

DYNAMIC ANALYSIS OF A ROCKFILL DAM

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DYNAMIC ANALYSIS OF A ROCKFILL DAM

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Synopsis ;

This model dam is a 93.0-m-high rockfill dam with a total storage capacity of 109 million m³ and a central core zone type. In this report an investigation on the stability of a rockfill dam during earthquakes was carried out using dynamic analysis.

Keywords ;

Rockfill Dam, Fill Material, Shear Strength, Shear Modulus, Triaxial Test, Dynamic Analysis

Introduction

This model dam is a 93.0-m-high rockfill dam with a total storage capacity of 109 million m³ and a central core zone type. In this report an investigation on the stability of a rockfill dam during earthquakes was carried out using dynamic analysis.

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1. The Flowchart of Analysis

The investigation on the stability of a dam during earthquakes is composed of static analysis, which includes embankment analysis, reservoir filling analysis, and dynamic analysis.

The properties of rockfill materials in the static and dynamic analyses were obtained from large-scale triaxial tests using rock materials transported from the quarry site of the model dam. In the embankment analysis, the body forces calculated from seepage analysis about the impervious core zone were applied to the core zone. The stresses of the upstream rock zone were decreased in a certain ratio, considering the effects of uplift and the void ratio of materials.

These initial static stresses were superposed on the dynamic stresses from the dynamic analysis, and then the evaluation of the stability of the dam body was carried out. The stability of the dam body was investigated about the point safety factor and the sliding, and the stability of surface sliding was also confirmed. The flowchart of analysis is indicated in Fig.-1.1.1.

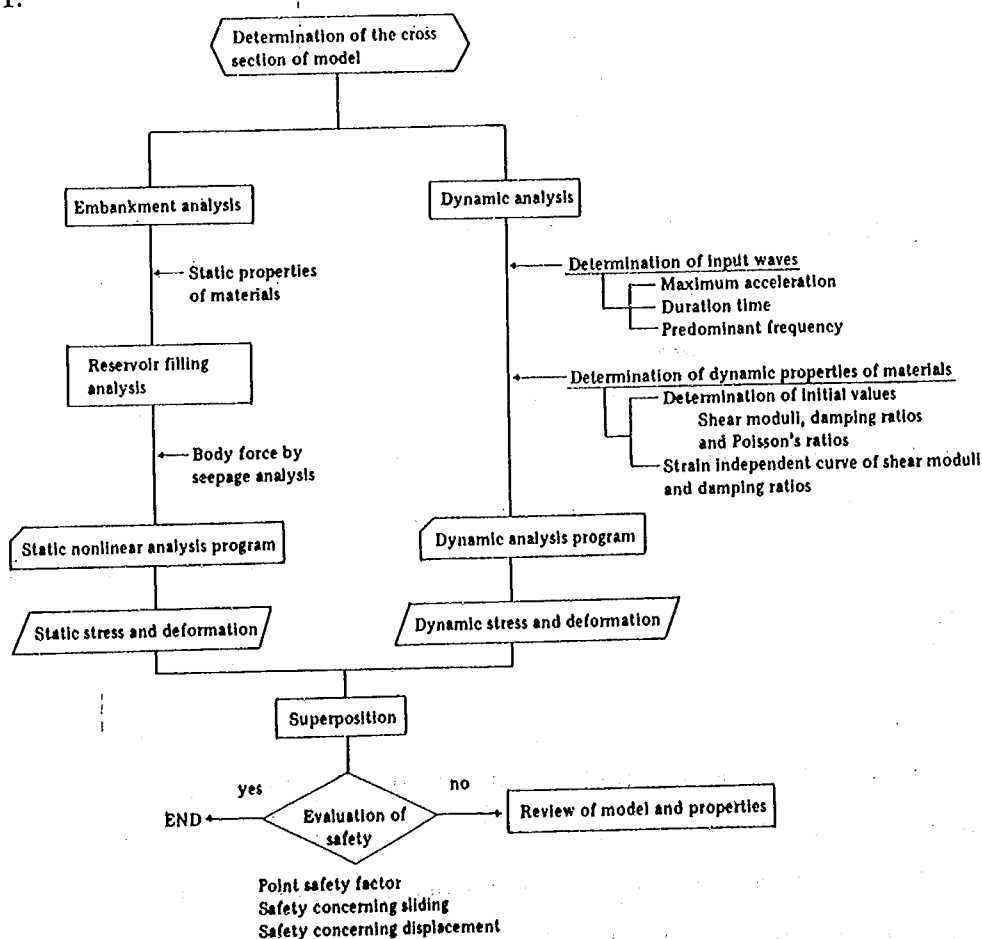


Fig.-1.1.1 Flowchart of analysis

2. Embankment Analysis

2.1 General Description of Analysis

The stress-strain relationships of soil and coarse grained granular materials show non-linearity. The principal factors that affect the stress-strain relationship are the void ratio of the material, the confining pressure, and the level of shear strain. The confining pressure of the element of a dam body increases according to the banking, and then the moduli of deformation also increase. Duncan-Chang applied a parabolic curve to the relationship of the confining pressure and elastic modulus using the results of laboratory tests. Kuhrhawy-Duncan-Seed also applied the same curve about Poisson's ratio. After them, tangent elastic modulus " E_t " and tangent Poisson's ratio " ν_t " are given as the following.

$$E_t = K \cdot P_a \left(\frac{\sigma_3}{P_a} \right)^n \cdot \left\{ 1 - \frac{R_f \cdot (1 - \sin \phi) (\sigma_1 - \sigma_3)}{2 \cdot c \cdot \cos \phi + 2 \cdot \sigma_3 \sin \phi} \right\}^2 \quad \dots (2.1)$$

$$\nu_t = \frac{G - F \log(\sigma_3/P_a)}{\left[1 - \frac{D \cdot (\sigma_1 - \sigma_3)}{K \cdot P_a \left(\frac{\sigma_3}{P_a} \right)^n \cdot \left\{ 1 - \frac{R_f \cdot (1 - \sin \phi) (\sigma_1 - \sigma_3)}{2 \cdot c \cdot \cos \phi + 2 \cdot \sigma_3 \sin \phi} \right\}} \right]^2} \quad \dots (2-2)$$

where

P_a ; Atmospheric pressure (the same unit as a stress)

c ; Cohesion

ϕ ; Internal friction angle

σ_1 ; The maximum principal stress

σ_3 ; The minimum principal stress

K, n, R_f ; Coefficients related to elastic modulus

G, F, D ; Coefficients related to Poisson's ratio

The process of leading the above mentioned equations (2.1) and (2.2) is explained in the next paragraphs.

2.1.1 Elastic Modulus

Duncan-Chang adopted the method of an approximation by parabolic curve after Kondner. The results of triaxial tests shown in Fig.-2.1.1 approximate a parabolic curve. In equation (2.1), R_f is the ratio of a breaking deviatoric stress $(\sigma_1 - \sigma_3)_f$ at a triaxial test and the ultimate breaking stress $(\sigma_1 - \sigma_3)_{ult}$ at a parabolic curve approximation. Therefore it means a higher approximation that R_f is near 1.0.

where

$\sigma_1 - \sigma_3$; Deviatoric stress

ϵ_a ; Axial strain

a, b ; Constants obtained from experiments

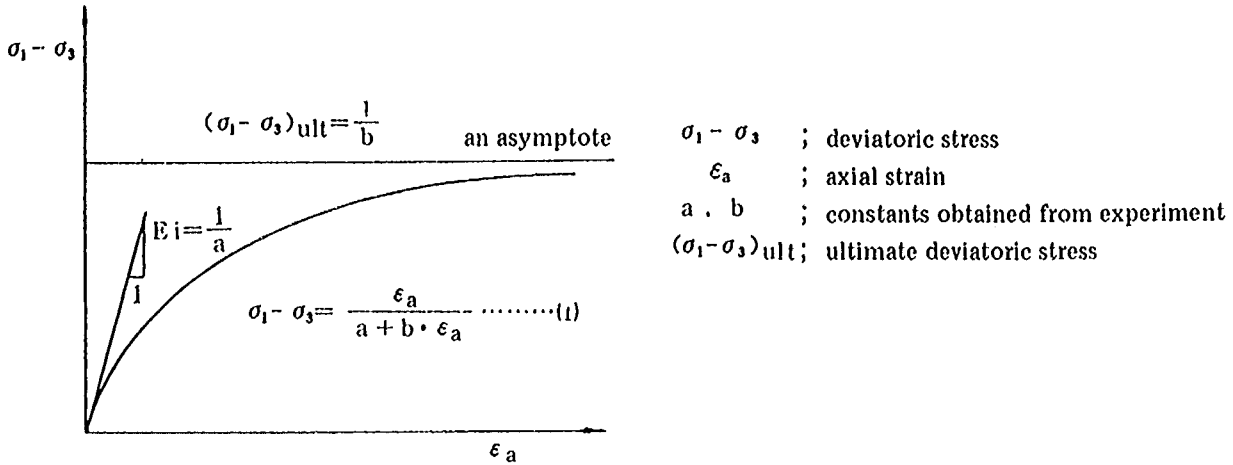


Fig.-2.1.1. Parabolic approximation of triaxial test

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \quad 0.75 \leq R_f \leq 1 \quad \dots\dots (2.3)$$

Equation (2.4) is an approximate one.

$$\sigma_1 - \sigma_3 \approx \frac{\epsilon_a}{a + b \cdot \epsilon_a} \quad \dots\dots (2.4)$$

Equation (2.5) is gained from equations (2.3) and (2.4).

$$\sigma_1 - \sigma_3 = \frac{\epsilon_a}{\left\{ \frac{1}{E_i} + \frac{R_f \cdot \epsilon_a}{(\sigma_1 - \sigma_3)_f} \right\}} \quad \dots\dots (2.5)$$

Constant a is obtained from the relationship between $\epsilon_a/(\sigma_1 - \sigma_3)$ and ϵ_a , and constant b is also the result of the inclination of the same relationship (see Fig.-2.1.2.).

The initial elastic modulus " E_i " depends on a confining pressure. Janbu presented the equation (2.6) and concluded that the relation of the initial elastic modulus and the confining pressure is linear, both in logarithmic scale.

$$E_i = K \cdot P_a \left(\frac{\sigma_3}{P_a} \right)^n \quad \dots\dots (2.6)$$

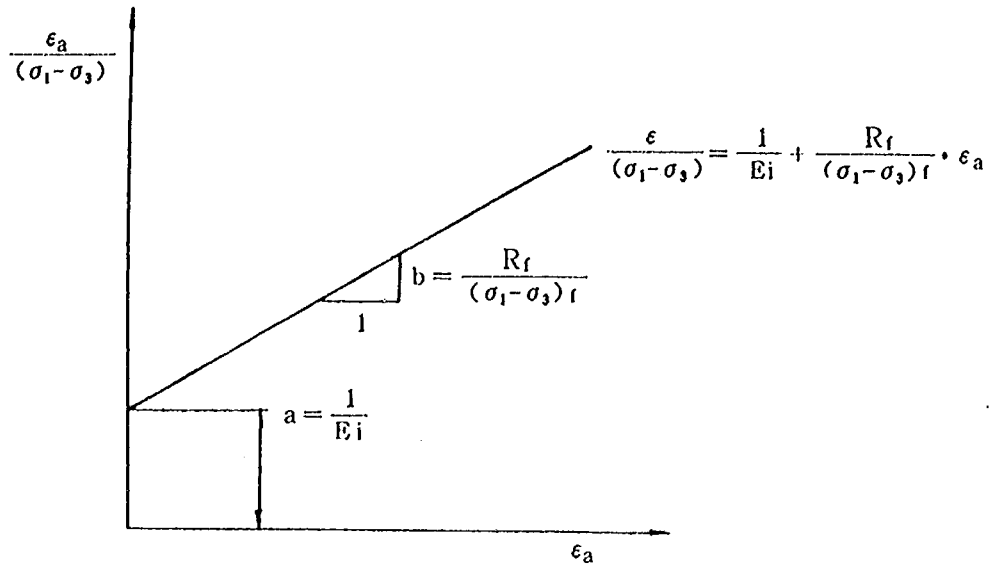


Fig.-2.1.2 $\frac{\epsilon_a}{\sigma_1 - \sigma_3} \sim \epsilon_a$

where

E_i ; Initial elastic modulus

K, n ; Constants obtained from the experiments

P_a ; Atmospheric pressure

σ_3 ; Confining pressure at triaxial testing

(Minimum Principal stress)

K and n are obtained from Fig.2.1.3 as the following.

$$\log E_i = \log K + \log P_a + n \cdot \log (\sigma_3 / P_a)$$

$$\log P_a = 0 (\because P_a = 1 \text{ Kgf/cm}^2)$$

$$\log E_i = \log K + n \cdot \log \sigma_3$$

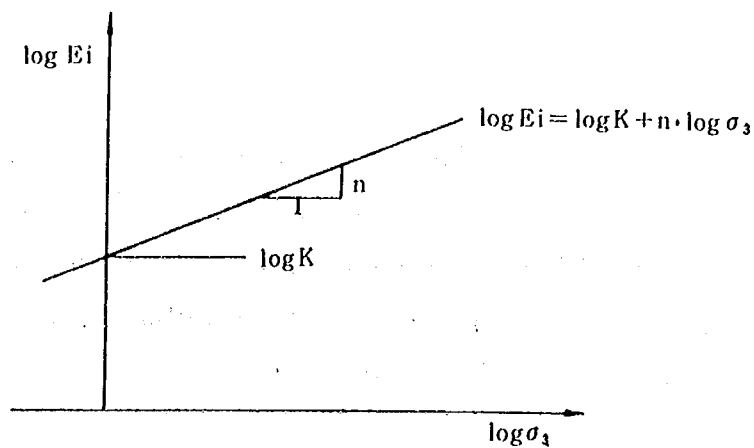


Fig.-2.1.3 $\log E_i \sim \log \sigma_3$

The deviatoric breaking stress is given from Mohr-Coulomb's failure criterion as the following.

$$(\sigma_1 - \sigma_3)_f = \frac{2 \cdot c \cdot \cos \phi + 2 \cdot \sigma_3 \sin \phi}{1 - \sin \phi}$$

The tangent elastic modulus " E_t " (equation 2.1) is obtained from equation (2.4), (2.5), and (2.6) using the stress ~ strain relationship.

$$\begin{aligned} E_t &= \frac{\partial(\sigma_1 - \sigma_3)}{\partial \epsilon_a} = \frac{\partial \frac{\epsilon_a}{a + b \cdot \epsilon_a}}{\partial \epsilon_a} = \frac{a}{(a + b \cdot \epsilon_a)^2} \\ &= \frac{\frac{1}{E_i}}{\left\{ \frac{1}{E_i} + \frac{R_f \cdot \epsilon_a}{(\sigma_1 - \sigma_3)_f} \right\}^2} \\ &= \frac{(\sigma_1 - \sigma_3)_f^2}{E_i} \cdot \frac{E_i^2 \left\{ 1 - \frac{R_f (\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} \right\}}{(\sigma_1 - \sigma_3)^2} \\ &= K \cdot P_a \cdot \left(\frac{\sigma_3}{P_a} \right)^n \cdot \left\{ 1 - \frac{R_f (1 - \sin \phi) (\sigma_1 - \sigma_3)}{2 \cdot c \cdot \cos \phi + 2 \cdot \sigma_3 \cdot \sin \phi} \right\}^2 \end{aligned} \quad \dots (2.1)$$

2.1.2 Poisson's Ratio

In general, the relations between volumetric strain, axial strain, and radial strain at the triaxial specimens are expressed as the following.

$$\epsilon_r = \frac{1}{2}(\epsilon_v - \epsilon_a)$$

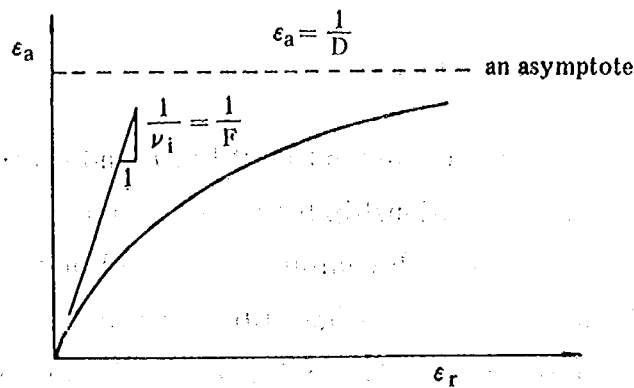


Fig.-2.1.4 $\epsilon_a \sim \epsilon_r$

where

ϵ_r ; Radial strain

ϵ_a ; Axial strain

ϵ_v ; Volumetric strain

Equation (2.7) is the relation of ϵ_a to ϵ_r , approximated by a parabolic curve. (see Fig.-2.1.4)

$$\epsilon_a = \frac{\epsilon_r}{F + D \cdot \epsilon_r} \text{ or } \frac{\epsilon_r}{\epsilon_a} = F + D \cdot \epsilon_r \quad \dots (2.7)$$

Equation (2.8) shows the tangent Poisson's ratio.

$$\nu_t = -\frac{\partial \epsilon_r}{\partial \epsilon_a} \text{ or } \frac{1}{\nu_t} = -\frac{\partial \epsilon_a}{\partial \epsilon_r} \quad \dots (2.8)$$

$$\frac{1}{\nu_t} = -\frac{\partial \epsilon_a}{\partial \epsilon_r} = -\frac{F + D \cdot \epsilon_r - D \cdot \epsilon_r}{(F + D \cdot \epsilon_a)^2}$$

$$\nu_t = -\frac{\partial \epsilon_r}{\partial \epsilon_a} = -\frac{(F + D \cdot \epsilon_r)^2}{F}$$

ϵ_r is expressed by equation (2.9) from equation (2.7).

$$\epsilon_r = \frac{F \cdot \epsilon_a}{(1 - D \cdot \epsilon_a)} \quad \dots (2.9)$$

therefore

$$\nu_t = -\frac{F \cdot \epsilon_a / (1 + D \cdot \epsilon_a)^2}{\epsilon_a^2 \cdot F} = -\frac{F}{(1 - D \cdot \epsilon_a)^2} \quad \dots (2.10)$$

The relation between F and D is indicated in Fig.-2.1.5. F and D are given as a constant and an inclination each from the relationship between ϵ_r/ϵ_a and ϵ_r .

