TENNESSEE VALLEY AUTHORITY

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830 Power Building

August 2, 1978

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Director of Nuclear Reactor Regulation
Attention: Mr. S. A. Varga, Chief
Light Water Reactors Branch No. 4
Division of Project Management
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Mr. Varga:

In the Matter of the Application of	)	Docket Nos. 50-390
Tennessee Valley Authority	)	50-391

Enclosed are ten copies of the Tennessee Valley Authority's response to the Corps of Engineers' questions on the Watts Bar Nuclear Plant application transmitted to us by your letter dated January 27, 1978, to G. Williams, Jr. This transmittal includes TVA responses to questions a through e.

Very truly yours,

J E Gilleland

J. E. Gilleland
Assistant Manager of Power

Enclosure

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ENCLOSURE

RESPONSES TO THE

CORPS OF ENGINEERS' QUESTIONS

ON THE

WATTS BAR NUCLEAR PLANT

Received by Letter Dated
January 27, 1978
from
S. A. Varga to Godwin Williams, Jr.

COE Question:
(a)

It is our opinion that the investigations of the soil along the alignments of the Class IE conduits and ERCW pipelines, as described in the FSAR, are not adequate to demonstrate that these soils would be stable under seismic loading. Logs of borings on the ERCW pipeline alignment have not yet been received by WES; however, we infer that the information that has been developed is similar to that obtained for the Class IE conduits. Specific reasons for our concern are as follows:

(1) Soil borings are at excessive intervals (about 200 feet), and no attempt has been made to correlate soil units or to develop a coherent geological cross section of the soils.

(2) Nearly total reliance is placed on blow counts and on split-spoon samples to determine soil properties. The few borings in which undisturbed samples were taken penetrated only the upper part of the overburden. Consequently, no direct information is available on the densities of granular soils and their susceptibility to liquefaction. Also, many of the granular soils encountered are described as gravelly; blow counts in such materials are difficult to interpret.

(3) Reliance is placed on 24-hour water level readings in the split-spoon boreholes to indicate the long-term water table conditions.

(4) Reliance is placed on gradation of granular soils to provide their nonsusceptibility to liquefaction. It is our opinion that gradation alone is not a sufficient basis to conclude that any cohesionless soils are safe against liquefaction.

The applicant should supplement these investigations to adequately define the soil and groundwater conditions along the Class IE conduit and ERCW pipeline alignments, or alternatively, demonstrate that liquefaction of the foundation soils would not endanger these Category I facilities.

Response:

Logs of the borings along the route of the ERCW pipelines were submitted in Amendment 24. The evaluation of the soils along that route is discussed below. That discussion is followed by the response to parts 1 through 4 of this question.

ERCW Pipelines

The criteria used to assess the potential for liquefaction along the ERCW pipeline route is similar to that used for the Class IE conduits. The ERCW pipelines follow the route defined by SS-87 through SS-101 of the two possible routes shown to the east of the cooling towers on figure 2.5-185 (revised by Amendment 28). Silty sand was encountered above the water table at various locations along the pipeline route. These silty sands were evaluated (1) using the empirical rules of Section 2.5.4.8, (2) by comparing the gradation characteristics to those of materials known to liquefy, such as the silty sand encountered in the intake channel area, and (3) by the procedures described in references 1 and 2. Only those silty sands above the top of weathered shale were considered since all materials below that point are merely rock fragments which have been given a soil classification.

The only borings in which silty sands were encountered in sufficient quantities to warrant evaluation were SS-88, -90, and -92. The silty sands encountered between elevations 699 to 712 in SS-88 and 709 to 718 in SS-90 show blowcounts as high as 50 and typically between 20 to 40. Evaluating these materials using the procedures of references 1 and 2 showed no potential for liquefaction.

Figures a-1 and a-2 are gradation curves for the silty sands found between elevations 714 to 722 in boring SS-92. These figures show the silty sands to be well-graded materials whereas liquefaction is normally associated with uniformly graded materials. In addition, the percentage of silt and clay present in these samples (42 percent) is significantly above the range of 10 percent or less specified in the criteria of Section 2.5.4.8 of the FSAR. A comparison of these materials with the liquefiable silty sands of the intake channel shows that the materials are not similar in their gradation characteristics. The gradation characteristics of the intake channel sand are shown in figure a-3. The material in the intake channel is a uniform silty sand which meets the criteria for liquefaction potential of Section 2.5.4.8; whereas, the materials along the pipeline route do not meet that criteria. This dissimilarity of materials in conjunction with the pattern in which silty sands were encountered along the pipeline route establishes that these sands along the route are not extensions of the liquefiable sand found in the intake channel. The intake channel soils profile is discussed in Section 2.5.5. A distinct, continuous layer of silty sand is visible in the graphic logs for the exploration in the intake channel. However, the borings show that this layer tapers out as one leaves the flood plain which substantiates that the liquefiable sand of the intake channel is confined to the flood plain.

The elevation of the water table is not shown on the graphic log for boring SS-92. However, it is possible to infer the elevation of the water table from its location in other borings around SS-92 and along the alternate route just east of the cooling towers (see Figure 2.5-185). Specifically, the water table elevation was determined in borings SS-65, -67, -87, -88, -93 through -95, -104, -105, -107, and -108. In those borings, the elevation of the water table varies from a minimum elevation of 693 to a maximum of approximately 704. From figure 2.5-185, it is apparent that the borings mentioned above bracket SS-92. Furthermore, the area in which SS-92 was taken has no apparent topological features which could result in an unusually high water table. Therefore, it is reasonable to infer that the elevation of the water table in SS-92 is approximately 702 and certainly no more than elevation 705.

Section 2.4.13 of the FSAR contains a discussion of sources of groundwater at the site and a description of investigations to determine the variation in groundwater elevations with time. Figure 2.4-104 shows the location of six observation wells which have been used to observe groundwater fluctuations over a 3-year period. Figure

2.4-103 graphically displays those fluctuations for the 3-year period. The maximum variation in any observation well is approximately 8 feet with typical variations of only 3 to 5 feet. Figure 2.4-105 shows a generalized water table contour map for an area within a 2-mile radius of the site and based on 48 water-level measurements made in January 1972.

If we use a water-table elevation of 703 in boring SS-92 and allow for a 3-4 feet fluctuation, the SM material between elevations 714 and 722 is typically 10 to 11 feet above the water table and a minimum of approximately 6 to 7 feet above it when long-term fluctuations are considered. Therefore, the SM material in boring SS-92 will normally be in a unsaturated state and cannot liquefy.

<u>Response to Parts 1 Through 4</u>

(1) The soils exploration programs for both the class IE conduits and ERCW pipelines were formulated as shown on figure 2.5-185. The programs called for split-spoon borings at 200-foot intervals unless additional or fewer borings were needed to define the materials encountered or to determine the extent of unexpected materials. The initial borings were reviewed for completeness and additional borings taken as needed. In the case of these testing programs, the borings shown on figure 2.5-185 adequately define the materials along both routes.

The graphic logs of the borings for the Class IE electrical conduits show that the SM and G-SM materials encountered are not present in extensive layers. The electrical conduits follow the route defined by borings 49, 50, 51, 52, 63, 57, and 58. Rather, the pattern is that these materials are present in isolated pockets. For instance, the SM and GG-SM material in SS-50 extending from elevation 689 to 699 is not encountered in either SS-49, SS-51, or SS-59. Rather, thin layers of silty sand no more than 1 to 1-1/2 feet thick are found at elevations which do not correlate with boring 50. Approximately 5 feet of SM and G-SM material is found above top of weathered shale in boring SS-53. However, the undisturbed boring US-53 which was taken only 5 feet away encountered no SM or G-SM material above top of weathered shale. One therefore concludes that the silty sand in SS-53 is not an extensive layer. The layers of silty sand in borings SS-60 and SS-63 are not extensions of the same layers since the silty sand of SS -53 is an isolated pocket. Similarly, the remainder of the borings show no evidence to suggest extensive layering of silty sands.

Borings 59 and 51 clearly establish that the silty snads encountered in the other borings for the electrical conduits are not extensions of the liquefiable sands encountered in the intake channel since only a thin layer of silty sand is encountered in 51 and none in 59. The silty sand of boring 49 was addressed in TVA's response question 362.14 in which that sand was shown to be 12-20 feet above the water table. The fluctuation of the water table is described above in the discussion of the ERCW pipeline route. Finally, a comparison of the gradation characteristics of the liquefiable sands of the intake channel (a typical gradation curve is presented in figure a-3) with those of the silty sands along the conduit and pipeline routes, shown in figures 362.14-1 through -7, a-1, and a-2 shows that these sands are not similar to the sand in the intake channel.

(2) The undisturbed samples for the routes of the electrical conduits and ERCW pipelines extend to top of weathered shale in all cases. Any material shown in the graphic logs as being below weather shale are actually rock materials which have been fragmented by the split-spoon sampler and to which a soil classification has been given. Tables 2.5-10 and 2.5-11 present the results of laboratory tests on the undisturbed samples of the route for the Class IE conduits and includes natural densities. Table 2.5-24 presents similar information for the route of the ERCW pipelines.

(3) Fluctuations of the water-table are discussed above for the route of the ERCW pipelines. Please refer to that discussion for references to specific portions of the FSAR.

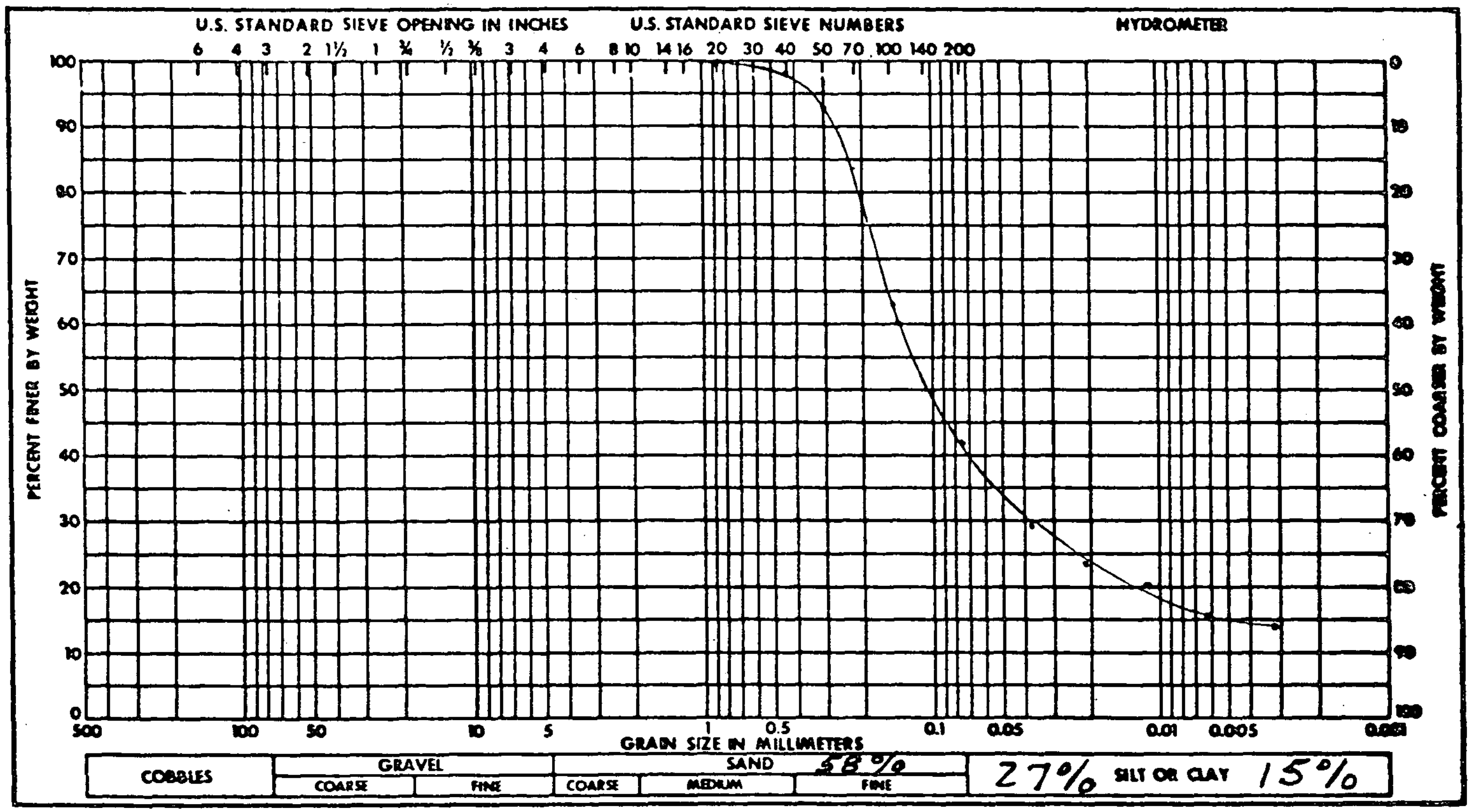
(4) We concur that gradation alone is not a sufficient criteria to assess liquefaction potential. The gradation information developed has been used to supplement the information discussed above. In addition, an evaluation of the soils along the route of the Class IE conduits using the simplified procedures of Seed and Idriss (reference 1) and the information of reference 2 for an SSE acceleration level of 0.18 g shows that the silty sands will not liquefy.

The soils along the routes for the Class IE conduits and ERCW pipelines have been evaluated for liquefaction potential. This evaluation has considered (a) the graphic logs for the materials encountered, (b) the results of the laboratory tests on undisturbed samples of these materials, (c) the location of and fluctuations in the water table, (d) the location of top of weathered shale, (e) a determination of the extent of the liquefiable sands of the intake channel, (f) the simplified procedures of Seed and Idriss (reference 1), and (g) the information presented by Shannon & Wilson and Agbabian Associates (reference 2). Based on these criteria it is concluded that the soils along both routes will not liquefy.

