Chapter 4 Earthquakes

Public Notice

Earthquake Ground Motions

Article 16

Level 1 earthquake ground motions shall be appropriately set in terms of the probabilistic time history wave profiles based on actual measurements of earthquake ground motions and by taking into consideration the hypocenter characteristics, the propagation path characteristics, and the site characteristics.

2 Level 2 earthquake ground motions shall be appropriately set in terms of the time history wave profiles based on actual measurements of the earthquake ground motions, scenario parameters of earthquake hypocenters, and/or others, and by taking into consideration the hypocenter characteristics, the propagation path characteristics, and the site characteristics.

[Technical Notes]

1 Ground Motion

1.1 General

The three important factors that affect ground motion are the effect of the rupture process on the fault surface, namely source effects, the effect of the propagation path from the source to the seismic bedrock, namely propagation path effects, and the effect of the sediments on the seismic bedrock, namely site effects (see **Fig. 1.1.1**). Here the seismic bedrock is strata generally made of granite having an S wave velocity of 3km/s or more. The acceleration Fourier spectrum $O(f)$ of the ground motion measured on the ground surface is given in general by the product of the source effects $S(f)$, the propagation path effects $P(f)$, and the site effects $G(f)$.

$$O(f) = S(f)P(f)G(f) \tag{1.1.1}$$

Here f is the frequency. Also, the group delay time $t_{gr}^{O}(f)$ measured on the ground surface is given by the sum of the source effects $t_{gr}^{S}(f)$, the propagation path effects $t_{gr}^{P}(f)$, and the site effects $t_{gr}^{G}(f)$.[1)]

$$t_{gr}^{O}(f) = t_{gr}^{S}(f) + t_{gr}^{P}(f) + t_{gr}^{G}(f) \tag{1.1.2}$$

Here the group delay time is the derivative of the Fourier phase with respect to the angular frequency $\omega = 2\pi f$, having the units of time, and is approximately the arrival time of the frequency component f. In this case the arrival time is the time measured from the start of the time history used in the analysis. The superscripts in equation **(1.1.2)** have the following meanings: O is the actual measured value on site, S is the source effect, P is the propagation path effect, and G is the site effect. The existence of sediments affect both the Fourier amplitude and phase of the ground motion as shown above, but in this part the term used for the effect on the Fourier amplitude, in other words $G(f)$, is “site amplification factors”, and in this part the term used for the effect on the ground motion overall is the “site effects”.

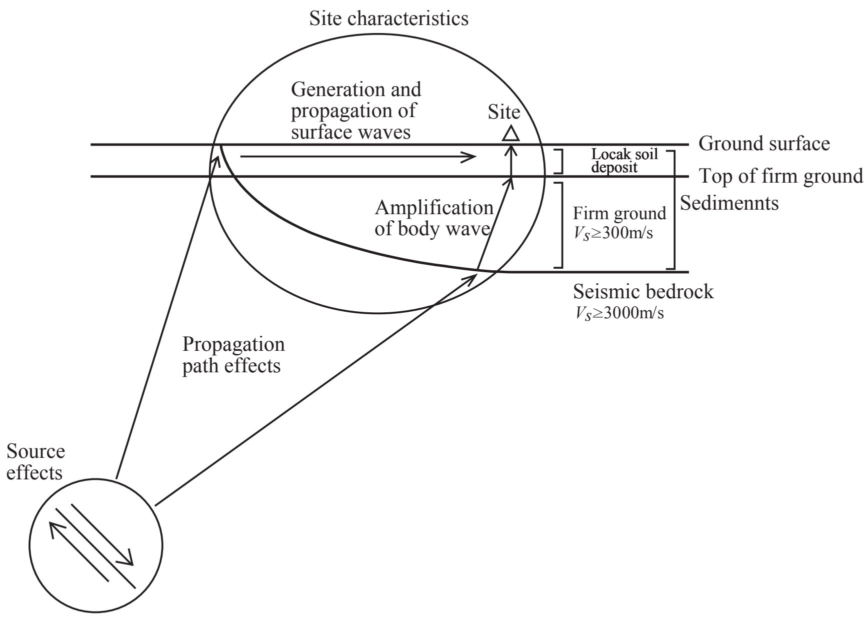


Fig. 1.1.1 Source Effects, Propagation Path Effects, and Site Effects

1.1.1 Source Effects

(1) ω^{-2} Model (Omega Squared Model)

A generally accepted model for the source effects of ground motions is the ω^{-2} model.[2)] In the ω^{-2} model, the acceleration Fourier amplitude spectrum of the seismic wave radiating from the source, the acceleration source spectrum, is expressed by the following equation

$$S(f) = C\frac{M_0}{4\pi\rho V_S^3}\frac{(2\pi f)^2}{1+(f/f_c)^2} \tag{1.1.3}$$

where

M_0 : seismic moment
f_c : corner frequency
ρ : density of the medium of the seismic bedrock
V_s : S wave velocity in the seismic bedrock
C : constant (see equation **(1.3.5)**).

Fig. 1.1.2 illustrates source spectra of the displacement, velocity, and acceleration in accordance with the ω^{-2} model. As can be understood from equation **(1.1.3)** and **Fig. 1.1.2**, the acceleration source spectrum depending on the ω^{-2} model is proportional to the square of the frequency for frequencies lower than f_c, and is flat for frequencies higher than f_c. This corner frequency f_c is the frequency corresponding to the bend in the source spectrum. The seismic moment M_0 is a physical measure to express the size of the earthquake, and is defined by the following equation.[3)]

$$M_0 = \mu A D_0 \tag{1.1.4}$$

where

μ : shear modulus of the rock in the source region
A : area of the source fault
D_0 : average value of the final of slip on the fault surface

On average the corner frequency f_c is inversely proportional to M_0 to the power of 1/3. Therefore in the ω^{-2} model, the Fourier amplitude spectrum of the seismic wave radiating from the source is proportional to the seismic moment on the long period side, and is proportional to the seismic moment to the power of 1/3 on the short period side. Every time the Magnitude is increased by 1, M_0 increases by a factor of about 30, so the long period component of the ground motion radiating from the source, that is proportional to M_0, becomes about 30 times and the short period component that is proportional to M_0 to the power of 1/3, becomes about 3 times. In other words, as the magnitude of the earthquake increases, the long period component increases most of all. When analyzing long period structures, such as high rise buildings, long span bridges, oil tanks, base isolated structures, etc. that are easily affected by the long period component of ground motions, it is particularly necessary to pay attention to large Magnitude earthquakes.

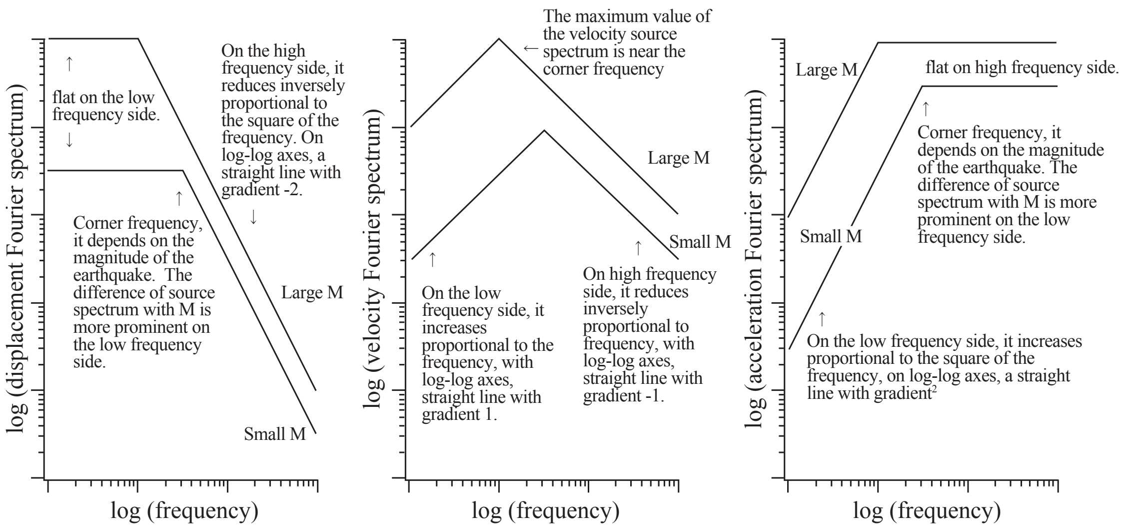


Fig. 1.1.2 Displacement, Velocity, and Acceleration Source Spectra Depending on the ω^{-2} Model

(2) Directivity

The source of a large earthquake is not a single point, but is a fault surface having a definite extent of spread. Rupture starts at a point on the fault surface, and spreads to the surroundings. At this time, the S wave velocity in the source region and the rupture propagation velocity are about the same, so at a harbor in the direction of propagation of the rupture, the energy of the seismic waves successively released from the fault surface arrive at about the same time, so the amplitude becomes large. This phenomenon is referred to as the directivity of the ground motions.

Associated with this, it is known that in the areas where the amplitude is large as a result of the effect of directivity, it has been reported that the oscillations in the direction normal to the direction of strike of the fault tend to be strong.[5), 6), 7), 8)]

(3) Asperities

It is known that the slip on the fault surface of a large earthquake is not uniform, but non-uniform. The area on the fault surface where the slip is particularly large is referred to as an asperity. Models that express the non-uniform distribution of the final slip on the fault surface include the variable slip model, which expresses of final slip by a continuous function, and the characterized source model which arranges several rectangular asperities on the fault surface, and within these asperities the amount of slip is uniform.

1.1.2 Propagation Path Effects

The effect of propagation path on the amplitude of ground motion is frequently given by the combination of attenuation (1/*r*) as the wave spreads from the source in a spherical form, and inelastic damping. The following expresses this in the form of an equation

$$P(f) = \frac{1}{r}\exp\left(-\pi f r / Q V_S\right) \tag{1.1.5}$$

where

r : distance from the source
Q : Q value on the propagation path.

The Q value is a quantity expressing the magnitude of inelastic damping caused by scattering and conversion to heat of the seismic wave on the propagation path. The larger the value of Q, the smaller the inelastic damping on the propagation path. It is necessary to be aware of situations where geometric attenuation in the form above does not apply due to the effect of L_g waves, a type of seismic wave propagated by reflection within the earth's crust at a distance from the source.[12)]

1.1.3 Site Effects

The sediments near the ground surface, see **Fig. 1.1.1**, have a large effect on the amplitude of the seismic waves, period characteristics, duration, etc. The effect of the sediments is referred to as the site effects.

1.1.4 Nonlinear Behavior of Local Soil Deposit

Normally the properties of local soil deposit vary with the level of applied strain, and when strong ground motions are acting, the shear modulus reduces, and the damping coefficient increases. This phenomenon is referred to as the nonlinear behavior of the local soil deposit.

1.2 Level 1 Earthquake Ground Motions used in Performance Verification of Facilities

The Level 1 earthquake ground motion is normally set using a probabilistic seismic hazard analysis taking into consideration the source effects, the propagation path effects, and the site amplification factors between the seismic bedrock and the top of firm ground. The set ground motion is a wave whose amplitude is double that of the seismic wave incident on the top of the firm ground from below (2E wave).[14] In the probabilistic seismic hazard analysis, if a probabilistic Green function method is used to evaluate the ground motion for each expected earthquake, it is desirable that the site amplification factors estimated from earthquake observation records obtained at the harbor, or seismic observation records obtained from K-NET, [15] KiK-net, [16] or other seismic networks, near the harbor, within 2km of the harbor, are used as the site amplification factors, after confirming by microtremor measurements that the ground motion characteristics at the observation point do not differ greatly from those at the location of the facilities. If such site amplification factors cannot be used, it is desirable that short term seismic observations are made at the harbor, see **ANNEX 3 Evaluation of Site Amplification Factors (1)**, and the site amplification factors are evaluated using the method stated in **ANNEX 3 Evaluation of Site Amplification Factors (3)**. If these seismic observations cannot be made due to the imminent construction period, etc., the site amplification factors of the harbor may be estimated from the site amplification factors of nearby observation points, using empirical relationships. However, it is necessary to be aware that the evaluation accuracy of the ground motion in this case is greatly reduced compared with estimates based on the seismic observations.

1.3 Level 2 Earthquake Ground Motions used in Performance Verification of Facilities

1.3.1 Outline

The Level 2 earthquake ground motion is mainly set to determine whether the seismic resistance is at a rational level from the viewpoint of safety of the public, and is the must damaging ground motion among the estimated ground motions at the site from scenario earthquakes. The Level 2 earthquake ground motion is normally set by a strong motion evaluation taking into consideration the source effects, the propagation path effects, and the site amplification factors between seismic bedrock and top of firm ground. The term “safety of the public” used here is a concept that includes maintenance of the function of facilities that are necessary for emergency measures after an earthquake, and is a broader concept than “safety”, which is a concept in contrast to “usability” or “reparability”. The set ground motion is a so called 2E wave having double the amplitude of the seismic wave incident on the top of the firm ground from below.[14] If probabilistic Green functions are used in the strong motion simulation, it is desirable that site amplification factors estimated from earthquake observation records obtained at the harbor, or earthquake observation records obtained from observation points near the harbor, within 2km of the harbor, such as K-NET, [15] KiK-net, [16] or other networks, are used as the site amplification factors, after confirming using microtremor measurements that the ground motion characteristics at the observation points do not differ greatly from those at the facility location. If these site amplification factors cannot be used, it is desirable that short term seismic observation, see **ANNEX 3 Evaluation of Site Amplification Factors (1)**, be carried out at the harbor, and the site amplification factors are evaluated by the method described in **ANNEX 3 Evaluation of Site Amplification Factors (3)**. If seismic observations cannot be carried out due to the imminent start of construction, for example, the site amplification factors at the harbor may be estimated from the site amplification factors at nearby observation points, using empirical relationships. However, in this case it is necessary to be aware that the evaluation accuracy of the ground motions is greatly reduced compared with estimates based on the seismic observations. The procedure for calculating the Level 2 earthquake ground motion is shown in **Fig. 1.3.1**.

The evaluation results of the ground motion from the method described below and the evaluation results of the ground motion by another organization assuming a similar scenario earthquake may not be the same, but this is mainly caused by differences in the method of evaluating the site effects. The following method may be used for calculating the ground motion for seismic performance evaluation of harbor facilities.