The initial Poisson's ratio " ν_i " (at each confining pressure) depends on the confining pressure, like the initial elastic modulus. Fig.-2.1.6 shows the relationship of Poisson's ratio to the minimum principal stress. The initial Poisson's ratio connected with the confining pressure is obtained from the relation of ν_i to $\log \sigma_3$.

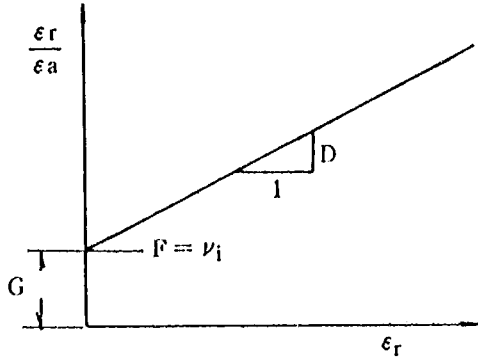


Fig.-2.1.5 $\epsilon_r \sim \epsilon_i / \epsilon_a$

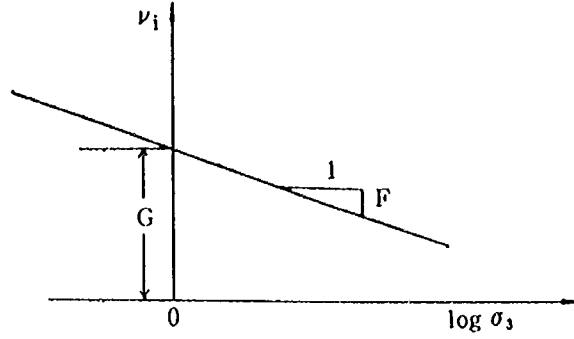


Fig.-2.1.6 $\log \sigma_1 \sim \nu_i$

$$\begin{aligned}\nu_i &= G - F \cdot \log(\sigma_3 / P_a) \\ &= G - F \cdot \log \sigma_3\end{aligned}\quad \dots (2.11)$$

where

$$P_a = 1 \text{ Kgf/cm}^2$$

from equation (2.5)

$$\begin{aligned}\epsilon_a &= \frac{\frac{\sigma_1 - \sigma_3}{E_i}}{\left\{1 - \frac{R_f \cdot (\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f}\right\}} \\ &= \frac{(\sigma_1 - \sigma_3)}{K \cdot P_a \left(\frac{\sigma_3}{P_a}\right)^n \cdot \left\{1 - \frac{R_f (\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f}\right\}}\end{aligned}\quad \dots (2.12)$$

Equation (2.2) is finally obtained from equations (2.10) and (2.12).

2.1.3 The Process of Calculation

It is a basic principle for embankment analysis that elastic moduli and Poisson's ratios depend on the confining pressure. These deformation properties are used in equation (2.13), which shows the stress-strain relation of the isotropic material in a plain-strain condition.

$$\begin{Bmatrix} \Delta \sigma_x \\ \Delta \sigma_y \\ \Delta \tau_{xy} \end{Bmatrix} = \frac{E_t}{(1 - \nu_t)(1 - 2\nu_t)} \cdot \begin{bmatrix} 1 - \nu_t & \nu_t & 0 \\ \nu_t & 1 - \nu_t & 0 \\ 0 & 0 & \frac{1 - 2\nu_t}{2} \end{bmatrix} \cdot \begin{Bmatrix} \Delta \epsilon_x \\ \Delta \epsilon_y \\ \Delta \gamma_{xy} \end{Bmatrix}\quad \dots (2.13)$$

A calculation is advanced using the method of load increment. The whole load is divided into several small-load increments. In each load increment, the behavior of soil material is assumed to be linear. Thus the leastic modulus and Poisson's ratio of each element in each load increment are evaluated in accordance with the calculated stress. The average stress in each load increment is used to calculate the tangent elastic modulus values and the tangent Poisson's ratio values. Each increment is analyzed twice, the first time using the modulus and Poisson's ratio values based on the stresses at the beginning of the increment, and the second time using the shear modulus and Poisson's ratio values based on the average stresses during the increment. If the stress level decreases during the increment, the unloading-reloading elastic modulus E_{ur} is used in the second evaluation.

$$E_{ur} = K_{ur} \cdot P_a \cdot \left(\frac{\sigma_3}{P_a} \right)^n \quad \dots (2.14)$$

in which K_{ur} is the unloading-reloading modulus number.

The initial stresses in each element of the dam body are estimated as follows.

$$\begin{aligned} \sigma_x &= \gamma \cdot h \cdot \nu / (1 - \nu) \\ \sigma_y &= \gamma \cdot h \\ \tau_{xy} &= 0 \end{aligned} \quad \dots (2.15)$$

Where ϵ_r is 0 and " $\gamma \cdot h$ " is the effective overburden pressure at the center of an element. The initial stresses of the triangular elements under the slope surface are calculated using equation (2.16)

$$\begin{aligned} \sigma_x &= \gamma \cdot h \cdot \nu / (1 - \nu) \\ \sigma_y &= \gamma \cdot h \\ \tau_{xy} &= 0.5 \cdot \gamma \cdot h \cdot \sin \alpha \end{aligned} \quad \dots (2.16)$$

in which " α " is the angle of the surface, " γ " is the unit weight of the soil, and " h " is the depth from the slope surface to the center of an element.

2.2 Model of Finite Element Mesh

A typical cross-section of a rockfill dam is used for the analysis.

The model used for analysis is a two-dimensional finite element mesh (see Fig.-2.2.1). A general description is shown in Table-2.2.1. The model has 16 layers for banking, including 4 layers for a cofferdam. The numbers of the element and nodal point are 267 and 288 each.

As for the boundary condition, the foundation is assumed to be rigid and the analysis is carried out only for the rockfill dam body.

Table-2.2.1 General description of analyzed model

Dam height	93 m		
Crest width	12 m		
Slope	Upstream 1:2.6 Downstream 1:2.0		
Zone section	Slope of core	Upstream	1:0.22
		Downstream	1:0.22
	Slope of filter	Upstream	1:0.47
		Downstream	1:0.47
Design water level	EL 304.5 m (89.5 m)		

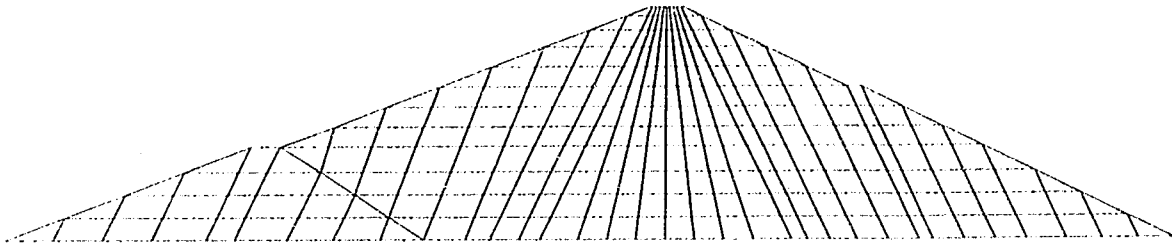


Fig.-2.2.1 Finite element idealization

2.3 Properties of Rockfill Materials for Analysis

The properties used in the embankment analysis of a rockfill dam are indicated in Table-2.3.1. Table-2.3.2 shows the cases analyzed.

The observed maximum strain of settlement of a rockfill dam during banking is about 2%, and, on other hand, elastic moduli in triaxial laboratory tests begin to change when the axial strain becomes 2~4%. So the parabolic approximations of the stress-strain relations from the triaxial test results were carried out at an axial strain of less than 4%. Poisson's ratio was approximated from $\epsilon_r/\epsilon_a \sim \epsilon_r$ relations under the radial strain " ϵ_r " of 2%.

The properties of rockfill materials are obtained from Fig.-2.3.1~Fig.-2.3.8. Fig.-2.3.1 shows the relationships of the confining pressure to " K " and " n " those concern elastic moduli of rock and filter materials. " K " and " n " are presented as the constant and the inclination of the straight line respectively.

The axial strain of data used for making out the input properties range from 0 to 4%. From Fig.-2.3.1, it is recognized that the elastic modulus increases according to the increase in the confining pressure. Fig.-2.3.2 shows the relationship between the confining pressure and " G " and " F " concerning Poisson's ratio value. In this case, the axial strain of data used for processing the test results ranges from 0 to 2%, and the Poisson's ratio value decreases in accordance with the increase in the confining pressure. Fig.-2.3.3 indicates the relations of the confining pressure to K , n , G , and F of the core materials.

**Table-2.3.1 Static physical properties of rockfill materials
(Embankment analysis)**

Embankment analysis	E_1			ν_1			γ_t g/cm ³	C Kg/cm ²	ϕ_d
	K	n	R_f	G	F	D			
							1.88	0	42
Rock ① SR02	850	0.371	0.487	0.324	0.269	13.82	γ_d 2.045	1.56	46
② SR04	1086	0.218	0.767	0.221	0.200	14.50	γ_d 1.883	1.53	44
							1.81	0	36
Filter ① SF01	772	0.248	0.869	0.271	0.163	10.74	2.049	0.55	38
② SF02	608	0.419	0.998	0.252	0.173	11.16	2.019	0.44	37
							1.85	0	35
Core SC10	141	0.941	1.039	0.397	0.098	7.96	2.171	0.31	36

Table-2.3.2 Case for embankment analysis

Calculation case of reservoir filling analysis		E Elastic moduls			ν Poisson's ratio			γ_t g/cm ³	C Kg/cm ²	ϕ_d
		K	n	R_f	G	F	D			
Case 1	Rock ① SR02	850	0.371	0.387	0.324	0.269	13.82	1.88	0	42
	Filter ② SF02	608	0.419	0.998	0.252	0.173	11.16	1.81	0	36
	Core SC10	141	0.941	1.039	0.397	0.098	7.96	1.85	0	35
Case 2	Rock ① SR02	850	0.371	0.487	0.324	0.269	13.82	1.88	0	42
	Filter ① SF01	772	0.248	0.869	0.271	0.163	10.74	1.81	0	36
	Core SC10	141	0.941	1.039	0.397	0.098	7.96	1.85	0	35
Case 3	Rock ② SR04	1086	0.218	0.767	0.221	0.200	14.50	1.88	0	42
	Filter ② SR02	608	0.419	0.998	0.252	0.173	11.16	1.81	0	36
	Core SC10	141	0.941	1.039	0.397	0.098	7.96	1.85	0	35
Case 4	Rock ② SR04	1086	0.218	0.767	0.221	0.200	14.50	1.88	0	42
	Filter ① SF01	772	0.248	0.869	0.271	0.163	10.74	1.81	0	36
	Core SC10	141	0.941	1.039	0.397	0.098	7.96	1.85	0	35

The relations between $\epsilon_a/(\sigma_1 - \sigma_3)$ and ϵ_a of rock and filter materials are shown in Fig.-2.3.4 and Fig.-2.3.5, and they are the so-called relationship of the reciprocal number of the elastic modulus to the axial strain.

The approximation using a straight line in the initial range of the axial strain is not proper for the filter materials from Fig.-2.3.5. For rock materials, it is not noticeable that the relation of the reciprocal number of the elastic modulus and the axial strain brings close to a straight line in the initial strain range, but it brings far from a straight line according to the increase of the axial strain.

Figs.-2.3.6 and 2.3.7 explain the relation of Poisson's ratio " ν " ($\nu = -\epsilon_r/\epsilon_a$) to the axial strain " ϵ_a " of rock and filter materials. Poisson's ratio decreases according to the rise of the confining pressure, but increases corresponding to the increase of the radial strain. Fig.-2.3.8 indicates the relationships of $\epsilon_a/(\sigma_1 - \sigma_3)$ to ϵ_a and ϵ_r/ϵ_a to ϵ_r that concern core

