the specifications of the section described below, the Eurocode-based method means the cases which were performed with a downward inertia force acting on a soil mass in permeable condition.

When we compare the two design methods in evaluating the section of the sheet pile, we need to consider the following:

① the difference in the bending moment based on differences in the calculation methods of the earth pressure
② the difference in the method for verifying the bending resistance of the section

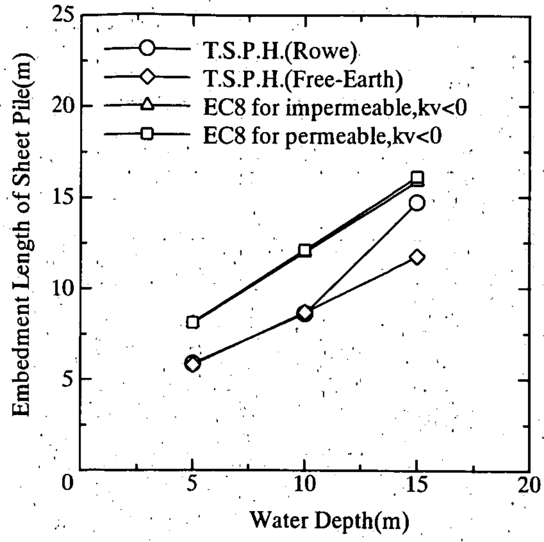

Figure 6.2 Embedment Length of Sheet Pile (Comparison of the results calculated from the T.S.P.H. to those calculated from the Eurocode 8 for permeable and impermeable ground)

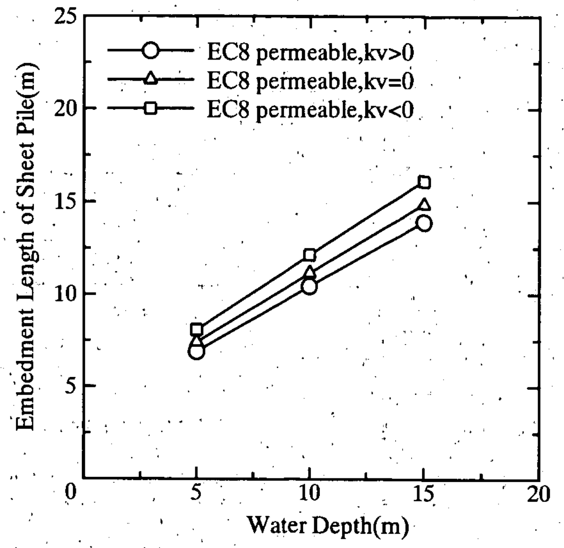

Figure 6.3 Embedment Length of Sheet Pile (Comparison of the effect of the vertical seismic coefficient on permeable ground calculated from the Eurocodes)

Table 6.1 Embedment Length of Sheet Pile (unit: m)

	T.S.P.H.		Eurocodes, Impermeable Ground			Eurocodes, Permeable Ground		
Water Depth	Free-earth	Rowe	k_v>0	k_v =0	k_v <0	k_v >0	k_v =0	k_v <0
5.0m	5.78	5.85	6.88	7.39	8.07	6.92	7.43	8.10
10.0m	8.68	8.61	10.28	11.01	11.98	10.43	11.14	12.11
15.0m	11.73	14.68	13.63	14.59	15.85	13.88	14.83	16.09

k_v>0: the inertia force acting on the soil mass is upward (the weight of the soil appears lighter).
k_v=0: the vertical inertia force does not act. k_v<0: the inertia force acting on the soil mass is downward.

The difference in the bending moment based on differences in the calculation methods for earth pressure can be explained as follows:

Using the same equivalent beam method in the two cases, **Table 6.2** shows that the maximum bending moment of the sheet pile calculated from the Eurocodes is 1.19 to 1.22 times larger than that calculated from the T.S.P.H..

The difference in the methods for verifying the bending resistance of the section can be explained as follows:

The examination of earthquake conditions based on the T.S.P.H. gives the following allowable unit stress after the multiplication:

$$\sigma_a=1.5\times 0.6\,\sigma_y=0.9\,\sigma_y \tag{6.9}$$

The bending moment of resistance M_1 based on the T.S.P.H. is therefore given by the following equation:

$$M_1=\sigma_a Z_y=0.9\,\sigma_y Z_y \tag{6.10}$$

Since the material factor based on the Eurocodes is 1.1, the design value used for calculating the fully plastic bending moment of resistance is expressed as follows:

$$\sigma_d=\frac{\sigma_y}{\gamma_m}=\frac{\sigma_y}{1.1}=0.91\sigma_y \tag{6.11}$$

The plastic bending moment of resistance M_2 based on the Eurocodes is therefore given by the following equation:

$$M_2=\sigma_d Z_y=0.91\,\sigma_y Z_p \tag{6.12}$$

The ratio of the bending moment of resistance based on the T.S.P.H. to that based on the Eurocodes can hence be expressed by the shape factor from the following equation:

$$\frac{M_2}{M_1}=\frac{0.91\sigma_y Z_p}{0.90\sigma_y Z_y}\approx\frac{Z_p}{Z_y}=f \tag{6.13}$$

where

Table 6.2 Comparative Study of Sheet Pile Section (1)