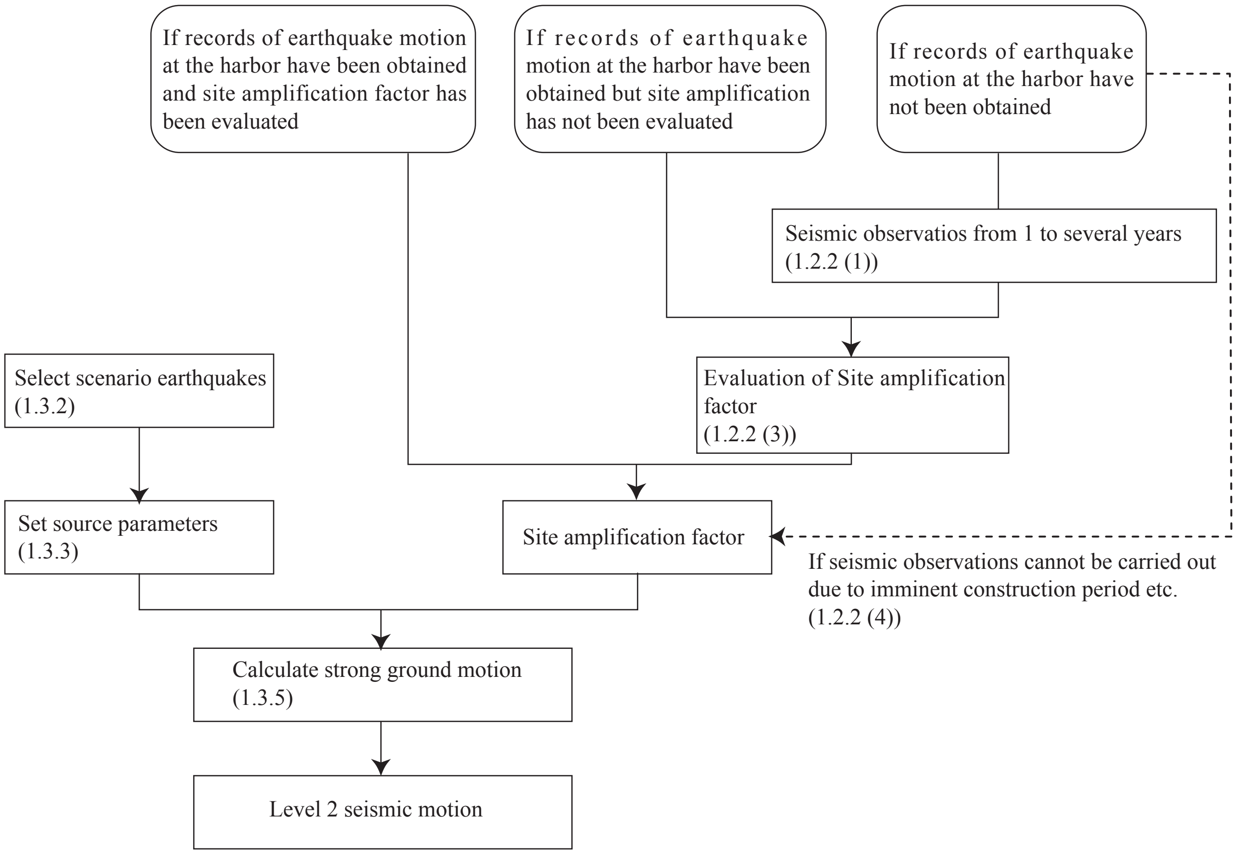


Fig. 1.3.1 Procedure for Calculating the Level 2 Earthquake Ground Motion

1.3.2 Scenario Earthquakes for the Level 2 Ground Motion

It is necessary to select the scenario earthquake for the level 2 ground motion comprehensively taking into consideration information on past earthquakes and information on active faults. In particular, at the time of performance verification, the active faults should be based on the latest survey results. Regarding past earthquakes, references 53) and 54) are comprehensive documents. Reference 35) is a document that summarizes the fault parameters for the main past earthquakes. References 33) and 34) are comprehensive documents regarding active faults. In addition to these, after the 1995 Hyogo-ken Nambu Earthquake, active faults were surveyed, and the results were made public by the Headquarters for Earthquake Research Promotion and local governments. By referring to the above documents, the following should be considered:

(a) The recurrence of earthquakes that have caused significant damage in the past

(b) Earthquakes due to the activity of active faults

(c) Other earthquakes for which there is a concern over occurrence from a seismological or geological viewpoint

(d) Earthquakes postulated by the national organizations such as the Central Disaster Prevention Council and the Headquarters for Earthquake Research Promotion

(e) Earthquakes postulated in the local disaster plans

(f) M6.5 earthquakes [55)]

There may be some duplication within (a) to (f). From these, scenario earthquake for the level 2 ground motion should be selected as the earthquake capable of inducing the most damaging ground motion at the harbor. It can be difficult to decide which of the postulated earthquakes in (a) to (f) above can induce the most damaging ground motion at the harbor. For example, deciding which of a nearby comparatively small earthquake or a distant comparatively large earthquake can induce the most damaging ground motion at the harbor is not necessarily easy. Also, ground motions have various aspects, such as amplitude, frequency characteristics, duration etc., so determining which earthquake has the largest effect on a facility is sometimes only known after first evaluating the ground motions, and then carrying out seismic response analysis. Therefore, it is not necessary at this stage to make great efforts to shortlist the scenario earthquakes to a single earthquake, but several candidate earthquakes should be selected. In this case, the ground motion that has the greatest effect on the facility based on the results of the seismic response analysis will ultimately become the level 2 earthquake. When the number of earthquakes to be considered is large, one method is to carry out in advance a simple evaluation of the ground motions using attenuation equations, and eliminate earthquakes whose effect is clearly small. For the earthquakes postulated in (d), refer to the following homepages:

Central Disaster Prevention Council: http://www.bousai.go.jp/jishin/chubou/index.html

Headquarters for Earthquake Research Promotion: http://www.jishin.go.jp/main/p_hyoka02.htm

The reasons for considering M6.5 right below earthquakes are as follows.[55] An active fault is the trace of an earthquake fault, referred to as a surface fault trace, that has appeared in the ground surface due to a large earthquake in the past. However, in the case of comparatively small scale earthquakes, surface fault trace do not appear, so even in locations where there is no active fault, there is the possibility of occurrence of a comparatively small scale earthquake. Takemura et al.[56] investigated the relationship between the scale of an earthquake and the probability of appearance of surface fault trace, and the relationship between the scale of an earthquake and the extent of damage, [32] see **Fig. 1.3.2**, for earthquakes within the earth's crust of M>5.8 occurring in Japan between 1885 and 1995. According to their results, earthquakes of M<6.5 have a very low probability of appearance of surface fault trace, but earthquakes of M≥6.8 have a probability of appearance of surface fault trace of nearly 100%. Also, focusing on the fact that earthquakes of M = 6.6 and 6.7 are very few, it is inferred that this is because the earthquake fault penetrates to the ground surface. Therefore it is considered appropriate that the scale of the earthquake postulated at locations where there is no active fault should be about M6.5.

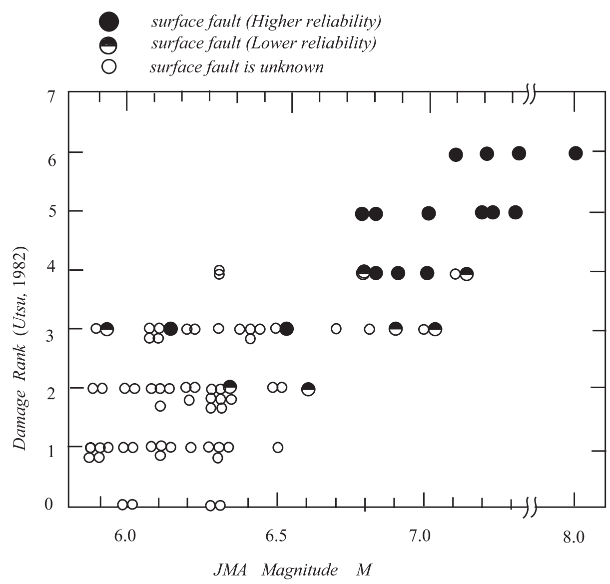
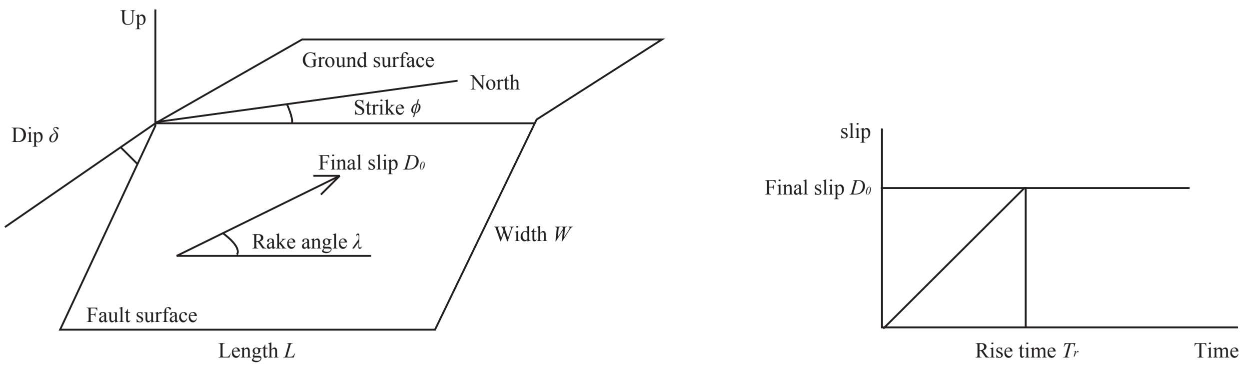


Fig. 1.3.2 Relationship between Scale of Earthquakes and Probability of Appearance of Surface Faults [56]

Among harbor facilities, there are some for which it is required that a tsunami be expected following the ground motion, and the performance in these circumstances is prescribed. In this case, the ground motion to be combined with the tsunami does not necessarily have to be the most damaging ground motion, i.e., Level 2 earthquake ground motion, expected for the harbor. For example, at a certain harbor, both an earthquake at an active fault on land and an subduction-zone earthquake may be expected, and it may be expected that the earthquake at the active fault on land will bring the most damaging seismic motion. In this case, a tsunami does not accompany the earthquake at the active fault on land, so it is not rational to expect that immediately after the ground motion of the earthquake at the active fault on land, a tsunami will attach, and this would result in excessive investment. Therefore, there may be situations where it is necessary to evaluate the ground motions that precede a tsunami, apart from the level 2 earthquake motion. In this case the method of evaluating the ground motions may be to simply change the earthquake from that for the level 2 earthquake motion to the earthquake that is the cause of the tsunami, and apply the following evaluation method as it is.

1.3.3 Setting the Source Parameters

The source parameters necessary for evaluating the Level 2 earthquake ground motion include macroscopic hypocenter parameters, such as position of the base point, strike, dip, length, width, area and seismic moment, microscopic source parameters, such as number of asperities, area of asperities, seismic moment of asperities and rise time, etc., and other parameters, such as rupture starting point, rupture velocity and rupture propagation type. The meaning of these parameters is shown in **Fig. 1.3.3**. The source parameters may be set in accordance with the standard method of setting the parameters shown below, or they may be set by carrying out a separate detailed survey.



(a) Explanation of fault parameters (1) (b) Explanation of fault parameters (2)

Fig. 1.3.3 Meaning of Source Parameters

(1) When the Recurrence of an Earthquake that has Caused Significant Damage in the Past is Expected

If the recurrence of an earthquake that has caused significant damage in the past is postulated, such as the Tonankai and Nankai earthquakes, it is desirable that documents concerning the earthquakes that have actually occurred in the past, referred to as past events, be used as much as possible.

Regarding the macroscopic source parameters, if parameters of the past events are known, those parameters may be used. The macroscopic source parameters of many past earthquakes are contained in Reference **35)**. When only one of the seismic moment M_0 and the fault area S is given and the other must be estimated, the following equation [57), 58)] may be used. By combining equation **(1.3.1)** and Esherby's equation for a circular crack, [59)] the average stress drop for the entire fault surface is 3MPa.

$$S(\mathrm{km}^2) = 1.88 \times 10^{-15} \times M_0^{2/3}(\mathrm{dyne \cdot cm}) \tag{1.3.1}$$

Regarding the microscopic source parameters such as asperity location, etc., an appropriate approach should be taken depending on the volume of data for the past events. Firstly, if the microscopic source parameters for the past events have been investigated well using wave profile data, etc., those parameters may be used. For example, this is the case when considering the recurrence of the 1923 Kanto Earthquake, [60)] recurrence of the 1968 Tokachi Oki Earthquake, [44)] or recurrence of the 1978 Miyagi Ken Oki Earthquake.[44)] Next, if wave profile data from the past event is not available, and if the earthquake intensity distribution is known from historical documents, microscopic source parameters set to be consistent with this earthquake intensity information may be used. For example, this is the case when considering the recurrence of the Hoei Earthquake, the Ansei Tokai Earthquake, or the Ansei Nankai Earthquake. As an example of the microscopic source parameters defined so as to be compatible with the earthquake intensity distribution, there are the microscopic source parameters for the expected Tonankai and Nankai earthquakes the Central Disaster Prevention Council, see **Fig. 1.3.4**.

The other parameters such as rupture start point etc. are dealt with in the same way as the microscopic source parameters.

In the case of an earthquake occurring at an active fault, the average interval between activities is long, so in almost all cases it is not possible to refer to the past events. However, as an exception, if expecting a recurrence of the 1995 Hyogo-ken Nambu Earthquake or similar, the above consideration may be used, without using **(2) When an Earthquake is Expected to Occur at an Active Fault**.

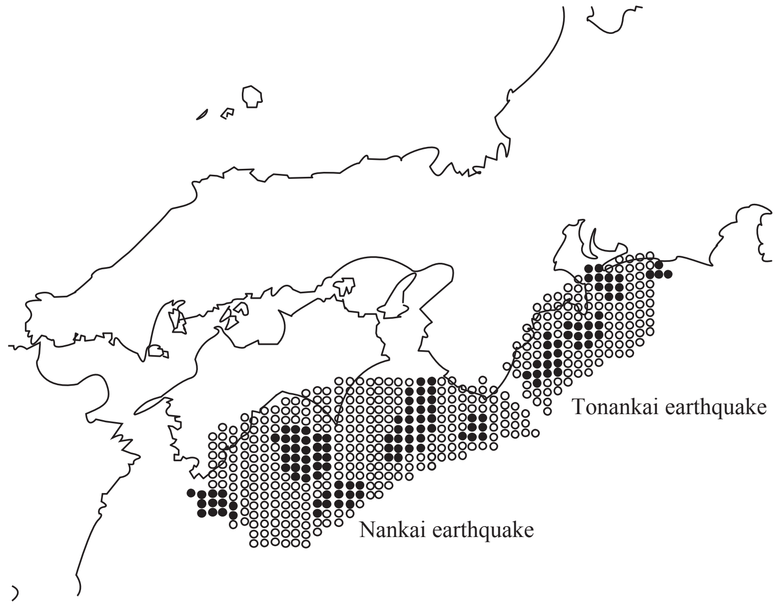


Fig. 1.3.4 Source Model of the Tonankai and Nankai Earthquakes Expected by the Central Disaster Prevention Council [61]

(2) When an Earthquake is Expected to Occur at an Active Fault

The macroscopic source parameters for an earthquake occurring at an active fault may be set in accordance with the following concepts. First, the strike ϕ and dip angle δ of the fault are obtained based on the results of geological, topographical, and geographical surveys. Also, the total length of fault segments with a high possibility of simultaneous occurrence is set as the fault length L. If the dip angle δ is unknown, in case of a strike slip fault, it may be assumed 90°, in case of a high angle reverse fault, it may be assumed 60°, in case of a low angle reverse fault, it may be assumed 30°, and in case of a reverse fault that is neither high angle nor low angle, it may be assumed 45°. The fault width W of an earthquake occurring at an active fault is limited by the thickness H of the seismogenic layer in the upper earth's crust, so when $L<H/\sin\delta$, $W=L$, and when $L>H/\sin\delta$, $W=H/\sin\delta$ may be assumed.[58), 62)] When the thickness H of the seismogenic layer is unknown, 20km may be assumed. The fault area S is obtained from the estimated fault length L and fault width W. The seismic moment M_0 may be obtained from the fault area S using the following empirical equation.[63)]

$$S(\mathrm{km}^2) = 2.23\times10^{-15}\times M_0^{\,2/3}(\mathrm{dyne\cdot cm}) \qquad (1.3.2)$$

The microscopic source parameters for an earthquake occurring at an active fault may be defined as follows. First, the total area of asperities as a percentage of the total fault area is assumed to be 22%.[58), 62), 63), 64), 65)] The number of asperities is assumed to be 1 or 2.[58)] If the magnitude of the postulated earthquake is M7 or larger, the number of asperities is assumed to be 2. When the number of asperities is assumed to be 2, the larger one is assumed to be 16% of the total fault area, and the smaller one is assumed to be 6%.[58), 64)] The shape of the asperities are taken to be rectangular as much as possible.[58), 63)] The seismic moment of the asperity is assumed to be 44% of the total seismic moment.[58), 63), 64)] When there are two asperities, the seismic moment of the larger one is assumed to be 36% of the total seismic moment, and the seismic moment of the smaller one is assumed to be 8%.[58), 64)] The rise time τ of the asperity is defined from the width W_a of the asperity and the rupture velocity V_r using the following equation.[58)]

$$\tau = \left(W_a/V_r\right)/4 \qquad (1.3.3)$$

The layout of the asperity is arranged in relationship to the rupture strong point, which is discussed later, so that the rupture of one of the asperities propagates towards the harbor. This is because due to the effect of directivity, a particularly strong ground motion is generated in the direction of propagation of the rupture of the asperity, and a strong ground motion generated in this way resulted in devastating damage in the 1995 Hyogo-ken Nambu Earthquake.[4)] Specifically, the asperity is arranged as shown in **Fig. 1.3.5**. The depth of the center of the asperity is taken to be 10km.