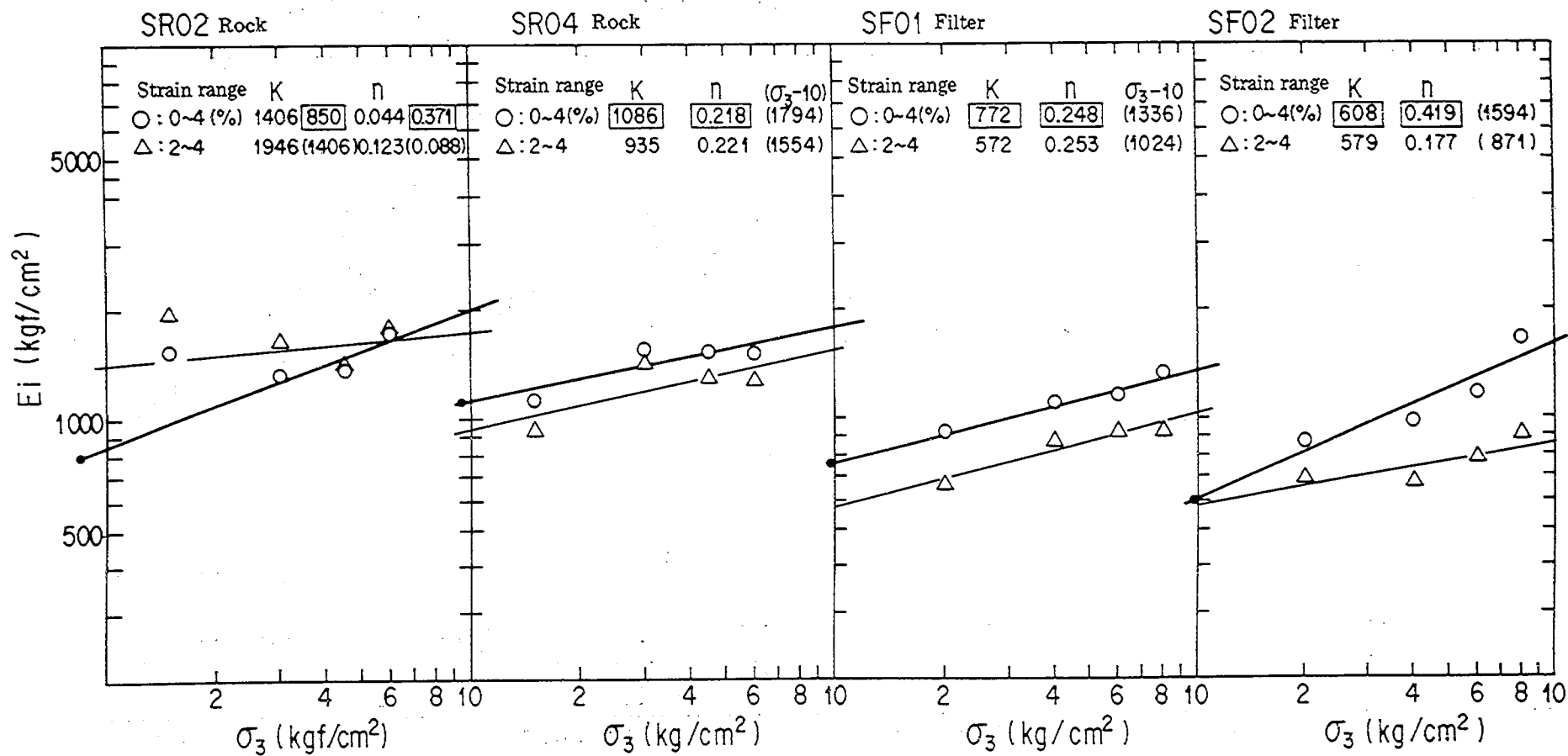


Fig.2.3.1 Relation between confining pressure and K , n

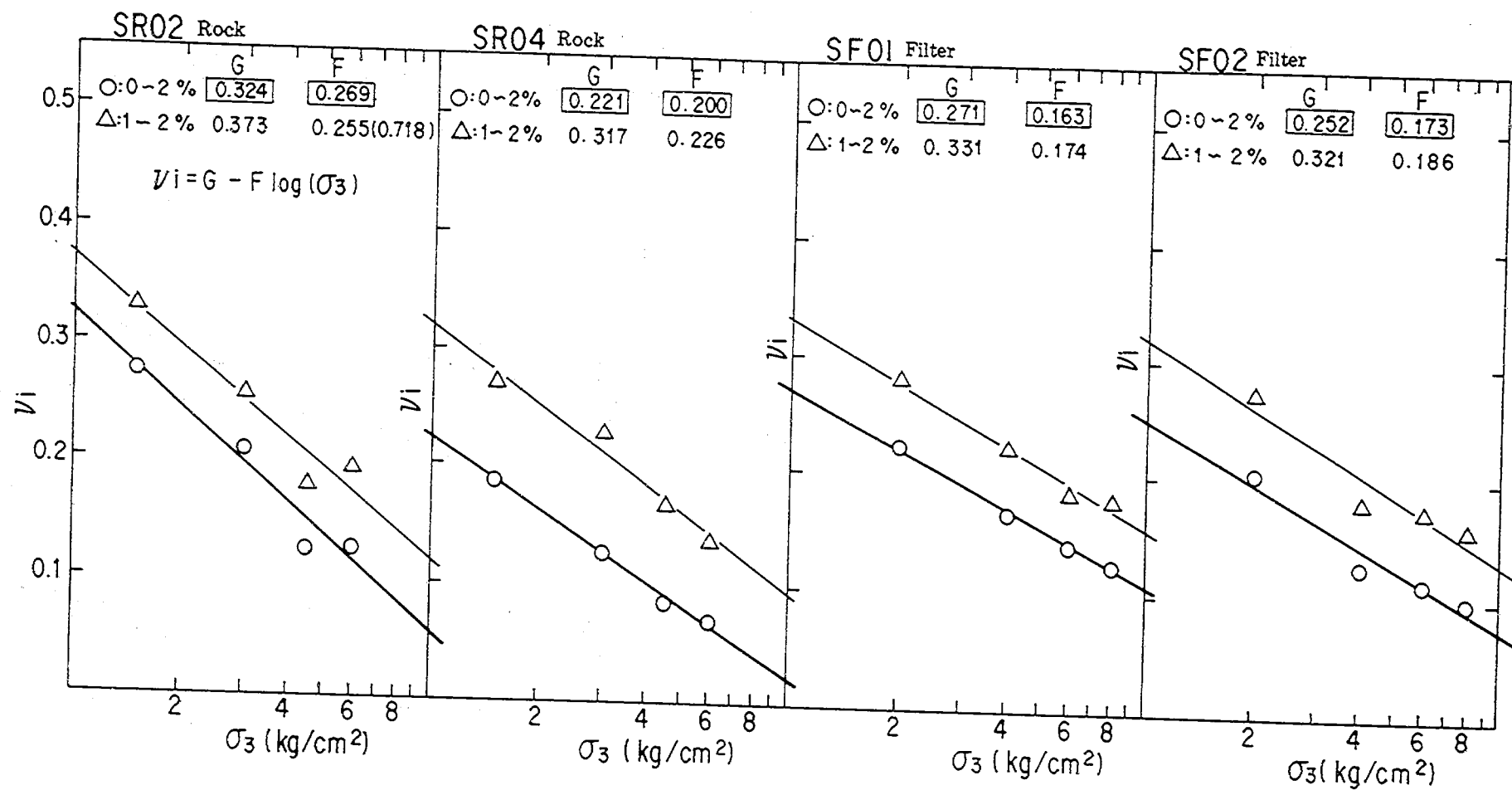
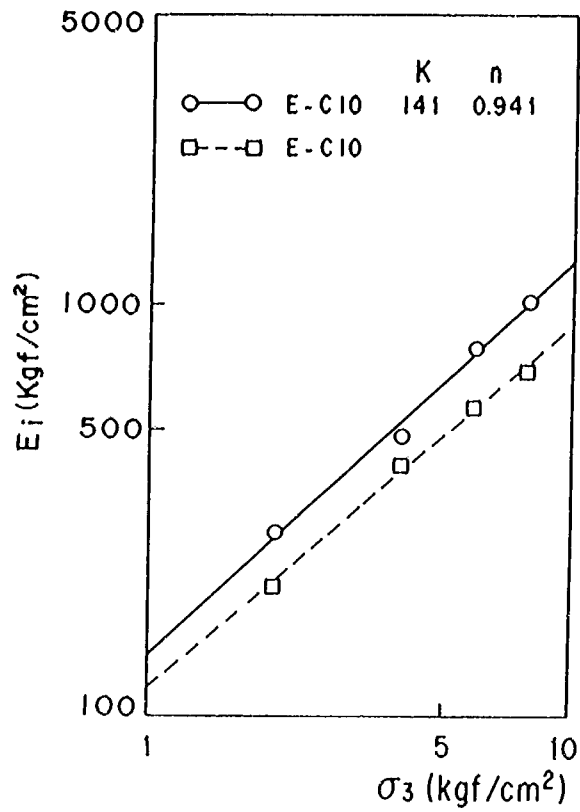
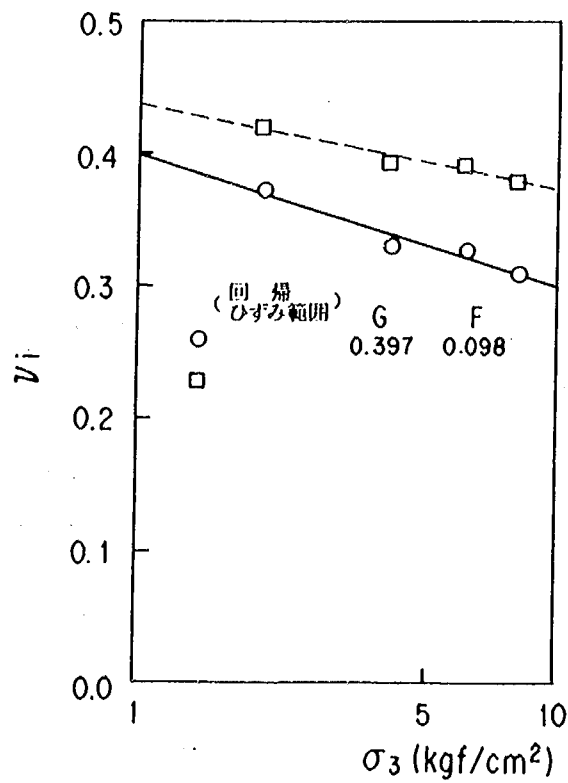


Fig.-2.3.2 Relation between confining pressure and G,F



Relation between confining pressure and K,n



Relation between confining pressure and G,F

Fig.-2.3.3 Constants concerning the deformability of core material

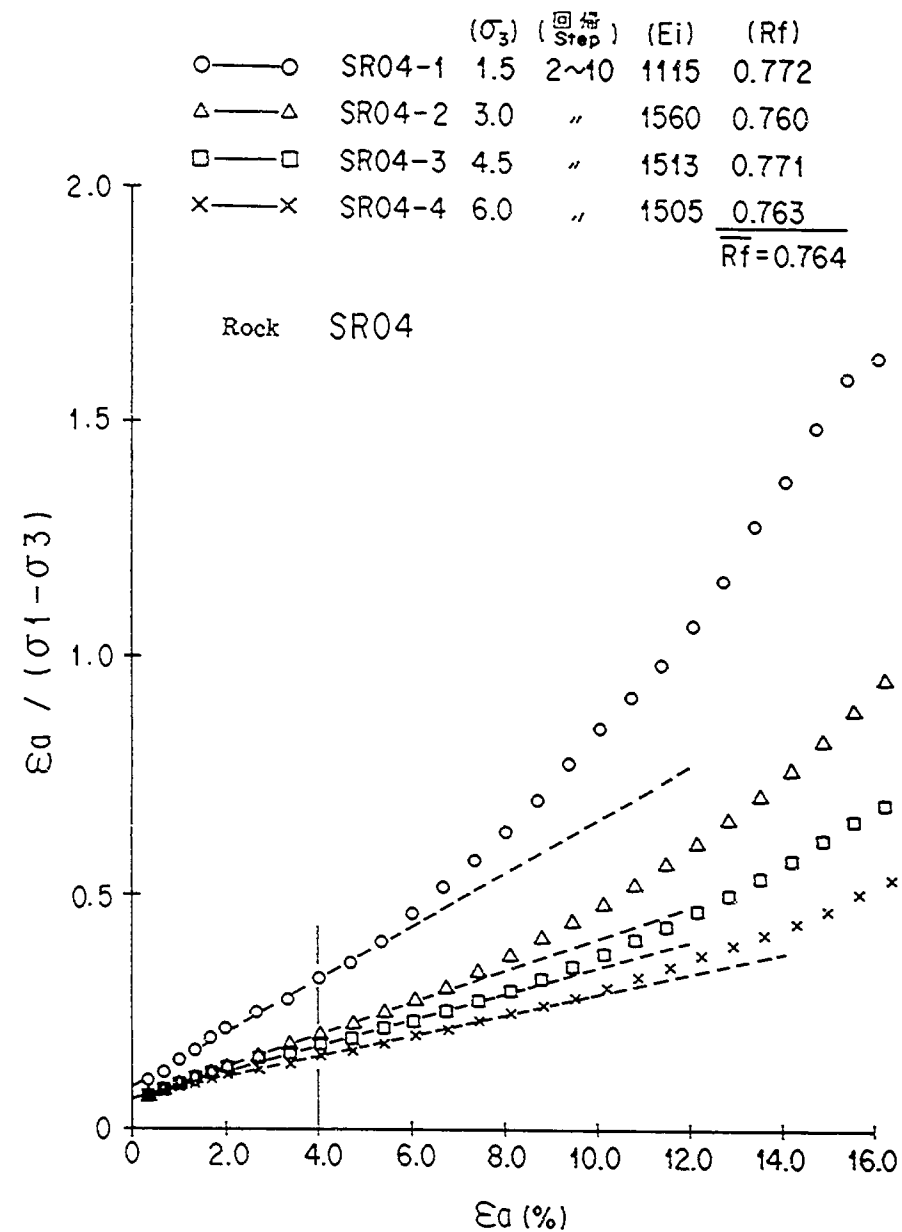
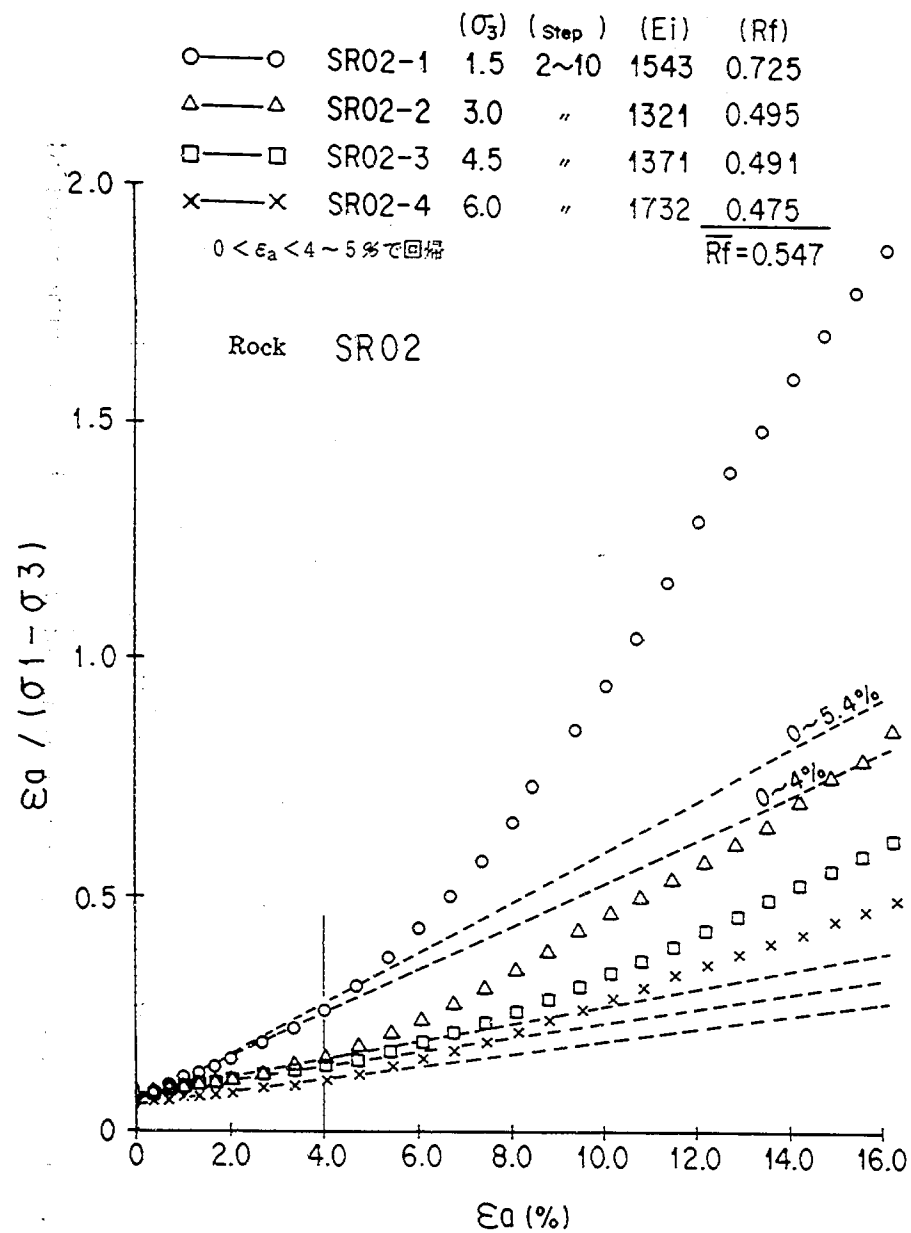


Fig.-2.3.4 $\epsilon_a / (\sigma_1 - \sigma_3) \sim \epsilon_a$ (Rock)

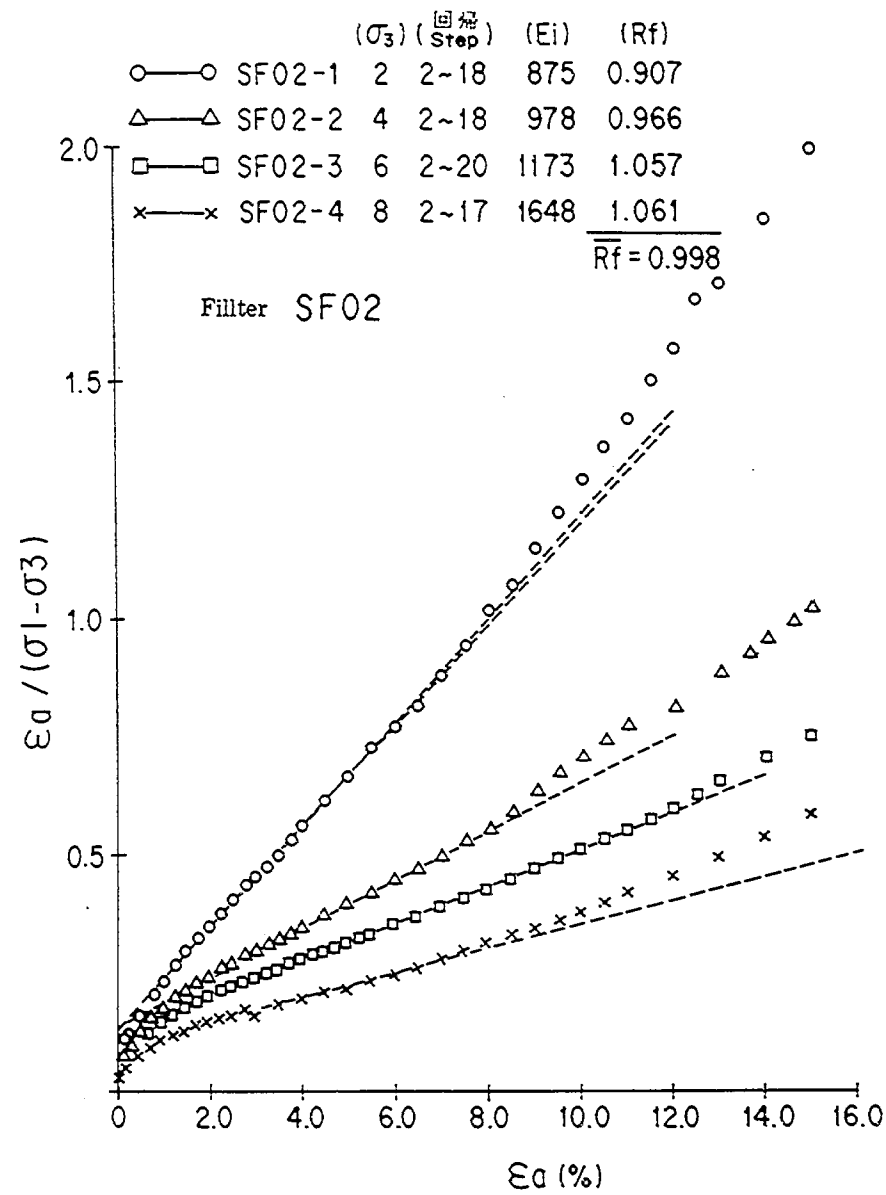
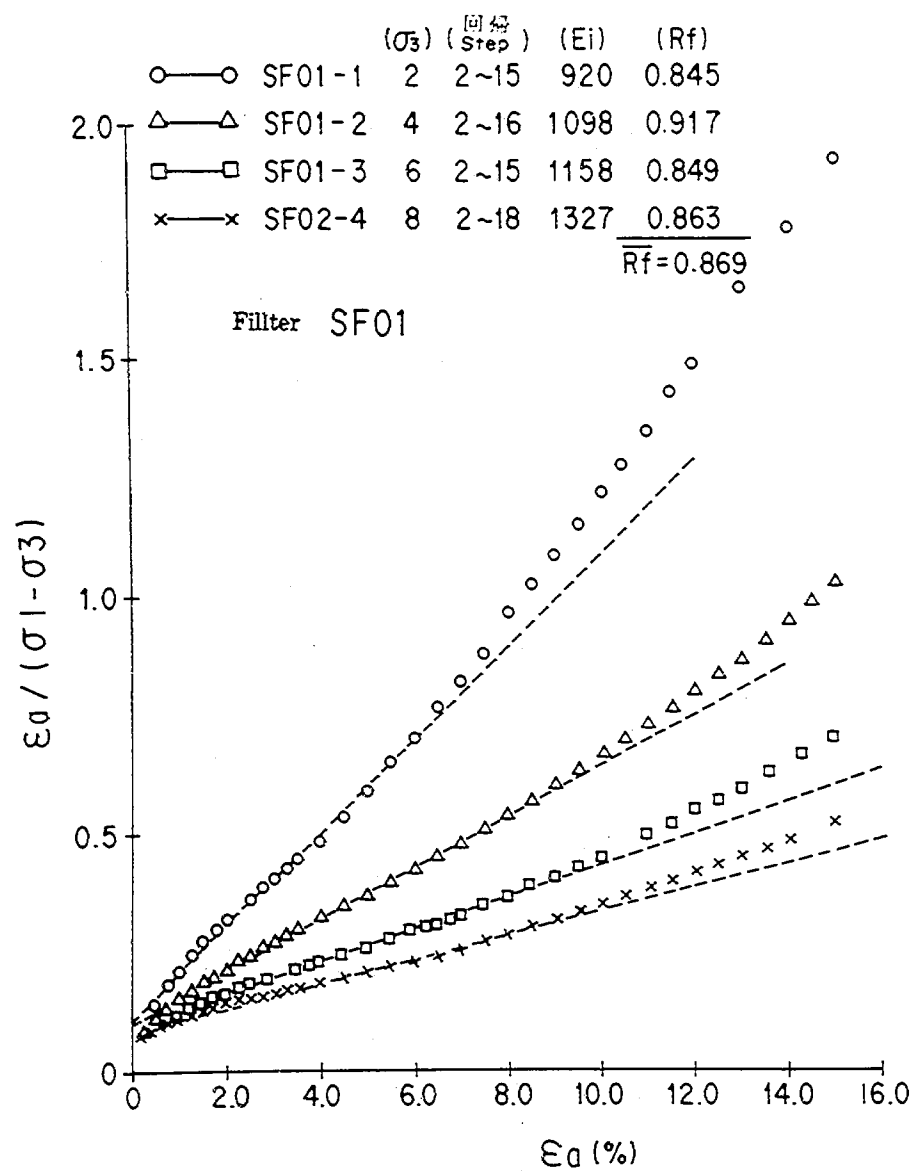