Design Method			T.S.P.H. Equivalent Beam Method		Eurocodes (permeable, k_v<0) Equivalent Beam Method	
Water Depth	Type of Sheet Pile		Action	Resistance	Action	Resistance
5.0m	Ⅱ$_W$ SY295	Bending Moment (kN·m/m)	164.0 (1.00)	270.0	194.9 (1.19)	305.7
		Stress Intensity (N/mm^2)	164.0	270.0		
10.0m	V$_L$ SY295	Bending Moment (kN·m/m)	779.9 (1.00)	850.5	954.4 (1.22)	969.6
		Stress Intensity (N/mm^2)	247.6	270.0		
15.0m	ϕ900mm (t=12mm) SM490	Bending Moment (kN·m/m)	2128.8 (1.00)	2260.1	2557.1 (1.20)	2775.0
		Stress Intensity (N/mm^2)	261.3	277.5		

Table 6.3 Comparative Study of Sheet Pile Section (2)

Front Water Depth (Specifications of Sheet Pile)	Action/ Resistance		②/①
	Method Based on T.S.P.H. Equivalent Beam Method ①	Eurocode-based Method Equivalent Beam Method ②	
5.0m (Ⅱ$_W$ SY295)	0.61	0.64	1.05
10.0m (V$_L$ SY295)	0.92	0.98	1.07
15.0m (ϕ 900 t=12mm SM490)	0.94	0.92	0.98

Table 6.4 Comparative Study of Sheet Pile Section (3)

Check Method			T.S.P.H. Rowe's Method		Eurocode (permeable ground, k_v<0) Equivalent Beam Method	
Water Depth	Type of sheet Pile		Action	Resistance	Action	Resistance
5.0m	Ⅱ$_W$ SY295	Bending Moment (kN·m/m)	203.2 (1.00)	270.0	194.9 (0.96)	305.7
		Stress Intensity (N/mm^2)	203.2	270.0		
10.0m	V$_L$ SY295	Bending Moment (kN·m/m)	832.1 (1.00)	850.5	954.4 (1.15)	969.6
		Stress Intensity (N/mm^2)	264.2	270.0		
15.0m	ϕ900mm (t=12mm) SM490	Bending Moment (kN·m/m)	2528.8 (1.00)	2260.1	2557.1 (1.01)	2775.0
		Stress Intensity (N/mm^2)	310.5	277.5		
	ϕ1100mm (t=12mm) SM490	Bending Moment (kN·m/m)	2528.8 (1.00)	2775.0	2557.1 (1.01)	3453.0
		Stress Intensity (N/mm^2)	269.9	277.5		

Table 6.5 Comparative Study of Sheet Pile Section (4)

Front Water Depth (Specifications of Sheet Pile)	Action/ Resistance		②/①
	Method Based on T.S.P.H. Rowe's Method ①	Eurocode-based Method Equivalent Beam Method ②	
5.0m (Ⅱ$_W$ SY295)	0.75	0.64	0.85
10.0m (V$_L$ SY295)	0.98	0.98	1.00
15.0m (ϕ 900 t=12mm SM490)	1.12	0.92	0.82
(ϕ1100 t=12mm SM490)	0.91	0.74	0.81

Z_y : Section modulus (m^3)
Z_p : Plastic section modulus (m^3)
f : Shape factor

From our study, the bending moment of resistance of the steel sheet pile calculated from the Eurocodes is about 1.13 to 1.14 times larger than that calculated from the T.S.P.H., and that the moment for the steel-pipe sheet pile is about 1.23 times larger.

The comparison study of the sheet pile section using the same equivalent beam method shown in **Table 6.3**. We may say that both methods give the almost same section because the difference is relatively small.

Table 6.4 indicates that the Eurocode-based method gives values for the bending moment that are about 0.96 to 1.15 times larger than those based on the T.S.P.H. using the Rowe's method as the evaluation model. **Table 6.5** shows that the method based on the T.S.P.H. tends to give more conservative results.

Comparing the steel-pipe sheet pile and the steel sheet pile as shown in **Tables 6.3** and **6.5**, the steel-pipe sheet pile tends to give safer results than the steel sheet pile using the method based on the T.S.P.H.. The reason is that the bending moment of resistance of the steel-pipe sheet pile is relatively larger because of the difference in the shape factors between the steel-pipe sheet pile and the steel sheet pile. For the case of a front water depth of 15.0m shown in Table 6.5, the method based on the T.S.P.H. and the Eurocode-based method give optimum sections of ϕ 1100 (t=12mm) and ϕ 900 (t=12mm), respectively. This is an example resulting from the difference in the bending moment of resistance because there is no large difference in the bending moment between the two methods.

7. Comparison of Methods for Calculating Design Seismic Coefficient

(1) Calculation Method

a) Eurocode 8

① Fundamental Requirements

For the No Collapse Requirement, Eurocode 8 assumes that the earthquake motion has an excess probability of 10% in 50 years (475 year return period). For the Damage Limitation Requirement, it assumes that the earthquake motion has an excess probability of 10% in 10 years (95 year return period).

② Design Seismic Acceleration

The reference acceleration of the earthquake motion is defined as the maximum acceleration (a_g) exerted on rocky or hard ground during the reference return period. Eurocode 8, however, does not define the type of strongmotion seismograph to be used for observing this acceleration or the method to be used for estimating a_g from the observed value.