References

1. H. Bolton Seed and I. M. Idriss, "Simplified Procedure for Evaluating Soil Liquefaction Potential, ASCE, Journal of the Soil Mechanics and Foundations Division, September 1971.

2. NUREG-0026, "Evaluation of Soil Liquefaction Potential for Level Ground During Earthquakes," Prepared by Shannon & Wilson, Incorporated, and Agbabian Associates for the U.S. Nuclear Regulatory Commission.



Soil Symbol	SM	Liquid Limit, %	28.0
Moisture Content, %		Plastic Limit, %	22.8
Specific Gravity		Plasticity Index, %	5.2
		Shrinkage Limit, %	

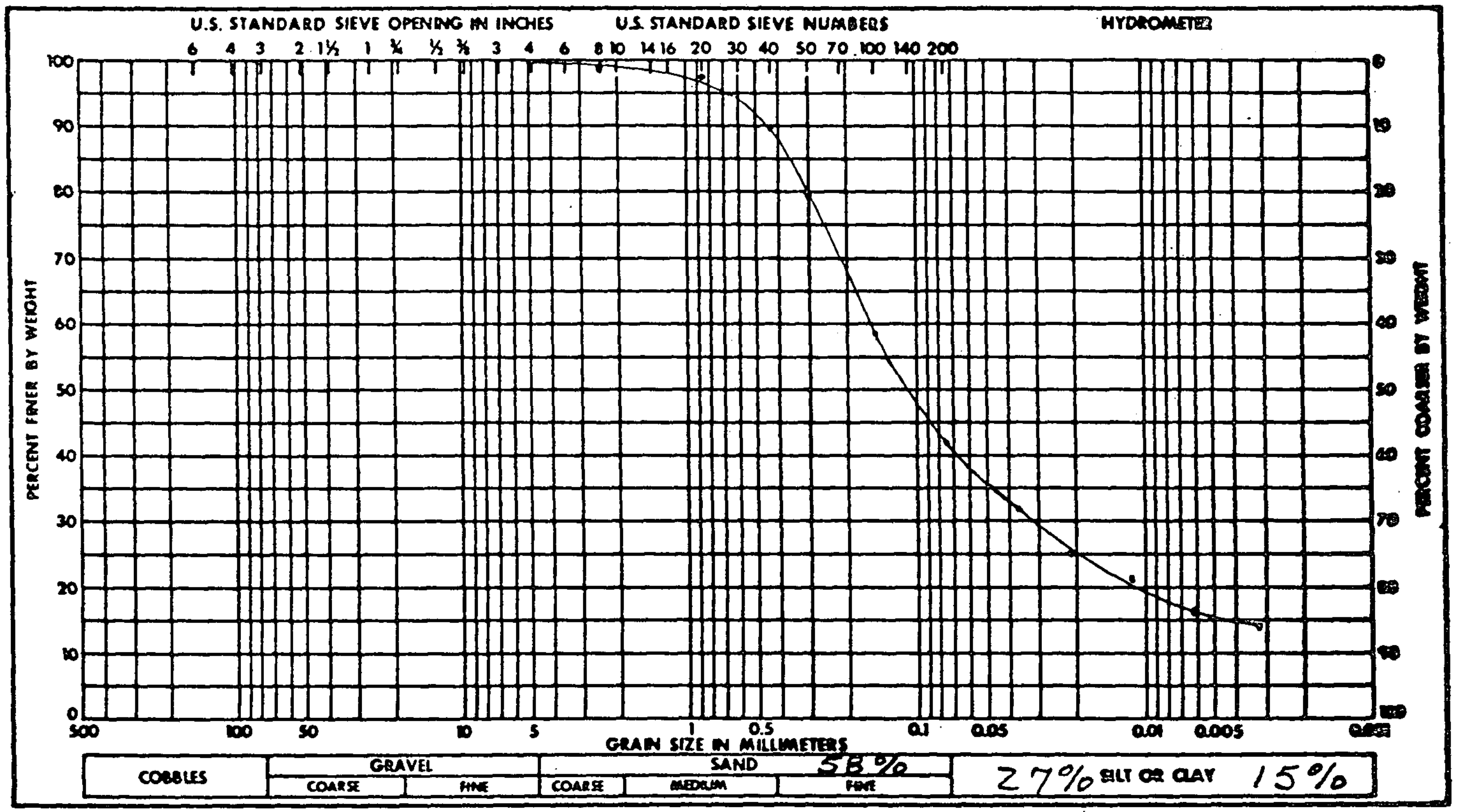
Remarks:
N=5

Project WATTS BAR N.P.	
Feature ERCW & HPFP SYSTEM	
Boring No. SS-92	Sample No. 3A,4A
Station	Offset
Date 11-26-75	Elevation 719,720.5

GRAIN SIZE ANALYSIS

Soil Form 8

Figure a-1



Soil Symbol	SM	Liquid Limit, %	26.0
Moisture Content, %	20.1	Plastic Limit, %	22.1
Specific Gravity		Plasticity Index, %	3.9
		Shrinkage Limit, %	

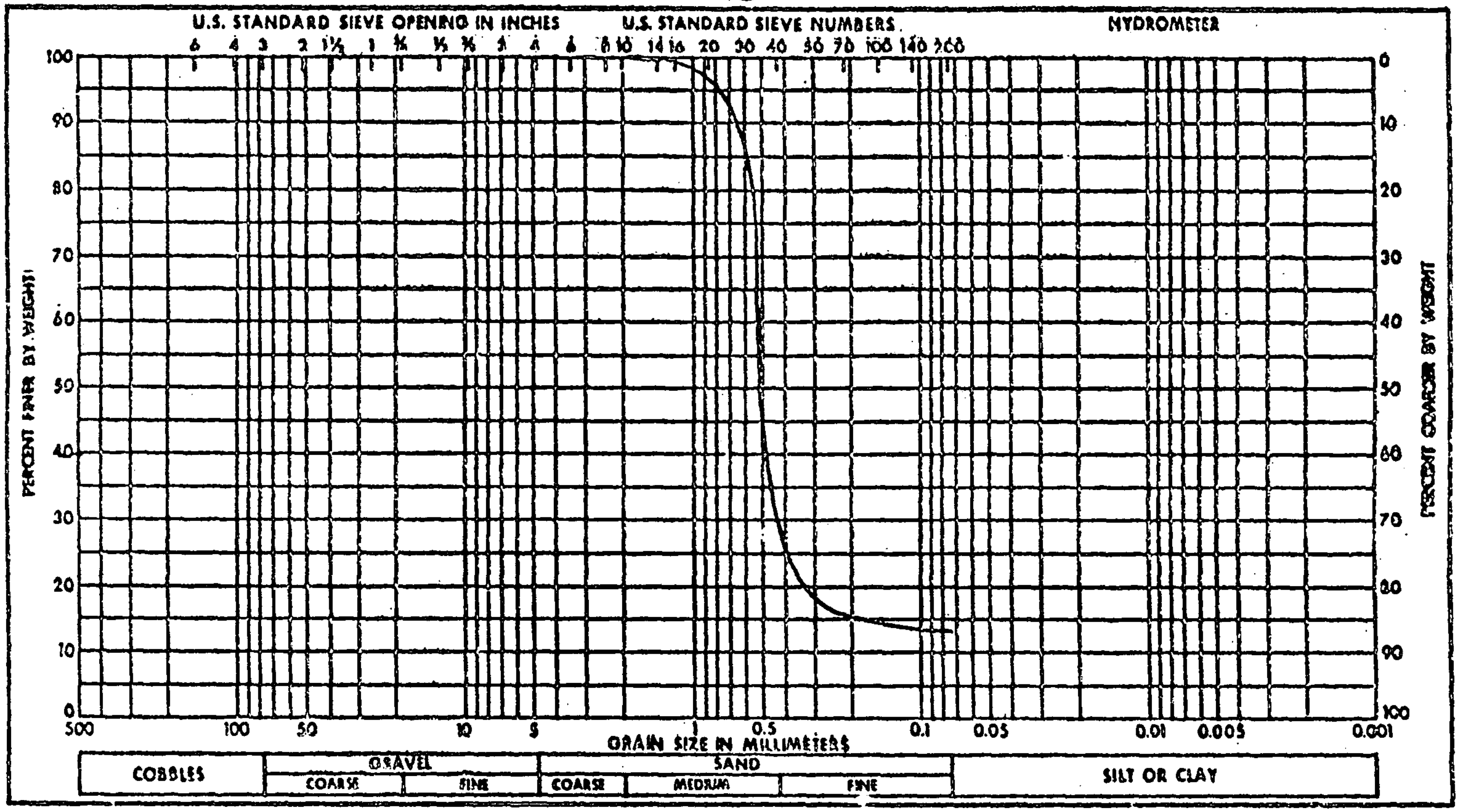
Remarks:

Project WATTS BAR N.P.	
Feature ERCW & HPFP SYSTEM	
Boring No. SS-92	Sample No. 5A
Station	Offset
Date 11-26-75	Elevation 716.9

GRAIN SIZE ANALYSIS

Soil Form 8

Figure a-2



Soil Symbol		Liquid Limit, %	
Moisture Content, %		Plastic Limit, %	
Specific Gravity		Plasticity Index, %	
		Shrinkage Limit, %	

Remarks:

$D_{60} = 0.53$

γ_D 96.1 pcf

w 12.4 %

Project Watts Bar N.P	
Feature Intake Channel	
Boring No.	Sample No. 2
Station	Offset
Date 10-22-76	Elevation

GRAIN SIZE ANALYSIS

Soil Form 8

Figure a-3

COE Question:
(b)

The method used for measurement of shear-wave velocities is questionable. Because of refraction through high-velocity zones or through the water column in the borehole, the indicated velocities are higher than true average velocities even with a homogeneous soil layer. Also, the influence of possible low-velocity zones in the lower part of the depth interval represented by the boring cannot be detected. The applicant should consider variations of ± 50 percent in the shear-wave velocity values, and determine the effects of such variation on safety related structures.

Response:

Adequacy of the method used for measurement of shear wave velocity is addressed in TVA's response to question 362.12 which was submitted in Amendment 30. The investigation program to determine dynamic soil properties was and is still considered state of the art for the thickness of overburden and for the soil types present. Refraction in high-velocity zones and influence of low-velocity zones can create problems in interpreting the data; however, these problems are not significantly reduced by use of cross-hole techniques. Standard design practice at TVA is to consider the effects of a variation in shear wave velocity of $\pm$ 30 percent which results in a variation of shear modulus of at least $\pm$ 50 percent. This range of shear wave velocity is considered to provide adequate design conservatism. This range of variation is used widely throughout the industry.

As stated in TVA's response to question 362.12, the parametric study for the diesel generator building and the waste packaging building was conducted using soil springs and a lumped-mass model of the structures. The shear wave velocity used to compute the stiffness values of the soil springs was varied over a sufficient range to cause the natural frequency of each structure to fall in the peak of the amplified response spectrum for the surface motion. The structural responses for the structures were then computed based on those peak earthquake accelerations. Therefore, a variation in shear wave velocity exceeding that performed for the parametric study would be meaningless as the loads which resulted would be lower than those used for design.

The shear wave velocities used in the seismic analysis of the Category I pipelines and Class IE conduits resulted in both being designed for amplification from the peak of the amplified response spectrum for motion of the soil deposit at the corresponding height of the pipelines and conduits. Therefore, varying the shear wave velocities beyond the range already considered would not affect the design.

COE Question:
(c)

Several Category I structures (e.g. the Diesel Generator Building) overlie in situ granular soils described as silty gravel, for which only data from split-spoon borings are presented. We would like to review any test data on this material (especially density data) that were obtained at the time the foundation excavations were open.

Response:

The in situ testing on the basal gravel material was limited to the splitspoon boring (SPT) results. There were no other in situ tests run on the basal gravel either during the field investigation program or at the time of excavation. The basal gravel was easily recognizable during excavation with gradation sizes ranging up to 6 inches. The Diesel Generator Building founded on granular fill above the basal gravel has experienced 0.025 feet settlement since it was constructed in 1975.

COE Question:
(d)

We would like to review the procedures used to determine earth pressure due to seismic loading on sheet pile and concrete retaining walls (Q362.9). For this purpose, the applicant should be requested to furnish copies of Reference 1 and the relevant portions of Reference 2, cited in Section 3.7.2.1.1. (Reference 1 is a 1939 TVA report, and Reference 2 is material presented at a short course at UCLA).

Response:

Copies of Reference 1 (Appendix E of TVA report, Dynamic Effect of Earthquake on Engineering Structures) to Section 3.7.2.1.1 were provided in response to question 130.14. Copies of Reference 2 are attached to this question.

Syllabus For

EARTHQUAKE ENGINEERING FUNDAMENTALS

Instructor in Charge: C. M. DUKE

Instructors:

R. W. BINDER   D. J. LEEDS
J. M. ENGLISH   R. B. MATTHIESEN
W. C. HURTY   M. F. RUBINSTEIN
K. L. LEE   W. T. THOMSON
S. HAYASHI

SHORT COURSE 800
August 22— September 2, 1966

ENGINEERING EXTENSION • DEPARTMENT OF ENGINEERING
UNIVERSITY OF CALIFORNIA, LOS ANGELES

Response to COE question d

SOIL STEM

Analysis and Design
of Earth
Structures
and Foundations

Current Procedures for Earthquake Resistant Design of Harbour Structures in Japan

Satoshi Hayashi

CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

Among various harbour structures, earthquake resistant design of quaywalls and piers is reviewed and discussed in this lecture because quaywalls and piers suffered more damages in past earthquakes than other harbour structures and most of the problems encountered in earthquake resistant design are also related to quaywalls and piers.