Of the other parameters, the rupture starting point is located as shown in **Fig. 1.3.5** in relation to the location of the asperity. The rupture velocity is assumed to be 80% of the S wave velocity in the source region.[58)] The rupture is assumed to propagate radially.

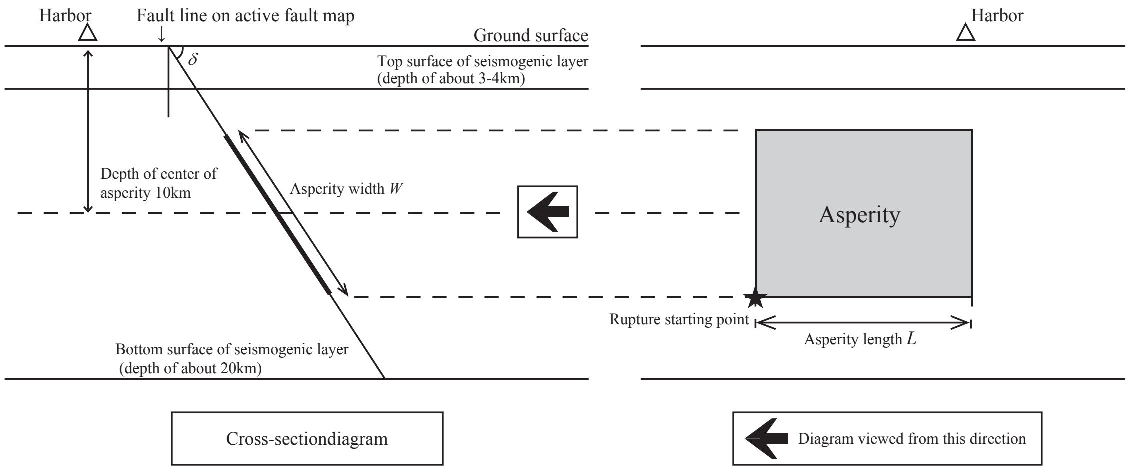


Fig. 1.3.5 Arrangement of Asperity and Rupture Starting Point

(3) When M6.5 Earthquake is Expected to Occur just Beneath the Site
The seismic moment M_0 can be calculated from the Magnitude using the following equation.[66)]

$$\log M_0 = 1.17M + 17.72 \ (\text{dyne} \cdot \text{cm}) \tag{1.3.4}$$

Therefore, the fault area S may be obtained from equation **(1.3.2)**. The dip angle δ may be assumed to be 90°. What follows is the same as in **(2) When an Earthquake is Expected to Occur at an Active Fault**. The number of asperities is taken to be 1.

1.3.4 Evaluation of Site Amplification Factors

The site amplification factors can be evaluated in accordance with **ANNEX 4 Analysis of Seismic Motion** for the Level 1 earthquake ground motions.

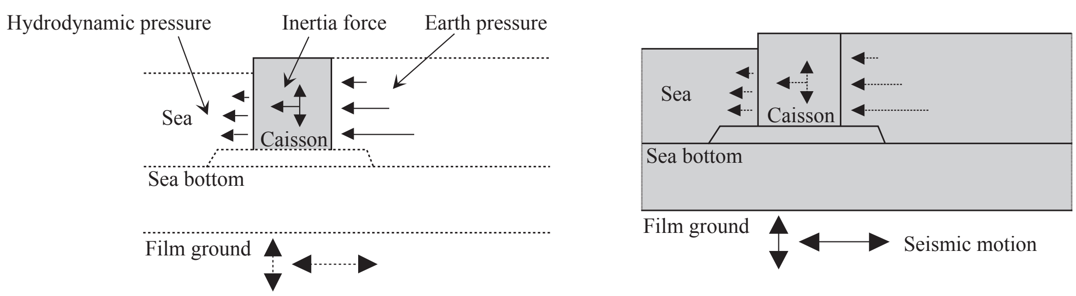
2 Seismic Action

2.1 Modeling and Seismic Action of the Ground - Structure System

The ground motions described in **Section 1 Ground Motion** are ground motions that are independent of the facilities, and do not depend on the type of the facilities or analysis method. This is referred to as a "reference ground motion" in ISO 23469.[1] In contrast, the seismic action, the term used in ISO 23469, necessary for performance verification of port facilities is defined differently depending on the facility or analysis method as stated below. When setting the seismic action for seismic performance verification, firstly the ground motion is evaluated by **Section 1 Ground Motion** for the case where the facilities do not exist, and next the seismic action is evaluated corresponding to the type of facilities or analysis method.

Normally, the analysis methods used in seismic performance verification of port facilities can be classified as equivalent static analysis and dynamic analysis. Also, the analysis methods can be classified as simple analysis or detailed analysis depending on whether ground-structure interaction is taken into consideration. As a result the analysis methods used in seismic performance verification can be classified into 2×2=4 categories. Here, simplified analysis focuses on a part of the ground – structure system, and analyzes its behavior, and the seismic action is defined as the effect on the part under consideration from outside its boundary. On the other hand, in detailed analysis, the total behavior of the ground – structure system, for example the gray part in **Fig. 2.1.1(b),** is analyzed, and in this case the seismic action is defined as the ground motion input to the bottom end of the analysis domain. For example, in a simplified equivalent static analysis, namely seismic coefficient method, of a caisson type quaywall, as indicated in gray in **Fig. 2.1.1(a)**, the part of the whole on which the focus is applied is the wall, and analysis of its behavior is carried out. In this case the seismic action is the inertia forces, earth pressure and hydrodynamic pressure during the earthquake acting on the wall from the external domain. In a detailed dynamic analysis, mainly effective stress analysis, of the caisson type quaywall, as indicated in gray in **Fig. 2.1.1(b)**, the focus is on the entire system comprising the caisson, the backfill, the seawater, and the foundation grounds below the caisson, and its behavior is analyzed. In this case the seismic action is the ground motion input to the bottom end of the analysis domain. In detailed dynamic analysis, the earth pressure and hydrodynamic pressure during the earthquake acting on the caisson wall are produced as the response analysis results, and are not set as an action.

The types of analysis method used for seismic performance verification of port facilities and the method of defining the seismic action in accordance with the analysis method are discussed below.



(a) Simplified Equivalent static analysis (seismic coefficient method)

(b) Detailed dynamic analysis (effective stress analysis)

Fig. 2.1.1 Seismic Action in the Seismic Coefficient Method and Effective Stress Analysis (Example of a Caisson Type Quaywall)

2.2 Seismic Action in the Seismic Coefficient Method [2)]

As shown in **Fig. 2.2.1**, this method is considered when a rigid object is on a rigid ground. Assume the mass of the object is m, and its weight is W. If the ground moves to the right with an acceleration α, an inertia force αm acts on the object to the left. At this time a friction force of αm must act on the bottom surface of the object, in order that it will not slide. If the static friction coefficient on the bottom surface is not sufficiently large, the object will slide, and in most cases, depending on the changes of the acceleration force afterwards, a residual displacement will occur. At this time, when checking whether sliding will occur, it is possible to apply a static force $\alpha\, m$ to the object. This is the fundamental idea of the seismic coefficient method.

The following equation shows the magnitude of the inertia force acting in the seismic coefficient method.

$$F = (\alpha / g) W \tag{2.2.1}$$

If k_h is written instead of α / g, the following equation is obtained.

$$F = k_h W \tag{2.2.2}$$

In other words, the inertia force due to the ground motions is obtained by multiplying the weight of the facility by the coefficient k_h. This k_h is referred to as the seismic coefficient. The seismic coefficient set for performance verification is referred to as the seismic coefficient for verification.

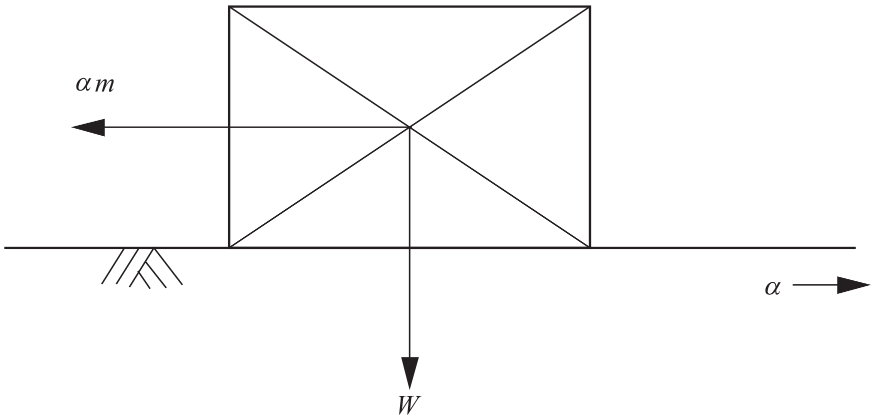


Fig. 2.2.1 Concept of the Seismic Coefficient Method

In the classification of analysis methods given in **2.1 Modeling and Seismic Action of the Ground - Structure System**, The seismic coefficient method is a simplified equivalent static analysis. Problems of the stability of facilities in an earthquake can be converted into static equilibrium problems and conveniently analyzed, so the method is used widely, not only for ports. In the field of ports, this method is used for the performance verification of gravity quaywalls, sheet pile quaywalls, and cell type quaywalls subject to the Level 1 earthquake ground motions. When applied to gravity type quaywalls, it is necessary to consider the inertia forces acting on the wall, as well as the earth pressure and hydrodynamic pressure during the earthquake, as shown in **Fig. 2.1.1(a)**.

For the level 1 earthquake motion, when carrying out seismic performance verification using the seismic coefficient method, it is not necessary to take the value of the expected maximum acceleration of the ground divided by the acceleration of gravity as the seismic coefficient for verification to be applied to the structure. For example, substituting α =215Gal into equation **(2.2.1)** gives k=0.22. However, it is known from experience [2), 4)] that when a ground motion with a maximum acceleration exceeding 215Gal acts on a quaywall with a seismic coefficient for verification of 0.22, a residual deformation does not necessarily occur. The reasons for this have not been phenomenologically explained sufficiently, but it is considered that one of the reasons is that even if a 215Gal acceleration acts on the quaywall, if the action is instantaneous, it is difficult to cause a visible residual deformation to the quaywall. The method of converting the acceleration time history of the scenario Level 1 earthquake ground motions to the seismic coefficient for verification varies depending on the structural form of the mooring facility. For gravity quaywalls refer to **Part III Chapter 5, 2.2.2 Actions**, and for sheet piles quaywalls refer to **Part III Chapter 5, 2.3.2 Actions**.

When carrying out a seismic performance verification using the seismic coefficient method, the earth pressure during the earthquake and the foundation ground properties are as discussed later. However, with the seismic coefficient method normally it is assumed that liquefaction does not occur in the ground behind the wall or in the foundations, and the earth pressure during the earthquake and foundation ground properties are set based on this assumption. Therefore, when carrying out seismic performance verification by the seismic coefficient method for the Level 1 earthquake ground motion, an analysis to predict whether liquefaction will occur in the ground behind the wall or in the foundations is carried out, and if it is determined that liquefaction may occur, it is necessary to take measures against it.

As can be understood from its principle, the seismic coefficient method is a method for determining whether deformation will occur in specific modes, such as sliding, overturning, insufficient bearing capacity of the foundation ground etc., based on static equilibrium of forces. If deformation does occur, it is not possible to calculate by the seismic coefficient method how much residual deformation is caused. This is a limitation of the seismic coefficient method, and because of this limitation it is not practical to apply the seismic coefficient method to the Level 2 earthquake ground motion. Normally, for very strong ground motions, such as Level 2 earthquake ground motions, it is assumed that the facility will suffer some damage, and it is necessary to investigate the process of this damage when carrying out the seismic performance verification.[5), 6)] The same applies to port facilities such as mooring facilities etc., in which it is assumed that deformation will be caused by the Level 2 earthquake ground motion, and it is required to carry out the design to limit the deformation to be equal to or less than the allowable amount. In order to meet this requirement, it is necessary to carry out a seismic response analysis of the ground – structure system, as described later, not the analysis by the seismic coefficient method.

2.3 Seismic Action in the Modified Seismic Coefficient Method [2)]

In the case of the seismic coefficient method, the acceleration acting on the facility is equal to the acceleration acting on the ground. In contrast, in the case of a flexible structure as shown in **Fig. 2.3.1**, the acceleration α' acting on the facility is not the same as the acceleration α acting on the ground. In this case, if the dynamic characteristics of the facility, such as the natural periods, etc., and the time history of the ground acceleration are given, it is possible to calculate the response acceleration of the facility. By applying to the facility the equivalent static force obtained by multiplying the maximum value of the response acceleration of the facility by its mass m, it is possible to replace the actual phenomenon with static equilibrium of forces to carry out the seismic design. When the scope of the seismic coefficient method is expanded in this way to structures with flexibility, it is called the modified seismic coefficient method. Using the time history of the expected ground acceleration, if a response calculation is carried out in advance for facilities with various natural periods, and if the maximum value of the response acceleration of the facility is arranged as a function of the natural period, the result is referred to as an acceleration response spectrum.

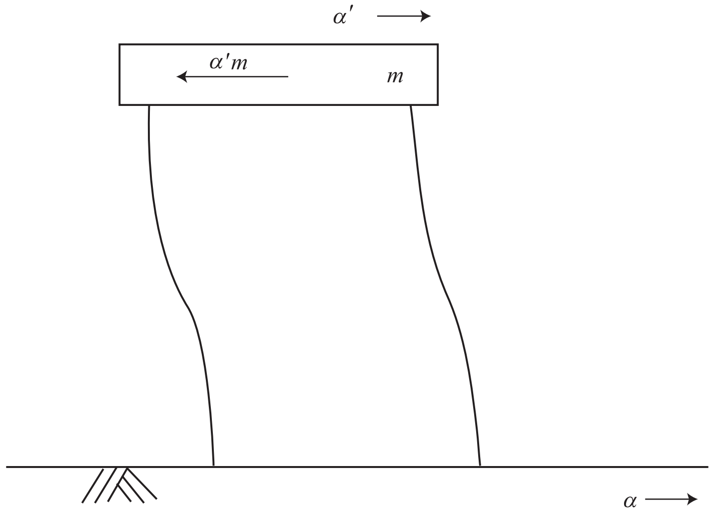


Fig. 2.3.1 Concept of the Modified Seismic Coefficient Method

The modified seismic coefficient method is classified as a simplified equivalent static method in the classification of analysis methods in **2.1 Modeling and Seismic Action of the Ground - Structure System**. For obtaining the response acceleration of the facility in the modified seismic coefficient method, it is frequently assumed that the restoring force characteristics of the facility are linear. However, when a very strong earthquake acts on the structure, the restoring force characteristic of the facility actually becomes nonlinear, as a result of plasticity in the structural members. Therefore, the response acceleration obtained under the assumption of linearity becomes meaningless. Therefore the modified seismic coefficient method is unsuitable for very strong ground motions, such as the Level 2 earthquake ground motion.