③ Difference between Eurocode 8, part 1 and Eurocode 8, part 5

The method for calculating the design seismic coefficient described in Eurocode 8, part 1 differs from the one described in Eurocode 8, part 5. The methods for calculating the design horizontal seismic coefficient k_h and the design vertical seismic coefficient k_v specified in Eurocode 8, parts 1 and 5, and in the T.S.P.H. are shown below.

b) Eurocode 8, part 1

① Classification of Soil

Eurocode 8, part 1 classifies ground into the soil classes shown in **Table 7.1**.

② Elastic Response Spectrum

The horizontal seismic action is described by two orthogonal components considered as independent and represented by the same response spectrum.

Eurocode 8 divides the target earthquake motion into the following two types according to the value of the surface-wave magnitude M_s:

TYPE I : large earthquake (surface-wave magnitude $M_s \geqq 5.5$)
TYPE II: small earthquake (surface-wave magnitude $M_s < 5.5$)

③ Estimation of Elastic Response Spectrum

The design horizontal seismic coefficient is calculated from the estimated elastic response spectrum. The elastic response spectrum $S_e(T)$ divided by the acceleration of gravity g gives the design horizontal seismic coefficient k_h.

$$0 \leqq T \leqq T_B \ : \ S_e(T) = a_g S\left[1 + \frac{T}{T_B}(2.5\eta - 1)\right] \tag{7.1}$$

$$T_B \leqq T \leqq T_C \ : \ S_e(T) = 2.5 a_g S\eta \tag{7.2}$$

$$T_C \leqq T \leqq T_D \ : \ S_e(T) = 2.5 a_g S\eta\left[\frac{T_C}{T}\right] \tag{7.3}$$

$$T_D \leqq T \leqq 4s : S_e(T) = 2.5 a_g S \eta \left[\frac{T_C T_D}{T^2} \right] \quad (7.4)$$

where T is the vibration period of a linear single degree of freedom system, a_g is the design ground acceleration (i.e., the maximum acceleration in rocky or hard ground during the reference return period), T_B and T_C are the limit values of the constant spectral acceleration branch, T_D is the value defining the beginning of the constant displacement response range of the spectrum, S is the soil parameter corresponding to the subsoil class of the target ground, and η is the damping correction factor. The η is 1.0 for viscous damping below 5% and is calculated from the following equation for viscous damping of 5% or more:

$$\eta = \sqrt{10/(5+\xi)} \geq 0.55 \quad (7.5)$$

where ξ is the viscous damping ratio of the structure expressed as a percentage.

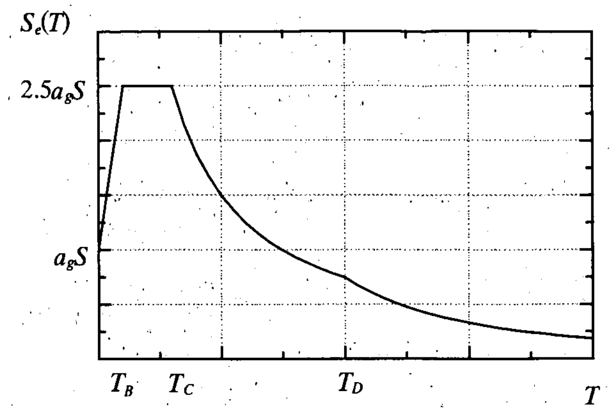

Figure 7.1 Example of the elastic Response Spectrum

Table 7.1 Classification of Subsoil Class

Subsoil Class	Description of Stratigraphic Profile	Parameters		
		$V_{s,30}$ (m/s)	N Value	c_u (kPa)
A	Rocky or other rock-like geological formation, including at most 5m of weaker material at the surface	> 800	-	-
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of m in thickness, characterized by a gradual increase of mechanical properties with depth	360-800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with $V_{s,30}$ values of class C or D and thickness varying between about 5m and 20m, underlain by stiffer material with $V_{s,30} \geqq$ 800m/s			
S_1	Deposits consisting-or containing a layer at least 10 m thick-of soft clays/silts with high plasticity index (PI>40), and high water content	< 100 (Indicatively)	-	10-20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in classes A - E or S_1			

$V_{s,30}$: average shear wave velocity in surface layer of 30m deep

Tables 7.2 and **7.3** show values of the soil parameter S and the parameters T_B, T_C, and T_D for elastic response spectra of TYPE I and TYPE II, respectively. The parameters T_B, T_C, and T_D determine the shape of the spectrum.