Needless to say, it is desirable to work out earthquake resistant design of a structure basing on dynamical behaviours of the structure during earthquakes. Unfortunately, however, from lack of knowledge of such dynamical behaviours, it is still common practice in earthquake resistant design for civil engineering structures to replace the whole effect of an earthquake by a static force which is obtained by multiplying seismic coefficient to the mass in question. This method of earthquake resistant design, which is called the "seismic coefficient method", is adopted in the current standards of design procedures for harbour structures.

In some exceptional cases, however, relatively simple characteristics of proposed structure allowed some dynamical modification of the "seismic coefficient method" (for example, No. 7 pier of Kobe Port).

The paper titled "Aseismic design of quaywalls in Japan"[1] which was presented to the First World Conference on Earthquake Engineering, 1956, made a considerable contribution to the rationalization of earthquake resistant design of quaywalls, by discussing all important problems in design, analysing the damages of qyaywalls caused by past earthquakes and introducing many examples of recent earthquake resistant designs. After that the "Japan Harbour Engineers' Manual"[2] was published in 1959 and standards of earthquake resistant design were established. However, these standards are still tansitional ones having many problems to be solved in future.

In 1966, "Draft Standards for Design of Harbour Structures in Japan[3)]" was published. These new standards correspond to the revised ones of the "Japan Harbour Engineer's Manual" published in 1959. Into these new design standards, results of the intensive research activities in the fields of harbour engineering, hydraulics, coastal engineering, soil mechanics, foundation engineering and earthquake engineering have been brought.

Here, the current procedures on earthquake resistant design of harbour structures are described based on "Draft Standards for Design of Harbour Structures in Japan", since the author believes that it represents the most typical procedures on earthquake resistant design of harbour structures. The subjects common to several types of structure such as earthquake load, earthpressure, bearing capacity, slope stability during earthquakes are dealt in the former part, from Section 2 to Section 6, and the procedures on earthquake resistant design of various type of the structure are described in the latter part from Section 7.

In the "Draft Standards for Design of Harbour Structures in Japan", it is legislated that the allowable stresses of construction materials, i.e. steel and concrete during earthquake could be as same as 1.5 times of the allowable stresses in normal condition.

2. EARTHQUAKE LOAD

2-1 Calculation of Earthquake Load

The earthquake load is obtained by multiplying dead loads and live loads by the seismic coefficient, and acts horizontally. The live loads here only mean the weight of crane which is travelling on the rail on the quaywall and the earthquake load of which has influence upon the stability of quaywall.

2-2 Determination of Seismic Coefficient

Since the "seismic coefficient method" is used in the earthquake resistant design of harbour structures, proper estimation of the coefficient, which is the ratio of seismic force to gravity force, is very important problem in earthquake resistant design. Many factors such as (i) regional probability of occurence of destructive earthquake, (ii) condition of foundation soil, (iii) structural feature of the proposed structure and (iv) importance of the structure should be considered in estimating seismic coefficient to be used in design.

According to "Draft Standards for Design of Harbour Structures in Japan", the seismic coefficient is determined regardless dynamic characteristics of the structures, because the most of the harbour structures have relatively short natural periods and large damping and effect of dynamic response of the structure to the stability of it is considered to be small. In Section 2 of Chapter 9, it is described that the seismic coefficient is determined by the following formula, taking regional probability of occurrence of earthquake, condition of foundation soil and importance of the structure into consideration. Seismic coefficient of structural design = Regional seismic coefficient × Factor for subsoil condition × Importance factor. *No reduction or increase for type of structure — such as a frame, shear wall, etc..*

Seismic coefficient is calculated down to two places of decimals and the last place larger than or equal to 0.08 is counted as 0.1, that between 0.07 ~~0.08~~ and 0.03 as 0.05 and cut away the rest.

As the exception, it is determined that a flexible structure such as a large pier made of piles or a tall structure should be designed by dynamic procedure.

(1) Regional seismic coefficient

Regional seismic coefficient basing on the regional probability of occurrence of earthquake is assigned as shown in Table-1 and Fig. 1. The result of the statistical study on the probability of occurrence of earthquake by Dr. H. Kawasumi[4)] shown in Fig. 2 was refered to the seismic zoning map shown in Fig. 1.

(2) Factor for subsoil condition

Factors determined by the kinds of subsoils are assigned as shown in Table-2. Classification of subsoil is illustrated as shown in Table-3 considering the thickness of the quaternary deposit.

If the ground are formed by alternate strata, kind of subsoil is determined by the thickness of the individual deposit, which forms the alternate strata. In the classification of subsoil illustreted in Table-3, sand stratum of which N value is less than 4, and clay stratum of which q_u value is less than 0.2 kg/cm^2 are called soft ground.

(3) Factor depending on the importance of structures.

Structures are divided into three kinds as shown in Table-4, and factors depending upon the importance of the structure are assigned as shown in Table-4.

Degree of importance of the structures are determined refering to the following items.

(i) Effect of the damage of the structure upon the social living.

(ii) Effect of the loss of the port function due to the damage of structure upon the reconstruction of environs.

(iii) Cost and time required to the reconstruction of structures.

2-3 Apparent Seismic Coefficient

Seismic coefficient in the air should be increased in the water due to buoyancy. This increased coefficient is called "apparent seismic coefficient" and given by the following equation.

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

where, k' : apparent seismic coefficient in the water

k : seismic coefficient in the air

γ : unit weight of the mass in the air ton/m^3

(for soil, it should include the weight of water which is saturating soil)

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

3. LATERAL EARTHPRESSURE AND DYNAMIC WATER PRESSURE IN EARTHQUAKES

3-1 Lateral Earthpressure in Earthquakes

Lateral earthpressure of sandy soil in earthquakes is computed by using the Mononobe-Okabe Formula [5),6)] which is derived from Coulomb's formula by statically applying a seismic force to the mass in question. For horizontal ground surface, the formula is given by the following expression (Fig. 3).

$\gamma_1 h_1$

$\gamma_2 h_2$

$\gamma_3 h_3$

$$p = (w + \Sigma\gamma \cdot h)K \qquad \cdots \quad (2)$$

$\Sigma(\gamma \cdot h)$

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

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

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

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

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

for general case .... 30°

for particularly good backfill .... 40°

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

above water table in backfill .... 1.8 t/m^3 $1m^3 = 35.3145\ ft^3$

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

h: depth from the ground surface (m)

K: coefficient of lateral earthpressure

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

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

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

ζ: angle between failure surface and horizon (°)

In Eqs. (3) and (4), upper signs are for active case and lower signs for passive case. Coefficients of active and passive earthpressure and angles between failure surface and horizon ζ for typical values of φ and δ are shown in Fig. 4 and Fig. 5 respectively.

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

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

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

(iii) Straight lines connecting these values of lateral earthpressures distribution under water.

3-2 Dynamic Water Pressure in Earthquakes

Dynamic pressure of water in backfill is not taken into consideration in current design procedure, because dynamic water pressure in earthquakes is to be included in lateral earthpressure in earthquakes, when the latter is computed by employing the apparent seismic coefficient given in Eq. (1) which, as mentioned in section 2-3, is based on the assumption of combined movement of water and soil mass. Dynamic pressure of water in front of a wall is not taken into consideration, because it is recognized that the effect of dynamic water pressure in front of the wall is compensatated by the other factors in the whole course of design calculation. As to the water in or between structures of a quaywall, mass force of the water due to earthquake should be considered instead of dynamic water pressure.

4. BEARING CAPACITY IN EARTHQUAKES

Current procedure of computing bearing capacity in earthquakes is similar to that for static state and no consideration is made on the effects of dynamic load and of dynamical properties of soil. Lower limit of safety factor for bearing capacity in earthquakes is 1.0.

(1) The bearing capacity of shallow foundation in sandy soil is computed by either Meyerhof's solution[7] or Tateishi's solution[8], which are for inclined and eccentric loading in static state.

(i) Meyerhof's solution for bearing capacity of shallow foundation in sandy soil (Fig. 6).

$$q_{av} = \frac{1}{F}\left[(1-\frac{2e}{B})(1-\frac{\alpha}{90^\circ})\ \gamma_2 D\, N_q d_q + (1-\frac{2e}{B})^2\cdot(1-\frac{\alpha}{\varphi})^2\cdot\frac{\gamma_1}{2} B\, N_\gamma\, d_\gamma\right]$$

$$+ (1-\frac{2e}{B})(1-\frac{\alpha}{90^\circ})^2\cdot \gamma_2 D \qquad \cdots \qquad (5)$$

where, q_{av} : vertical component of allowable bearing capacity (t/m^2)

F : safety factor

B : breadth of foundation (m)

e : eccentricity of resultant load (m)

α : angle of obliquity of resultant load (deg)

D : embedment depth (m)

φ : angle of internal friction (deg)

γ_1 : unit weight of soil below the base of foundation (submerged unit weight is taken below the water level) (t/m^3)

γ_2 : unit weight of soil above the base of foundation (submerged unit weight is taken below the water level) (t/m^3)

N_q, N_γ: bearing capacity factor[9] (modified Terzaghi's solution) (Fig.7)

d_q, d_γ: correction factor for embedment

$$\left.\begin{array}{l} d_\gamma = 1 + 0.6\ (D/B) \\ d_q = 1 + 0.2\ (D/B) \end{array}\right\}\ \text{for } D/B \leq 1.0$$

(ii) Tateishi's solution for bearing capacity of foundation on the horizontal ground surface. (Fig. 8)

$$q_{av} = \frac{1}{F} \cdot \frac{\gamma_1}{2} B\ N \qquad \cdots \quad (6)$$

where the symbols are the same to those in (i), and the bearing capacity factor N by Tateishi is shown in Fig. 9.

(iii) In the case when an actual measurement of angle of internal friction is not performed, the following values may be used in the evaluation of bearing capacity factors.

loose sand	$\varphi = 30°$
medium sand	$\varphi = 35°$
dense sand	$\varphi = 40°$

(2) The bearing capacity of shallow foundation in cohesive soil is computed by Meyerhof's solution as follows (Fig. 1):

$$q_{av} = (1 - \frac{2e}{B})\ (1 - \frac{\alpha}{90°})^2\ [\frac{5.14}{F} c\ f\ \gamma_2\ D] \qquad \cdots \quad (7)$$

where c is an apparent cohesion of cohesive soil (t/m^2), and other symbols are the same to those in (i).

5. LATERAL RESISTANCE OF PILES AND WELLS

5-1 Lateral Resistance of Vertical Piles

In the design of a laterally loaded piles, three values, namely, deflection at the pile head, bending moment induced in the pile, and necessary length of pile embedment, should be estimated in advance. At present so-called Chang's method[10], in which elastic behaviours of pile and soil are assumed, is most widely used because of its relative simplicity and reliability.

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

above the ground surface: $EI\dfrac{d^4 y_1}{d x^4} = 0 \quad (0 \geq x \geq -h)$

$$\cdots (8)$$

below the ground surface: $EI\dfrac{d^4 y_2}{d x^4} + E_s \cdot y_2 = 0 \quad (x \geq 0)$

where, EI: flexural rigidity of pile

x: depth from the ground surface

y: deflection of pile at depth x

B: width of pile

Es: elastic modulus of soil → SEUDO FDN. MODULUS

h: height of the point of load application

In Chang's method, Eqs. are solved with an assumption that Es = constant. The solutions can give necessary data for the design of a laterally loaded pile. For example, deflection at the pile head is given by the following expression.

$$\left.\begin{aligned} y_{top} &= \frac{H}{2EI\beta^3} \quad \text{..... "free-head" pile, h=0} \\ y_{top} &= \frac{H}{4EI\beta^3} \quad \text{..... "fixed-head" pile, h=0} \end{aligned}\right\} \cdots (9)$$

$$y_{top} = \frac{Hh^3}{3EI}\psi_\Delta(\beta h) \quad \text{..... "free-head" pile, } h>0$$

$$y_{top} = \frac{Hh^3}{12EI}\overline{\psi}_\Delta(\beta h) \quad \text{..... "fixed-head" pile, } h>0$$

(note: "fixed-head" means no rotation at the pile head)

where, $\beta = \sqrt[4]{\frac{Es}{4EI}}$ $\qquad \beta x = \frac{\pi}{4}$

H = lateral load

$$\psi_\Delta(\beta h) = \frac{(1+\beta h)^3 + \frac{1}{2}}{(\beta h)^3}$$

$$\overline{\psi}_\Delta(\beta h) = \frac{(1+\beta h)^3 + 2}{(\beta h)^3}$$

The soil modulus $Es = k_h \cdot B$ = constant ..... (10)

Foundation Modulus

The value of k_h, which is called "coefficient of horizontal subgrade reaction", can be estimated from the standard penetration value N of the site. The relationship between k_h and N based on field test data is shown in Fig. 10. pg 45.

Recently, a new method of lateral resistance estimation was proposed by Shinohara and Kubo[11), 12), 13)], in which all of the necessary computations can readily be made by means of computation charts.