Fig.-2.3.5 $\epsilon_a/(\sigma_1-\sigma_3) \sim \epsilon_a$ (Filter)

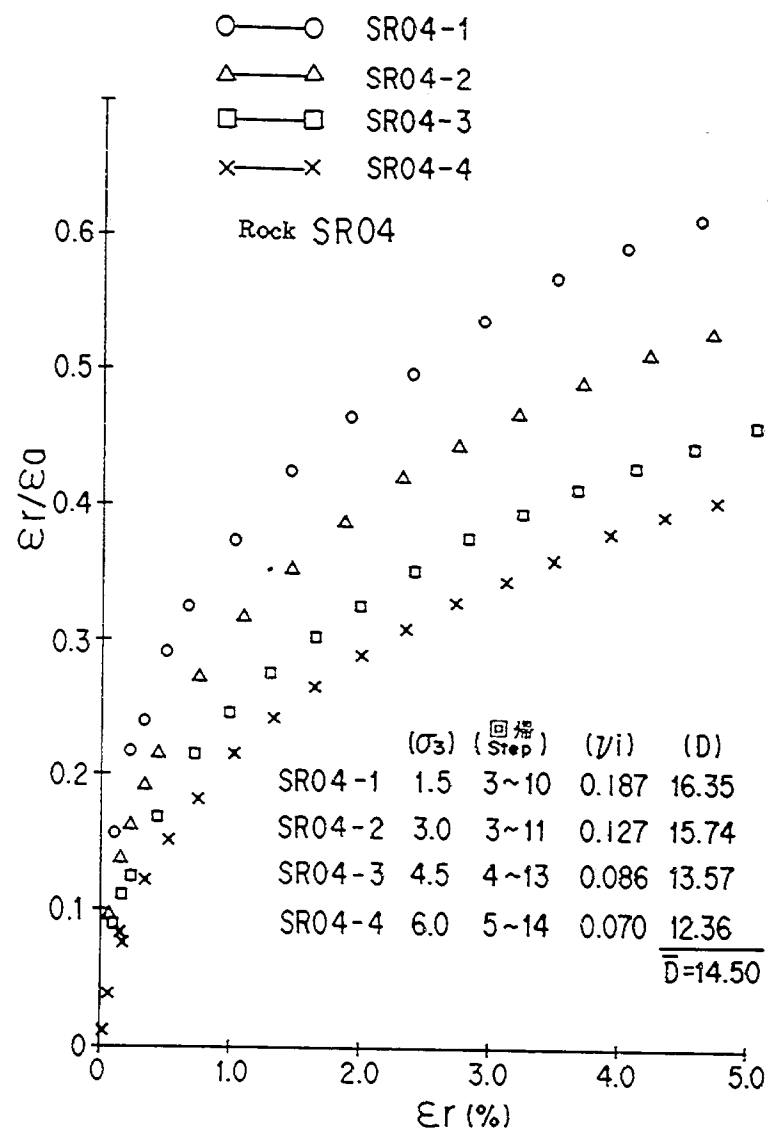
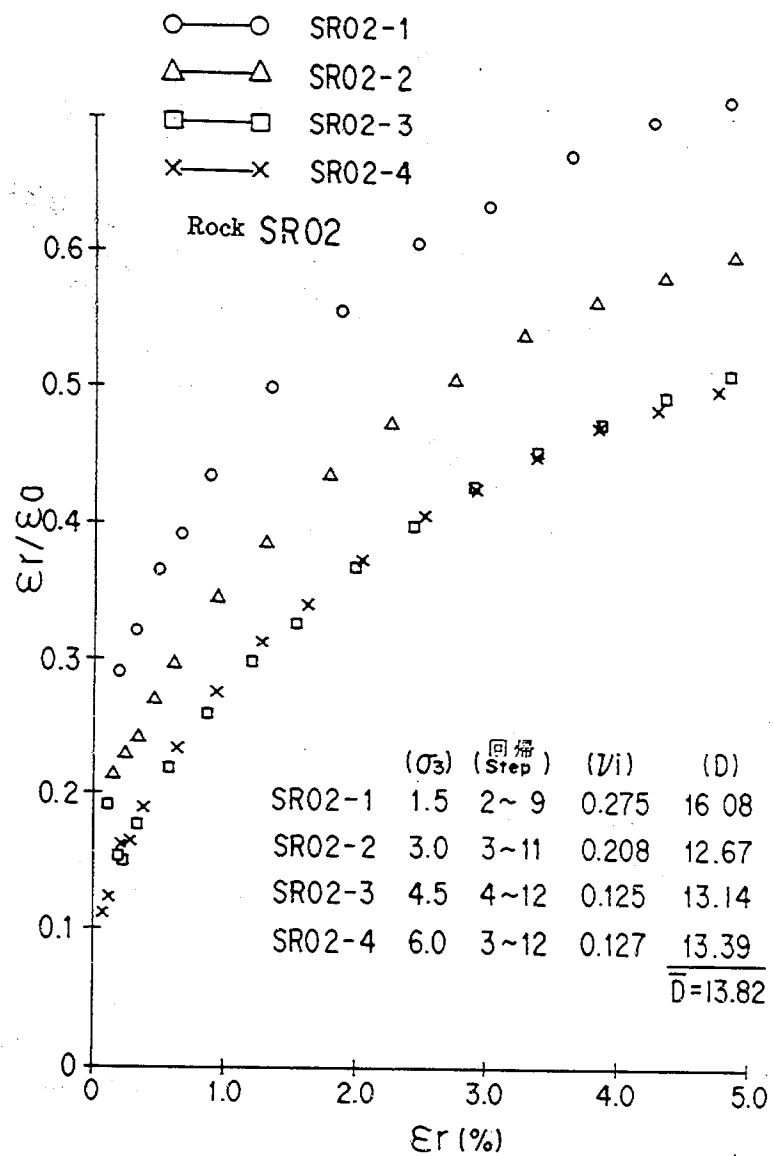


Fig.-2.3.6 $\epsilon_r/\epsilon_a \sim \epsilon_r$ (Rock)

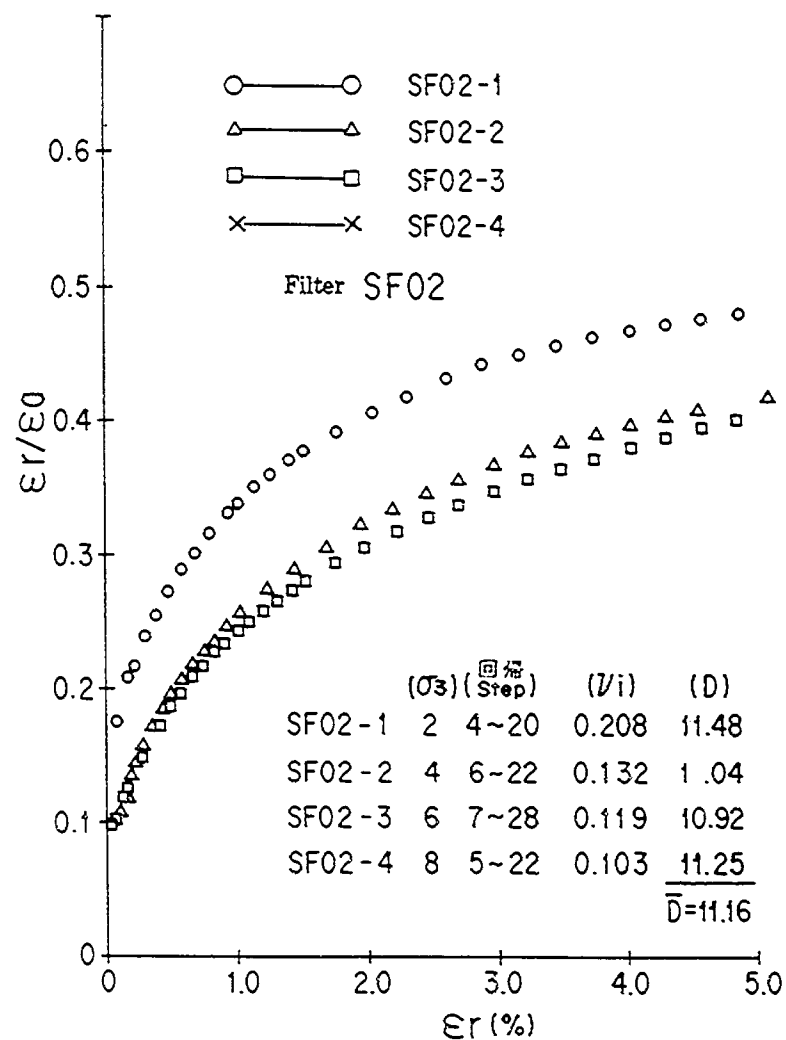
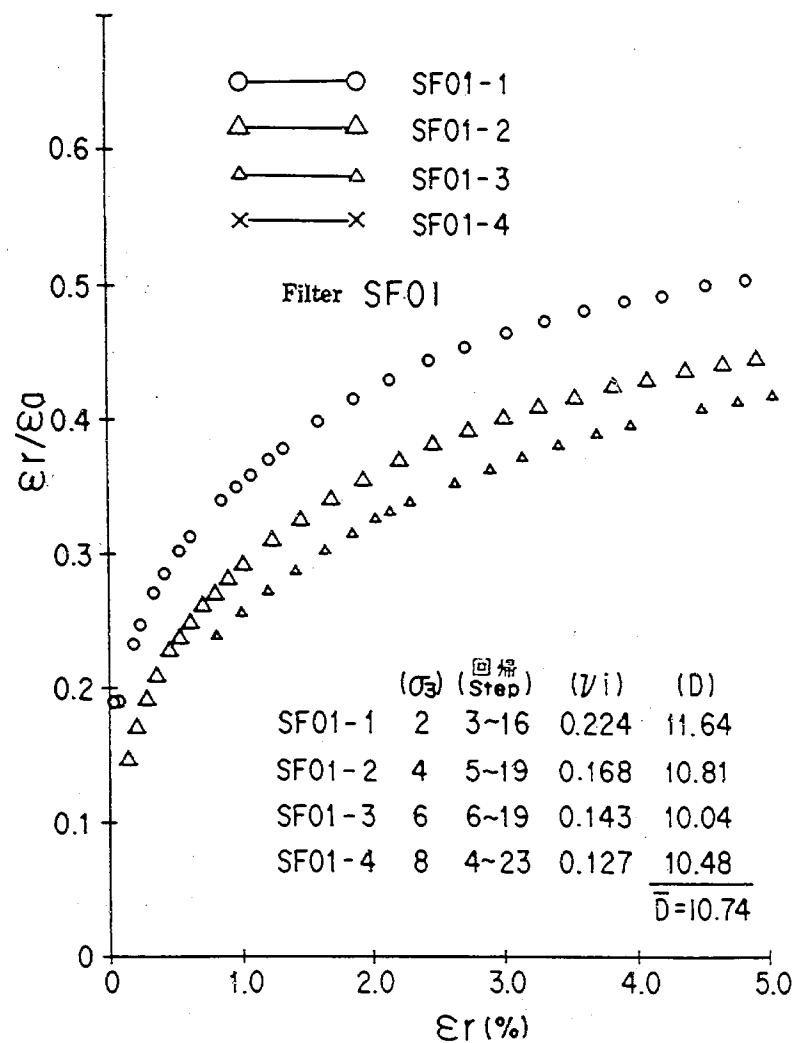


Fig.-2.3.7 $\epsilon_r/\epsilon_0 \sim \epsilon_r$ (Filter)

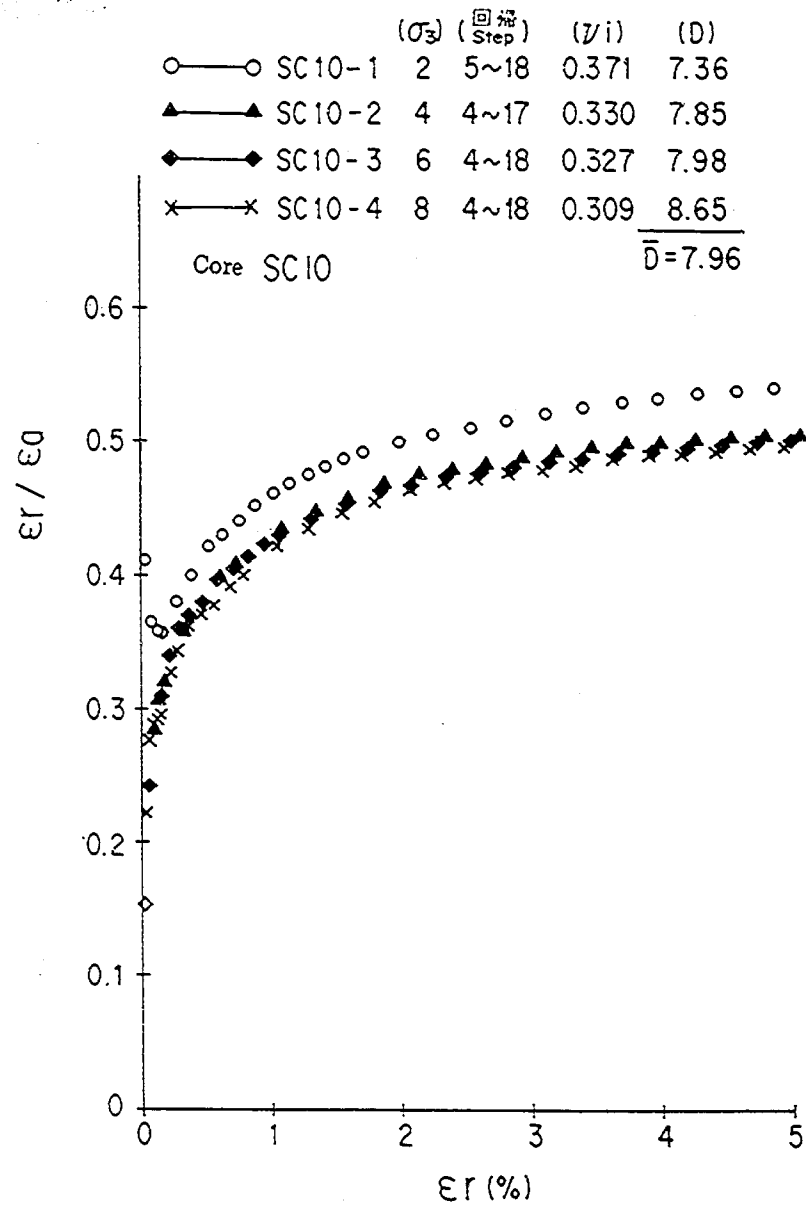
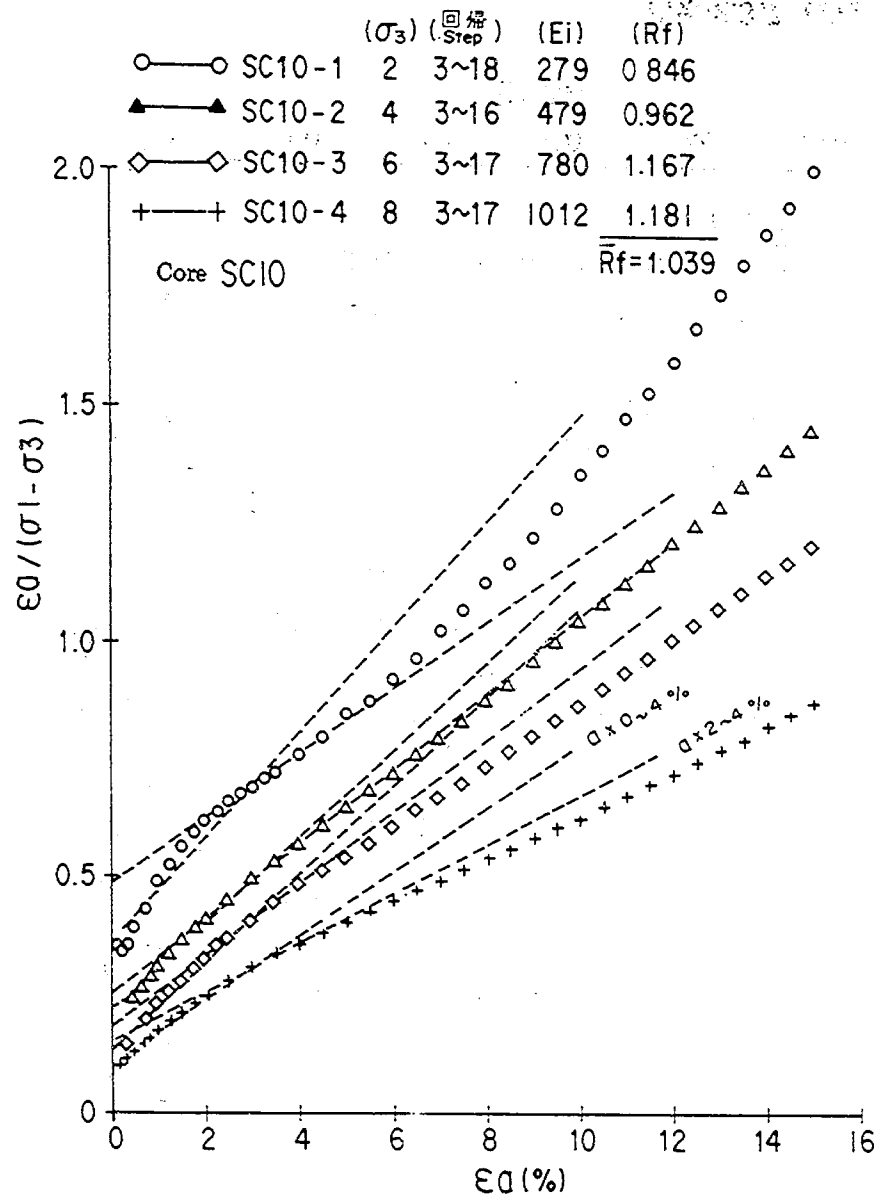