Table 7.2 Parameter Values of Elastic Response Spectrum of TYPE I

class	S	TB	TC	TD
A	1.0	0.10	0.4	2.0
B	1.1	0.15	0.5	2.0
C	1.35	0.20	0.6	2.0
D	1.35	0.20	0.7	2.0
E	1.4	0.15	0.5	2.0

Table 7.3 Parameter Values of Elastic Response Spectrum of TYPE II

class	S	T_B	T_C	TD
A	1.0	0.05	0.25	1.2
B	1.1	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.5	0.05	0.25	1.2

④ Estimation of Vertical Elastic Response Spectrum

The design vertical seismic coefficient is calculated from the estimated vertical elastic response spectrum. The vertical elastic response spectrum $S_{ve}(T)$ divided by the acceleration of gravity g gives the design vertical seismic coefficient k_v.

$$0 \leqq T \leqq T_B : \ S_{ve}(T) = a_v \left[1 + \frac{T}{T_B}(3.0\eta - 1) \right] \tag{7.6}$$

$$T_B \leqq T \leqq T_C : \ S_{ve}(T) = 3.0 a_v \eta \tag{7.7}$$

$$T_C \leqq T \leqq T_D : \ S_{ve}(T) = 3.0 a_v \eta \left[\frac{T_C}{T} \right] \tag{7.8}$$

$$T_D \leqq T \leqq 4s : \ S_{ve}(T) = 3.0 a_v \eta \left[\frac{T_C T_D}{T^2} \right] \tag{7.9}$$

Table 7.4 shows values of the vertical elastic response spectrum parameters.

Table 7.4 Vertical Elastic Response Spectrum Parameters

Spectrum	a_v/a_g	T_B	T_C	TD
TYPE I	0.90	0.05	0.15	1.0
TYPE II	0.45	0.05	0.15	1.0

The values of a_v/a_g are still under discussion at CEN.

⑤ Design Spectrum for Elastic Analysis

For structures that resist seismic actions in the non-linear range, the design horizontal seismic coefficient is calculated from the design spectrum. The design spectrum $S_d(T)$ divided by the acceleration of gravity g gives the design horizontal seismic coefficient k_h. (Eurocode 8, part 1 uses the dimensionless design spectrum divided by the acceleration of gravity g. However, we used the design spectrum with dimensions of acceleration to make it consistent with the elastic response spectrum.)

$$0 \leqq T \leqq T_B : \ S_d(T) = a_g S \left[1 + \frac{T}{T_B} \left(\frac{2.5}{q} - 1 \right) \right] \tag{7.10}$$

$$T_B \leqq T \leqq T_C : \ S_d(T) = \frac{2.5 a_g S}{q} \tag{7.11}$$

$$T_C \leqq T \leqq T_D : \ S_d(T) \begin{cases} = \dfrac{2.5 a_g S}{q} \left[\dfrac{T_C}{T} \right] \\ \geq [0.20] a_g \end{cases} \tag{7.12}$$

$$T_D \leqq T \leqq 4s : \ S_d(T) \begin{cases} = \dfrac{2.5 a_g S}{q} \left[\dfrac{T_C T_D}{T^2} \right] \\ \geq [0.20] a_g \end{cases} \tag{7.13}$$

where q is the behavior factor. Eurocode 8, part 1 specifies that the q is given in each chapter of Eurocode 8 and is equal to 1.0 for structures classified as non-dissipative.

c) Eurocode 8, part 5

In Eurocode 8, part 5, the soil condition is not considered, and the design horizontal seismic coefficient k_h and the design vertical seismic coefficient k_v are calculated from the following equations:

$$k_h=a_g / g / r,\ k_v=0.5\,k_h \qquad (7.14)$$

where a_g is the design ground acceleration (i.e., the maximum acceleration in rocky or hard ground during the reference return period), g is the gravity acceleration, r is a parameter related to the allowable displacement of the structure, as shown in **Table 7.5**.

Table 7.5 Parameters related to Allowable Displacement of Structure

Type of retaining structure	r
Free gravity walls that can accept a displacement d_r<300 α (mm)	2
As above with d_r<200 α (mm)	1.5
Flexible RC walls, anchored or braced walls, RC walls founded on vertical piles, restrained basement walls, and bridge abutments	1

where α is a_g / g.

d) Technical Standards for Port Structures in JAPAN (T.S.P.H.)

It is necessary to assure the seismic resistance of port structures according to their degree of importance. All structures must retain their safety and servisability against level 1 earthquake motions (earthquake motion with a 75 year return period). Structures with special seismic resistance must retain expected functionality against level 2 earthquake motions (earthquake motion caused by a earthquake in the plate with a return period of more than several hundred years or earthquake motions near land caused by a large-scale earthquake at a plate boundary).

Except for structures with long natural periods such as large-scale bridges and immersed tunnels, the design seismic coefficient (consisting horizontal only in the T.S.P.H.) is calculated from the following equation:

(design seismic coefficient) = (regional seismic coefficient) x (factor for subsoil condition) x (coefficient of importance)

① Regional Seismic Coefficient and Expected Value of Bedrock Acceleration for Return Period of 75 Years

Table 7.6 shows values of the regional seismic coefficient and the expected values of the acceleration of bedrock for a return period of 75 years.

The basic equation for estimating earthquake motions (distance damping equation) is given by the following:

$$\log_{10}A_{smac} = 0.53M - \log_{10}\ (X + 0.0062 \cdot 10^{0.53M}) - 0.00169X+0.524 \qquad (7.15)$$

Table 7.6 Resional Seismic Coefficient and Expected Value of Bedrock Acceleration for 75 Year Return Period.

Area	Resional Seismic Coefficient	Expected Value of Bedrock Acceleration for 75 Year Return Period (gal)
A	0.15	350
B	0.13	250
C	0.12	200
D	0.11	150
E	0.08	100

where A_{smac} is the maximum acceleration of bedrock as recorded by a SMAC strong motion seismograph (gal), M is the magnitude of the target earthquake, and X is the fault plane distance (km).