2.4 Seismic Action in the Seismic Deformation Method [2)]

In extended, long facilities such as buried pipelines or immersed tunnels, etc., where the apparent weight per unit volume and stiffness are comparatively small, the acceleration applied to the facility is seldom a problem. The weight and stiffness of these facilities is small, so the effect of the existence of these facilities on the surrounding ground is small, and the displacements in the facility tend to be governed by the displacements in the surrounding ground. When the displacement in the surrounding ground is not uniform, strain is caused in the facility. This is a problem for seismic design.

In the seismic deformation method, first the displacement of the ground for the case where the facility does not exist there is obtained, and next the displacement and stress in the facility is obtained based on the assumption that the displacement of the facility is the same as the displacement of the ground. In other words, in contrast to the seismic coefficient method in which the equivalent static load is applied to the facility as the seismic action, in the seismic deformation method the displacement of the ground is applied to the facility as the seismic action. In cases where the stiffness of the subsurface structure is quite high, and the error in the assumption that the facility deforms exactly the same as the ground is large, the displacement of the ground can act on the facility via springs. The seismic deformation method is classified as a simplified equivalent static analysis in the classification of analysis methods given in **2.1 Modeling and Seismic Action of the Ground - Structure System**.

2.5 Seismic Action in the Seismic Response Analysis of Ground - Structure Systems

Each of the methods described so far simplify the actual phenomena, but seismic response analysis that more truely reproduces the overall ground - structure system behavior can also be carried out. This is classified as detailed dynamic analysis in the classification of analysis methods given in **2.1 Modeling and Seismic Action of the Ground - Structure System**. Seismic response analysis of ground - structure systems is frequently based on the finite element method, in particular the effective stress method, as shown in **Fig. 2.5.1**. In this case the seismic action is the ground motion input at the bottom end of the analysis domain.

In general, the ground motion at the bottom end of the analysis domain is the sum of an upcoming wave E and a downgoing wave (F). Methods of applying the input ground motions to the bottom end of the analysis domain include the method in which the actual seismic wave motions E+F are applied to the bottom end of the analysis area, and the method in which an seismic wave having an amplitude twice that of the seismic wave incident from below is applied to the bottom end of the analysis domain, namely 2E wave input method. When carrying out a calculation to reproduce damage actually incurred, or when carrying out a simulation of a shake table rest, there may be measurements of the ground motions at the bottom end of the analysis domain, including the upcoming wave and the downgoing wave, and in these cases the E+F wave input method can be used. However, for seismic response analysis of ground structure systems carried out for seismic performance verification the 2E wave input method is used. In this case, if directly below the analysis domain there is ground that can be considered to be firm ground, the ground motion at the firm ground obtained in **Section 1 Ground Motion** may be used as it is. However, if directly below the analysis domain there is ground that cannot be considered to be the firm ground, it is necessary that the ground motion defined at the firm ground be converted to a 2E wave directly below the analysis domain by a seismic response analysis for the local soil deposit, and this 2E wave is then inputted.

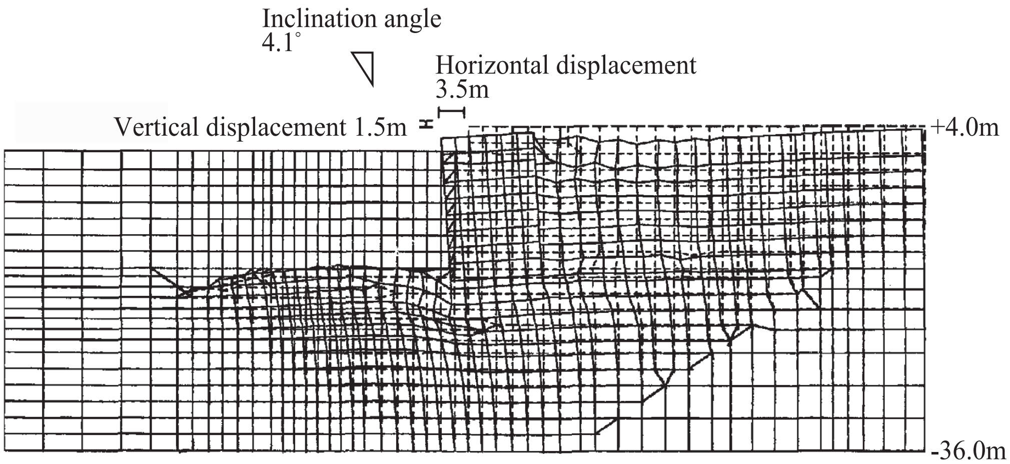


Fig. 2.5.1 Example of Residual Displacement of a Gravity Quaywall Calculated by Effective Stress Analysis

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ANNEX 3 Evaluation of Site Amplification Factors

1 Evaluation of Site Amplification Factors

The following is an explanation of the fundamental method of evaluation of the site amplification factor based on seismic observation records, for the case that there is an observation point on the rock that can be regarded as the seismic bedrock near the harbor, based on **Fig. A-3.1**. The site amplification factor between seismic bedrock and ground surface at the observation point at the harbor can be obtained from the ratio of the Fourier amplitude spectra at the observation point at the harbor and an observation point on the nearby rock. When the site amplification factors between seismic bedrock and the top of firm ground are necessary the amplification factors from the top of firm ground to the ground surface are evaluated by linear multiple reflection theory, [14), 17)] based on ground data at the observation point in the harbor. Then by dividing the site amplification factors between seismic bedrock and ground surface by the amplification factors from the top of firm ground to the ground surface, the site amplification factors between seismic bedrock and top of the firm ground can be obtained. In this case the damping factor may be taken to be 3%.

However, normally there is no observation point on the rock that can be considered to be the seismic bedrock near the harbor, so normally the amplification factors from the seismic bedrock to the ground surface is evaluated using techniques such as spectral inversion techniques, as described later.

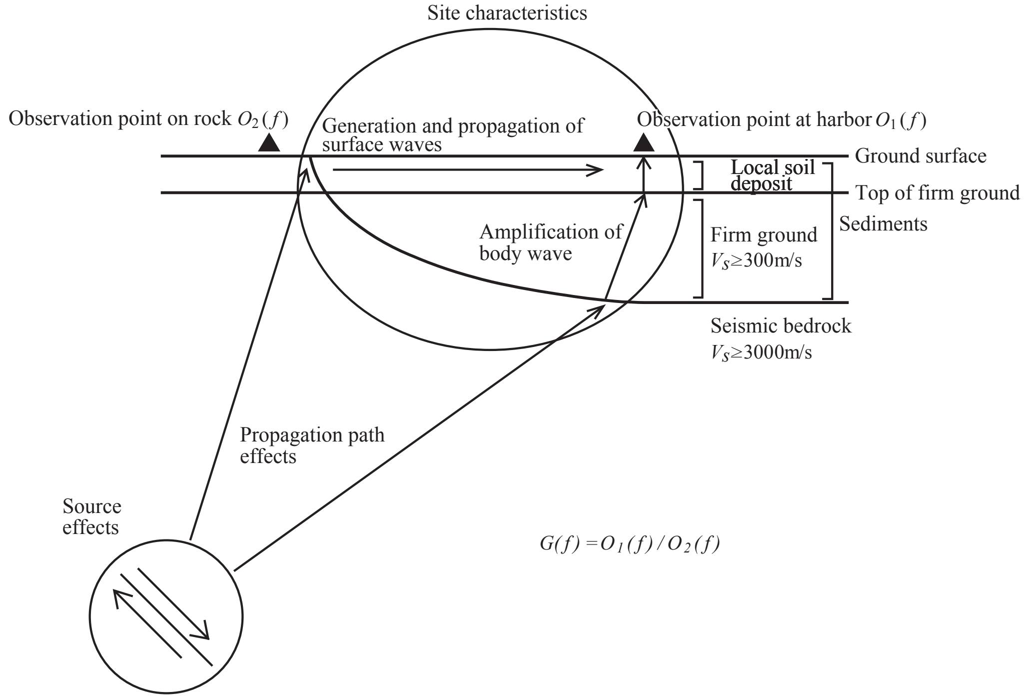


Fig. A-3.1 Fundamental Concept Regarding Evaluation of Site Amplification Factors

(1) Seismic Observation for Evaluation of Site Amplification Factors

It is desirable that the site amplification factors be evaluated based on seismic observation records for the harbor. Observation of strong seismic motion is carried out at the major harbors in Japan, see **Fig. A-3.2**, and the site effects can be evaluated by using these records. Observation of strong ground motion is a type of seismic observation which uses equipment that will withstand very strong motion from damaging earthquakes. Observation records of strong motion earthquake observation in Japanese harbor areas can be downloaded from the National Institute for Land and Infrastructure Management homepage (http://www.eq.ysk.nilim.go.jp).

If the harbors are not subject to designated strong ground motion observation points and if no seismic observation records can be obtained in nearby points within 2km of the harbor in advance of performance verification of an important facility, it is desirable that seismic observation records be obtained for evaluation of site effects by carrying out seismic observations. In this case, it is desirable to confirm that the ground motion characteristics at the observation point do not differ greatly from those at the facilities installation location, by microtremor measurement carried out in advance. The time period necessary for seismic observations depends on the seismicity of that area, but in general in Japan's case, if from one to several years' observation is carried out, it is possible to obtain sufficient records of medium and small earthquakes or distant large earthquakes in order to evaluate the site effects. In order to obtain many records in a short observation period, normally the trigger level, the level of vibration that initiates the seismometer observation, is set lower than that normally used for observation of strong earthquakes. In order to avoid the effect of extraneous vibrations from nearby, one method is to use a mechanism in which the trigger is operated when the velocity exceeds a certain level, not the acceleration. Another method is to carry out continuous measurement regardless of whether there is an earthquake or not, and to extract the data later after an earthquake has occurred.

Fig. A-3.2 Strong Motion Earthquake Observation in Japanese Harbor Areas

(2) Spectral Inversion

Assuming that M earthquakes have been observed at N observation points, the Fourier amplitude spectrum of the observation records can be expressed by the following equation as the product of the source effects, the propagation path effects, and the site effects.[18)]

$$O_{ij}(f) = S_i(f) P_{ij}(f) G_j(f) \tag{A-3.1}$$

where

$S_i(f)$: source effects of the i^{th} earthquake

$P_{ij}(f)$: propagation path effects from the hypocenter of the i^{th} earthquake to the seismic bedrock of the j^{th} observation point
$G_j(f)$: site amplification factors of the j^{th} point

The propagation path effects $P_{ij}(f)$ can be expressed by the following equation, taking into consideration geometric attenuation, $1/r$, of the wave spreading in spherical form from the hypocenter and inelastic damping.

$$P_{ij}(f) = \frac{1}{r_{ij}} \exp\left(-\pi f r_{ij} / Q V_S\right) \tag{A-3.2}$$

where

r_{ij} : distance from the hypocenter of the i^{th} earthquake to the j^{th} observation point
Q : Q value on the propagation path

Substituting equation **(A-3.2)** into the right hand side of equation **(A-3.1)**, and taking common logarithms of both sides, the following equation is obtained.

$$\log O_{ij} = -\log r_{ij} + \log S_i + \log G_j - (\log e)\, \pi f r_{ij} / Q V_S \tag{A-3.3}$$

In order to simplify the expression shown here, the f which indicates dependence on frequency has been omitted. Equation **(A-3.3)** includes $M+N+1$ number of unknowns, including the source effects S_i, the site amplification factors G_j, and the Q value. Therefore, if there are more equations, namely the number of records that can be used, than the number of unknowns, it is possible to obtain the combination of unknowns for each frequency f, by the method of least squares so that the residual error of equation **(A-3.3)** is minimized. The above is the basic concept of spectral inversion. It is also possible to have the Q value as a known quantity, and obtain $M+N$ number of unknown quantities.

However, there is a trade-off relationship between the source effects S_i and the site amplification factors G_j in equation **(A-3.3)**. For example, assuming that a certain combination of S_i and G_j is a solution, the combination $S_i/2$ and $2G_j$ is also a solution. As a method for avoiding this, there is the method of assuming that the site amplification factors are 1 at a rock observation point, referred to as the standard observation point, selected in advance. At this time it is necessary to carefully consider the selection of the reference point. The following points[19] are useful for selecting the reference point. Firstly, select the point with the smallest site amplification characteristics for each frequency as the reference point based on the results of preliminary analysis. However, as the amplification in the high frequency range in weak grounds is small, the point selected as the reference point should be limited to points with sufficiently large S wave velocities. Specifically, the reference point should be selected from points for which the average S wave velocity from the ground surface to the depth of 10m is 400m/s or higher. Also, in order to avoid the characteristics of each individual record greatly affecting the results, the reference point should be limited to those points for which records of several, about 5 earthquakes, measurement records have been obtained. Besides basing the selection of the reference point on the above criteria, it is necessary to make the decision based on an examination of whether the low frequency part of the source effects S_i obtained from the actual inversion results is compatible with Centroid Moment Tensor, CMT, solution [20], for example, that of the F-net of the National Research Institute for Earth Science and Disaster Prevention.

In addition, the points to note when actually carrying out the spectral inversion are as follows:

In spectral inversion it is normally assumed that there is geometric attenuation, 1/r, of the wave spreading in spherical form from the hypocenter. However, at distant observation points, geometric attenuation in the form above becomes inapplicable as a result of the effect of Lg waves transmitted by reflection within the earth's crust.[12] In order to avoid this, it is necessary to exclude records of earthquakes that occur far away, about 150 - 200km or farther.

The records of small scale earthquakes frequently do not have good S/N ratio in the low frequency range. When considering harbor facilities, there are times when it is necessary to ensure accuracy down to 0.2Hz on the low frequency range, so it is necessary to use records of M4.5 or larger. Also, it is desirable to check the S/N ratio on the low frequency range of each of the records used in the analysis.[21] On the other hand, the records of large scale earthquakes are affected by the rupture process of the fault, so it becomes inappropriate to consider a single source effect S_i, unaffected by direction. Therefore, it is desirable to avoid records for M6.0 or larger. As a result of the above, earthquakes in the range M4.5 – M6.0 are frequently used in spectral inversion.

In order to avoid nonlinear behavior of the local soil deposit, it is desirable to avoid the use of records with large amplitude. It is also necessary to pay attention to the length of the records used in the analysis. It is also possible to extract by some method the "S wave part" of the observed ground motion, and use its Fourier spectrum in the analysis. However, when considering harbor facilities, it is necessary to obtain the amplification factors of the Fourier spectrum including later phases by analyzing not only the S wave, but also surface waves.

Nozu and Nagao [22] applied spectral inversion to a data set that contained strong motion earthquake records in Japanese harbor areas as well as K-NET, KiK-net, and other strong ground motion records, and obtained the site amplification factors between seismic bedrock and ground surface of the strong ground motion observation points in each area, in particular harbors. The results are available on CD-ROM.[22)]

(3) Method of Evaluating the Site Amplification Factors from Simultaneous Records from the Harbor and its Surroundings

If records have been obtained for the same earthquake at the harbor and a nearby observation point, and if the site amplification factors have already been evaluated at the nearby observation point, the site amplification factors at the harbor can be evaluated by the following method. Firstly, in order to be able to explain the record at the nearby observation point, the source effects of the earthquake under consideration are appropriately set. Next, it is possible to obtain the site amplification factors at the harbor by dividing the Fourier amplitude spectrum at the harbor by the source effects and by the propagation path effects. [23)] It is necessary to be aware that if the harbor and the nearby observation point are in fairly different directions from the hypocenter, then it is possible that the accuracy of the evaluation will be reduced by the dependence on direction of the source effects of the earthquake.