Fig-2.3.8 $\epsilon_{\theta}/(\sigma_1-\sigma_3) \sim \epsilon_{\theta}$, $\epsilon_r/\epsilon_{\theta} \sim \epsilon_r$ (Core)

materials.

2.4 Results of Analysis

The amount of settlement at the center of a core is shown in Fig.-2.4. 1 to Fig-2.4.4. The maximum values of the settlement at the center of the core zone range from 64.7cm to 82.8cm in each case. The maximum one is 82.7cm in Case 3.

The relationship between the maximum and the minimum principal stress is shown in Fig.-2.4.5. The change of the stress ratio " σ_1/σ_3 " accompanying the embankment is not noticed in each case, and ranges from 0.35 to 0.55.

Fig.-2.4.6 indicates the relationship of Poisson's ratio and the minimum principal stress. Poisson's ratio is 0.4 for core material, 0.25~0.3 for filter material, and 0.2~0.35 for rock material. Poisson's ratio has an inclination to become a downward convex curve according to the increase of the minimum principal stress.

The deformation and the principal stress of a dam body just after an embankment are shown in Fig.-2.4.7 and Fig.-2.4.8. The minimum principal stresses of a dam body point toward the crest and indicate larger values in the core zone. However, the minimum principal stresses show slightly smaller values in the filter zone due to the difference of elastic moduli between a filter zone and the circumference.

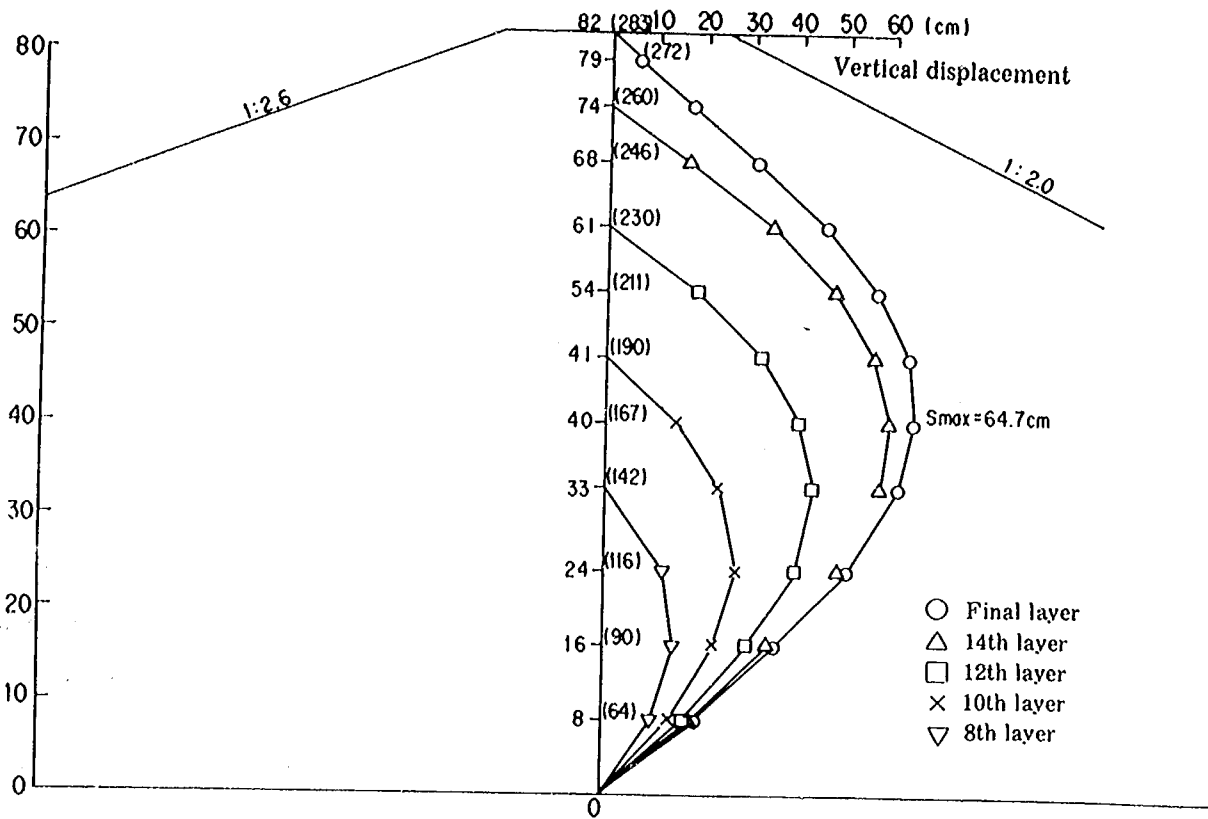


Fig.-2.4.1 Vertical displacement at core zone (Settlement at nodal points) Case 1

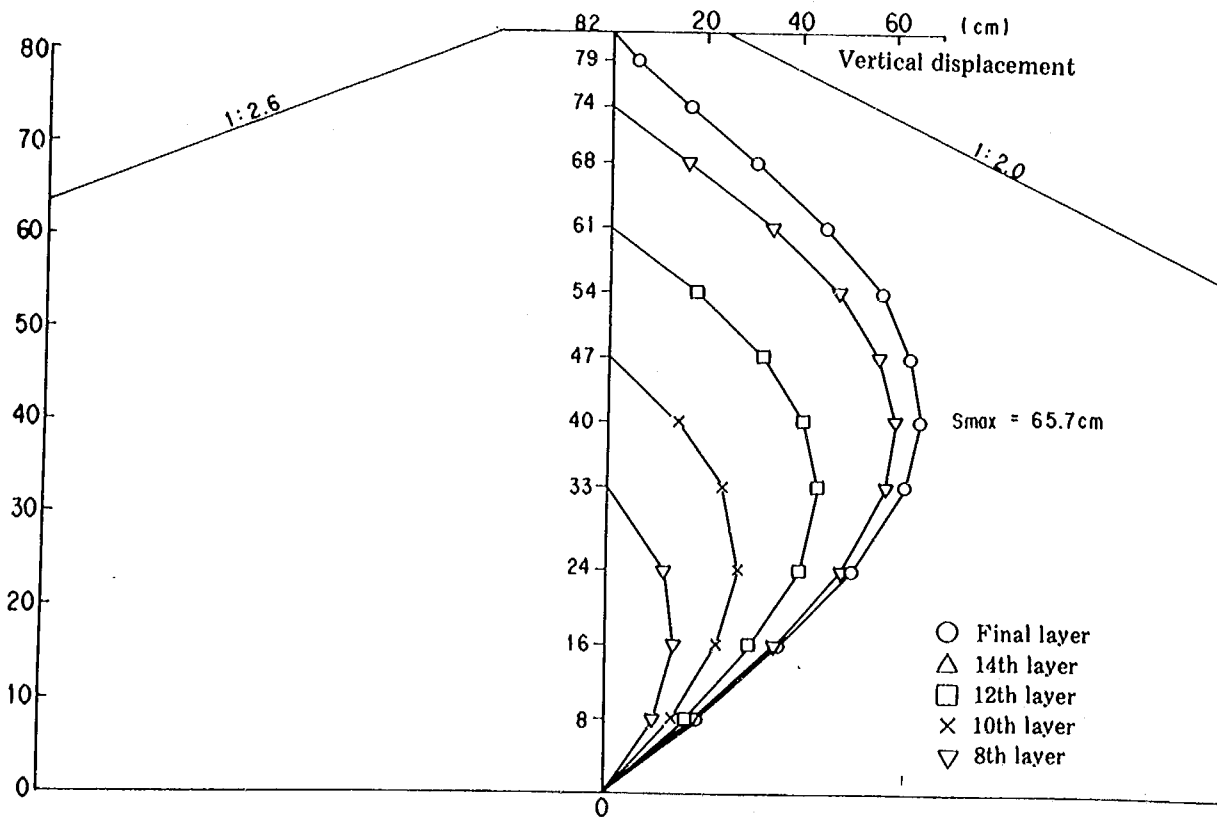


Fig.-2.4.2 Vertical displacement at core zone (Settlement at nodal points) Case 2

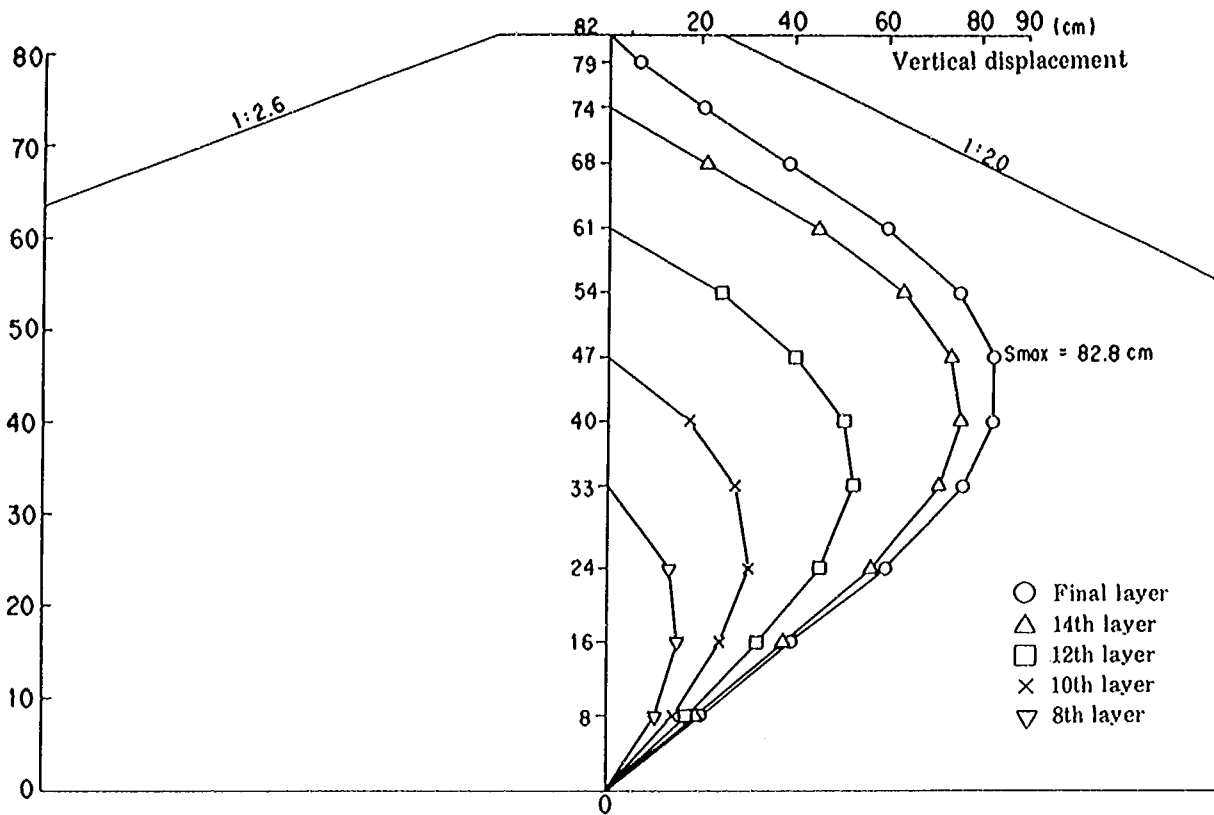


Fig.-2.4.3 Vertical displacement at core zone (Settlement at nodal points) Case 3

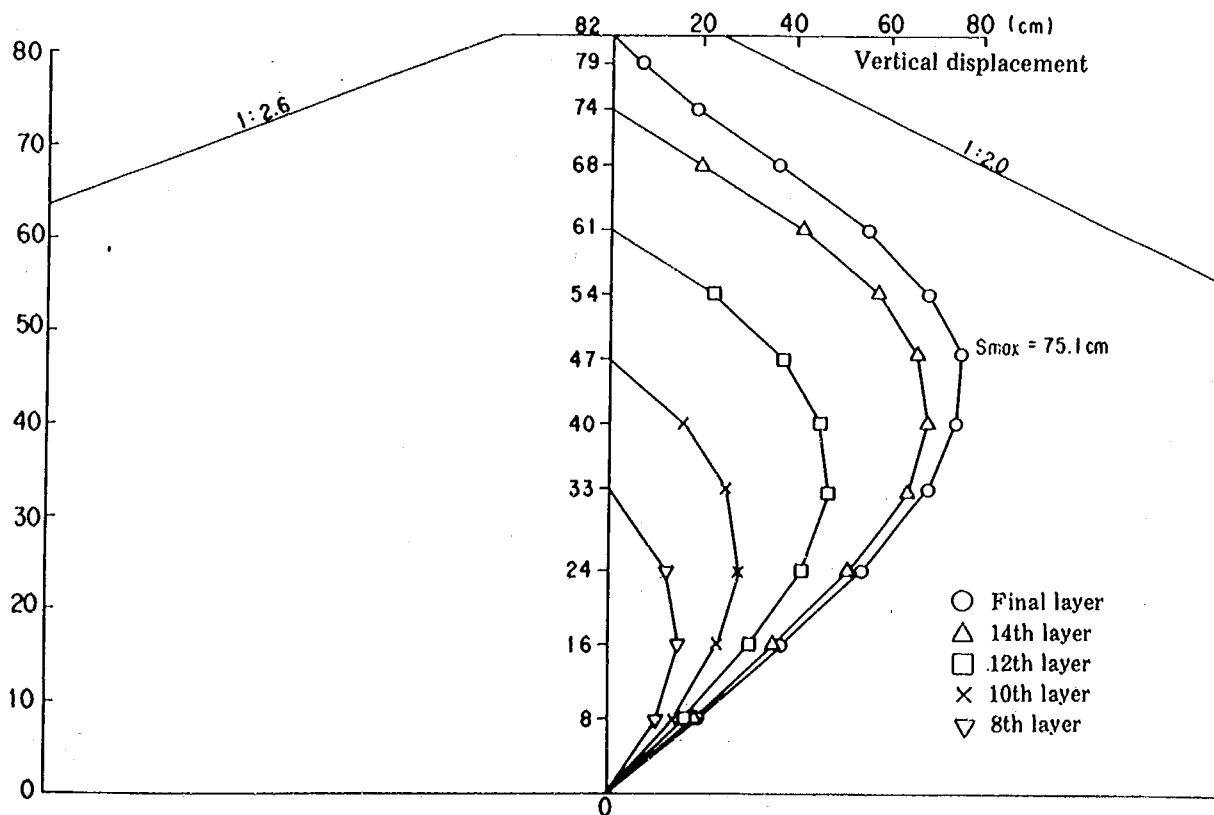


Fig.-2.4.4 Vertical displacement at core zone (Settlement at nodal points) Case 4