② Regional Seismic Coefficient

The area-wise seismic coefficients are calculated from the relational equation between the maximum ground surface acceleration α_s and the design seismic coefficient k_h applied at the design time, which is obtained from analysis of examples of earthquake damage to gravity quaywalls and sheet-pile quaywalls.

$$k_h= \alpha_s/g \quad \text{at} \quad \alpha_s\leqq 200\text{gal}$$
$$k_h=1/3\cdot(\alpha_s/g)^{1/3} \quad \text{at} \quad \alpha_s>200\text{gal} \qquad (7.16)$$

where g is the gravity acceleration. Since general subsoil condition is considered in the above equations, the relation between the maximum ground surface acceleration α_s and the acceleration of bedrock a_g is expressed the following equation:

$$\alpha_s = a_g/0.8 \qquad (7.17)$$

where 0.8 is the ratio of the subsoil condition factor of class 1 and class 2.

Equation (7.16) is close to the upper limit of the design seismic coefficient for the estimated value of the maximum ground surface acceleration that acted on the damaged structures. We used a conversion factor of 0.59 considering that an average relationship between the design seismic coefficient and the ground surface acceleration has been used for the design of general port structures.

③ Factor for Subsoil Condition

Table 7.7 shows values of the factor for subsoil condition. The factor for subsoil condition is defined in

Table 7.8.

Table 7.9 shows the correspondence of the subsoil class from Eurocode 8, part 1 and the subsoil class of the T.S.P.H..

Table 7.7 Factor for Subsoil Condition

Subsoil Class	Class 1	Class 2	Class 3
Factor for Subsoil Condition	0.8	1.0	1.2

Table 7.8 Subsoil Class

Thickness of Quaternary Deposit	Sand Gravel Layer	Ordinary Sandy, Clay, Subsoil	Poor Subsoil
equal to or less than 5m	Class 1	Class 1	Class 2
more than 5m and less than 25m	Class 1	Class 2	Class 3
equal to or more than 25m	Class 2	Class 3	Class 3

Table 7.9 Correspondence of Subsoil Class of Eurocode 8, part 1 and T.S.P.H.

Subsoil Class of T.S.P.H.	Subsoil Class of Eurocode 8, part 1
Class 1	A
	B
	C
Class 2	E
Class 3	D

④ Coefficient of Importance

Table 7.10 shows the coefficient of importance.

Relation between the coefficient of importance and the return period were examined by Yamamoto and Uwabe (1999) as follow. It was assumed for the coefficient of importance and the return period that the maximum acceleration of earthquake motion is linearly proportional to the return period on logarithmic axes. Regression coefficients of their linear relation equation were calculated using data for each port in Japan of 278. Relation between the coefficient of importance and the return period were estimated by substituting averaged regression coefficient for the linear relation equation. As a result, considering on the basis of a Class B structure having a coefficient of importance of 1.0, the design seismic coefficients of Class S, Class A, and Class C structures correspond to earthquake motions for return periods of approximately 150, 100, and 50 years, respectively.

⑤ Setting of Level-2 Earthquake Motion

The scale of an earthquake is the magnitude of the target earthquake. The magnitude of the active fault may be estimated from the following equation:

$$\log_{10}L = 0.6M - 2.9 \qquad (7.18)$$

where L and M are the length of the ground surface seismic fault (km) and its magnitude, respectively.

If the application of Equation (7.18) is difficult, the magnitude may be set at 7.2 in accordance with the Hyogoken-Nanbu (Kobe) Earthquake (1995).

Table 7.10 Coefficient of Importance

Classification of Structure	Characteristics of Structure	Coefficient of Importance
Class S	Structures having more effects for ①～④ then Class A structures	1.5
Class A	①Structures that may cause a huge loss of human life/ property if damaged by an earthquake ②Key structures designed serviceable for recovery from earthquake disaster ③Structures handling hazardous materials ④Structures, if disrupted, devastating the economic and social activities of the earthquake damage ⑤Structures, if damaged, being difficult to restore	1.2
Class B	Structures other than those of Class S, Class A, and Class C.	1.0
Class C	Small easily restorable structures other than those of Class S and Class A	0.8

Designer uses **Figure 7.2** to determine if the seismic motion is caused by an earthquake directly above its epicenter. The maximum acceleration of bedrock at the construction position of the facilities is calculated from Equation (7.15).