If the earthquake has occurred sufficiently far away, the source effects and the propagation path effects of the harbor and the nearby measurement point can be considered to be common, so evaluation of the source effects may be omitted, and the site amplification factors of the harbor may be evaluated by taking the ratio of the spectra of the two points. The records of large earthquakes that have occurred particularly far away are not suitable for spectral inversion, but the S/N ratio is frequently good down to the low frequency range, so they can frequently be used in this manner. **Fig. A-3.3** shows a comparison of the ratio of the site amplification factors obtained from SZO013, K-NET Shimizu, and SZO014, K-NET Shizuoka, from the records of the Kii Hanto Nanto Oki Earthquakes with M7.1 and M7.4, which occurred on 5th September 2004, and the ratio of the site amplification factors based on spectral inversion. It can be seen that the ratios of the site amplification factors obtained by the two methods agree well.

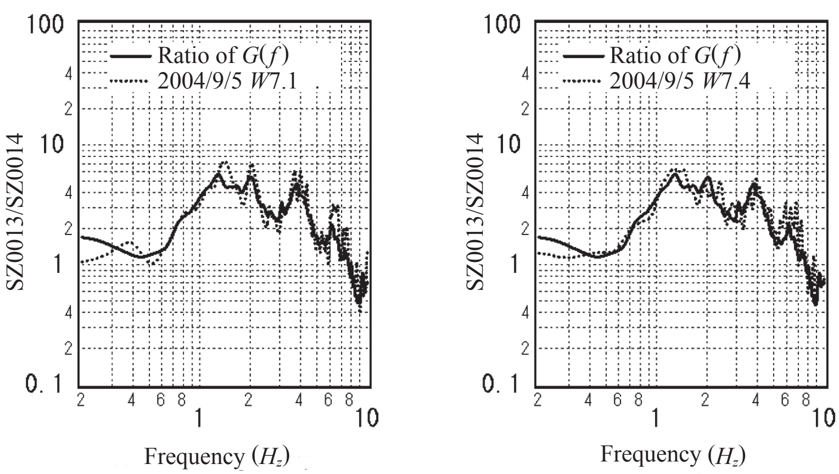


Fig. A-3.3 Comparison of the Ratio of Site Amplification Factors Evaluated by Two Methods

(4) Evaluation of the Site Amplification Factors when Seismic Observation Records have been Obtained at Several Locations Near the Harbor

If ground motion records can be obtained at several locations near the harbor, it is possible to obtain several site amplification factors. In this case, it is necessary to carry out zoning on the several site amplification factors. In coastal areas, sudden changes are sometimes seen in the bedrock depths, due to the basin structure, so it is necessary to be aware that if zoning is carried out according to whether the physical distance is long or short, it is possible to make the evaluation on the dangerous side. The use of microtremors can be considered as a means of carrying out simple zoning. There are many examples of research into the use of microtremors to determine the subsurface structure. Among them is research focused on the ratio of the spectra of the horizontal component and the vertical component, hereafter referred to as the H/V spectrum, obtained by measurement of three components of microtremors, [26)] and research focused on the average S wave velocity obtained from array measurements, [27)] however these are mainly for investigating the shallow subsurface structure. Also, research examples focused on the deep subsurface structure using microtremors frequently use phase velocity by array measurement.[28)] There are comparatively few examples of research on the deep subsurface structure using the H/V spectrum from 3 component measurements, but for Sato et al.[29)] have indicated that the microtremor spectral peak appearing in the range with period equal to or greater than 1 second can be explained by the H/V spectral peak in the Rayleigh wave down to the seismic bedrock, based on measurement records at Sendai.

It is considered that of the microtremor measurements, 3-component measurement is suitable for investigating the subsurface structure because of the simplicity of the measurements. For the deep subsurface structure, it is also possible to consider zoning by focusing on the peak in the long period side of the microtremor H/V spectrum.

2 Probabilistic Seismic Hazard Analysis

The uniform hazard Fourier spectrum at the top of firm ground and the corresponding time history can be calculated in accordance with the procedure shown in **Fig. A-3.4**.[31] The following is an explanation of the flow.

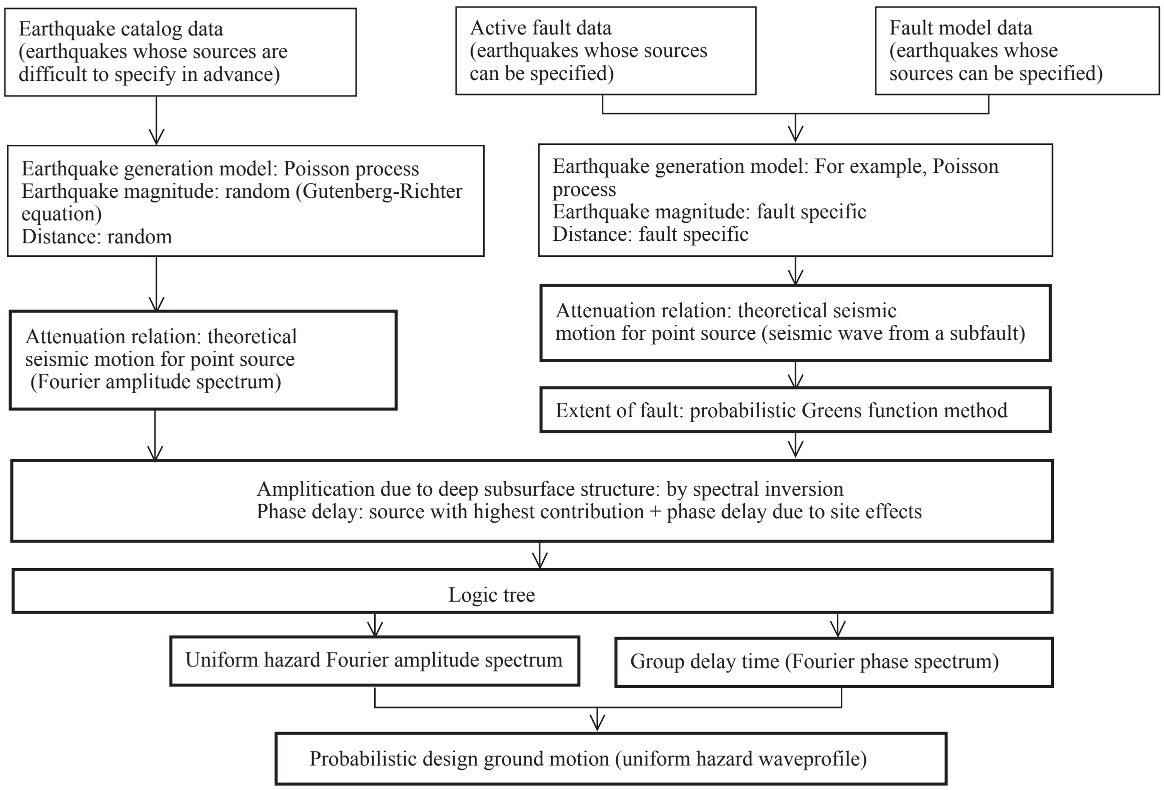
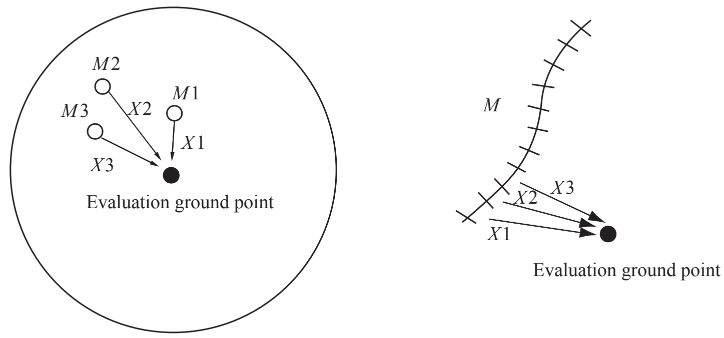


Fig. A-3.4 Method of Calculating the Uniform Hazard Fourier Spectrum and the Corresponding Time History [31]

Firstly, the sources of earthquakes that could occur in the future near the harbor are classified into those that cannot be easily defined and those that can be defined, and each of the sources are modeled. Here modeling the sources means setting the position and size of the sources. To model the former, earthquake catalog data [32] that records earthquakes that have occurred near the site in the past are used. To model the latter, active fault data [33], [34] obtained from topographical and geological surveys and fault model data for past earthquakes [35] are used. For sources that cannot be easily defined in advance, the sources may be equally spread over an area that appears to be seismically active, hereafter referred to as a seismic area, or sources may be randomly set within the seismic area, see **Fig. A-3.5(a)**. On the other hand, for the sources that can be defined, the position and size of the source is set, see **Fig. A-3.5(b)**.

After modeling the sources, the earthquake Magnitudes that could occur at these sources in the future and the frequency of their occurrence are evaluated. In the case of sources that are difficult to define in advance, the model of the Gutenberg-Richter equation, namely *b* value model, see **Fig. A-3.6(a)**, is assumed in which specifies relationship between the logarithm of the frequency of occurrence of an earthquake, *N,* and the Magnitude, *M*. The earthquake Magnitudes are the Magnitude values obtained from the earthquake Magnitude-frequency relations. Also, the frequency of occurrence within the seismic area can be obtained from the number of occurrences of earthquakes in the earthquake catalog data and their time of measurement. In the case of sources that can be defined, the maximum Magnitude model, maximum moment model, see **Fig. A-3.6(b)**, in which the magnitude of the earthquakes that occur is constant is frequently used. The Magnitude and frequency of earthquakes occurring on active faults are frequently calculated from information on the length of the active fault, the average slip rate, and other topographical and geological information.



(a) Earthquakes for which the sources are difficult to define in advance

(b) Earthquakes for which the sources can be defined

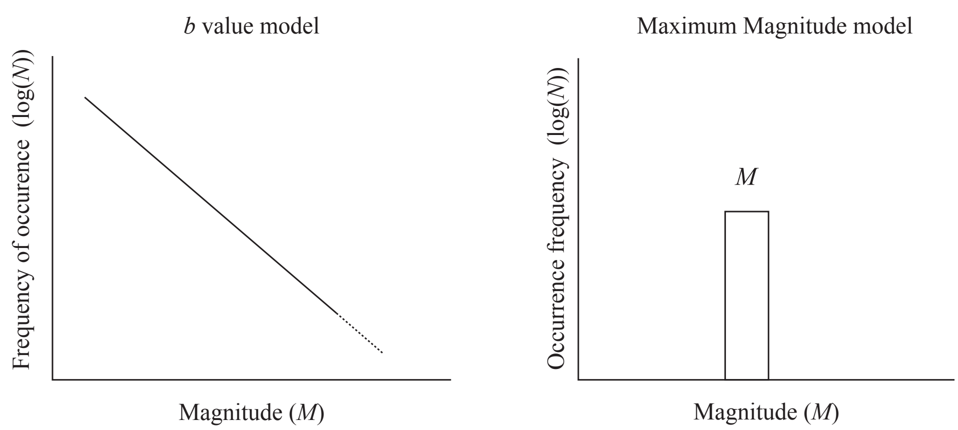
Fig. A-3.5 Modeling of Sources

For each of the postulated earthquakes, the Fourier amplitude spectrum at the top of firm ground is calculated taking into consideration the source effects, the propagation path effects, and the site amplification factors between seismic bedrock and top of firm ground. For sources that can be defined, it is desirable that the Fourier amplitude spectrum is calculated by a method capable of taking into account the finiteness of the fault, such as the probabilistic Green function method. For sources that cannot be easily defined in advance, it can be assumed that the source effects of the earthquakes follow the ω^{-2} model.

As a result of the above, many Fourier amplitude spectra are evaluated with probabilities, see top figures of **Fig. A-3.7**. Therefore, these can be arranged so that the relationship between the Fourier amplitude spectrum and the annual probability of exceedance hazard curve, can be obtained for each frequency, see **Fig. A-3.7**. When these are overlayed the hazard surface is obtained, see **Fig. A-3.7**, so that focusing on a particular annual probability of exceedance a uniform hazard Fourier amplitude spectrum is obtained, see **Fig. A-3.7**. There are 4 samples in the top figures of **Fig. A-3.7**. This means that although these are earthquakes from the same source, their manner of occurrence is not the same.

In order to investigate the extent of uncertainty in the evaluation results due to the selection of assumptions and models used in the above evaluation process, a logic tree may be used. In a logic tree, the combinations of model and parameter values are appropriately set, and analysis is carried out, and the reliability is evaluated from the variation in the analysis results.

To obtain the time history corresponding to a uniform hazard Fourier amplitude, information regarding Fourier phase is necessary. In this case it is desirable that the Fourier phase be defined taking into consideration the characteristics fo Fourier phase at the evaluation point.



(a) Earthquakes for which the sources are difficult to define in advance

(b) Earthquakes for which the sources can be defined

Fig. A-3.6 Evaluation of Magnitude of Earthquake and Frequency of Occurrence

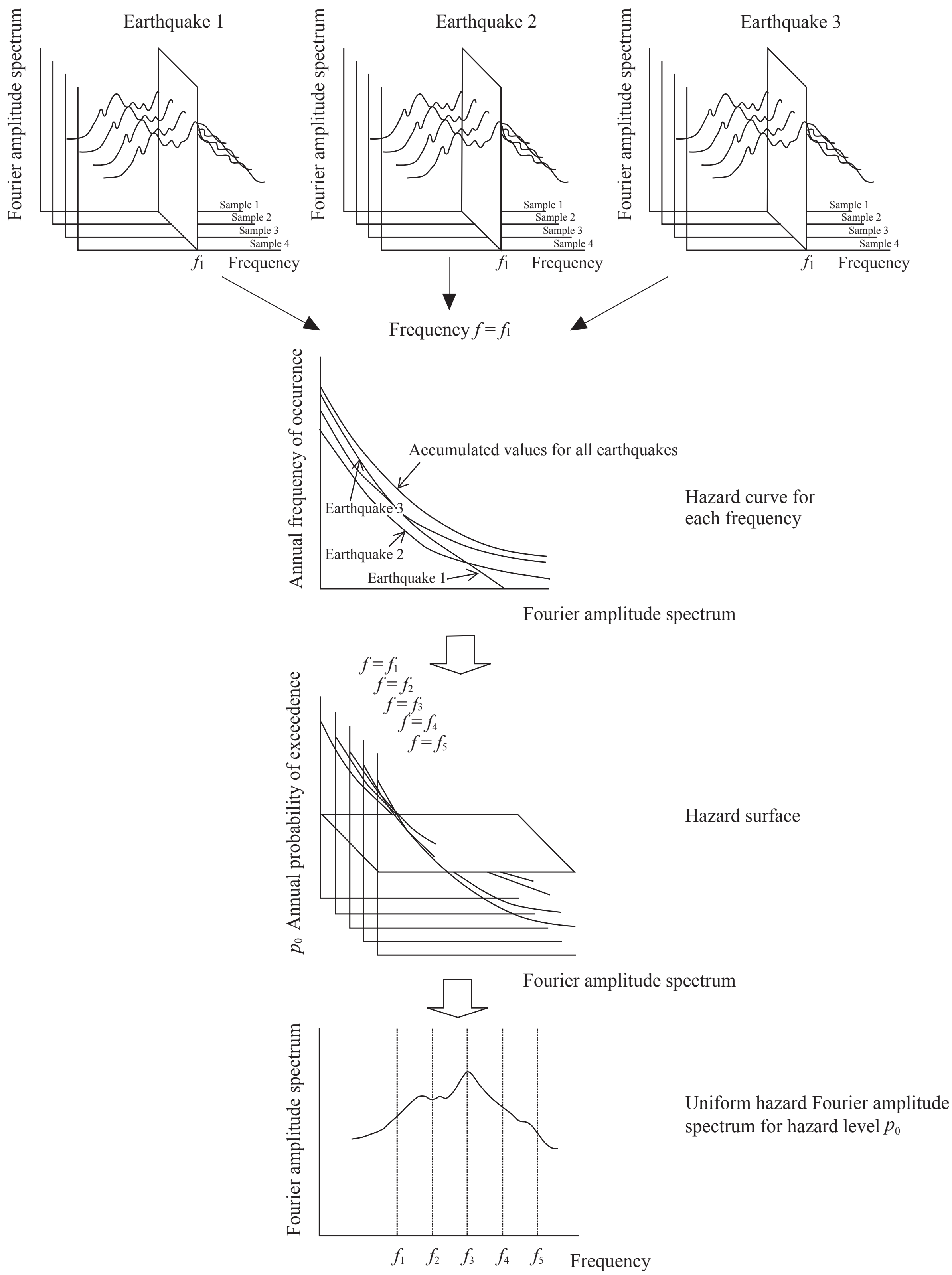


Fig. A-3.7 Procedure for Calculating the Uniform Hazard Fourier Amplitude Spectra

ANNEX 4 Analysis of Seismic Motion

1 Seismic Response Analysis of Local Soil Deposit

Normally the Level 1 earthquake ground motion is set as the incident wave, 2E waves, on the top of the firm ground. However, when the acceleration, velocity, displacement, shear stress, shear strain, etc., are needed at other depths of the local soil deposit, they can be obtained by a seismic response analysis of the local soil deposit. The following is a description of the seismic response analysis for this purpose. For seismic response analysis for performance verification of seismic-resistant, see **Part III, Chapter 5 Mooring Facilities**. Normally seismic response analysis of the local soil deposit is carried out by modeling the local soil deposit above the firm ground. However, the range in S wave velocity of the firm ground can be considerable, so it is necessary to confirm that the S wave velocity of the firm ground considered when setting the Level 1 earthquake ground motion and the S wave velocity of the firm ground considered for the seismic response analysis are consistent to a certain extent. Also, when carrying out seismic response analysis for the purpose of predicting liquefaction, conventionally the wave was converted to correspond to SMAC-B2 before inputting. However, henceforth this conversion is not necessary. Conventionally a wave profile of past strong motion records was input with amplitude adjustment, and in this case the reference maximum acceleration was equivalent to SMAC-B2, so the wave profile was converted to correspond to SMAC-B2.