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

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

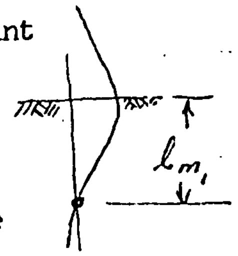
(ii) Soil resistance per unit area of pile surface decreases with increasing width of pile, but it becomes almost constant when pile width is larger than 20 cm.

$$EI\frac{d^4y}{dx^4} = -kx^m y^{0.5}$$

$m = 0$ $\quad N \approx$ const with Depth

$m = 1$ $\quad N \neq$ const. w/depth

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



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

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

5-2 Lateral Resistance of Coupled Piles

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

5-3 Lateral Resistance of Wells

The stability analysis of a wall which is subjected to a lateral force is done as follows (Fig. 11):

From the equilibrium of external forces:

$$\left.\begin{aligned} p_1 &= \frac{P_H}{(y_1 - \frac{h}{3})b}\left(\frac{y_1}{h}\right)^2 \\ y_1 &= \frac{\frac{3}{4}h - e}{2h - 3e}h \\ e &= \frac{M}{P_H} \end{aligned}\right\} \quad \text{... (11)}$$

The following relationship should be satisfied.

$$p_1 < K_P \cdot \gamma \cdot y_1 \quad \text{... (12)}$$

where p_1 : maximum value of lateral earthpressure acting in the opposite direction to horizontal component of external force (t/m^2)

y_1 : depth at which p_1 is acting (m)

h : embedded depth of well (m)

b : length of well in the direction perpendicular to horizontal component of external force (m)

P_H : horizontal component of external force (t)

M : overturning moment at the bottom of the sea due to external force (t-m)

K_P : coefficient of passive earthpressure (cf. section 3.1)

γ : unit weight of soil (t/m^3) (cf. section 3.1)

6. STABILITY OF SLOPES IN EARTHQUAKES

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

7. GRAVITY TYPE QUAYWALLS

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

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

Main external forces which should be considered in earthquake resistat design are lateral earthpressure (cf. section 3.1), water pressur (cf. section 3.2) and mass force of the wall itself. Stability analysis of a gravity type quaywall is done on three items as shown below.

(1) Sliding of the wall along its base

The ratio between horizontal and vertical components of the total external force should be smaller than the coefficient of friction between soil and base, which usually is 0.6. The safety factor is to be greater than 1.2 for static condition and can be reduced to 1.0 for seismic condition.

(2) Bearing capacity at the base

The reaction of soil or piles should not exceed their allowable bearing capacity. Computation is done according to the method shown in section 4.

(3) Sliding in the foundation soil

Sliding in the foundation soil is analyzed by following the procedure given in section 6.

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

8. SHEETPILE BULKHEADS

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

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

The fundamental principles of the current design method of sheetpile bulkheads are based on the extensive studies by G. P. Tschebotarioff[14], P. W. Rowe[15] and others on static state, and a special consideration is given to the seismic condition by utilizing field experiences in this country.

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

(1) Computation of lateral earthpressure and residual water pressure

The computation is done, following the procedures given in section 3-1 and 3-2.

(2) Estimation of necessary length of sheetpile embedment

Length of sheetpile embedment is to be 120% of that computed by free-earth support method. This length gives the safety factor against the failure of embedment about 1.5, which, according to experiences, is considered sufficient to keep the stability of bulkhead in an earthquake.

(3) Design of tie-rod

Tie-rod tension is computed on the assumption that the bulkhead is a simple beam which is supported at the sea bottom and the position of tie-rod connection and is carrying the load of lateral earthpressure and residual water pressure. Allowable stress of tie-rods is to be 900 kg/cm^2 and 1,400 kg/cm^2 for the static and the seismic condition repectively. These values correspond to 40% and 60% of the guaranteed yield point of mild steel. These relatively low values of allowable stress are adopted to have taken account of bending moment in the tie-rod due to surcharge. When some protection is provided to the tie-rod, greater allowable stress can be used.

(4) Design of sheetpile section

The maximum bending moment is computed for the simple beam mentioned in (3). This value of maximum moment, which is about 40~50% of that computed by free-earth support method, corresponds to the value computed by fully taking into account the moment reduction due to flexibility of sheetpile. The allowable stresses of sheetpile for the static and seismic conditions are 1,800~2,400 kg/cm^2 and 2,700~3,600 kg/cm^2 respectively.

(5) Design of anchor plate

Usually anchor plates are provided to support tie-rod tension, but in the case of a sheetpile bulkhead with relieving platform no anchor plate is used, because the horizontal force is supported by the platform and piles.

Lateral resistance of an anchor plate should be larger than 3 times and twice of the tie-rod tension for the static and seismic conditions respectively. Anchor plates should be placed behind the active failure wedge starting from the sea bottom. When passive wedge of anchor plate crosses the active wedge of sheetpile, the passive resistance of the soil before the point of intersection should be neglected in the computation of lateral resistance of anchor plate.

(6) Design charts

These computations for sheetpile bulkheads are illustrated in the design charts, and sheetpile section, tie-rod diameter, length of embedment etc., can readily be known from the charts when the design conditions such as soil condition, water depth, level of residual water, position of tie-rod, surcharge and seismic coefficient are given. As an example, charts for the condition, $H = 9$ m and $w = 3.0$ t/m^2, are shown in Fig. 12.

Symbols appearing in the charts have the following meanings.

- H : water depth in front of quaywall (m)
- H' : top level of quaywall (m)
- k : seismic coefficient to be used in the design
- w : surcharge (t/m^2)
- $\varphi = 25°$: angle of internal friction of soil $= 25°$, etc.
- φ_{100} : diameter of tie-rod = 100 mm, etc.
- M_{max} : maximum bending moment of sheetpile (t-m/m)
- A_P : tension of tie-rod (t/m)
- D : embedded length of sheetpile (m)
- $\sigma_s = 900$: allowable stress of steel = 900 kg/cm^2, etc.

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

In designing a sheetpile bulkhead it is necessary to take the following measures to prevent damages at the corner due to earthquakes; namely, using good backfill, increasing size of members, providing kingposts, or placing cellular bulkhead type structure.

9. CELLULAR BULKHEADS

Cellular bulkhead type quaywalls are constructed by driving straight-web sheetpiles to form cells and then filling them with soil (Fig. 13). This type of quaywall is becoming very popular because of simplicity and rapidity of construction, and low cost for greater depth quaywalls.

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

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

$$M_r = M_{rf} + M_{rs} \qquad \ldots (13)$$

Following expression have been proposed so far for the ultimate resisting moment.

a) Terzaghi-Krynine (1945)[16]

$$M_{rf} = \frac{1}{6}\gamma h^3 \, 2\, K_k \nu \tan\varphi$$
$$M_{rs} = \frac{1}{6}\gamma h^3 \, 2\, K_k \nu f \qquad \ldots (14)$$

b) Schneebeli (1957)[17]

$$M_{rf} = \frac{1}{6}\gamma h^3 (0.03)\cdot\nu\varphi \qquad \ldots (15)$$

where in degree

$0.6 < \nu < 1.2$

$16° < \varphi < 44°$

c) Cummings (1957)[18]

$$M_{rf} = \frac{1}{6}\gamma h^3 (\nu \tan\phi)^2 (3 - \nu \tan\varphi)$$

$$M_{rs} = \frac{1}{6}\gamma h^3 3 K_a \nu f \qquad \cdots \quad (16)$$

d) Kitajima (1962)[19]

$$M_{rfp} = \frac{1}{6}\gamma h^3 \cdot \frac{1}{12}\nu^2 (K_p - K_a)(3 - \nu\cos\varphi) \sin 2$$

... for a permanent structure

$$M_{rft} = \frac{1}{6}\gamma h^3 \cdot \frac{1}{4}\nu^2 (K_p - K_a)(3 - \nu\cos\phi)\cos^2\phi \qquad \cdots \quad (17)$$

... for a temporary structure

$$M_{rs} = \frac{1}{6}\gamma h^3 \cdot \frac{3}{2}\nu\ f \tan\varphi$$

Symbols

M_r : ultimate resisting moment of bulkhead

M_{rf} : ultimate resisting moment of fill material

M_{rs} : ultimate resisting moment of sheet-piling

γ : unit weight of fill material

h : height of bulkhead from the sea bottom

b : width of bulkhead

ν : b/h

ϕ : angle of internal friction of fill material

f : coefficient of friction between locks of sheetpiles (f=0.3)

K_a : Rankine's coefficient of active earthpressure

K_p : Rankine's coefficient of passive earthpressure

K_k : coefficient of earthpressure to be used in the Terzaghi-Krynine expression of resisting moment

Terzaghi .... $K_k = 0.4 \sim 0.5$

Krynine .... $K_k = \dfrac{1 - \sin^2\varphi}{1 + \sin^2\varphi}$

Japan Harbour Engineers' Manual.....

$K = 0.6$ for sandy soil

$K = 0.5$ for clayey soil

The Terzaghi-Krynine expression, which was adopted in the Japan Harbour Engineer's Manual, gives a too conservative value, and Kitajima's formula, which is based on an extensive experimental study, is considered to give more reasonable estimation. So, in the new standards, Kitajima's method is adopted.

Safety factor against shear failure should have the following value.

$F = 2.0 \sim 2.5$ for static condition

$F = 1.0 \sim 1.2$ for seismic condition

The lock tension T given in the following equation should be smaller than the allowable value, which is 250 t/m for the straight-web sheet pile procuded in Japan.

$$T = K_i \gamma h \cdot r \qquad \ldots \quad (18)$$

where, r : radius of cell

K_i : coefficient of earthpressure to be used in the computation of lock tention,

Terzaghi-Krynine ..... $K_i = K_k$

Cummings ...... $K_i = K_a$

Kitajima .... $K_i = \tan\varphi$

10. TRESTLE TYPE PIER

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

The most simple form of trestle type pier is piled pier. Piles supporting super structure are designed as "fixed-head" (cf. 5-1) subjected to lateral forces at the pile head in accordance with the procedure shown in section 5-1 (Fig. 14 (a), (b)). Main lateral forces to be considered in the design are ship impact, pulling force of moored ships, and earthquake force acting on the pier. When piles are braced, pile heads is assumed to be fixed at the lower connecting point with the bracings (Fig. 15).

For the sake of simplicity, computations of piles are sometimes carried out with an assumption that piles are vertical cantilever beams fixed at a certain depth below the ground level. The depth of the point of fixity is assumed to be α h, where h is the height of the pile from the sea bottom and α is a coefficient (Fig. 14 (c)).

Yokoyama[20] recommended the following formula for α, in which both of the characteristics of soil and pile are taken into consideration.

$$\alpha = \frac{1}{\beta h} \qquad \cdots \quad (19)$$

where, $\beta = \sqrt[4]{\frac{E_s}{4EI}}$

EI : flexural rigidity of pile

E_s : elastic modulus of soil

In the design of a pier with cylindrical trestles supported by piles, the trestles and super structure are considered to form a rigid frame with hinged supports at the pile-head. Piles supporting trestles are designed as "fixed-head" piles or "free-head" piles depending on the fixity at the pile-head (Fig. 16).

In the case of a pier which has trestles of coupled piles and trestles of large dimensions such as caissons, pneumatic caissons or wells, the design of trestles is performed following the procedure shown in section 5-2 and 5-3 respectively.

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

11. TRESTLE TYPE PIERS WITH SMALL RETAINING WALL

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

Trestles are designed by the method shown in section 10 and retaining wall is designed in accordance with the design procedure for a similar type quaywall.

In addition to the separate analysis of the each part, the stability of the whole structure including slope and retaining wall should carefully be studied with the procedure shown in section 6. It is desirable not to connect rigidly the main part and the retaining wall taking the possibility of differential settlement into account.

12. CONCLUSION

As stated before, earthquake resistant design of quaywalls and piers are described and discussed in this text. For some of other types of structures connected with port and harbour facilities such as breakwaters, coastal levees and so on, earthquake resistant design has not been neglected because of their structural features.

These are the recent tendency in the earthquake resistant design of harbour structures.

As the basis of the earthquake resistant design, strenuous research works on the earthquake resistant design of harbour structures are being carried out at research organs and universities in Japan. These research activities will be calssified into two categories. One is the adequate application of the advanced knowledge of soil mechanics to the earthquake resistant design of quaywalls which are essentially earthretaining structures. The other is the study of earthquake resistant design in consideration of the structural feature of quaywalls.

In addition to these research activity, it is quite important to know the characteristics of actual seismic forces. Then the observation of strong motion earthquakes has been started at principal ports over the country.

In 1962, eleven strong motion accelerographs of SMAC-B type were set at Yokohama and other ports. SMAC is the abbreviation of Strong Motion Accelerograph Committee. In 1963 another thirteen accelerographs were set at seven ports, four of the accelerographs were of SMAC-B_2 type and the other was newly deviced electro-magnetic type strong mothion accelerpgraphs. Some records of strong motion earthquakes have been obtained already by these accelerographs. In 1964, seventeen SMAC-B_2 type accelerographs and two electro-magnetic type accelerographs were set at fifteen ports. The network for observation of strong motion earthquake over all the harbour districts is to be established in near future.

Looking at the current method of designing, however, many things are found to be left not fully improved. This situation is largely due to the ambiguity of informations about the behaviour of structures in earthquakes. Above all it is urgently wanted to investigate the dynamic properties of soils, since the soil properties have an important bearing on the dynamic behaviours of harbour structures.