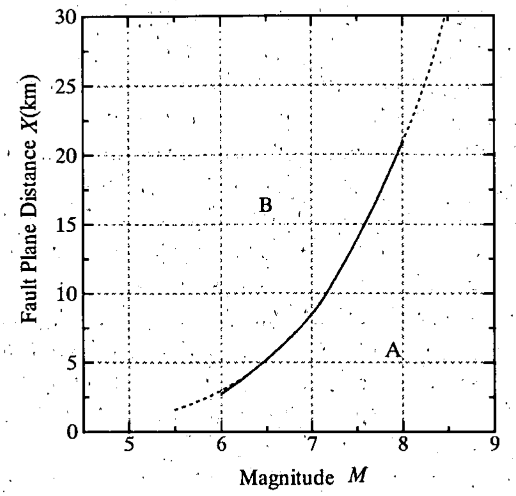

Figure 7.2 Classification of Active Fault Used for Judging Presence or Absence of Major Earthquake above Its Epicenter (An earthquake in the active fault belonging to domain A is considered to be a major earthquake above its epicenter)

⑥ Calculation of Design Seismic Coefficient for Level-2 Earthquake Motion

Designer first uses the ground condition for the target structures to calculate the earthquake response from the maximum acceleration of bedrock given by Equation (7.15) (e.g., SHAKE and others), and then estimate the design seismic coefficient from Equation (7.16). The following earthquake waveforms are used according to the classification of **Figure 7.2**:

Case A: Port Island Bedrock Wave and others

Case B: Earthquakes in the plate ; Port Island Bedrock Wave and others
Earthquakes at a plate boundary ; Hachinohe Wave, Ohfunato Wave, and others

In case that the design seismic coefficient calculated from Equation (7.16) is less than 0.25, the design seismic coefficient may be set at 0.25.

(2) Objective

The objective is to compare the calculation method for the design seismic coefficient based on the T.S.P.H. to those based on Eurocode 8, part 1 and part 5.

(3) Assumptions

We made the following assumptions to compare the calculation methods for the design seismic coefficient:

(a) A gravity quaywall was our design object. The natural period of the structure was assumed to be 0.0s.
(b) Assuming that the surface-wave magnitude $M_s \geqq 5.5$, the earthquake motion in the case of Eurocode 8, part 1 was assumed to be TYPE I. This is because the condition of the surface-wave magnitude $M_s \geqq 5.5$ holds everywhere in Japan.
(c) We set a port facility that is not a facility with special seismic resistance specified in the T.S.P.H. as our study target. It is a Class B structure with a coefficient of importance of 1.0.
(d) Since Eurocode 8, part 5 allows some displacement under earthquakes for a gravity quaywall, we assumed that r=2 and 1.5.

(4) Results

Figures 7.3 and **7.4** show the results of the comparison of the design seismic coefficients. In **Figure 7.3**, Noda/Uwabe's Formulas [Equations (7.14) and (7.15)] multiplied by 0.59 were plotted and compared to the results of Eurocode 8, part 1 and part 5. In **Figure 7.4**, the same formulas were plotted and compared to the results of Eurocode 8, part 1 and part 5. Noda/Uwabe's formulas multiplied by 0.59 give relatively small values of the design seismic coefficient compared to those of Eurocode 8, part 1, but they give almost similar values to Eurocode 8, part 5. For larger values of the acceleration, they give values of the design seismic coefficient that are smaller than those of Eurocode 8, part 5 under some conditions.

The T.S.P.H. specifies that the design seismic coefficient for a maximum ground surface acceleration of 200gal or more is proportional to the 1/3 power of the acceleration from Noda/Uwabe's Formula [Equation (7.15)]. The concept of the T.S.P.H. differs from that of the Eurocodes in this regard.

Where the bedrock is exposed on the ground surface and the structure receives the undamped seismic force from the bedrock, the following relationship holds because of the transmission mechanism of the seismic force:

(design horizontal seismic coefficient) = (acceleration of bedrock) / (gravity acceleration)

Where the structure receives the seismic force from the bedrock through a soil layer, however, it cannot receive a seismic force that is larger than the shear resistance because shear failure occurs in the soil layer. This means that the value of the design seismic coefficient acting on the structure does not increase proportionally with the increase of bedrock acceleration, but it reaches a limit.

Noda/Uwabe's Formula is based on average Japanese soils. It is implicitly assumed that the ground cannot transmit seismic force with 100% efficiency if the acceleration of the earthquake motion is large.

Does the soil parameter S in the Eurocode 8, part 1 consider the transmission mechanism of seismic force in the ground described above when converting acceleration in the bedrock into acceleration of the ground surface ?

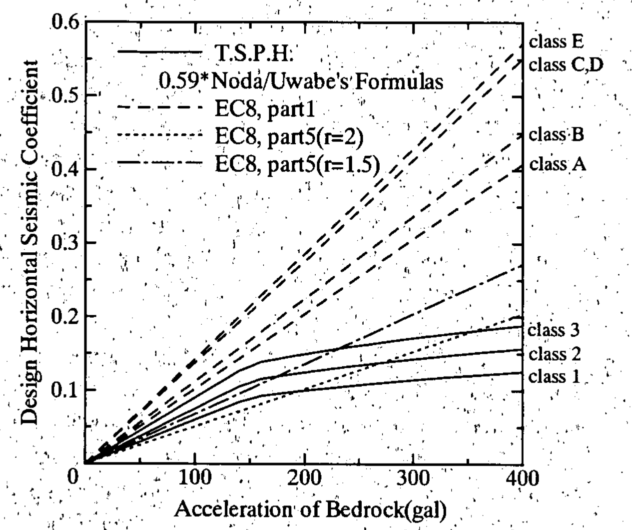

Figure 7.3 Comparison of Design Horizontal Seismic Coefficient (1)
(Noda/Uwabe's Formulas multiplied by 0.59 used for T.S.P.H.)