(1) Types of Seismic Response Analysis for Local Soil Deposit

(a) Classification according to the dimensions considered in the calculation

Depending on the dimensions considered in the calculation, seismic response analysis may range from 1-dimensional to 3-dimensional. Normally when investigating the seismic response of the ground only, for natural grounds or artificial grounds with a horizontally layered structure, 1-dimensional seismic response analysis is frequently carried out. In coastal areas it is frequently possible to assume that horizontal stratification is predominant. In these cases, it is considered that calculation results with sufficient accuracy for practical purposes can be obtained from a 1-dimensional model.

Also, related to this, frequently the type of seismic wave used in the calculation is the S wave that is propagated vertically. Normally in coastal areas, the S wave velocity in the ground near the surface is low, so the ray of the seismic wave near the surface becomes almost vertical, see **Fig. 1.1.1**. Also, the same tendency can be seen in the case of surface waves. Although this is a slightly detailed discussion, surface waves can be considered to be a superposition of P waves and S waves within the local soil deposit. At this time the rays of the P wave and S wave also approach the vertical near the surface. Therefore, by considering an S wave transmitting vertically, it is considered that the calculation will have sufficient accuracy for practical purposes.

(b) Classification according to modeling of the soil stress-strain relationship

Seismic response analysis of local soil deposit is classified into equivalent linear analysis and nonlinear analysis, from the viewpoint of modeling of the soil stress-strain relationship. Equivalent linear analysis takes account of the dependence of the shear modulus and the damping factor of the soil on the amplitude, strictly speaking the strain of the soil, of the ground motion, see **Chapter 3, 2.4.1 Dynamic Modulus of Deformation, Fig. 2.4.2**. However, in this calculation method it is assumed that during the time of the earthquake their values are constant. Of course this assumption is different from the reality, but at the time that equivalent linear analysis was developed, the performance of computers was not as high as today, so these assumptions were made for the convenience of the calculation. In contrast to this, in nonlinear analysis the calculation takes into consideration that the shear modulus etc. of the soil varies throughout the time of the ground motion. If the intention is to be as close as possible to the actual phenomena then it is necessary to carry out nonlinear analysis, but if the strain in the soil is not too large, it is considered that equivalent linear analysis can provide response analysis results that are close to the actual phenomena to a certain extent. The level of strain at which equivalent linear analysis can be applied depends on the method, but is about 0.5 to 1.0% or less.[36), 37)] Therefore, if as a result of carrying out equivalent linear analysis it is found that the strain obtained exceeds this amount, it is necessary to change the analysis method to nonlinear analysis.

In equivalent linear analysis the following repeated calculations are carried out. First, the effective shear strain is obtained from the maximum shear strain for each layer, in case of 2-dimensions or higher, for each element, calculated at a particular step, from the following equation.

$$\gamma_{eff} = \alpha\gamma_{\max} \tag{A-4.1}$$

where

$\gamma_{\max}$: maximum shear strain

γ_{eff} : effective shear strain

α : coefficient (normally 0.65)

Next, from the effective shear strain, the shear modulus and the damping factor are modified taking into consideration the strain dependence of **Fig. 2.4.2** of **Chapter 3, 2.4.1 Dynamic Modulus of Deformation**, and the routine proceeds to the next step. This operation is repeated until the shear modulus converges. The earliest equivalent-liner seismic response analysis program is SHAKE.[38] When SHAKE was first developed, there were no other competing programs, and it was widely used in design practice. Also, FLUSH, [39] the 2-dimensional version of SHAKE, is widely used. However, in recent years the problems with SHAKE have gradually become apparent, as a result of comparison of SHAKE calculation results with actual seismic observation records.[40] One of these problems is that the high frequency components are under-estimated. When trying to estimate the incident waves on the firm ground based on the seismic wave observed at the surface, the high frequency component is over-estimated). FDEL, [41] DYNEQ, [42] and other programs that are improved over SHAKE in this respect have been proposed. In these programs the problem of under- or over-estimation of the high frequency component is solved by using frequency dependent shear strain, instead of the effective shear strain obtained from equation **(A-4.1)**.

Nonlinear analysis is an analysis method that can be applied when the strain in the ground is large, about 0.5 – 1.0% or larger. However, whether the nonlinear analysis gives the correct result or not naturally depends on the constitutive equation used and whether the soil constants are appropriate or not. There are various types of analysis program for nonlinear analysis, using various constitutive models. It is important to use an analysis program that has successfully reproduced vertical array observation records obtained under similar conditions, soil properties and strain levels, in the past with good accuracy.[36] Nonlinear analysis can be classified into effective stress analysis and total stress analysis. When excess pore water pressure appears in a ground, the effective stress is reduced. As a result, the stress state of the soil changes, so the soil restoring force characteristics and damping characteristics are changed, so the response characteristics of the ground are also changed. Effective stress analysis is capable of expressing this type of situation, and is a method that is capable of directly obtaining the excess pore water pressure generated in a ground by calculation. On the other hand, in total stress analysis, the excess pore water pressure is not calculated in the calculation process, so it is not possible to take account of the change in seismic response due to the change in effective stress. If excess pore water pressure is generated more than a certain level, about 0.5 or higher in the effective stress ratio, there is a large possibility that the total stress analysis results will significantly differ from the actual seismic response. Therefore, if the intention is to analyze the actual phenomena truely, it is necessary to carry out an effective stress analysis.

One of the analysis programs for effective stress analysis is FLIP.[43] **Fig. A-4.1** shows the results of a calculation [44] using FLIPver.3.3 to reproduce the vertical array records obtained at Port Island in Kobe Port during the 1995 Hyogo-ken Nambu Earthquake. Port Island records were obtained at the four depths: GL-83m, GL-32m, GL-16m, and ground level. Here the NS component wave observed at GL-83m was used as the input wave, and the waves at the other levels, GL-32m, GL-16m, and ground level, were calculated and compared with the observed waves. The capability to reproduce the observed waves was very good. From this result, analysis results for the 1993 Kushiro Oki Earthquake, [45] and others, it is judged that FLIP is an analysis program that can give accurate results, provided the soil constants are appropriately set. However, in each individual case, whether the FLIP results are correct or not depends on whether the soil constants have been properly set or not.

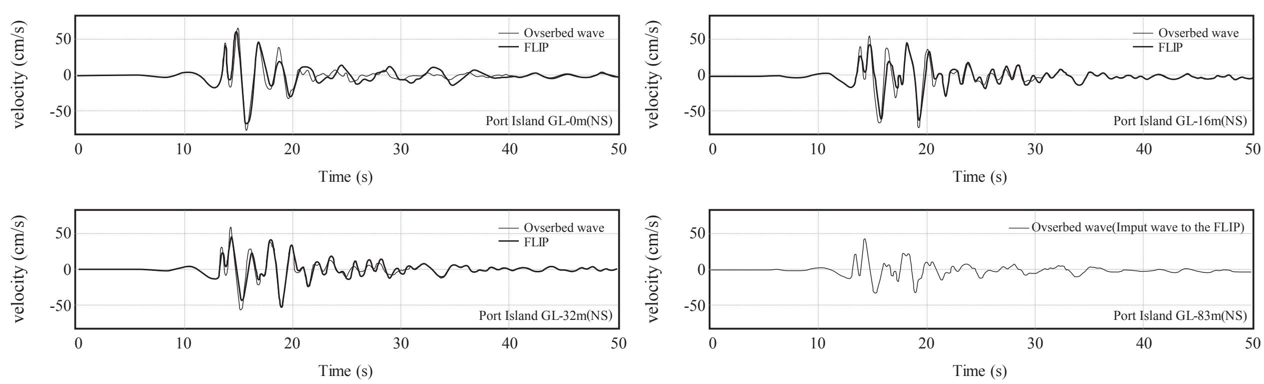
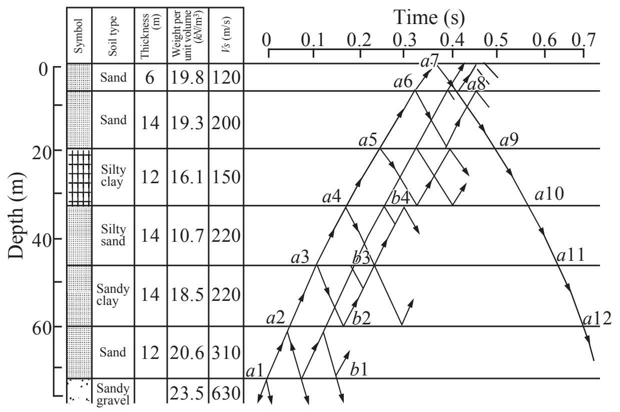


Fig. A-4.1 Reproduction of the Vertical Array Records at Port Island during the 1995 Hyogoken Nambu Earthquake using FLIP [44]

(c) Classification according to type of numerical solution method

Seismic response analysis of local soil deposit can be classified according to numerical solution method into multireflection theory and the finite element method.

As shown in **Fig. A-4.2**, multireflection theory is a method that considers a horizontally layered ground, and it is assumed that a shear wave incident on the ground propagetes upwards, and at the boundary of each layer reflection and transmission repeatedly occurs, and the coefficients of the analytical solution are determined at each layer, so that the boundary conditions are satisfied. The formulation is described in, for example, Reference **17)**, in an easy to understand manner. In multireflection theory, normally the soil stress-strain relationship is limited to linear or equivalent linear type. The calculation is normally carried out in the frequency domain. SHAKE, [38)] FDEL, [41)] DYNEQ, [42)] use multireflection theory as the numerical solution method.



(a) Soil boring log (b) Time sequence of reflection and transmission

Fig. A-4.2 Multireflection Theory [14)]

As shown in **Fig. A-4.3**, the finite element method is a method in which the ground is divided into a finite number of elements, and the differential equations governing the system is converted into algebraic equations in terms of the response of the system at the nodes and then solved. The finite element method is used not only for grounds, but is used in a wide range of fields. The characteristic of this method is that it is capable of dealing with the 2-dimentional and 3-dimentional changes in soil properties and layer thickness. FLUSH [39)], FLIP [43)], and others use the finite element method as the numerical solution method. The calculations are conducted in the domains of frequency and time.

(a) Ground cross-section (b) Cross-section on left divided into elements

Fig. A-4.3 Finite Element Method

(2) Ground Modeling for Seismic Response Analysis of Local soil deposit

The following is an explanation of the modeling of the ground and method of determining the parameters necessary for obtaining a solution for a seismic response analysis for the local soil deposit, focusing on 1-dimension.

① Outline

To carry out a seismic response analysis for the local soil deposit, the ground at the objective point is modeled by dividing it into several layers. For each layer, the layer thickness, density, and shear modulus under small strain are necessary parameters regardless of the solution method. In addition, in the case of equivalent linear analysis, the strain dependence of the shear modulus and the damping factor are necessary. The parameters necessary for nonlinear analysis depend on the method of modeling the stress-strain relationship of the soil, but in the case of FLIP, in addition to the above parameters, the modulus of volume, angle of shear resistance, the upper limit value of hysteretic damping factor, and parameters to define liquefaction are necessary characteristics. Of these, the parameters to define liquefaction characteristics are necessary only for carrying out an effective stress analysis.

② Modeling procedure

The engineering bedrock is set taking the results of the soil investigation into account. At this time the S wave velocity of the ground selected as the engineering bedrock must not differ significantly from the S wave velocity of the ground selected as the engineering bedrock when setting the ground motion. The ground is divided into layers corresponding to the changes in the soil properties. In this case, even if the soil classification is the same, if S wave velocity, *N* value, or q_u value differs significantly, they are considered to be separate layers. Each ground is classified into either sandy soil, clay, or gravel. In an actual ground, it is rare for a soil to comprise sand or clay only, and usually gravel, sand, silt, and clay are mixed in various proportions. Here soil with a fine, particle size 75μm or less, content of 20% or less is considered to be sandy soil, and others are considered to be cohesive soils. Rubble stones used in mounds and backfilling is considered to be gravel.

As for soil density of each layer for which undisturbed sampling of a soil has been carried out, the density of the soil is measured from the soil test sample, and this value is used. However, if the density has not been measured, the values shown in **Table A-4.1** may be used for convenience. The standard values shown in **Table A-4.1** are standard values for seismic response analysis, and it is necessary to be aware that they may not be used for other analyses in which density is a governing factor.

Table A-4.1 Standard Values of Soil Density [46)]

Soil type	Condition	Density (g/cm^3)
Cohesive soil	Water content ≥60%	1.5
	Water content ≥60%	1.7
Sandy soil	Higher than ground water level	1.8
	Below ground water level	2.0
Rubble stones backfilling		2.0

The shear modulus under small soil strain, shear strain about 10^{-6}, used in the response analysis can be calculated from the S wave velocity obtained by in-situ investigation.

$$G_0 = \rho V_S^2 \tag{A-4.2}$$

where

G_0 : elastic shear modulus under small soil strain

ρ : density

V_S : S wave velocity

If the S wave velocity has not been obtained by in-situ investigation for a sandy soil, the shear modulus can be estimated from the *N* value using the following equation.

$$G_0 = 14100 N^{0.68} \quad (kN/\mathrm{m}^2) \tag{A-4.3}$$

However, this equation shows the average value obtained from number of actual data, so it is necessary to be aware that there was a significant amount of variation [47)]. For details, refer to **Chapter 3, 2.4.1 (6) Simple Estimation of Shear Modulus and Damping Factors**.