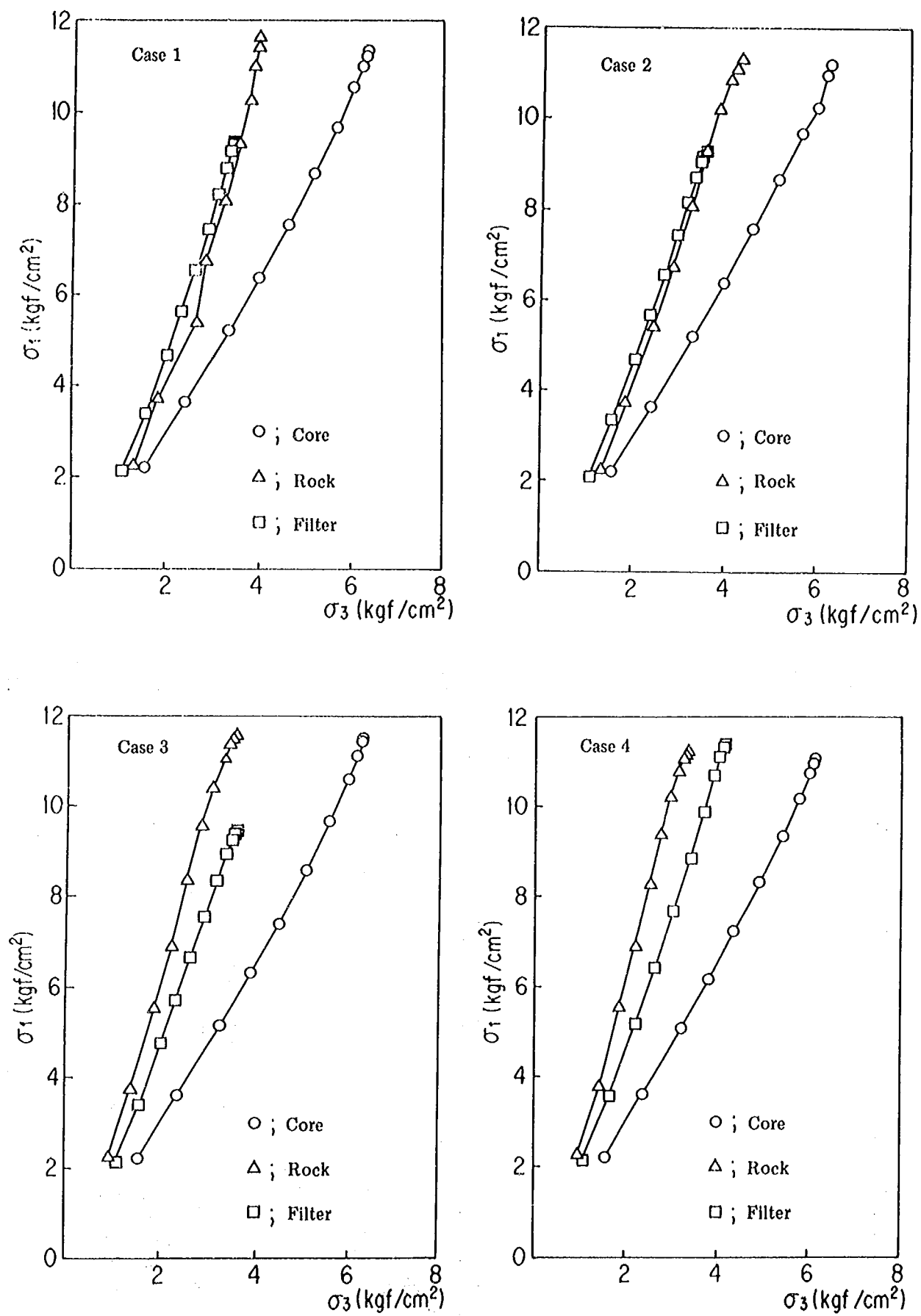


Fig.-2.4.5 Maximum principal stress ~ Minimum principal stress

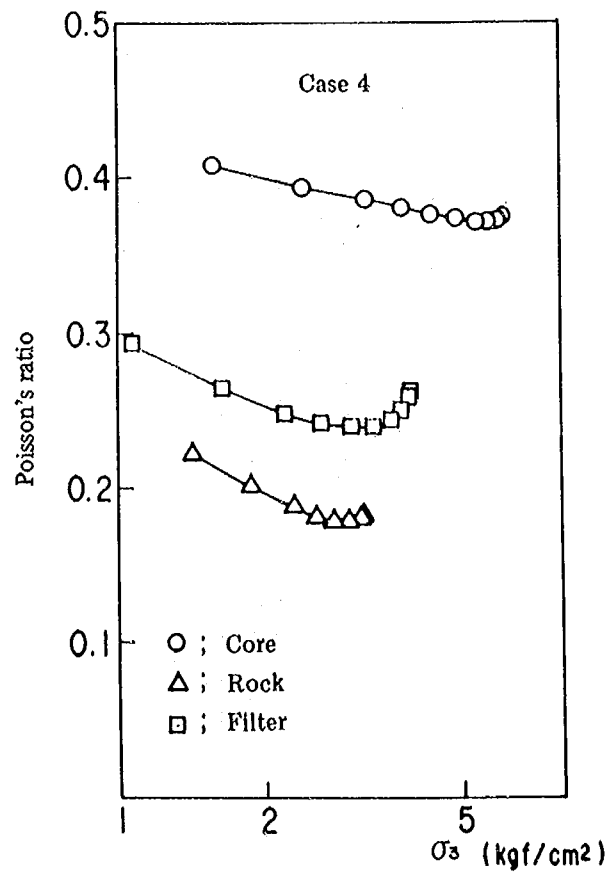
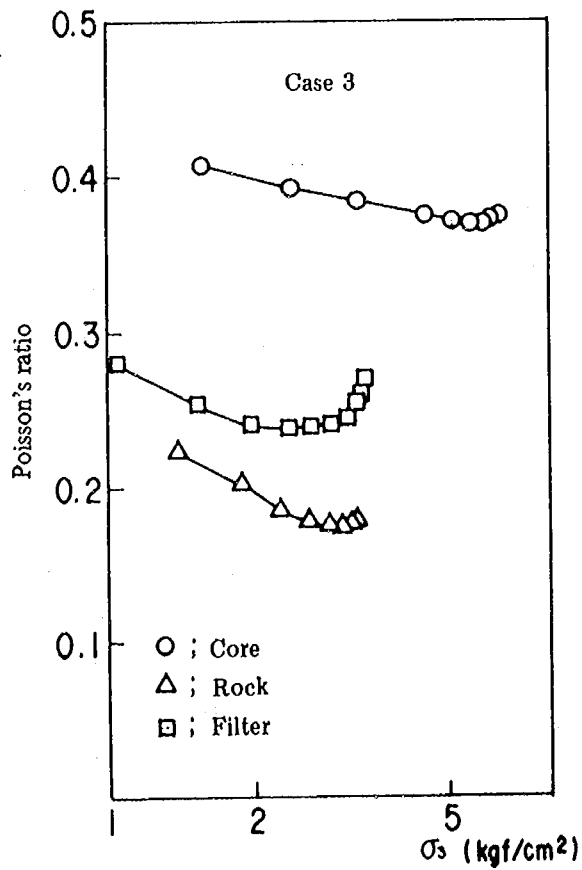
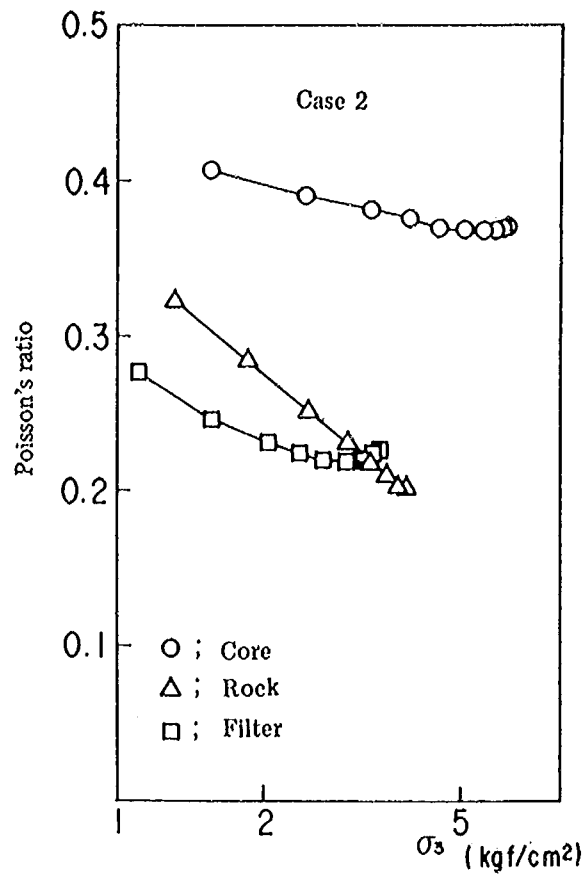
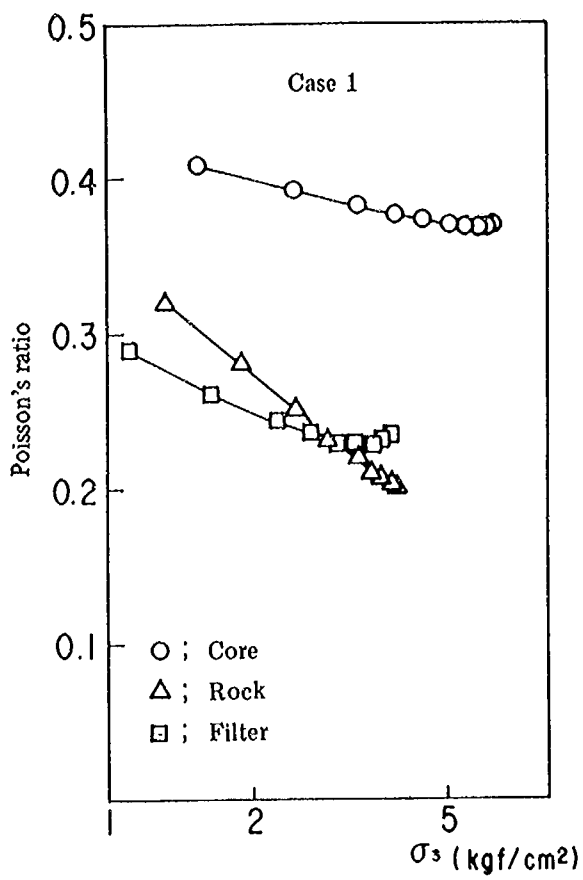


Fig.-2.4.6 Relation between Poisson's ratio and minimum principal stress

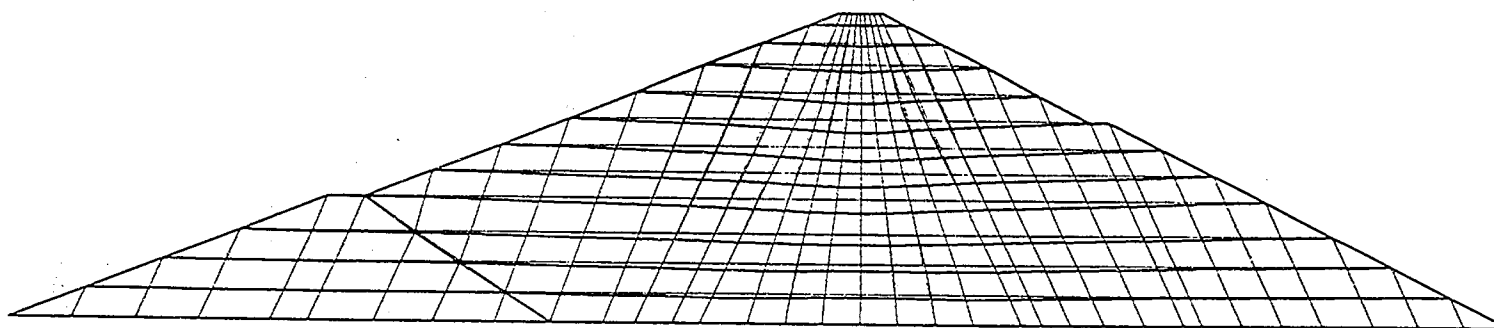


Fig.-2.4.7 Deformation after banking

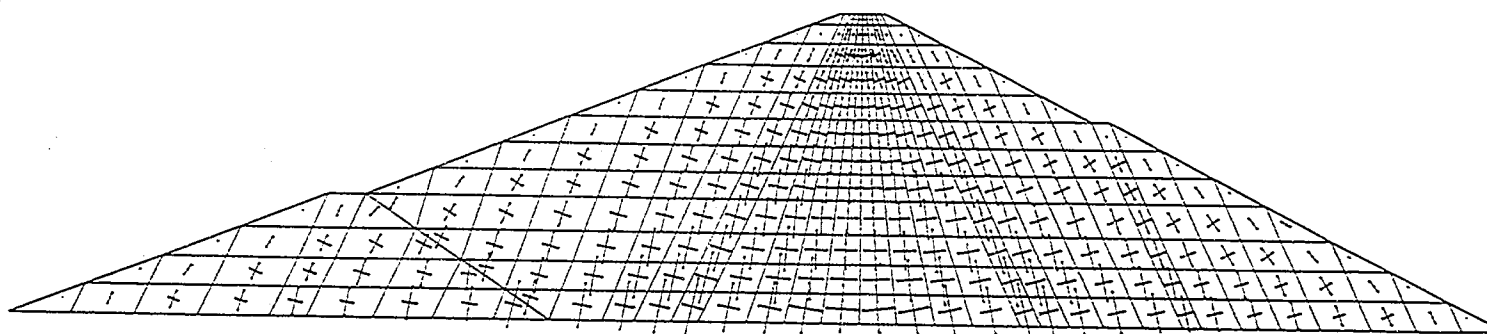


Fig.2.4.8 Stress distribution after banking

3. Reservoir Filling Analysis

3.1 General Description of Analysis

Accompanying reservoir filling, buoyancy can occur below the seepage line of the upstream pervious zone (rock and filter), and hydraulic pressure and seepage pressure in the impervious core zone. These pressures are treated as equivalent external nodal forces and used in the reservoir filling analysis just after the embankment analysis. Equivalent external nodal forces are applied to nodal points under the seepage line after the seepage analysis.

The buoyancy caused by reservoir filling appears to act on the solid part of the fill material. Therefore real buoyancy is obtained by eliminating the void part from the whole area of an element. The void part of a dam body seems to become smaller depending on the depth due to the compaction effect of its own weight, however the void is independent of the depth and assumed to be constant in this report. The porosity of rockfill materials of existing dams are 0.24 to 0.41 for the rock, 0.2 to 0.41 for the filter, and 0.1 to 0.54 for the core, respectively, as shown in Table-3.1.1.