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Table -1

Regional Seismic Coefficient

A region	Hokkaido (Nemuro, Kushiro, Tokachi) Kanto (Chiba, Tokyo, Kanagawa) Chubu (Shizuoka, Aichi) Kinki	0.15
B region	Hokkaido (Hidaka, Ishikari, Iburi, Shiribeshi, Hiyama, Oshima, Rumoi) Tohoku Kanto (Ibaragi) Chubu (Niigata, Toyama, Ishikawa, Fukui) Shikoku Chugoku (Tottori, Okayama, Hiroshima)	0.10
C region	Hokkaido (Soya, Abashiri) Chugoku (Shimane, Yamaguchi) Kyushu	0.05

Table - 2

Factor for Subsoil Condition

classification	1st kind	2nd kind	3rd kind
factor	0.8	1.0	1.2

Table - 3

Classification of Subsoil

thickness of quaternary deposit	gravel	sand or clay	soft ground
less than 5 m	1st kind	1st kind	2nd kind
5 $\sim$ 25 m	1st kind	2nd kind	3rd kind
more than 25 m	2nd kind	3rd kind	3rd kind

Table - 4

Importance Factor

degree of importance	I	II	III
factor	1.5	1.0	0.5

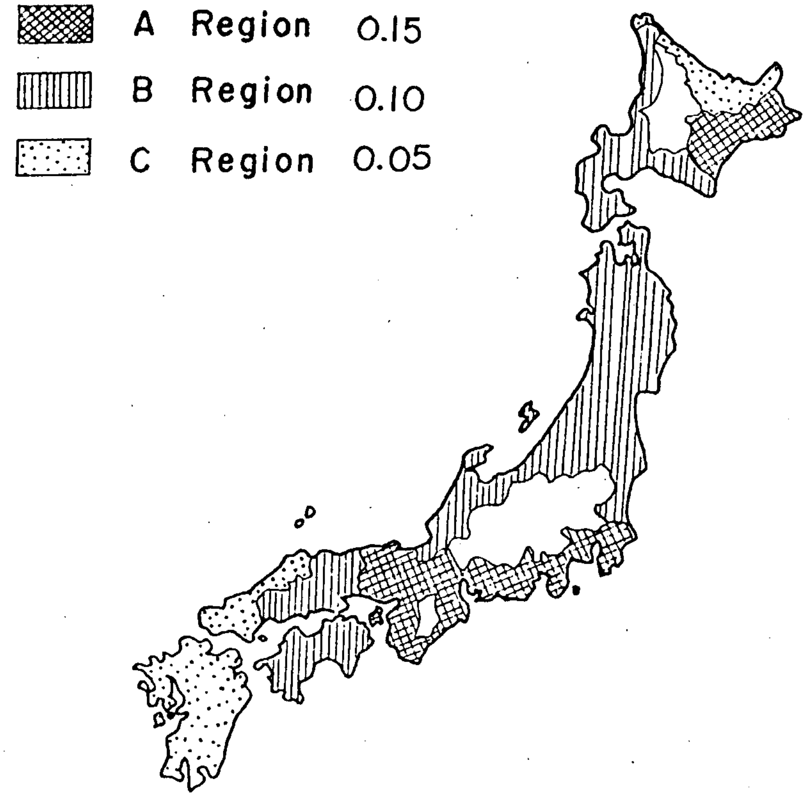


Fig. 1 Seismic Zoning Map for Harbour Structures

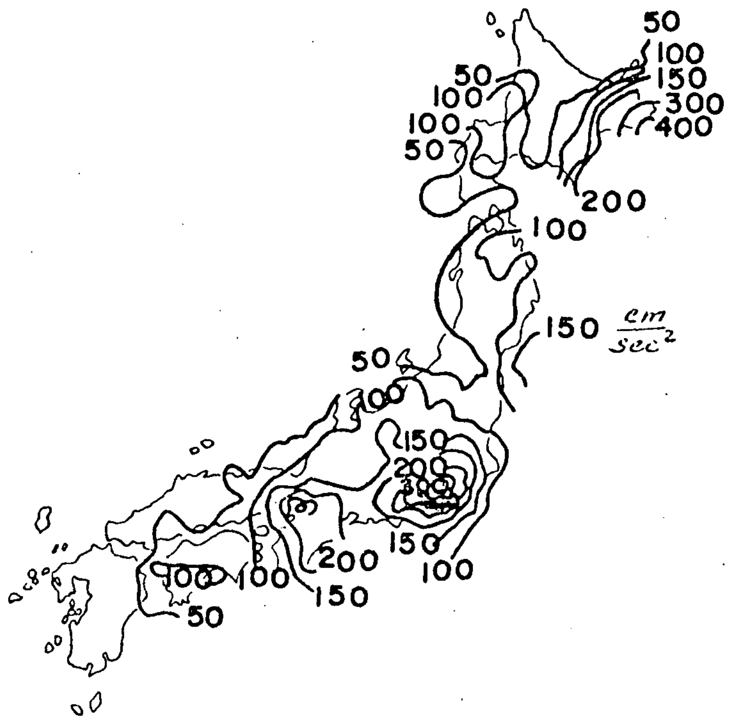


Fig. 2 Expectancy of Maximum Acceleration of Earthquakes in 75 Years (Kawasumi)

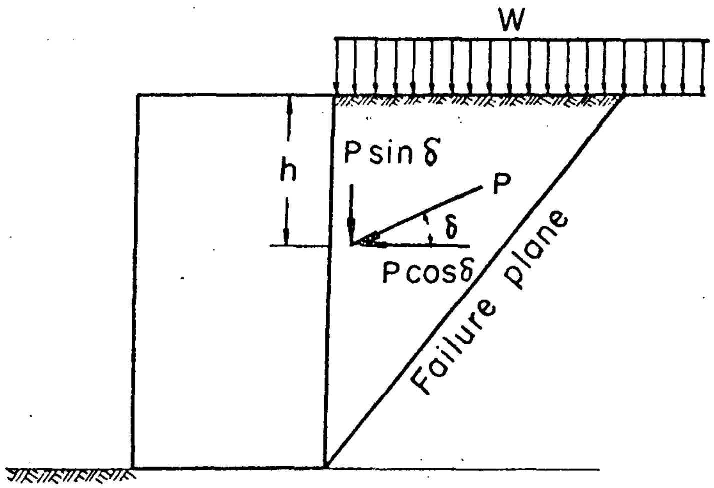


Fig. 3 Earthpressure Acting on a Vertical Wall

For earthquake
Japanese find
(by experiment)
That $\delta < 15°$.

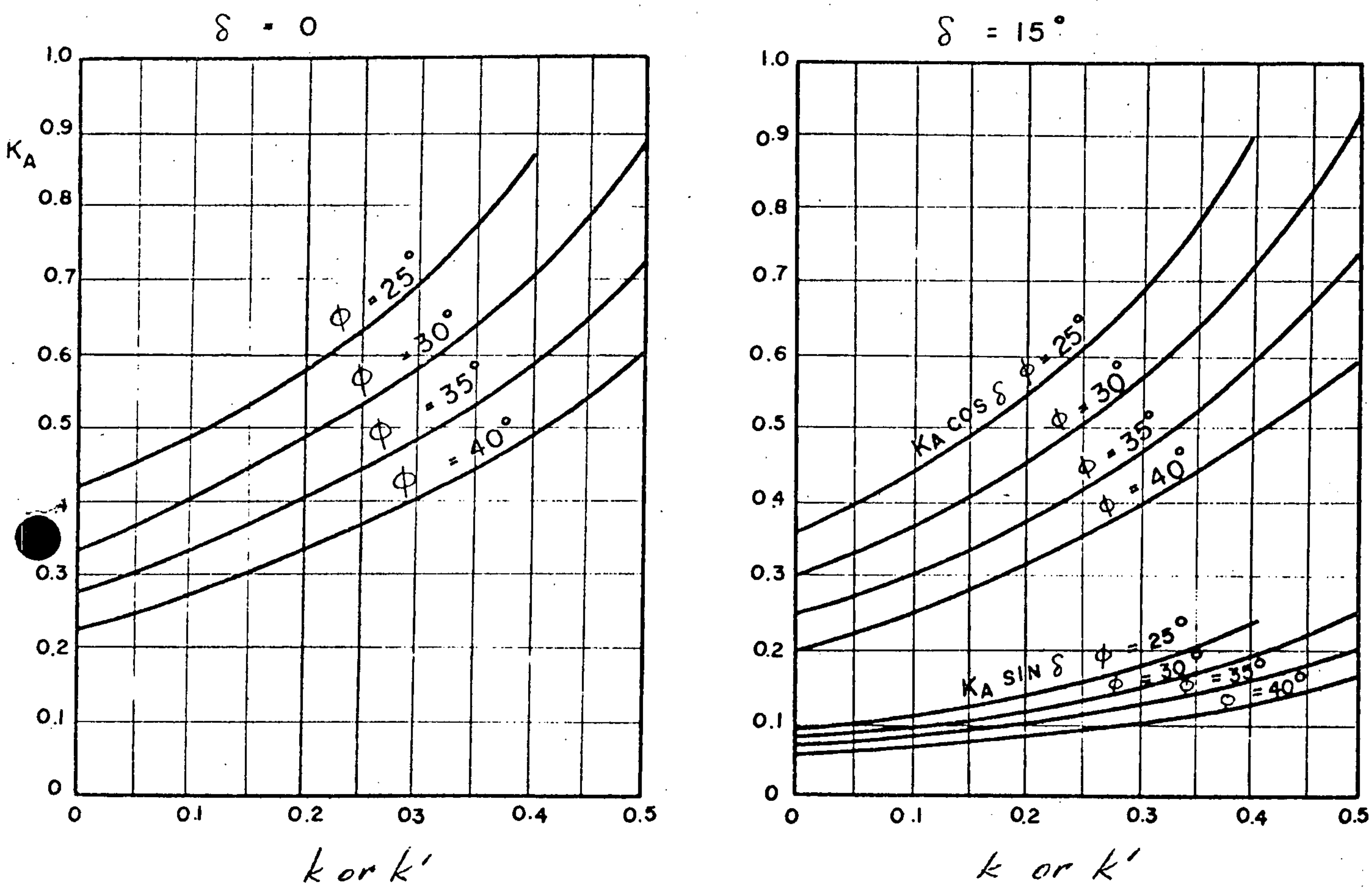


Fig. 4(a) Coefficients of Active Earthpressure

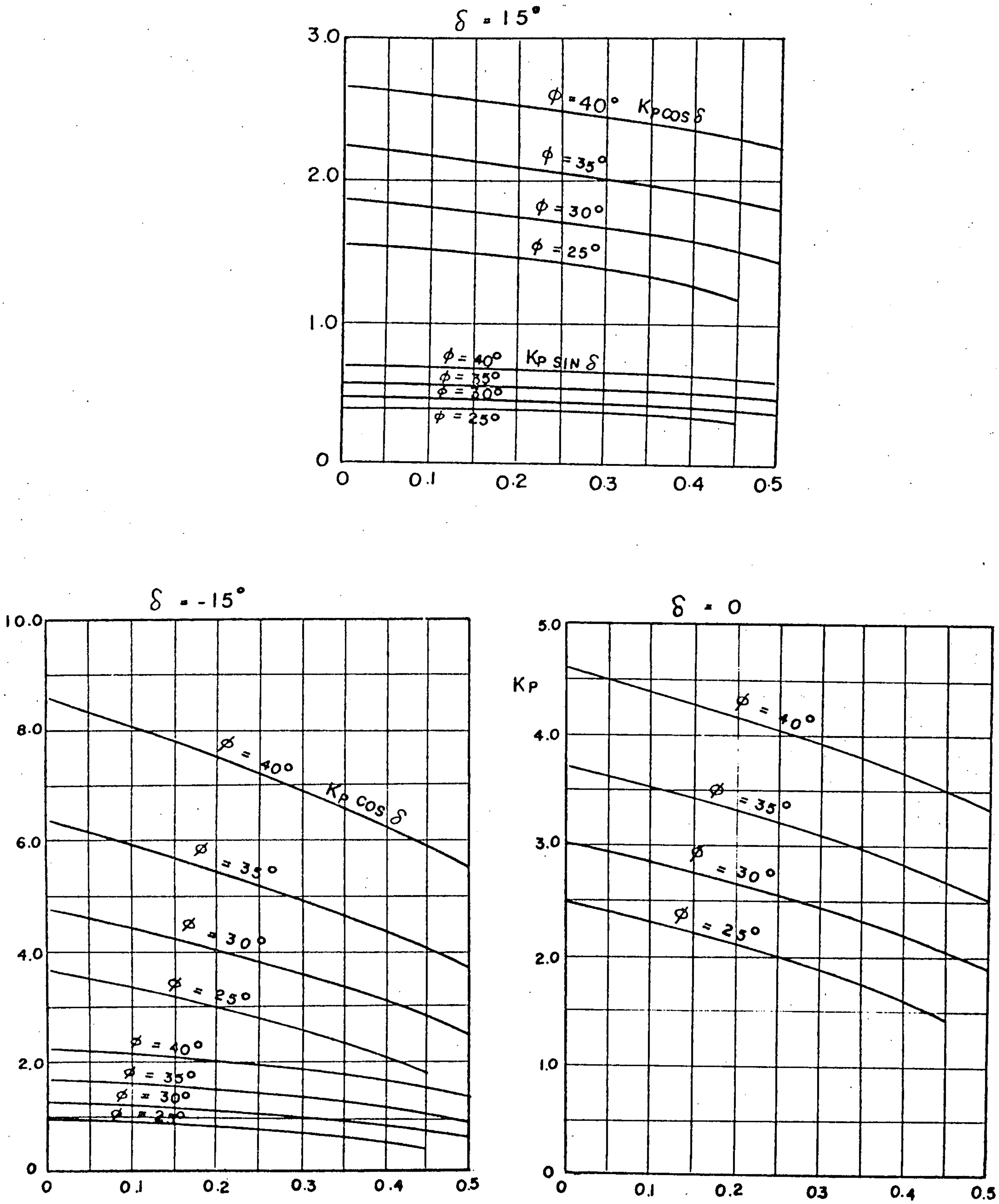
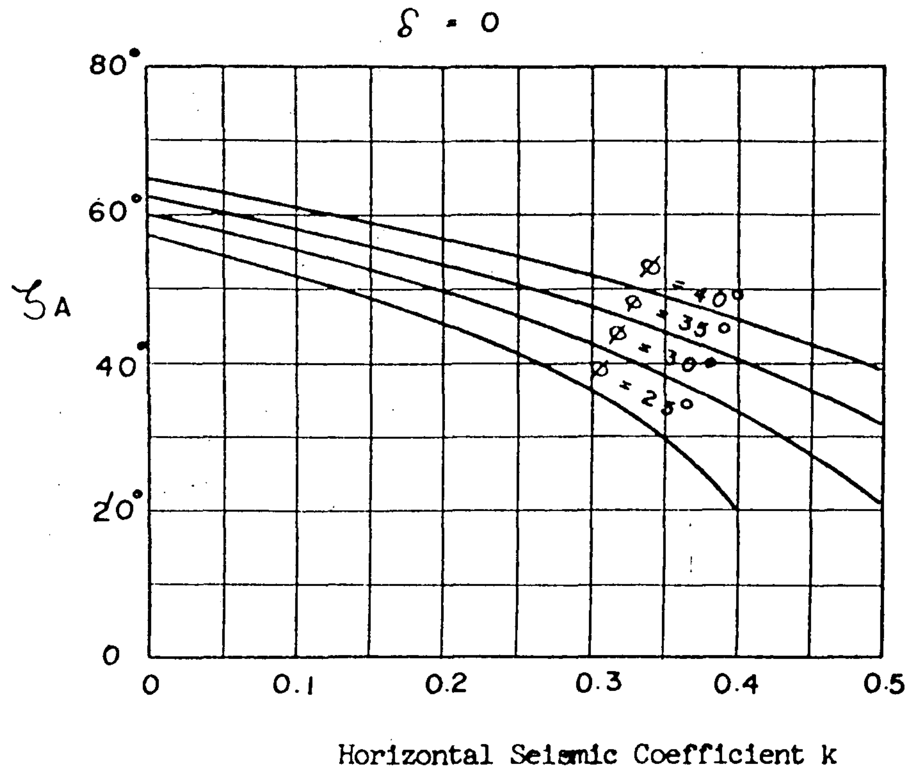