If the unconfined compressive strength (q_u) of cohesive soils has been obtained, the shear modulus may be estimated from the following equation [48)].

$$G_0 = 170 q_u \tag{A-4.4}$$

When the S wave velocity shall be estimated using the *N* value, if the *N* value prior to construction is only available, such as for the grounds underneath a wall, the *N* value after construction shall be estimated, taking into consideration the effect of the effective overburden pressure due to the wall or a mound. The following equation may be used in the estimation.

$$N = \frac{(0.0041\sigma_V' + 0.7355)N_0 + 0.019(\sigma_V' - \sigma_{V0}')}{0.0041\sigma_{V0}' + 0.7355} \tag{A-4.5}$$

where

N : N value after construction
N_0 : N value before construction
σ_v' : effective overburden pressure after construction (kN/m^2)
σ_{v0}' : effective overburden pressure before construction (kN/m^2)

Where the ground conditions change before and after the construction, and when PS logging is carried out only before construction, the S wave velocity after construction may be estimated from the following equation, which takes into account the effect of the change in effective overburden pressure, and uses the S wave velocity measured before the construction.

$$V_S = V_{S0}\left(\frac{\sigma_v'}{\sigma_{v0}'}\right)^B \tag{A-4.6}$$

where

V_S : S wave velocity after construction
V_{S0} : S wave velocity before construction
σ_v' : effective overburden pressure after construction (kN/m^2)
σ_{v0}' : effective overburden pressure before construction (kN/m^2)

B is 0.25 in the case of a sandy soil or a cohesive soil with a plasticity index I_p = 30 or less, and is 0.5 in the case of a cohesive soil with plasticity index I_p = 30 or higher.
The measurement of the S wave velocity of rubble mound and backfill is difficult, but for rubble mound and backfill to large quaywalls at a water depth of about -10m, the following values of S wave velocity obtained from the calculation equation [49] derived from the results of seismic observations on a composite breakwater may be used.

S wave velocity of the rubble mound : V_S=300 m/s
S wave velocity of the backfill : V_S=225 m/s

Also, there is an example [50] in which the value of the S wave velocity for both the rubble mound and the backfill is 300m/s for a reference mean effective confining pressure of 98kN/m^2.
When a caisson is considered as a kind of ground, the following values may be used as the S wave velocity of the caisson.

S wave velocity of caisson : V_S=2000 m/s

It is known that the shear modulus under small soil strain is proportional to the power of the effective confining pressure. Equation **(A-4.2)** shows a relationship between the shear modulus and the S wave velocity, so it may be inferred that the S wave velocity is proportional to the power of the effective confining pressure. This relationship is obtained as follows from past element tests.[48), 51)]

(a) The shear modulus for a cohesive soil with plasticity index of I_p = 30 or higher is proportional to the effective confining pressure to the power of 1.

(b) The shear modulus for a sandy soil or a cohesive soil with plasticity index of I_p = 30 or less is proportional to the effective confining pressure to the power of 0.5.

On the other hand, **Fig. A-4.4** shows a graph of the average S wave velocity for Toyoura standard sand obtained by centrifugal model tests corresponding to the confining pressure at the middle of sand ground. The solid line in the graph is the correlation curve $V_S = K(\sigma_c')^a$. The average value of the S wave velocity of a sand ground increases as the centrifugal acceleration increases, which demonstrates the significant dependence on the confining pressure. **Fig. A-4.5** shows the distribution of the S wave velocity with depth determined for the same test sample. The broken line in the graph is the curve of the case where the S wave velocity is proportional to the confining pressure to the power of 0.25, and this curve was obtained using the S wave at the middle in the sand ground as reference. In both cases the S wave velocity increases as the depth increases, and the change with respect to depth is approximately proportional to the power of 0.25. These results were obtained by applying centrifugal acceleration varying from 10G to 50G to a 24cm thick layer of soil to artificially vary the effective confining pressure.[52)]

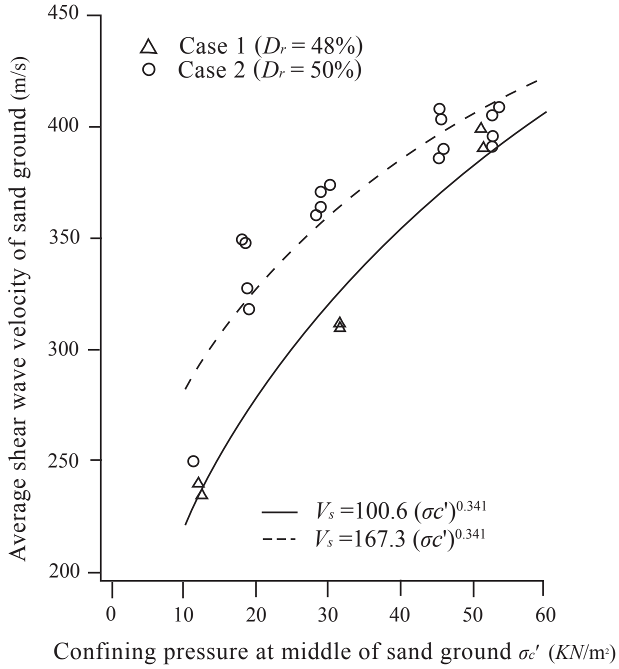


Fig. A-4.4 Relationship between the Average S Wave Velocity of a Sand Ground and the Confining Pressure [52)]

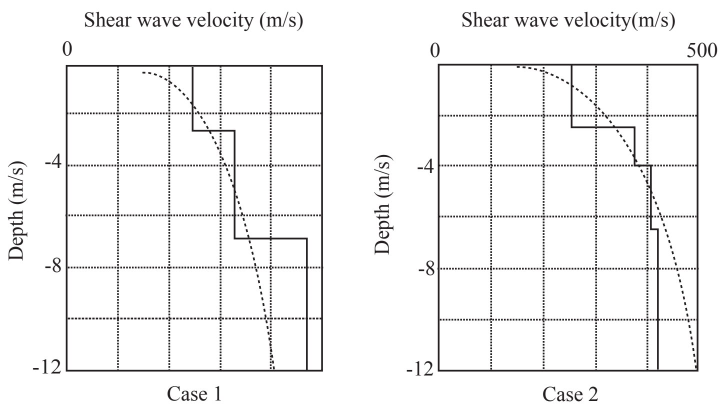


Fig. A-4.5 Distribution of S Wave Velocity in a Sand Ground [52)]

(c) Strain dependence of shear modulus and damping factor

Normally when the shear strain is small, the shear modulus is large and the damping factor is small, but as the strain increases, the former reduces and the latter increases, see **Chapter 3, 2.4.1 Dynamic Modulus of Deformation, Fig. 2.4.2**. The strain dependence characteristics of the shear modulus and the damping factor vary with the confining pressure and the soil properties of each stratum etc. Therefore, the shear modulus and damping factor values used should be obtained from soils tests corresponding to the shear strain level, as much as possible.

(d) Parameters necessary for nonlinear analysis

The parameters necessary for nonlinear analysis may be set based on **Part III, Chapter 5, 2.2.3 Performance verification (9) Performance Verification for Ground Motions (detailed method)**.

ANNEX 5 Evaluation of Ground Motion

1 Evaluation of Strong Ground Motion

(1) Outline

Methods for evaluating strong ground motions taking into consideration the source effects, the propagation path effects, and the site amplification factors include theoretical methods and semi-empirical methods. The theoretical methods model the medium from the source to the harbor as an elastic body, and evaluate the ground motions at the harbor based on the theory of elastodynamics. Among the semi-empirical methods, the empirical Green's function method is a method in which the measured ground motion from a medium or small earthquake, whose mechanism and propagation path is common with a large earthquake, is considered to be a Green function, and the ground motion for the large earthquake is synthesized by superposition.[67), 68), 69)] At this time, if a suitable medium or small earthquake record is not available to be used, the stochastic Green's function method has been proposed in which an artificial medium or small earthquake record is created, and then superposed, [70)] and this may be also classified as a semi-empirical method. In addition there are broad band hybrid methods in which the long period components of the ground motions are calculated by theoretical methods, the short period components are calculated using semi-empirical methods, and the two are superposed.[71)] In each of the above methods, it is known that if theoretical methods are applied to areas with comparatively well-known subsurface structures to periods longer than about 1 second, observed ground motions can be reproduced with good accuracy.[72)] However, although information on the subsurface structure has been collected activity, [73)] at present the areas for which the subsurface structure is sufficiently well known to enable theoretical methods to be applied are very limited. On the other hand, of the semi-empirical methods, in the empirical Green's function method, the effect of the subsurface structure included in the records of medium and small earthquakes is directly reflected in the prediction results. Also, in the probabilistic Green function method, which is likewise classified as a semi-empirical method, it is possible to utilize the site amplification factors evaluated from spectral inversion [18)] or similarly based on strong motion records at the site.[74)] Based on the above, at harbors for which there are comparatively plentiful strong motion records, it is desirable that semi-empirical methods are used in evaluating the design ground motion. In areas where the subsurface structure is comparatively well known, it is possible to use theoretical methods or broad band hybrid methods, but in this case the appropriateness of the subsurface structure model should be verified in advance from the viewpoint of consistency with the strong motion records.

Determining whether the results of a strong motion evaluation are valid or not should, as a rule, be carried out from the viewpoint of whether the calculation conditions and calculation procedure are in accordance with the contents of this section. However, comparison of the calculation results with strong motion records obtained under similar conditions is useful. For example, one can refer to strong motion records in the near source region of the 1995 Hyogo-ken Nambu Earthquake or the 2004 Niigata-ken Chuetsu Earthquake. However, normally the ground motions depend greatly on the source effects and the site effects, so because of these conditions it is possible for the amplitude of the calculated ground motions to differ greatly from the existing strong motion records. If a calculated result whose amplitude differs greatly from the existing strong motion records is obtained, it should be investigated whether it is possible to rationally explain the difference by the difference in the source effects, such as differences in the size of the earthquake or the site effects, and if it is possible to rationally explain the difference in this way, the results can be accepted. If a rational explanation is difficult, there should be a check for errors in inputs etc. In this way, comparison with the existing strong motion records is useful from the viewpoint of preventing simple errors. When carrying out a comparison with the existing strong motion records, comparison of the maximum acceleration should be avoided. This is because normally the maximum acceleration of a ground motion is easily affected by the high frequency component at 2Hz or higher. However, the high frequency component at 2Hz or higher does not have much effect on harbor facilities, so even though the maximum accelerations are compared, this does not verify the calculation results in the frequency range that has a large effect on the harbor facilities. Normally the maximum velocity is a better index than the maximum acceleration. Incidentally, the maximum velocity of the ground motions observed at the ground surface on sediments in the near source region of the 1995 Hyogoken Nambu Earthquake and the 2004 Niigata-ken Chuetsu Earthquake was about 100 to 150cm/s.

(2) Probabilistic Green Function Method

The probabilistic Green function method is a method in which first ground motions at the site are evaluated for a small earthquake, which are then superposed to obtain the motions for a large earthquake. The specific procedure is as follows.

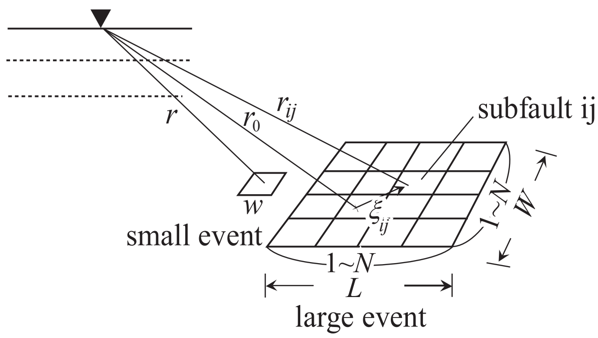


Fig. A-5.1 Probabilistic Green Function Method

First, focusing on one of the asperities, namely large event in **Fig. A-5.1** for the expected earthquake , the asperity is divided into $N \times N$. Then a small earthquake having the same area as each sub-fault of the asperity, namely small event in **Fig. A-5.1**, is considered. The Fourier amplitude of the ground motion from the small event at seismic bedrock which is equivalent to the probabilistic Green function at seismic bedrock, is defined as the product of the source spectrum of the small earthquake equation **(A-5.1)** and the propagation path effects equation **(A-5.2)**.[75)]

$$S(f) = R_{\theta\phi} FS \cdot PRTITN \cdot \frac{M_{0e}}{4\pi\rho V_S^3} \frac{(2\pi f)^2}{1+(f/f_c)^2} \tag{A-5.1}$$

$$P(f) = \frac{1}{r}\exp(-\pi f r / Q V_S) \tag{A-5.2}$$

where

- M_{0e} : seismic moment of small earthquake
- f_c : corner frequency of small earthquake
- ρ : density at the seismic bedrock
- V_S : S wave velocity at the seismic bedrock
- $R_{\theta\phi}$: radiation coefficient
- FS : amplification effect due to the free surface (=2)
- $PRTITN$: effect of partition of the ground energy into two horizontal components
- r : hypocentral distance of the small earthquake
- Q : Q value of the medium on the propagation path

When considering an earthquake occurring at an active fault, ρ =2.7g/cm^3 and V_S=3.5km/s may be assumed. An average value in all directions of 0.63 may be used for $R_{\theta\phi}$. When estimating the ground motion near the fault of an earthquake occurring at an active fault within about 10km from the fault, *PRTITN* =0.85 for the component normal to the strike direction, and 0.53 for the component parallel to the strike direction. These values are defined taking into consideration that in the near source region of an earthquake occurring at an active fault, the Fourier amplitude of the component normal to the strike direction is about 1.6 times [8)] that of the component parallel to the strike direction. When evaluating the ground motions far from an earthquake occurring at an active fault, and when evaluating the ground motions due to other earthquakes, *PRTITN* = 0.71 may be used assuming that the energy of the ground motion is equally distributed to the two horizontal components. In any case, it is necessary to set *PRTITN* so that the sum of squares of the two horizontal components is 1. **Table A-5.1** shows the standard values of *PRTITN*.

Table A-5.1 Standard Values of *PRTITN*

	Near the fault	Except near the fault
Subduction-zone earthquake	0.71	0.71
Inland active fault earthquake	0.85 (component normal to strike) 0.53 (component parallel to strike)	0.71
M6.5 earthquake just beneath the site	0.71	0.71

The seismic moment of the small earthquake M_{0e} can be obtained by dividing the seismic moment of the asperity by N^3. The corner frequency f_c of the small earthquake can be obtained from the following equation by Brune.[76), 77)]

$$f_c = 0.66 V_S \big/ \sqrt{S_e} \tag{A-5.3}$$

where

S_e : rupture area of the small earthquake

Equation **(A-5.3)** is "Brune's equation (36)" [76)] as it is. By combining equation **(A-5.3)** and Esherby's equation for a circular crack, [59)] it is possible to derive the well-known equation that expresses the corner frequency as a function of the seismic moment and the stress drop. In equation **(A-5.2)**, an appropriate regional Q value should be used .

The wave profile that satisfies the Fourier amplitudes at the seismic bedrock defined as above is obtained by Boore's method [75)] or by Nozu and Sugano's method, [44)] and this is taken to be the stochastic Green's function at the seismic bedrock.