Table-3.1.1 Porosity of rockfill material

	Rock	Filter	Core
Daisetsu	0.30	0.22	0.10
Iwaya	0.31	0.286	0.331
Terauchi	0.327	0.382	0.382
Miho	0.25	0.24	0.35
Nanakura	0.242 ~ 0.281	0.242	0.189
Takase	0.24	0.204	0.186
Narai	0.33 ~ 0.41	0.41	0.54
Maekawa	0.31 ~ 0.33	0.31	
Shichigashuku	0.26	0.24	0.26

Fig.-3.1.1 shows the change of soil pressure observed at Shirakawa Dam during the first reservoir filling. A soil pressure cell is installed at an elevation of 292m in the upstream rock zone. The wet and saturated unit weights of rock material at the Shirakawa Dam are 1.8 t/m³ and 2.1 t/m³, respectively. The overburden pressure coincides with the observed soil pressure of 6.3kgf/cm², because the earth covering to the location of the pressure cell is almost 35m deep. The pore water pressure starts

increasing at the gradient of unity when the water level exceeds the crest of the 11m-high cofferdam, which is constructed on the upstream side of the dam body. According to the first filling, the soil pressure also increases and reaches about 7.4 kgf/cm² when the water

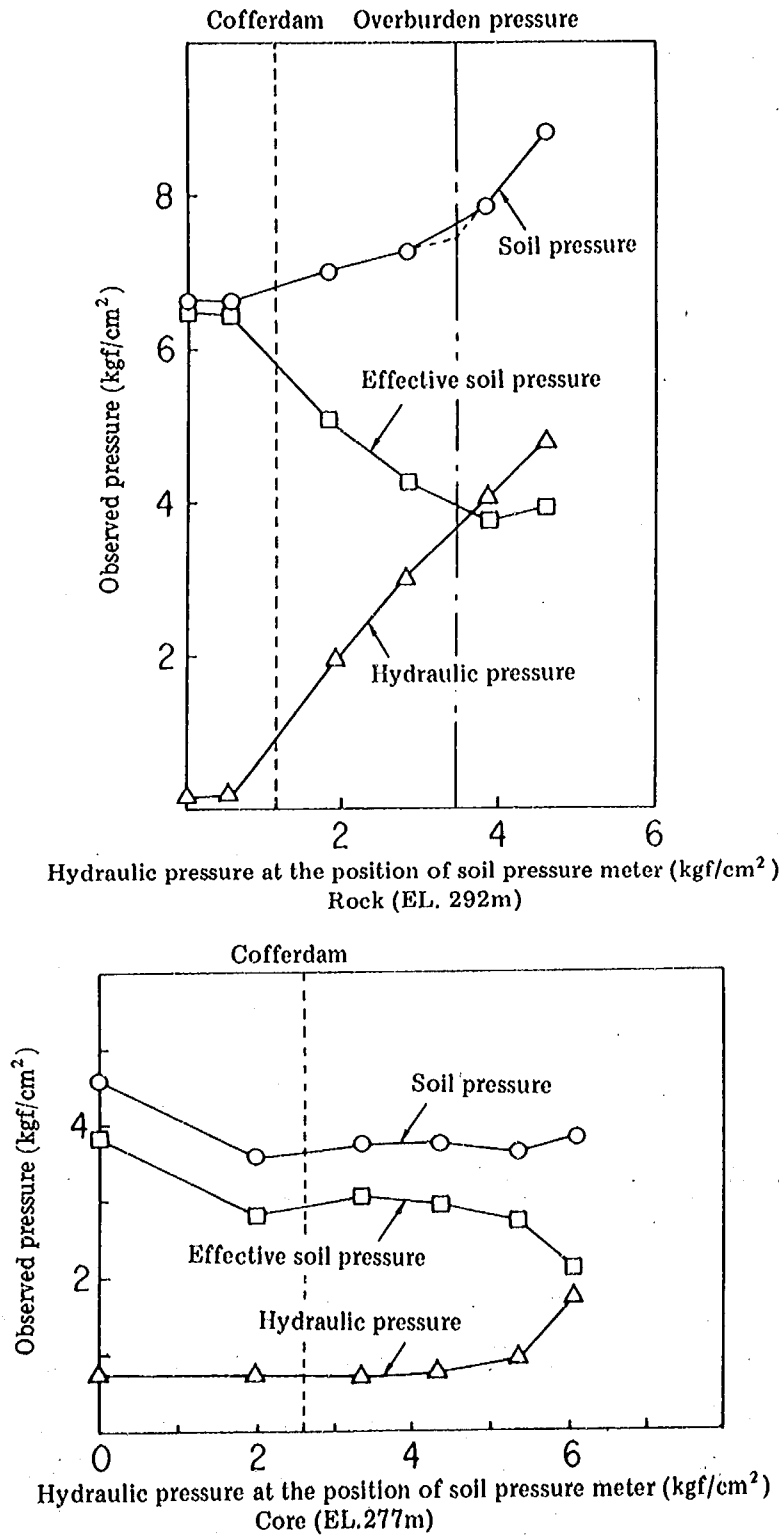


Fig.-3.1.1 Relation between hydraulic pressure and observed soil pressure (Shirakawa Dam)

level comes to the upstream slope just over the soil pressure cell. This value corresponds to a saturated overburden pressure of 7.35kgf/cm² at this location (Saturated weight 2.1t/m³ × depth 35m). As the effective soil pressure observed is 4.0kgf/cm², the decrease of effective stress after filling resulted in 0.62 of the effective stress 6.5 kgf/cm² before filling. The chain line in the figure explains the level of the upstream slope just over the soil pressure cell. The observed soil pressure rises at the gradient of unity when the water level surpasses the above-mentioned level, but the effective soil pressure cannot change.

It can be concluded that the buoyancy of the upstream side of the dam acts on the solid part of the body eliminating the void part, and the buoyancy suiting the dam's cross section of 100% is not proper for analysis.

The decrease of stresses of a rockfill dam by buoyancy is calculated by multiplying stresses before filling into K of the Watanabe - Baba method. K is obtained by equation (3.1), expressing the observation at the Shirakawa Dam.

$$K = \frac{\gamma_{sat} - \gamma_w}{\gamma'}, 0 < K < 1 \quad \dots\dots\dots (3.1)$$

where

- γ_{sat} ; unit weight of saturated soil
- γ_w ; unit weight of water
- γ' ; unit weight of submerged soil

The decrease of stress accompanying reservoir filling considered about the inundated pervious zone, dry pervious zone, and impervious zone. K is given as shown in Fig.-3.1.2. The stress due to the reservoir filling analysis results from seepage forces at the core zone after the embankment analysis.

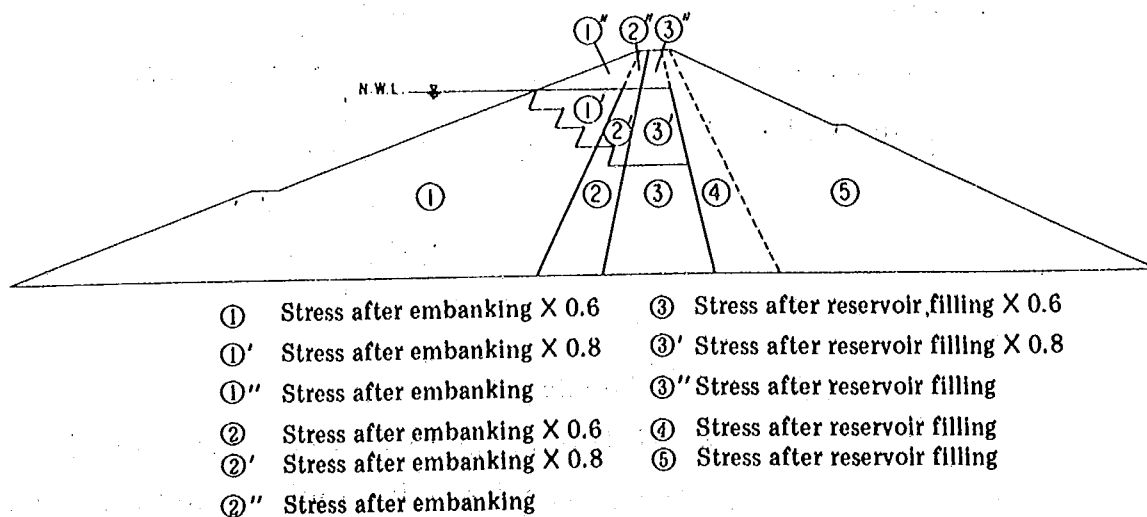


Fig.-3.1.2 Stress distribution after reservoir filling

The external nodal forces from the seepage flow are described in the following. The seepage analysis is only applied to the core zone. Fig.-3.1.3 illustrates an F.E.M. mesh of core zone used in seepage analysis. The permeability coefficient used is 5×10^{-6} cm/s.

The procedure of the analysis to gain static stress is as follows. The terms inside the parentheses means a computational program.

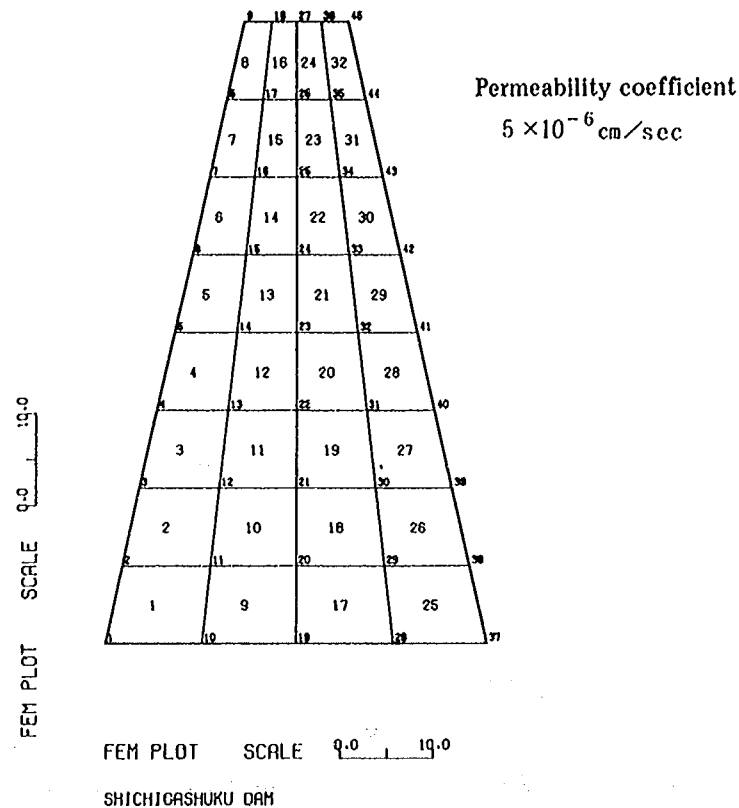
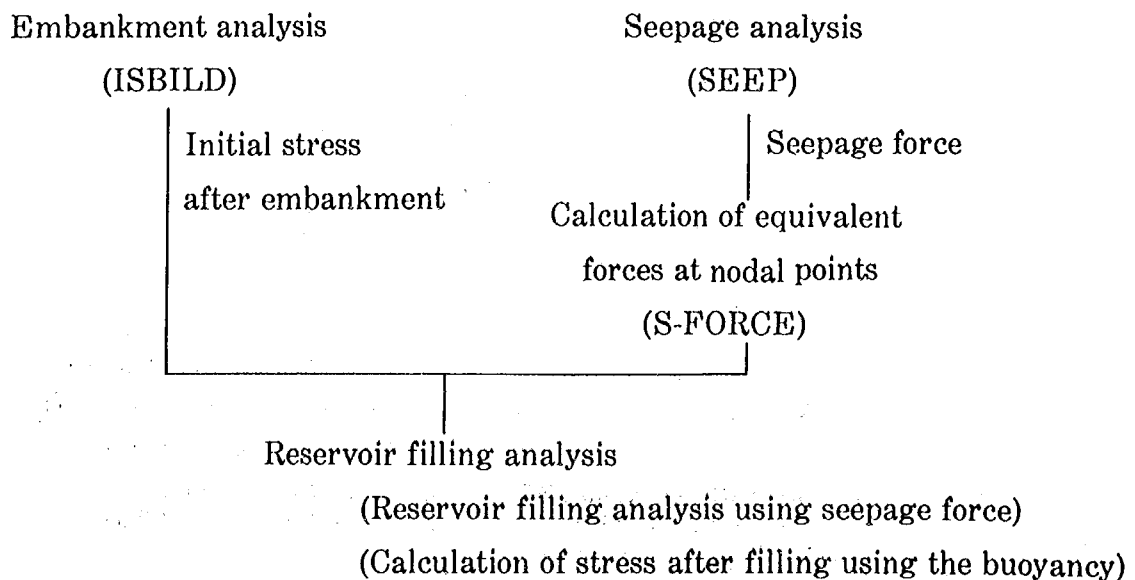


Fig.-3.1.3 Finite element mesh for seepage analysis



The procedure of seepage analysis is as follows.

Calculation of body-force

The piezo-head is given by equation (3.2), as shown in Fig.-3.1.4.

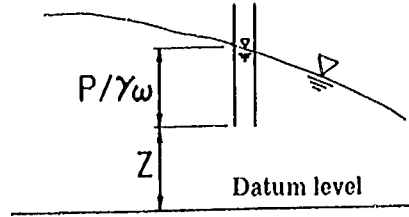


Fig.-3.1.4 Piezo head

$$h = \frac{P}{\gamma_w} + Z \quad \dots (3.2)$$

(Pressure head) + (Potential head)

$$\begin{aligned} \text{grad } h &= \text{grad} \left(\frac{P}{\gamma_w} + Z \right) \\ &= \text{grad} \frac{P}{\gamma_w} + j \end{aligned} \quad \dots (3.3)$$

j ; Vertical unit vector to upward direction

The seepage velocity V is obtained from the Darcy's law as follows.

$$\begin{aligned} V &= -K \cdot \text{grad } h \\ &= -\frac{K}{\gamma_w} \cdot \text{grad } P - K \cdot j \end{aligned} \quad \dots (3.4)$$

where the body-force acting on the soil particles of unit volume is the summation of the following F_1 , F_2 , and F_3 .

① Dry weight of soil particles : F_1

$$F_1 = - (1 - n) \cdot \gamma_s \cdot j \quad \dots (3.5)$$

n ; Porosity

γ_s ; Unit weight of soil particles

② Hydraulic pressure acting on soil particles: F_2

$$F_2 = - (1 - n) \cdot \text{grad } P \quad \dots (3.6)$$

$$\therefore \left(P - \frac{\partial p}{\partial x} \cdot \frac{1}{2} dx \right) dy \cdot dz \rightarrow \boxed{\begin{matrix} (x,y) \\ (u,v,w) \end{matrix}} \leftarrow - \left(P + \frac{\partial p}{\partial x} \cdot \frac{1}{2} dx \right) dy \cdot dz$$

$\therefore$ Pressure of x-direction

$$\text{Total pressure} = \left(\frac{\partial p}{\partial x} + \frac{\partial p}{\partial y} + \frac{\partial p}{\partial z} \right) = - \text{grad } P$$

③ Seepage force acting on soil particles : F_3

Resisting force of impervious layer of unit volume against seepage flow : R

$$R = \text{grad } h = - \frac{V}{K}$$

$$\begin{aligned} F_3 &= - n \cdot \gamma_w \cdot R \\ &= - n \cdot \gamma_w \cdot \text{grad } h \end{aligned} \quad \dots (3.7)$$

Resultant F is calculated as follows

$$\begin{aligned} F &= F_1 + F_2 + F_3 \\ &= - \gamma_w \cdot \text{grad } h - \gamma' \cdot j \end{aligned} \quad \dots (3.8)$$

($\gamma' = (1 - n) \cdot (\gamma_s - \gamma_w)$; Submerged unit weight)

First term: Force by flow acting on impervious zone of unit volume ... seepage force

Second term: submerged weight

in two-dimensional flow

$$\begin{aligned} \text{Body force (x)} &= \gamma_w \cdot \frac{V'_x}{K_x} \\ &= \frac{\gamma_w}{K_y} \cdot \beta_x \cdot V_x \end{aligned} \quad \dots (3.9)$$

$$\text{Body force (y)} = \frac{\gamma_w}{K_x} \cdot \beta_y \cdot V_y - \gamma' \quad \dots (3.10)$$

where

V'_x : Apparent velocity

$V_y = \frac{V'_x}{\beta_x}$: Real velocity

β_x : Average porosity over area

In the embankment analysis, F_1 is treated as external forces in the form of its own weight and the overburden load, so F_2 and F_3 are considered in the seepage analysis.

$$P_2 + P_3 = -\gamma_w \cdot \text{grad } h + \gamma_w \cdot j \cdot (1 - n)$$

$$= \gamma_w \cdot \frac{V}{K} + \gamma_w \cdot j \cdot (1 - n) \quad \dots (3.11)$$

Therefore,

$$\text{Body force (X)} = \gamma_w \cdot \frac{\beta_x}{K_x} \cdot V_x \quad \dots (3.12)$$

$$\text{Body force (Y)} = \gamma_w \cdot \frac{\beta_y}{K_y} \cdot V_y + \gamma_w \cdot (1 - n) \quad \dots (3.13)$$

These Body forces (X) and (Y) are added to the nodal points after the embankment analysis as equivalent forces.

The mean velocity of flow after seepage analysis is about 1.54×10^{-5} cm/s, as shown in Fig.-3.1.5.

3.2 Results of Analysis

As mentioned above, the buoyancy at the upstream rock and filter zone, and the body-force of seepage flow at the core are added to the nodal points after the embankment analysis as equivalent forces.

The vertical stress at the dam base after the reservoir filling is illustrated in Fig.-3.2.1. The vertical stresses at the boundary between the upstream rock and filter zone become 0.6 of the stresses before the filling. A disturbance of stress is recognized at the area of varied physical properties (the boundary of the rock, filter, and core zone) and the triangular element, however no extensional stress occurs as a result. Fig.-3.2.2 explains the stress distribution of a dam body. Stresses at the boundary of the downstream rock and filter zone are larger at each component of stress. The vertical stresses inside the upstream rock zone are 0.6 of those before filling, but no extensional stresses are recognized.