Fig. 4(b) Coefficients of Passive Earthpressure



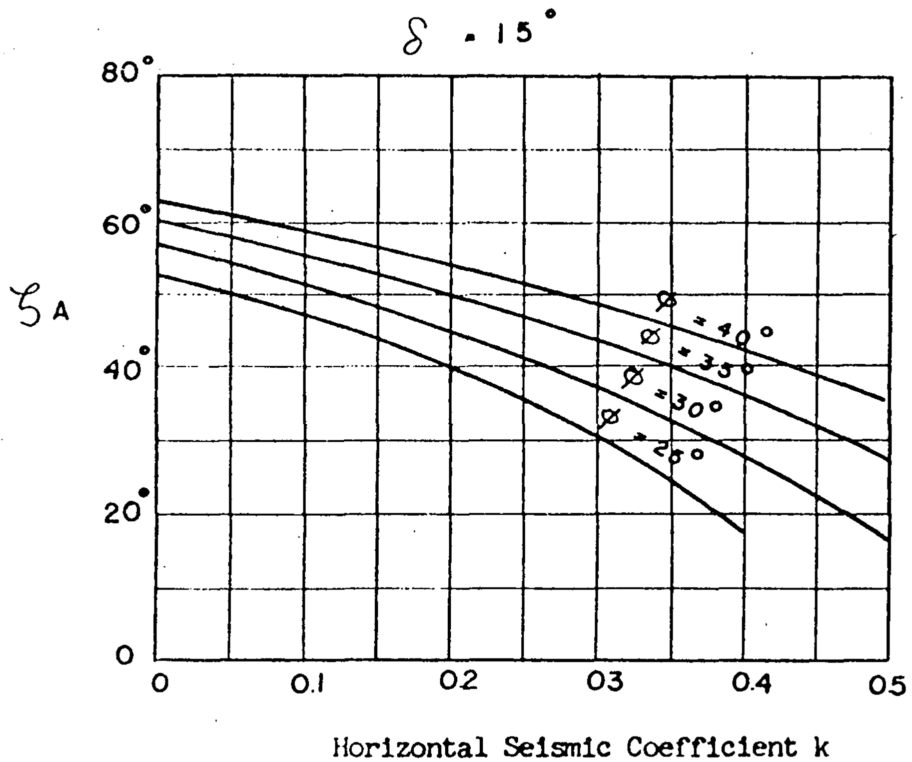


Fig. 5(a) Angles between Failure Surface and Horizon (Active State)

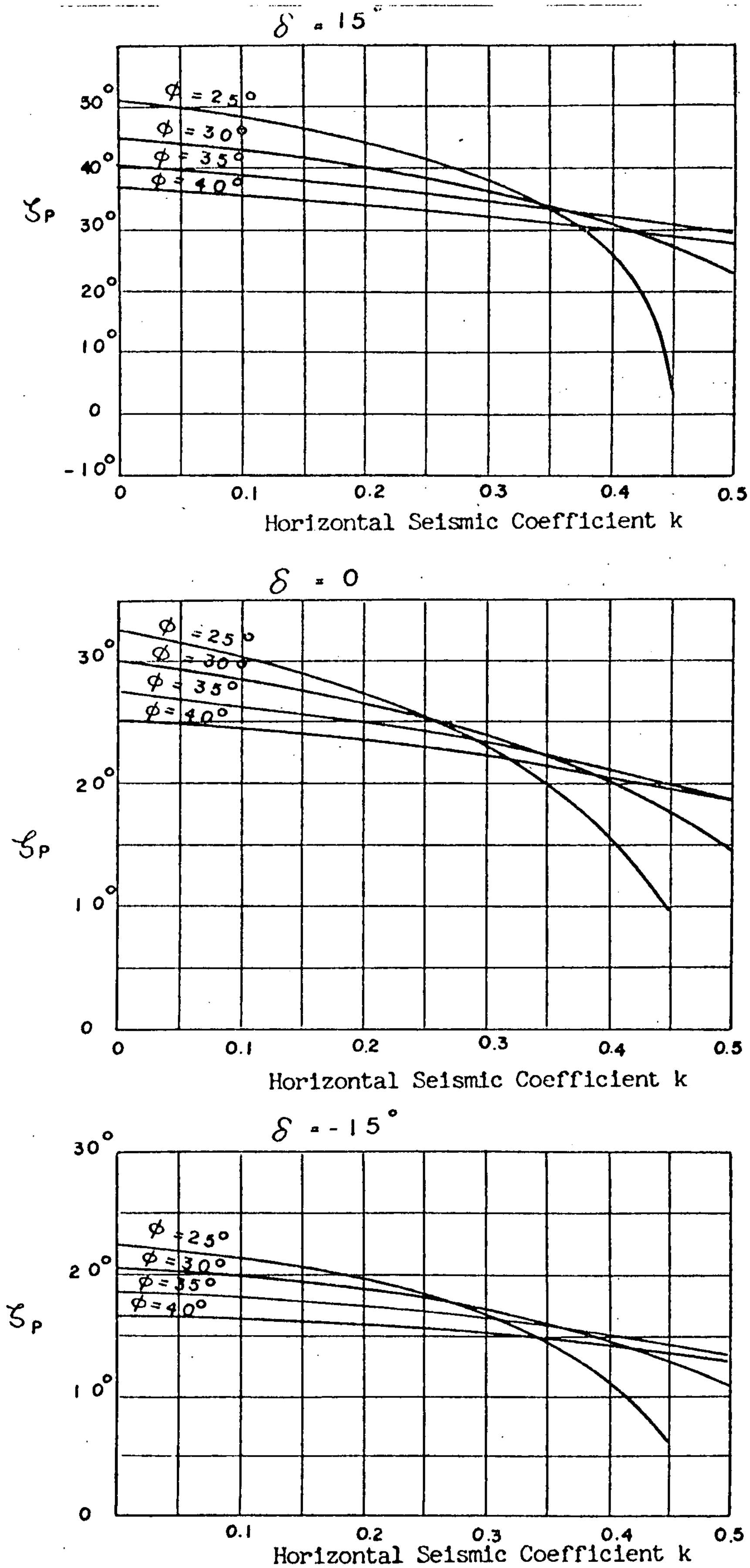


Fig. 5(b) Angles between Failure Surface and Horizon (Passive State)

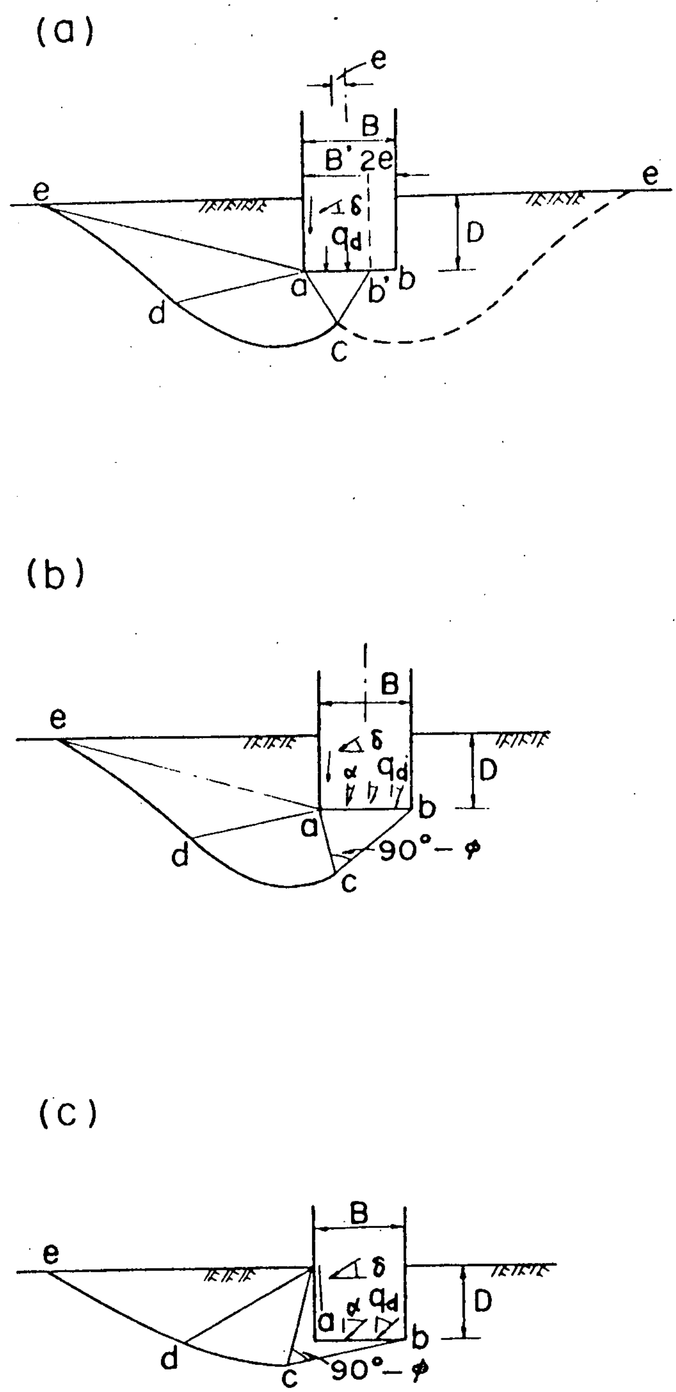


Fig. 6 Key Sketch to Equations (5) and (7) (Meyerhof)