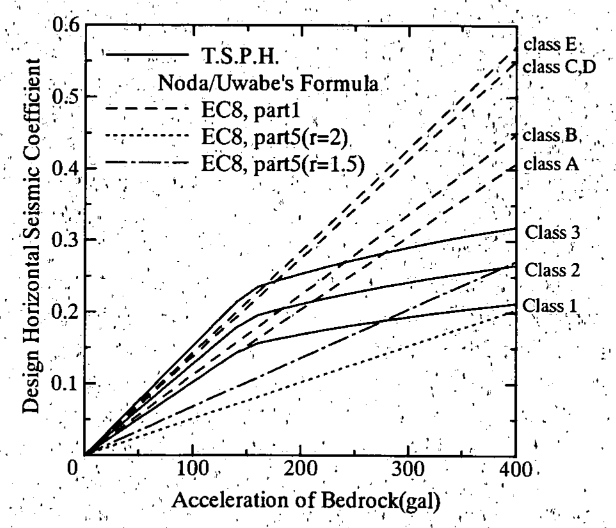

Figure 7.4 Comparison of Design Horizontal Seismic Coefficient (2)
(Noda/Uwabe's Formulas used for T.S.P.H.)

Figure 7.5 shows a comparison of the design seismic coefficients. The vertical seismic coefficient of the T.S.P.H. is 0.0 because it is not considered in the T.S.P.H..

Eurocode 8, part 1 specifies that the vertical seismic coefficient is equal to the horizontal seismic coefficient of subsoil class A multiplied by 0.9, whereas part 5 specifies that the multiplier is 0.5, which is significantly different. We used to adopted the value of the vertical seismic coefficient equal to one of the horizontal seismic coefficient multiplied by 0.5 in case of considering the vertical seismic force in Japan. In Eurocode 8, part 1, however, there is a description saying that "the ratio of the vertical seismic coefficient to the horizontal seismic coefficient is still under discussion." The above values may be revised in the future.

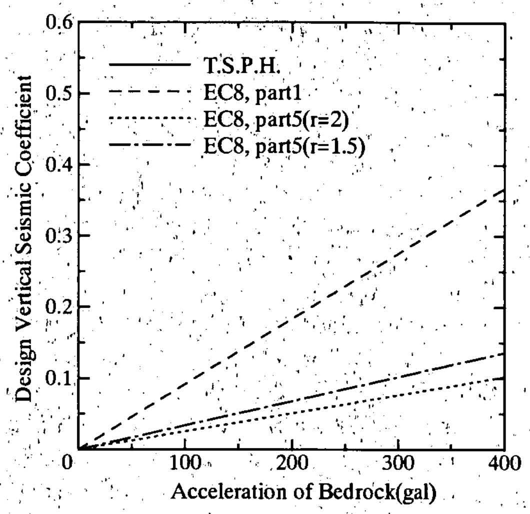

Figure 7.5 Comparison of Design Vertical Seismic Coefficients
(The vertical seismic coefficient for the T.S.P.H. is 0.0 because it is not considered in the T.S.P.H.)

8. Summary and Conclusions

[Concluding summary]

To clarify differences between the design methods based on the T.S.P.H. and the Eurocodes, we made comparative designs of example structures to study the following design issues: (1) slope stability, (2) bearing capacity of the pile foundation, (3) bearing capacity of the spread foundation, (4) sliding of the gravity quaywall, (5) stability of the sheet pile quaywall, and (6) estimation of the design seismic coefficient. The results are summarized below:

(1) Slope stability

We compared the degree of safety F_s (the ratio of the resisting moment to the driving moment) and clarified the relationship between the safety factor used in the T.S.P.H. and the partial factor proposed in Eurocode 7. In both cases, we calculated the degree of safety using the same calculation method.

The surcharge only slightly affects the difference in the degree of safety. The ratio of the safety factor used in the T.S.P.H. to the partial factor proposed in Eurocode 7 is nearly equal to the ratio of the degree of safety.

The Eurocode 7 seems to give somewhat safer results for slope stability compared to the T.S.P.H..

(2) Bearing Capacity of Pile Foundation

We calculated the bearing capacity of a pile foundation, compared the necessary embedment lengths in the push-in and the pull-out cases, and quantitatively estimated the difference between the formula for calculating the bearing capacity used in the T.S.P.H. and that used by Orr and Farrell (1999). We also clarified the relationship between the safety factor adopted in the T.S.P.H. and the partial factor proposed in the Eurocodes.

In the case of sandy-soil ground, the great difference in the embedment length arises from a difference in the calculation formulas for the bearing capacity. This difference in the calculation formulas for the bearing capacity greatly affects the design.

In the case of cohesive-soil ground, there are some differences in the formulas for calculating bearing capacity and in the safety factor and the partial factors, but their effects are balanced. We see no significant difference between the necessary embedment length calculated from the design based on the T.S.P.H. and that calculated from the Eurocode-based design such as was observed in the case of sandy-soil ground.

(3) Bearing Capacity of Spread Foundation

We calculated the stable width of the spread foundation, studied the effect of the ratio of the permanent load to the variable load of the vertical force, compared the ratio (degree of safety) of the action (the vertical force considering the partial factor) to the resistance (bearing capacity), and quantitatively estimated the difference between the formulas for calculating the bearing capacity used in the T.S.P.H. and the one adopted by Orr and Farrell (1999). We also clarified the relationship between the safety factor specified in the T.S.P.H. and the partial factor proposed in the Eurocodes.