Next, the ground motion from the small event, the stochastic Green's function, at the ground surface is obtained. In this case the effect of the sediments on both the Fourier amplitude and phase of the ground motion, namely the site effects, is taken into account. Specifically, it can be calculated by the following method.[74)] As stated previously, normally the Fourier amplitude of the ground motion is given by the product of the source effects, the propagation path effects, and the site effects, and the group delay time of the ground motion is given by the sum of the source effects, the propagation path effects, and the site effects.[1)]

$$O(f) = S(f)P(f)G(f) \qquad \text{(see 1.1.1)}$$

$$t_{gr}{}^{O}(f) = t_{gr}{}^{S}(f) + t_{gr}{}^{P}(f) + t_{gr}{}^{G}(f) \qquad \text{(see 1.1.2)}$$

Now if an earthquake whose size and hypocentral distance are sufficiently small is observed at the site under consideration, it is considered that expect for the shift along time axis, the group delay time in the record practically expresses the third term on the right side of equation **(1.1.2)**, in other words the site effects. Therefore, if the Fourier transform of the stochastic Green's function previously obtained for the seismic bedrock is multiplied by $G(f)$, and the Fourier transform of the record satisfying the above conditions after adjusting its amplitude in the frequency domain to 1, then its inverse Fourier transform gives the stochastic Green's function at the ground surface. Expressing this process in the form of a specific equation gives the following.

$$A(f) = A_b(f)\ G(f) \frac{O_S(f)}{\left| O_S(f) \right|} \tag{A-5.4}$$

where

$A(f)$: Fourier transform of the stochastic: C Green's function at the ground surface (complex number)
$A_b(f)$: Fourier transform of the stochastic: C Green's function at seismic bedrock (complex number)
$G(f)$: site amplification factors between seismic bedrock and ground surface (real number)
$O_S(f)$: Fourier transform of medium or small earthquake record obtained at the site (complex number)

It is desirable that the medium or small earthquake record for the site used at this time should have an incident angle to the site as similar to that of the scenario earthquake under consideration as possible. This is because in this way it is possible to more appropriately take into consideration the effect of the sediments on the riser phase of the ground motions.

When evaluating the stochastic Green's function at the ground surface by this method, it is necessary that the site amplification factors $G(f)$ be evaluated in advance. For evaluating the site amplification factors, there are two main approaches. One approach is to extract the S wave component from the ground motion by some kind of method, and obtain its amplification factors.[18)] The other approach is to include not just the S wave component but also the surface waves component in the analysis, and to obtain the amplification factors of the Fourier spectrum including later phases.[23)] Which approach is taken depends on the objectives, but when carrying out strong motion prediction taking the contribution not only of the S waves but also the surface waves into account, the latter approach is necessary. In particular, if the method described above is used, the contribution of the S wave and the contribution of the surface waves are blended together in the group delay time of the medium or small earthquake record obtained at the site, so it is necessary to consider the contribution of both to the amplitude also.

The ground surface for the ground motion from the asperity can be calculated by superposing the Green's function at the ground surface using the following equations, [81] see **Fig. A-5.1**. By carrying out this superposition, the directivity effect in the direction of rupture propagation is taken into account.

$$U(t)=\sum_{i=1}^{N}\sum_{j=1}^{N}\left(r/r_{ij}\right)f(t)*u(t-t_{ij}) \tag{A-5.5}$$

$$f(t)=\delta(t)+\left\{1/n'/\left(1-e^{-1}\right)\right\}\sum_{k=1}^{(N-1)n'}\left[e^{-(k-1)/(N-1)/n'}\delta\{t-(k-1)\tau/(N-1)/n'\}\right] \tag{A-5.6}$$

$$t_{ij}=\left(r_{ij}-r_0\right)/V_S+\xi_{ij}/V_r \tag{A-5.7}$$

where

$U(t)$: ground motion from asperity
$u(t)$: stochastic Green's function at the ground surface
$f(t)$: function to correct for the difference in the slip velocity time function between a large earthquake and a small earthquake
r : hypocentral distance of small earthquake
r_{ij} : distance from element ij to the site of interest
N : number of sub-divisions (**Fig. A-5.1**)
τ : rise time
n' : integer for removing the apparent periodicity that appears when superposing the waves
r_0 : distance from the rupture starting point of the asperity to the site of interest
ξ_{ij} : distance from the rupture starting point to the ij^{th} element
V_S : S wave velocity of the seismic bedrock
V_r : rupture velocity

When there are several asperities, the same procedure is followed for each asperity, and the linear Level 2 earthquake ground motion at the ground surface is calculated by adding the contributions from each asperity. Finally, the Level 2 earthquake ground motion, 2E wave, at the top of the firm ground is calculated by seismic response analysis of the local soil deposit. The contribution of the background areas can normally be ignored without problem for the purpose of performance verification of port facilities.

In the above calculation process, the linear Level 2 earthquake ground motion at the ground surface is calculated at once, but this does not include the nonlinear behavior of the local soil deposit during a large earthquake, so it is necessary to be aware that this normally results in an overestimation. In order to calculate more realistic Level 2 earthquake ground motion at the ground surface, normally the Level 2 earthquake ground motion at the top of the firm ground is obtained at once, then a seismic response analysis is carried out again taking into account the nonlinear behavior of the local soil deposit.

Examples of the application of the method described here to past earthquakes are introduced in Reference 44). Also, a calculation program for the method described here is available on CD-ROM.[44]

(3) Empirical Green Function Method

The empirical Green function method is a method for evaluating the ground motions at the site of interest due to a large earthquake for the case where the records of small earthquakes that have occurred near the large earthquake under consideration have been obtained at the site of interest, by superposing these records. The small earthquake records used for superposition at this time are referred to as empirical Green's functions. The records obtained at the site of interest naturally include the effect of the propagation path effects and the site effects, so a major feature of this method is that the ground motions due to a large earthquake can be evaluated with good accuracy without evaluating these propagation path effects and the site effects. However, this method cannot be applied if it is not possible to obtain suitable small earthquake records at the site of interest. Also, as stated below, there are some items that require rather specialized consideration.

For superposition of the wave profiles, equations **(A-5.5)** to **(A-5.7)** of the stochastic Green's function method can be applied virtually as they are. However, for equation **(A-5.5)** it is necessary to substitute the following equation which contains a correction coefficient C, in order that the small earthquake be appropriately reflected.[81]

$$U(t)=\sum_{i=1}^{N}\sum_{j=1}^{N}\left(r/r_{ij}\right)f(t)*\left(Cu(t-t_{ij})\right) \tag{A-5.9}$$

The parameters N and C associated with the superposition are defined so as to satisfy the following equation.

$$M_{0a} / M_{0e} = CN^3$$
$$S_a / S_e = N^2 \qquad \text{(A-5.10)}$$

where

M_{0a} : seismic moment of the asperity
M_{0e} : seismic moment of the small earthquake
S_a : area of the asperity
S_e : rupture area of the small earthquake

As can be understood from the above, when applying the empirical Green function method, it is necessary to appropriately estimate the parameters of the small earthquake. For the seismic moment M_{0e} of the small earthquake, it is possible to refer to CMT solutions, [20] for example, of the F-net by the National Research Institute for Earth Science and Disaster Prevention. The rupture area S_e of the small earthquake can be obtained from the corner frequency f_c using equation **(A-5.3)**. To obtain the corner frequency of the small earthquake, the method [82] of obtaining the ratio of the spectra of earthquakes with different magnitudes that occurred nearby may be used.

Other points to note when applying the empirical Green function method include the radiation coefficient problem. The radiation coefficient of the seismic wave from the source has direction dependence according to theory, [20, 83] and depending on the mechanism of the small earthquake such as strike, dip and rake angle, and, by chance, the site of interest may correspond to a trough in the radiation coefficient. In that case, if the records are superposed as they are, it is possible that the large earthquake ground motions will be underestimated. Therefore, it is necessary to pay sufficient attention to the mechanisms of the small earthquakes used.

As described above, evaluation of ground motions by the empirical Green function method requires several specialist type judgments, so it is necessary to pay attention to these points.

2 Seismic Response Analysis of Local Soil Deposit

Seismic response analysis of local soil deposit may be carried out in accordance with the description in **ANNEX 4, 1 Seismic Response Analysis of Local Soil Deposit**. However, when the Level 2 earthquake ground motion is acting, the strain level in the local soil deposit tends to become particularly large, so it is necessary to pay particular attention to the analysis method.

3 Spatial Variations in the Ground Motion Considered in Performance Verification of Facilities

If the ground is significantly non-uniform in the horizontal direction within the area occupied by a structure, spatial variation in the ground motion may be brought about as a result. Therefore, it is desirable that the non-uniformity in the horizontal direction of the ground conditions within the area occupied by the structure be evaluated, and if the non-uniformity is significant, the spatial variation in the ground motion should be evaluated taking the effect of the non-uniformity into account. For this, it is also desirable to take into consideration the effect of the non-uniformity in the horizontal direction of the ground below the top of the firm ground. If the non-uniformity of the ground in the horizontal direction is significant, the most effective specific method for evaluating the spatial variation of the ground motion is the method of evaluating the ground motion at several points using the method stated in **ANNEX 5, 1 Evaluation of Strong Ground Motion (2)** and **(3)**, based on the records of seismometers installed at several points. Another method when the subsurface structure is sufficiently well-known is to carry out an evaluation using an appropriate numerical analysis method such as the finite element method or the finite difference method. This is a rather specialist discussion, but when applying the method described in **ANNEX 5, 1 Evaluation of Strong Ground Motion (2)** , it is necessary to pay sufficient attention to ensure that the physical meaning of the phase differences in the ground motions evaluated at the several points is not lost. The physical meaning of the phase differences in the ground motions evaluated by the method described in **ANNEX 5, 1 Evaluation of Strong Ground Motion (2)** can be lost if the stochastic Green's functions at the seismic bedrock of several points are separately evaluated, and randomness is contained in the phases of the stochastic Green's functions.[75] It is also possible if the origin of the time axes of the medium and small earthquake records used in equation **(A-5.4)** have been shifted, for example, if the trigger time is different at different ground points. Methods for dealing with the former include the method of taking the same stochastic Green's functions at the seismic bedrock for two points that are not too far distant.

The above considerations can also be applied in cases where the non-uniformity of the ground conditions in the horizontal direction is not significant, but a more convenient approach can be applied as described below.
If the non-uniformity in the horizontal direction of the ground conditions is not significant, the main cause of spatial variation in the ground motion is the wave propagation effect in the horizontal direction. The strain $\varepsilon(\omega)$ in the ground caused by the wave propagation effect is a function of the amplitude of the ground motion velocity $v(\omega)$ and the apparent wave propagation velocity $c(\omega)$, where ω is the angular frequency.

$$\varepsilon(\omega) = v(\omega)/c(\omega) \qquad \text{(A-5.11)}$$

As can be seen from equation **(A-5.11)**, $\varepsilon(\omega)$ is a decreasing function of $c(\omega)$, so the smaller the value of $c(\omega)$, the more disadvantageous it is for the structure. The seismic waves that cause a wave propagation effect include the surface waves and S waves, but at an arbitrary ω, the phase velocity of surface waves is lower than the phase velocity of S waves. Also, among the surface waves, the phase velocity is lowest in the fundamental mode of the Love wave or the fundamental mode of the Rayleigh wave. Therefore, taking into consideration the fundamental mode of the Love wave or the fundamental mode of the Rayleigh wave is the most disadvantageous for the structure.

The phase velocity of the surface waves depends on the frequency. If it is assumed that $c(\omega)$ does not depend on ω, either the effect of the high frequency component on the ground strain will be underestimated, or the effect of the low frequency component will be underestimated. Therefore, it is important to evaluate the frequency dependence of the phase velocity. It is desirable that the frequency dependence of the phase velocity be evaluated based on the results of in-situ array measurements, on the microtremor or the earthquake motion, or based on the elastic wave velocity structure above and below the top of the firm ground. As an example, **Fig. A-5.2** shows the relationship between the phase velocity of the Love wave and frequency at a certain location in Tokyo Bay. The solid line is the theoretical phase velocity calculated from the S wave velocity structure model in **Table A-5.2**. The S wave velocity structure model in **Table A-5.2** includes the deep sediments down to the upper crust. If a model that excludes the deep sediments is used in the calculation, the phase velocity on the long period side is underestimated. The ■ mark in **Fig. A-5.2** are the phase velocities obtained from the array measurement results. At this point, the phase velocity of the fundamental mode of Love wave is about 400m/s at a period of 1 second, and about 750m/s at a period of 3 seconds. Therefore, if a constant value of 400m/s is used as the phase velocity which does not vary with frequency, the effect of the Love wave at a period of 3 seconds will be overestimated. Conversely, if a constant value of 750m/s is used, the effect of the Love wave at a period of about 1 second will be underestimated.

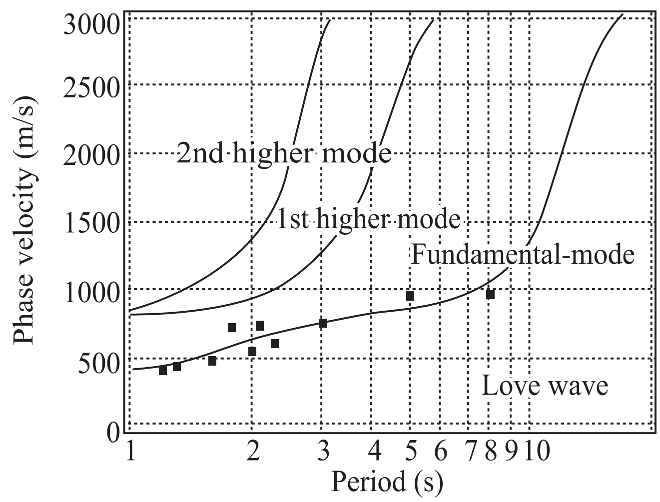


Fig. A-5.2 Relationship between the Phase Velocity of the Love Wave and Frequency at a Certain Location in Tokyo Bay [84)]

Table A-5.2 S Wave Velocity Structure Model [84)]

Thickness (m)	S-wave velocity (m/s)	Density (t/m^3)
50	250	1.8
120	410	1.9
1580	800	1.9
1250	1200	2.1
3100	2600	2.6
–	3400	2.6

Normally the ground motions evaluated by the methods of **1.2 Level 1 Earthquake Ground Motions used in Performance Verification of Facilities**, and **1.3 Level 2 earthquake ground motions used in Performance Verification of Facilities** include several frequency components, and each frequency component can cause a wave propagation effect. In this case, the spatial variation of a ground motion can be simply evaluated taking into consideration the frequency dependence of the phase velocity by the following method. Assume the ground motion time history evaluated at a reference point (x=0, y=0) at the relevant depth of the horizontally layered ground based on the methods of **1.2 Level 1 Earthquake Ground Motions used in Performance Verification of Facilities**, and **1.3 Level 2 earthquake ground motions used in Performance Verification of Facilities**, is $a_0(t)$. Also, assume the frequency dependent phase velocity corresponding to the point is $c(\omega)$. In these circumstances the ground motion time history $a(t)$ at an arbitrary point (x, y) at the same depth can be defined as follows:

(1) Take the Fourier transform of $a_0(t)$.

(2) Calculate the Fourier transform of $a(t)$ from the following equation.

$$A(\omega) = A_0(\omega)\exp[-i(k_x x + k_y y)] \tag{A-5.12}$$

$$k_x = (\omega/c(\omega))\cos\theta \tag{A-5.13}$$

$$k_y = (\omega/c(\omega))\sin\theta \tag{A-5.14}$$

where

$A_0(\omega)$: fourier transform of $a_0(t)$

$A(\omega)$: fourier transform of $a(t)$

θ : angle between the positive direction of the x-axis and the direction of propagation of the seismic wave

(c) Take the inverse Fourier transform of $A(\omega)$ to obtain $a(t)$.
Ideally, $c(\omega)$ should be defined taking into account the type of seismic waves included in the ground motions $a_0(t)$ evaluated at a certain point by the methods of **1.2 Level 1 Earthquake Ground Motions used in Performance Verification of Facilities**, and **1.3 Level 2 Earthquake Ground Motions used in Performance Verification of Facilities**. However, in reality the evaluated seismic wave frequently includes various types of wave, such as surface waves and S waves, etc., so it is not easy to extract the surface waves only. Therefore, considering the most disadvantageous conditions for the facilities, the smaller of the phase velocity of the fundamental mode of Love wave and the fundamental mode of Rayleigh wave may be used as the $c(\omega)$ in equation **(A-5.13)** and equation **(A-5.14)**. The angle θ may be taken to be the direction that is most disadvantageous for the facility.

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