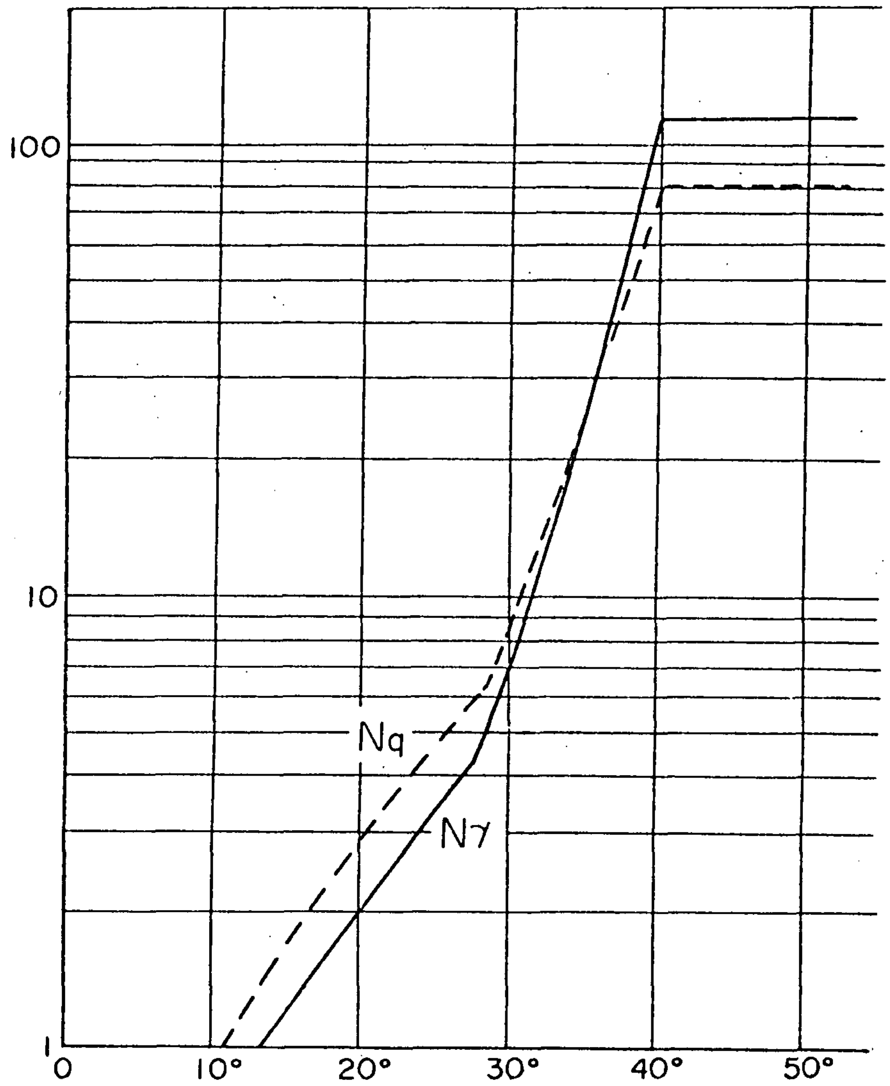


Fig. 7 Bearing Capacity Factor N and N

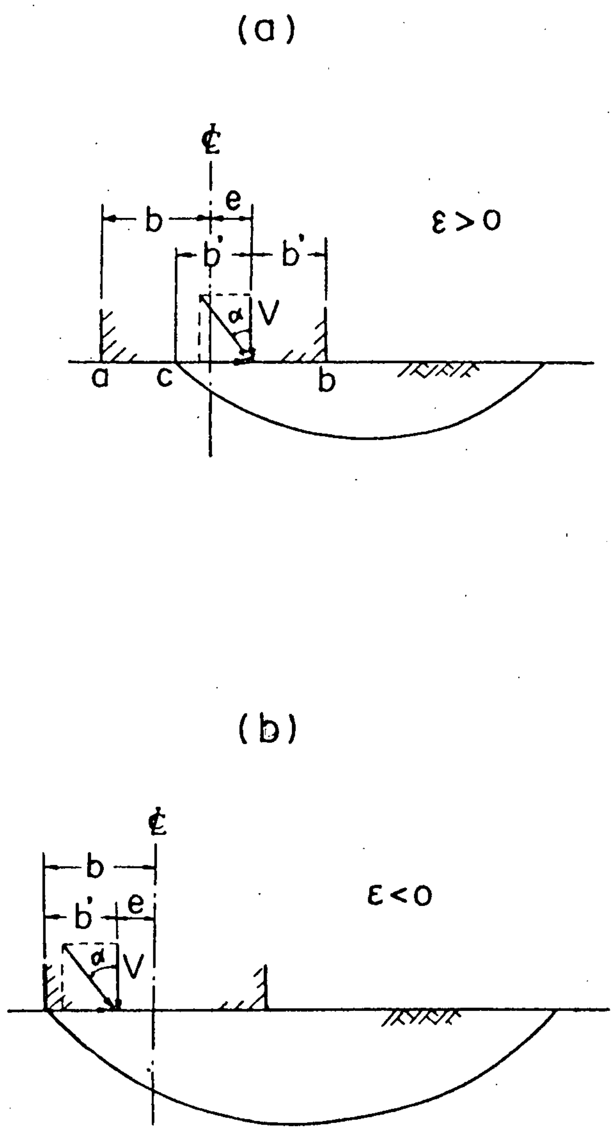


Fig. 8 Key Sketch to Equation (6) (Tateishi)

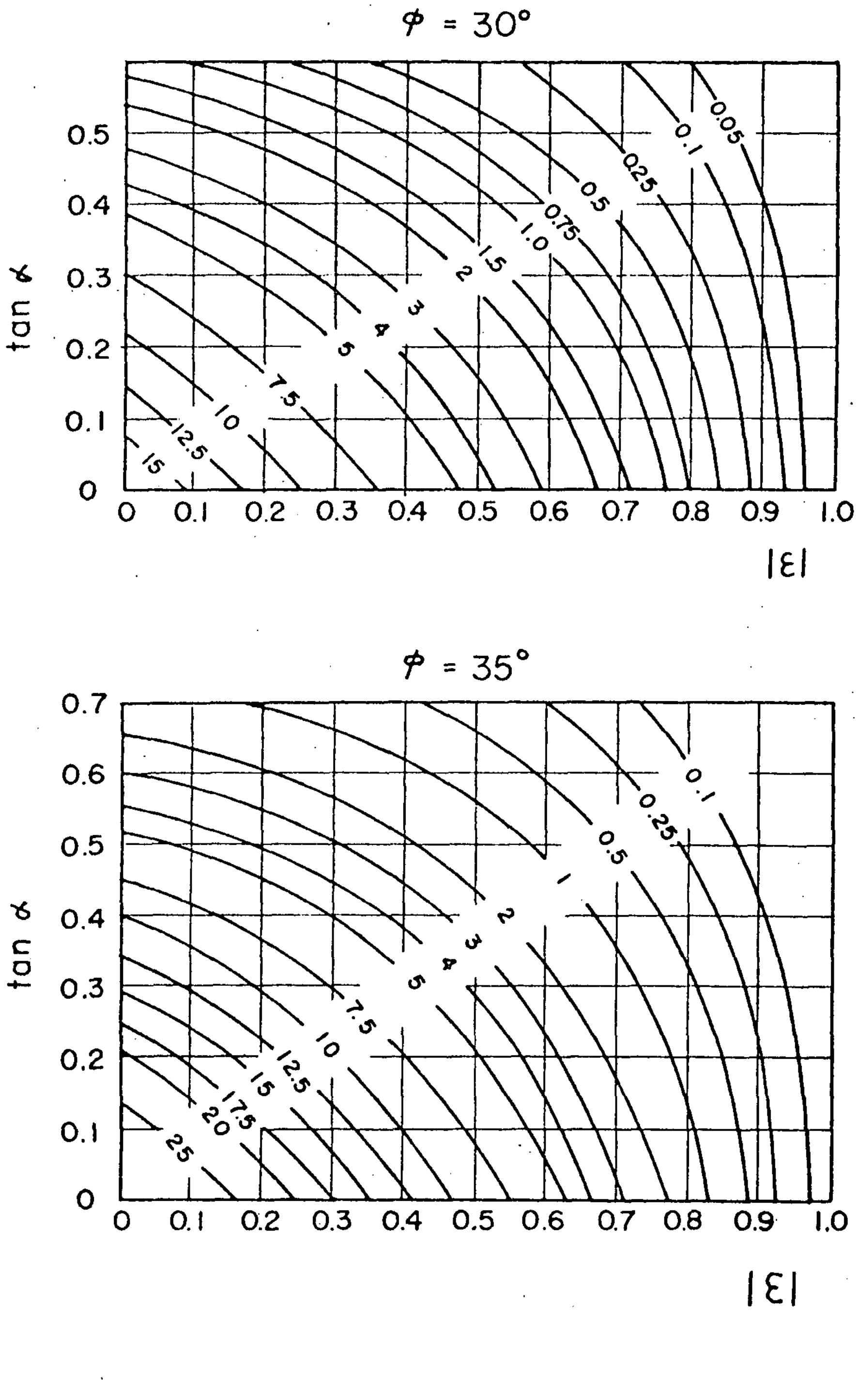
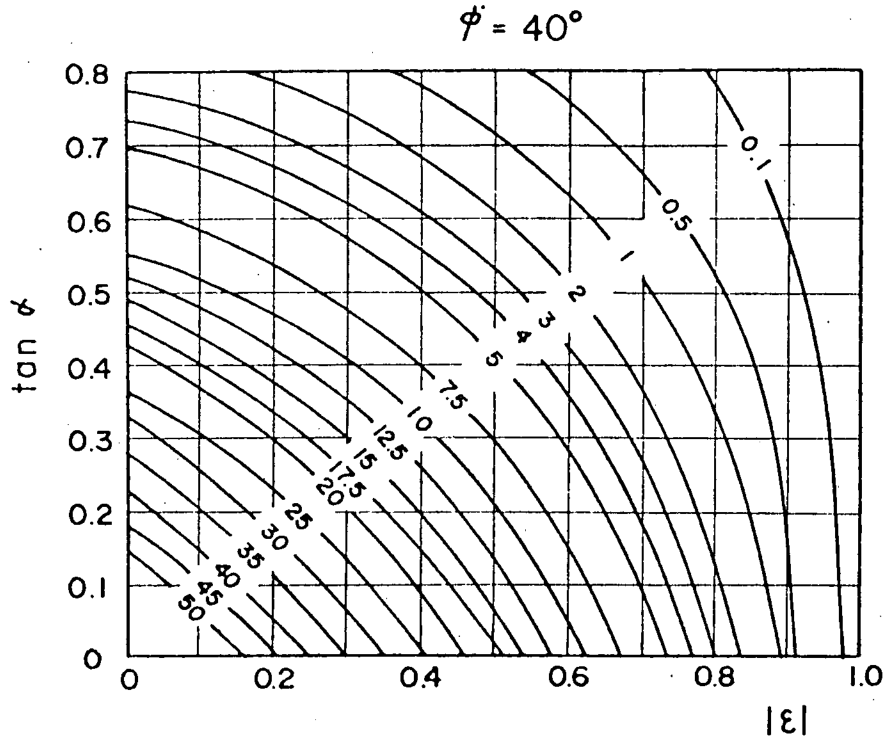


Fig. 9(a) Bearing Capacity Factor N (Tateishi)



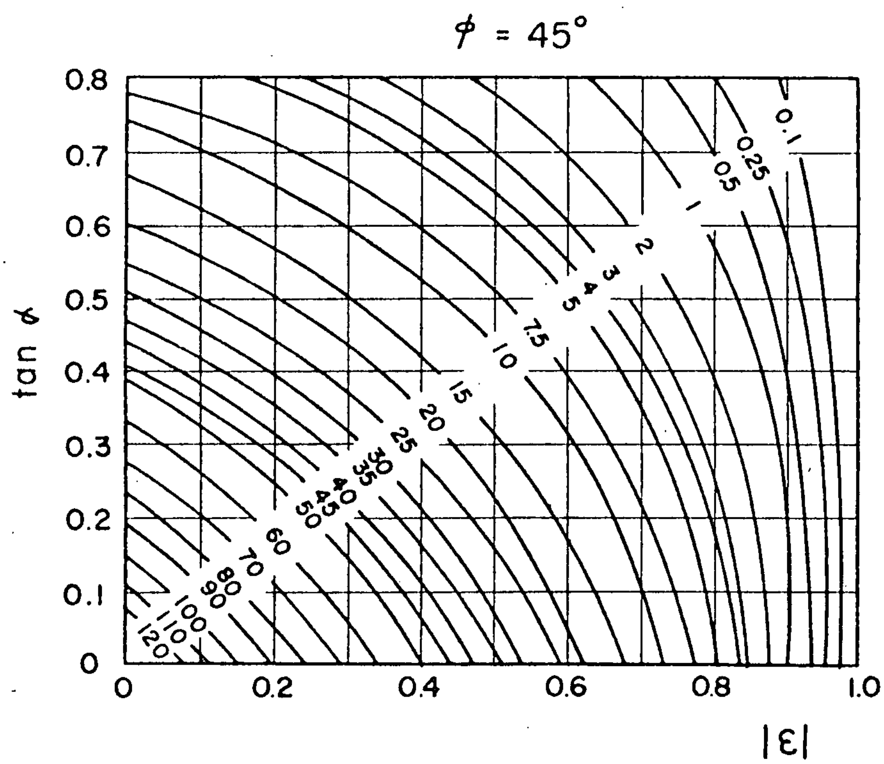
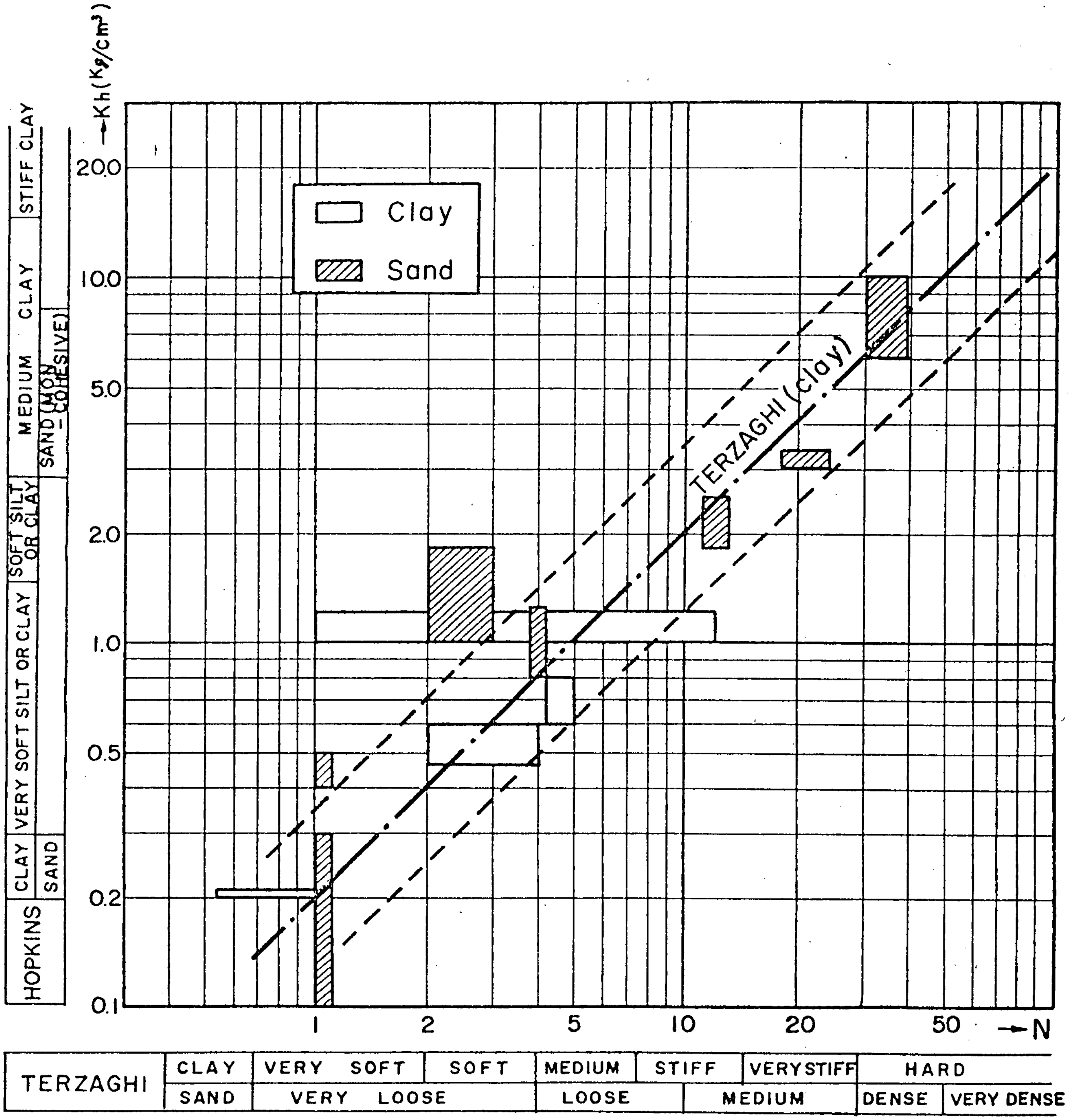