In the case of sandy-soil ground, the T.S.P.H. and the Eurocodes have different concepts regarding the calculation of the bearing capacity factor. Since the values of the bearing capacity factor used in the T.S.P.H. are smaller, the degree of safety of the T.S.P.H. is less than that of the Eurocodes. The difference in the bearing capacity factors greatly affects the design.

In the case of cohesive-soil ground, the formula for the bearing capacity used by Orr and Farrell (1999) is identical to the one used in the T.S.P.H..

Comparing the partial factor contributing to the determination of the section in Case C to the degree of safety calculated from the T.S.P.H., the values of the safety factor and the partial factor are balanced and have almost the same degree of safety when the ratio of the permanent load to the variable load is about 0.27. For different ratios of the permanent load and the variable load, the degree of safety of the T.S.P.H. differs from that of the Eurocodes.

(4) Gravity Quaywall Sliding

Regarding sliding of the gravity quaywall during an earthquake, we estimated the stable width of the wall body from design methods based on the T.S.P.H. and the Eurocodes, and studied the effect of the vertical seismic coefficient and permeability of the ground.

The effect of the vertical seismic coefficient is ignored by the T.S.P.H.. For the Eurocodes in the case of k_v>0 when the earthquake causes a downward acceleration to act on the ground, the required width of the wall body is 1.4 to 1.5 times wider than that calculated from the method based on the T.S.P.H..

The T.S.P.H. assume that the ground is impermeable because the soil particles and water behave as one body against the seismic force due to the very short action time of the seismic force. In the Eurocode-based design, we compared the widths of the wall body between the cases considering and not considering permeability. The necessary width of the wall body when permeability was considered was about 1.1 times larger than that when it was not.

(5) Sheet-Pile Quaywall Stability

Regarding the stability of a sheet-pile quaywall during an earthquake, we used the design methods based on the T.S.P.H. and the Eurocodes to estimate the stable embedment length of the sheet pile, and studied the effect of the vertical seismic coefficient and permeability of the ground as we did in the case of the gravity quaywall. We also examined the effect of differences in the evaluating methods of the section of the sheet pile.

The embedment length for permeable ground is generally longer than that for impermeable ground, but the difference is negligible.

The embedment length determined by the Eurocodes is a little longer than the one determined by the T.S.P.H., but the difference is very small. In our study, however, we set the value of the material factor in earthquake used for the Eurocode designs at 1.0. If we consider the material factor, the difference in the embedment lengths between the T.S.P.H. and the Eurocodes becomes larger.

The bending moment calculated from the Eurocode is about 1.2 times larger than that calculated from the T.S.P.H. because of the difference in the methods for calculating the earth pressure. The bending moment of resistance used for checking calculated from the Eurocodes is also about 1.13 to 1.23 times larger than that calculated from the T.S.P.H.. The optimum section are the almost same when the same equivalent beam method is used.

The value of the bending moment calculated from the T.S.P.H. using Rowe's method is almost the same as that calculated from the Eurocode using the equivalent beam method. However, the Eurocodes give smaller values for the optimum section size because of the difference in the bending moment of resistance. Especially in the case of a steel-pipe sheet pile, the Eurocodes give much smaller values.

(6) Comparison of Methods for Calculating the Design Seismic Coefficient

The method for calculating the design seismic coefficient based on the T.S.P.H. specifies that the design seismic coefficient for a maximum ground surface acceleration of 200gal or more is proportional to the 1/3 power of the acceleration. The concept of the T.S.P.H. differs from that of the Eurocodes in this regard. The methods give different values of the design seismic coefficient for the same value of bedrock acceleration.

The basis for the design seismic coefficient adopted in Eurocode 8, part 1 is different from that adopted in Eurocode 8, part 5. Especially in cases where the displacement of structures is allowed, Eurocode 8, part 5 rebates the design seismic coefficient by 2 or 1.5, depending on the allowable displacement. This operation greatly affects the estimated design seismic coefficient. The ratio of the vertical seismic coefficient to the horizontal seismic coefficient given by Eurocode 8, part 5 is 0.5, whereas the one given by Eurocode 8, part 1 is 0.9 in subsoil class A. From our past experience, we doubt the validity of using a vertical seismic coefficient as large as 0.9 times the horizontal seismic coefficient for port structures.

[Concluding remarks]

The quantitive difference of the degree of safety and the designed structural size between the design methods based on the T.S.P.H. and the Eurocodes was clarified. The some knowledge for revising current technical standards in JAPAN and for the correspondence to ISO standards were obtained.

We consider that the design technology for port facilities in JAPAN should be reported toward all over the world. The results of this study will contribute to expressing opinions against ISO standards. In the future, we will be revising T.S.P.H in cooperation with Port and Harbour Bureau in Ministry of Land, Infrastructure and Transport and each Regional Development Bureau, et al..

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本資料の転載・複写のお問い合わせは
〒239-0826 神奈川県横須賀市長瀬3-1-1
管理調整部企画調整課　　電話：0468-44-5018