Fig. 9(b) Bearing Capacity Factor N (Tateishi)



N = std. penetration test (no. of blows)

Fig. 10 Relation between k and N (Yokoyama)

Buckling of piles.
Clough & Penzine

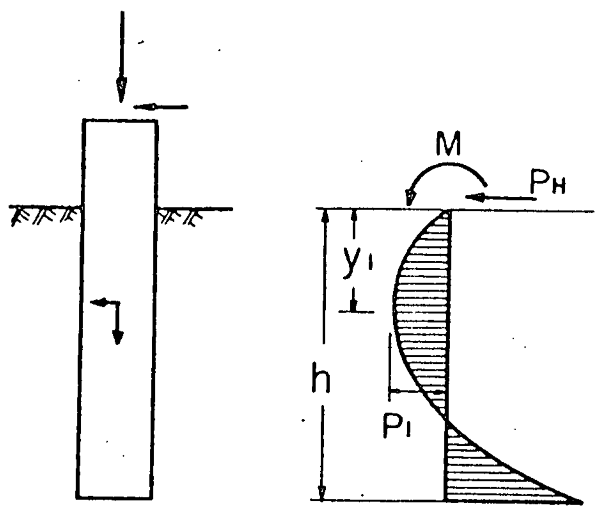


Fig. 11 Lateral Resistance of Well

H = 9.0   W = 3.0

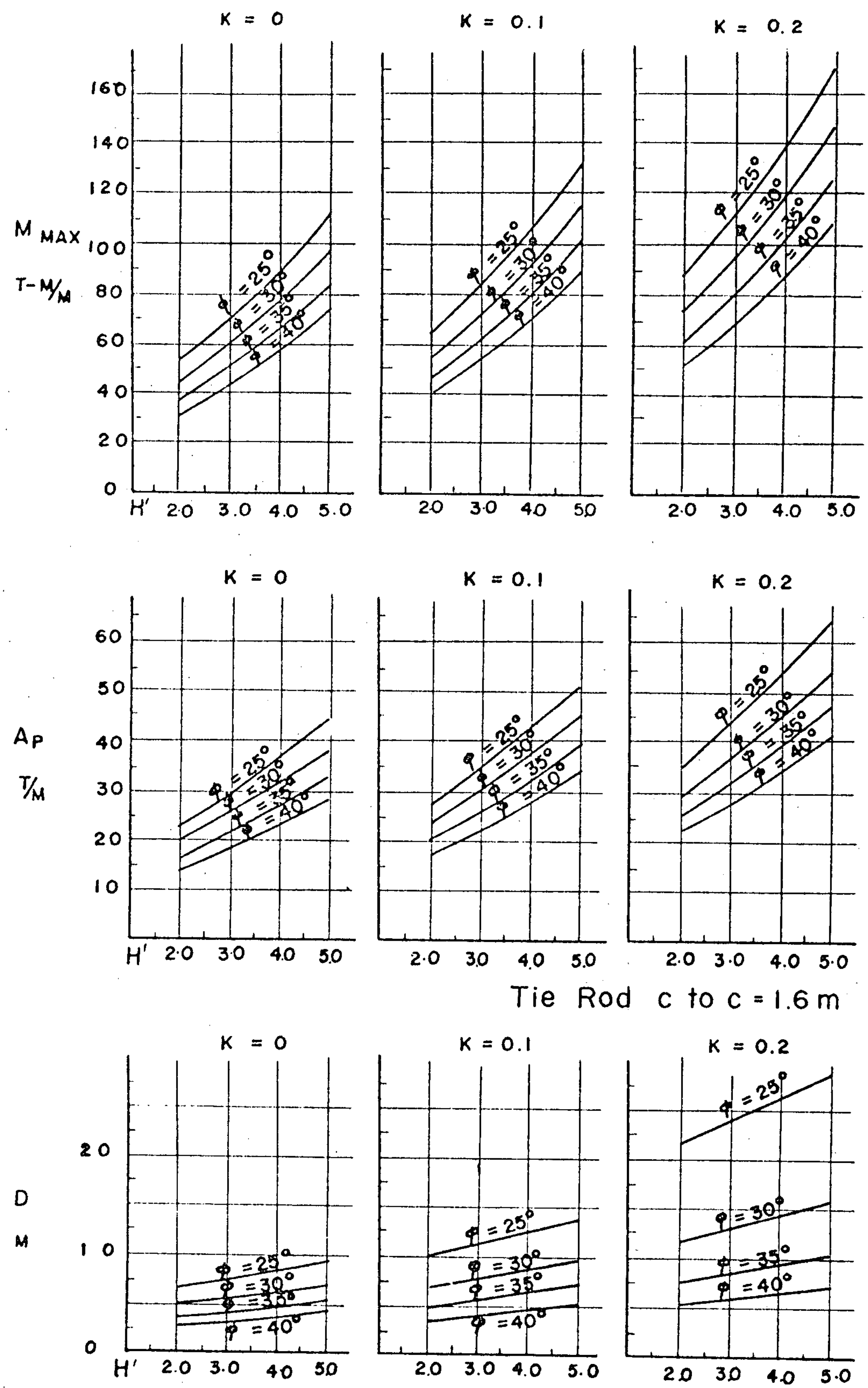


Fig. 12 Design Charts for Sheetpile Bulkheads

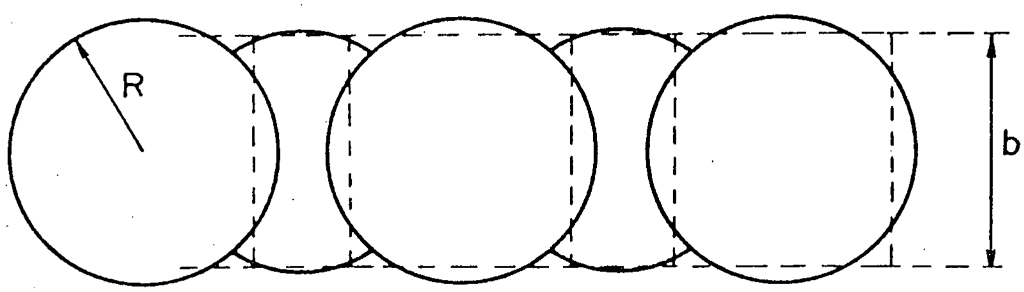


Fig. 13 Plan of Cellular Bulkhead

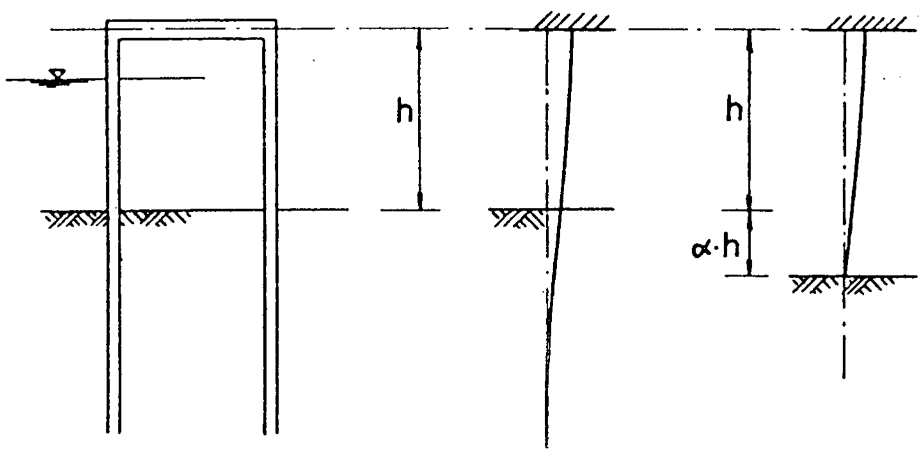


Fig. 14 Piled Pier

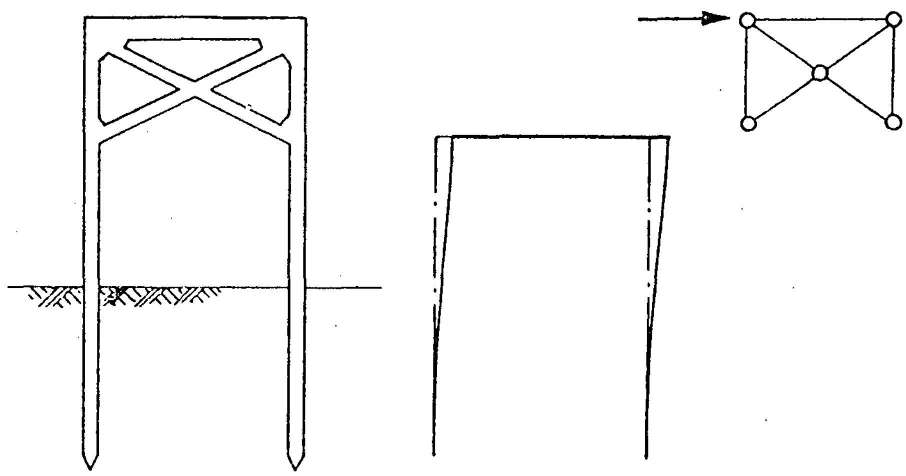


Fig. 15 Piled Pier with Bracings

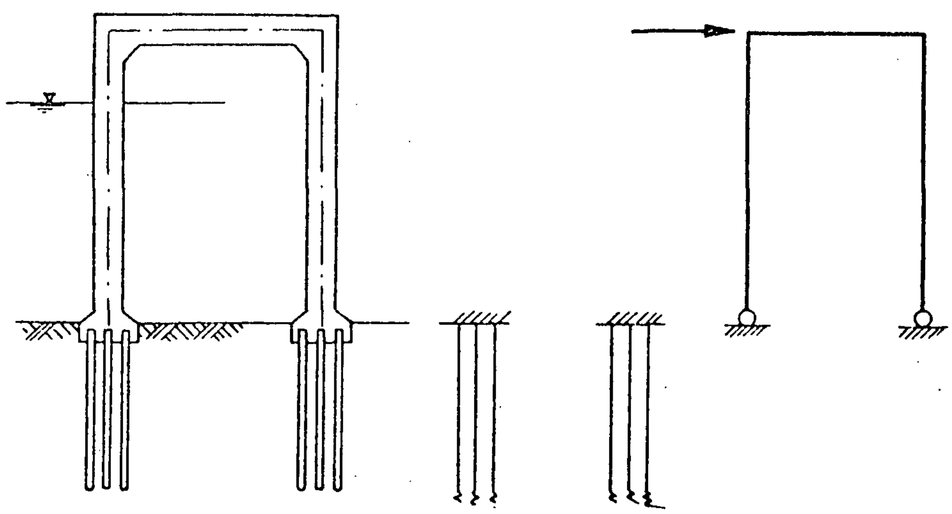


Fig. 16 Pier with Cylindrical Trestles Supported by Piles

COE Question:
(e)

With regard to the stability analyses for the intake channel, the information provided is not sufficient for us to judge the adequacy of the analyses. The applicant should provide descriptions or drawings indicating the forces considered in the analyses, and where appropriate, how they were computed.

Response:

The method used for the wedge analysis of the intake channel is described in NAVDOCKS DM-7 (Design Manual - Soil Mechanics, Foundations, and Earth Structures) issued by the Bureau of Yards and Docks of the Department of the Navy. The forces on the various wedges are computed in the manner shown in figures 7-5 and 7-6 of that publication.

The material presented in Sections 2.5.5.2.1 and 2.5.5.2.2 and the response to question 362.18 should provide sufficient information to verify the extremely conservative nature of the seismic analysis performed for the intake channel. Several areas of conservatism have been identified, primarily in the choice of seismic coefficients and soil resistance along failure planes passing through the zones of liquefied material.

The maximum acceleration level for the SSE at Watts Bar is 0.18 g and is defined as being at top of rock. A finite element analysis of the intake channel side slopes using 0.18 g as input at bedrock showed the maximum acceleration at ground surface to be 0.4 g. The conservative acceleration values, described in Section 2.5.5.2.1, of 0.3 g and 0.4 g for failure planes at elevation 665 and elevation 680, respectively, were applied to the entire respective wedges.

The SSE acceleration level of 0.18 g was selected based on a Modified Mercalli intensity of MM VIII for the Giles County, Virginia, earthquake of 1897 (see section 2.5.2.4 of the FSAR). The MM intensity actually defined ground surface motion. Therefore, the acceleration of

0.18 g could have been defined at the ground surface and a lower value used for motion at top of rock. TVA chose to conservatively use the SSE acceleration at top of rock. If we assume, for discussion purposes, that surface acceleration are approximately one and one-half times bedrock accelerations, the acceleration at top of rock would be 0.12 g for an SSE surface acceleration of 0.18 g. This acceleration distribution would result in accelerations on the wedges previously discussed with failure planes at elevations 665 and 680 of approximately 0.15 g and 0.18 g, respectively. Therefore, the conservative definition of SSE acceleration levels explained above results in inertia forces at least twice those which the less conservative, but acceptable, approach would yield.

In our response to question 362.18, a discussion was included which described the conservatism inherent in our assumption of zero strength along those portions of the failure planes which pass through the zones of liquefied material. In view of the discussion above concerning conservatism of the acceleration levels chosen, the assumption of complete and simultaneous liquefaction of the entire layer of silty sand becomes even more unlikely.

For the reasons discussed above one concludes that the analysis of the intake channel is both reasonable and conservative. That conservatism has continually been stressed in Section 2.5.5 of the FSAR and in the response to question 362.18.