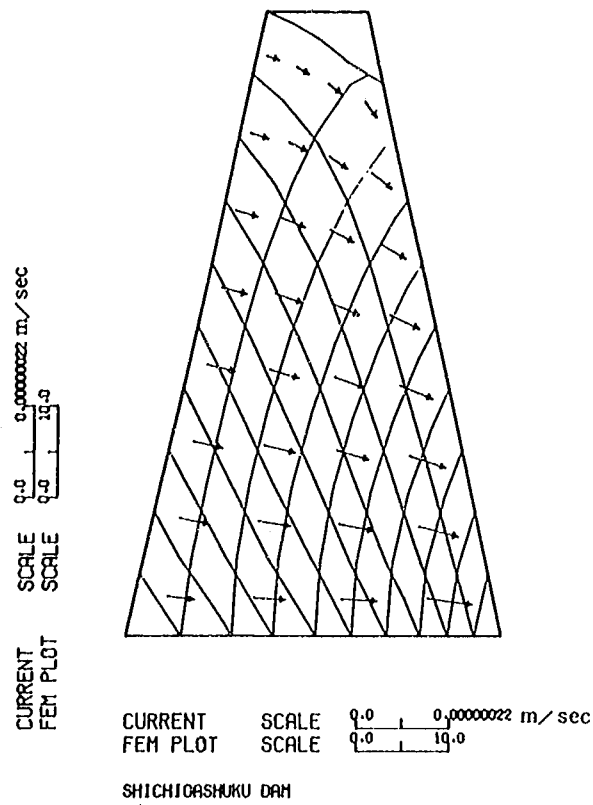


Fig.-3.1.5 Result of seepage analysis

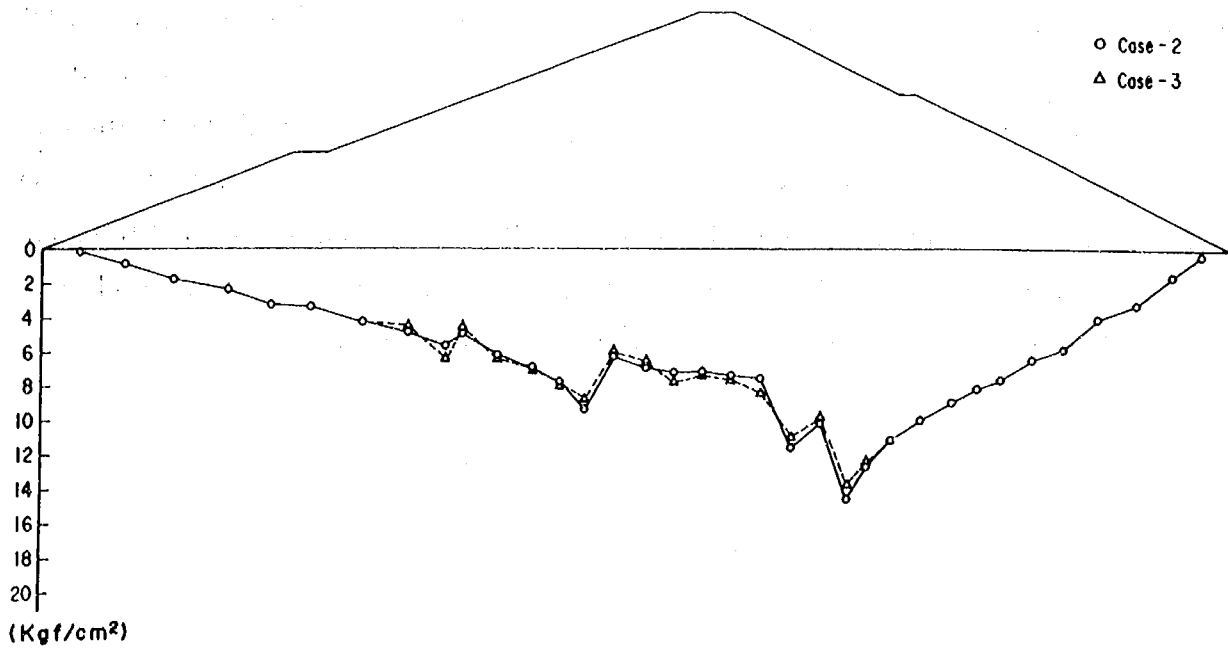


Fig.-3.2.1 Vertical stress at the base of dam after reservoir filling

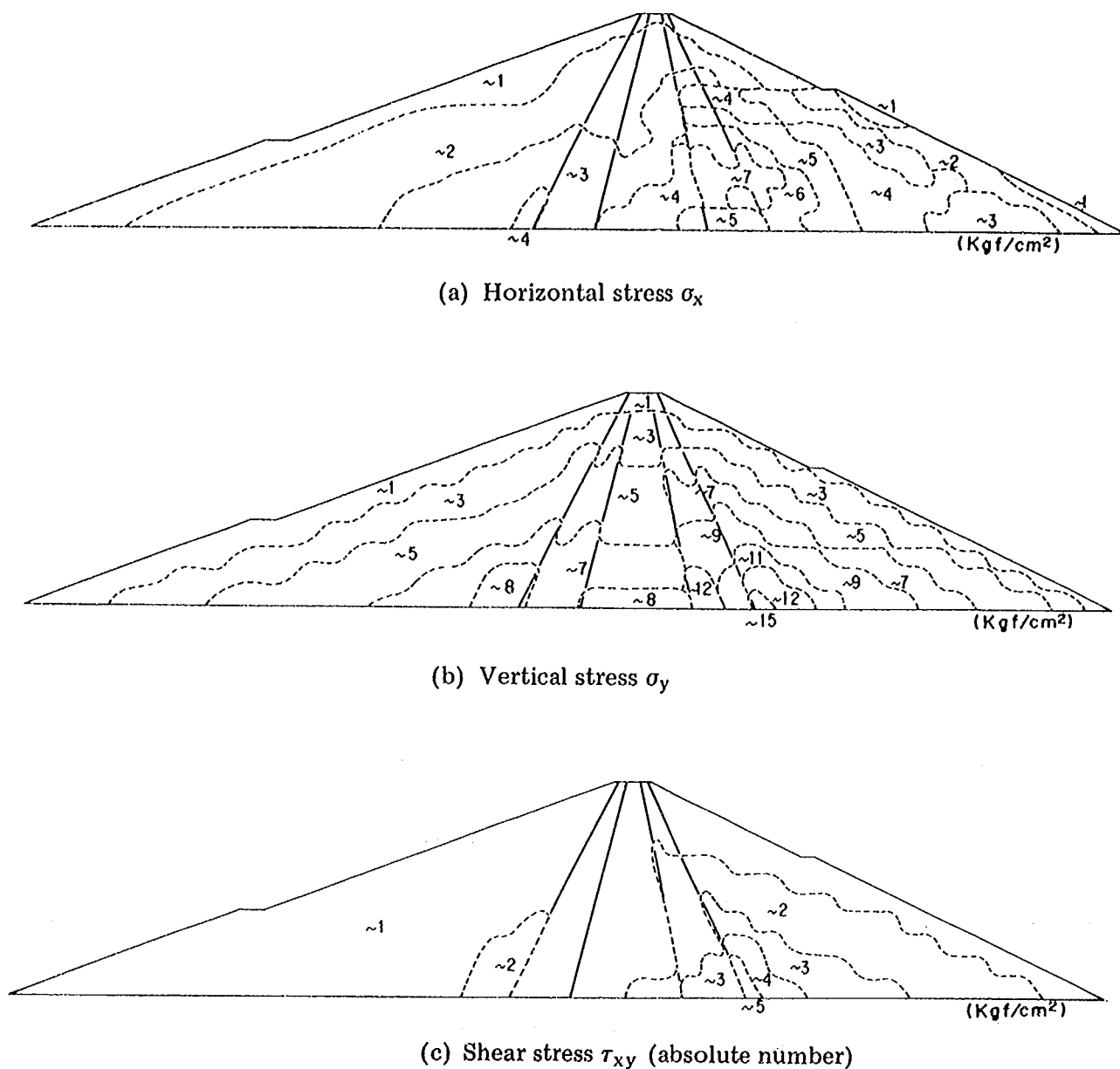


Fig.-3.2.2 Static stress after reservoir filling

Appendix A Comparisons of the Results between ISBILD and No-tension Analysis Method

Extensional stress is likely to occur at the boundary of the upstream pervious zone and the core zone, inside the core and upstream slope by reservoir filling analysis using the computational program ISBILD. (A buoyancy of 100% is applied to the analysis.) However, granular materials are not able to resist such extensional stress, so the extension should not occur due to a redistribution of the stress.

Assuming that rockfill material cannot transfer extensional stress, the stress after the

reservoir filling analysis by ISBILD is compared with that by the computational program NOTENSION for the no-tension analysis method.

A-1 No-tension Analysis Method

The initial stress method is taken up in non-linear analysis as for the no-tension analysis (see Fig.-A.1). The problem of nonlinear in this method is calculated by the initial stress S_0 and corresponding nodal force F_0 , not by changing the stiffness matrix $[K]$. S_0 and F_0 are expressed as a function of stress or strain, and formalized as follows by the finite element method.

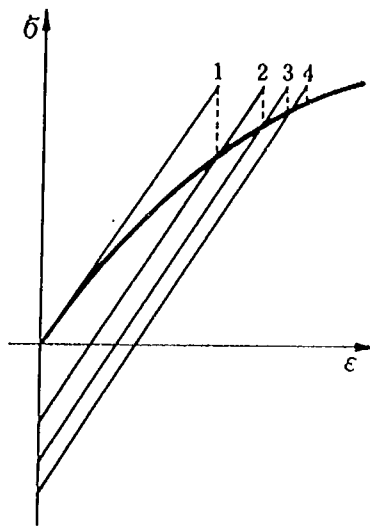


Fig.-A.1 Initial stress method

$$KU = F(U) \quad \dots (A.1)$$

where U is nodal displacement.

An approximation of load $F(U)$ for $U=0$ is determined as the first step, and the calculation is reiterated until the change of U is disregarded. The stiffness matrix $[K]$ is regarded as unchangeable in the initial stress method.

As for the formalization of the no-tension analysis method, the elastic equation using the triangular elements of three nodal points is expressed in the following.

$$\begin{aligned} F &= \Delta B^T E B U \\ e &= B U \\ S &= E B U \end{aligned} \quad \dots (A.2)$$

The elastic matrix E changes into $(E-E')$ to eliminate extensional stress, when extension occurs inside this triangular element. Therefore, equation (A.2) changes as follows.

$$\left. \begin{aligned} F &= \Delta B' (E - E') B U \\ e &= B U \\ S &= (E - E') B U \end{aligned} \right\} \quad \dots (A.3)$$

where

$$S' = E' B U \quad \dots (A.4)$$

Equation (A.3) is modified as follows

$$\left. \begin{aligned} F + \Delta B' S' &= \Delta B' E B U \\ e &= B U \\ S &= E B U - S' \end{aligned} \right\} \quad \dots (A.5)$$

Therefore, the no-tension stress problem is replaced as the elastic one by the initial stress $-S'$ and the corresponding nodal force $\Delta B' S'$, when S' is defined so that the occurred extensional stress becomes zero. However, S' cannot be determined beforehand, because S' is a function of displacement U , as indicated in equation (A.4). Consequently, the calculation is reiterated until the extension becomes under a permitted limit as an initial condition of $S' = 0$

The procedure of the reiterated calculation is expressed by the following equations.

$$\left. \begin{aligned} F + \sum \Delta B' \delta S'_n &= \Delta B' E B (U + \sum \delta U_n) \\ S &= E B (U + \sum \delta U_n) - \sum \delta S'_n \end{aligned} \right\} \quad \dots (A.6)$$

This means redistribution of the stress that is unsatisfied with the yielding condition, so such method is called the stress transference method. Assuming that S_n is the solution of the repeated calculation at the n -th step, the (Initial stress $\delta S'_{n+1}$) in a repeated calculation at the $(n+1)$ -th step is expressed by $\delta S'$ of Mohr's circle, as shown in Fig.-A.2. The principal stresses are given by σ_x , σ_y and τ_{xy} , which are the stresses inside the triangular element as follows.

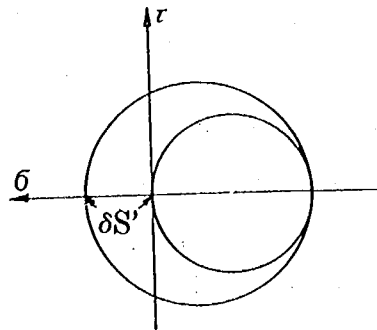


Fig.-A.2 Initial stress (Extensional stress is positive)

$$\left. \begin{aligned} \frac{\sigma_1}{\sigma_2} &= \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2} \\ \tan 2\theta &= \frac{2\tau_{xy}}{\sigma_x - \sigma_y} \\ -\frac{\pi}{2} &\leq \theta \leq \frac{\pi}{2} \quad (\sigma_x \geq \sigma_y) \\ \frac{\pi}{2} &\leq \theta \leq \frac{3}{2}\pi \quad (\sigma_x \leq \sigma_y) \end{aligned} \right\} \dots (A.7)$$

Therefore σ_1' (σ_2') is zero when σ_1 (σ_2) is compressive, and σ_1' (σ_2') is $-\sigma_1$ ($-\sigma_2$) when σ_1 (σ_2) is extensive by evaluating the positive and negative of σ_1 and σ_2 . Finally, the initial stress δS is expressed by the rectangular coordinates as follows.

$$\left. \begin{aligned} \sigma_x' &= \frac{\sigma_1' + \sigma_2'}{2} - \frac{\sigma_1' - \sigma_2'}{2} \cos 2\theta \\ \sigma_y' &= \frac{\sigma_1' + \sigma_2'}{2} + \frac{\sigma_1' - \sigma_2'}{2} \cos 2\theta \\ \tau_{xy}' &= \frac{\sigma_1' - \sigma_2'}{2} \sin 2\theta \end{aligned} \right\} \dots (A.8)$$

A-2 Comparisons of the Results between ISBILD and No-tension Analysis.

Fig.-A.3 shows a comparison between stresses by the computational program NOTENSION and ISBILD. The stresses at the base and middle level of a dam body are investigated for elements by ISBILD and for nodal points by NOTENSION. Case 3 is used for this calculation. Stress at the upstream rock zone is disturbed for ISBILD, but not for NOTENSION. Stresses at the core zone for NOTENSION are smaller than those for ISBILD. Two results for ISBILD and NOTENSION coincide with each other.

Fig.-A.4 shows elements where the minimum principal stress becomes an extension. The location of the element where tensional stress occurs for ISBILD differs much from that for NOTENSION. Tension arises at the downstream rock zone and the limited area between the cofferdam and filter zone for NOTENSION. Those tensions are less than 1.0 kgf/cm².

On the other hand, tension occurs at the surface and deep part of the upstream rock zone for ISBILD. A tension of 4.0 kgf/cm² to 6.0 kgf/cm² is in the vicinity of the dam crest, and the tension of less than 1.0 kgf/cm² is at the other elements. Tension is likely to be caused at the triangular elements for ISBILD. It seems to be one reason for the occurrence of tension

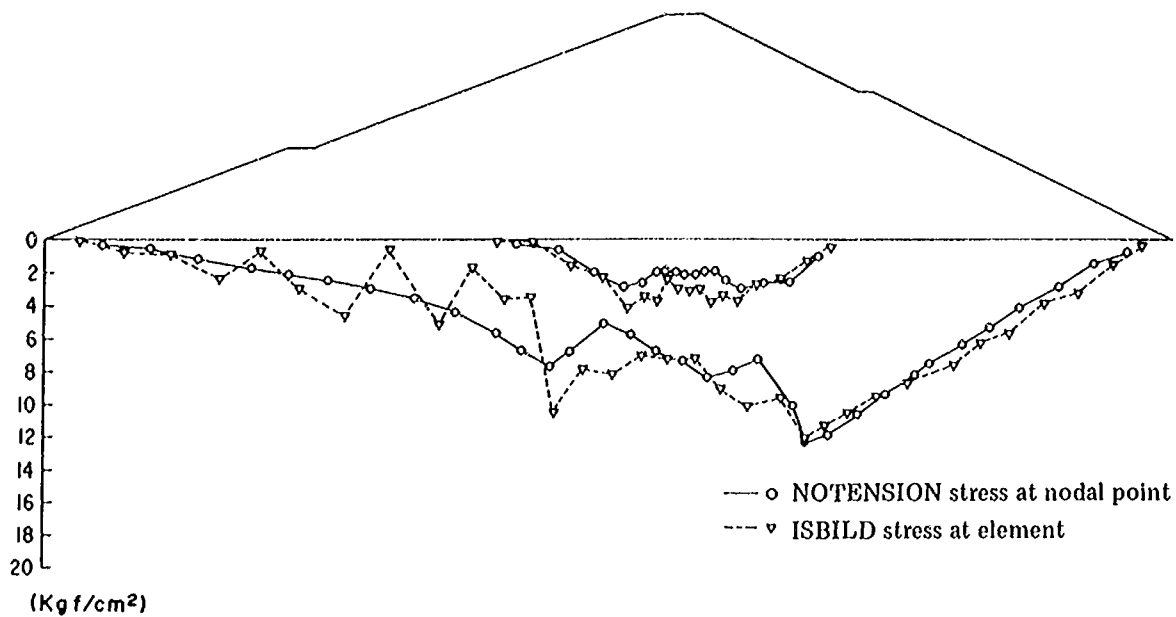
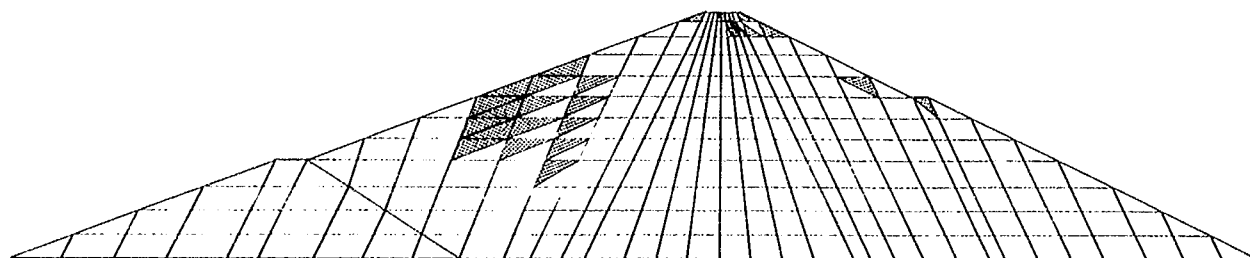
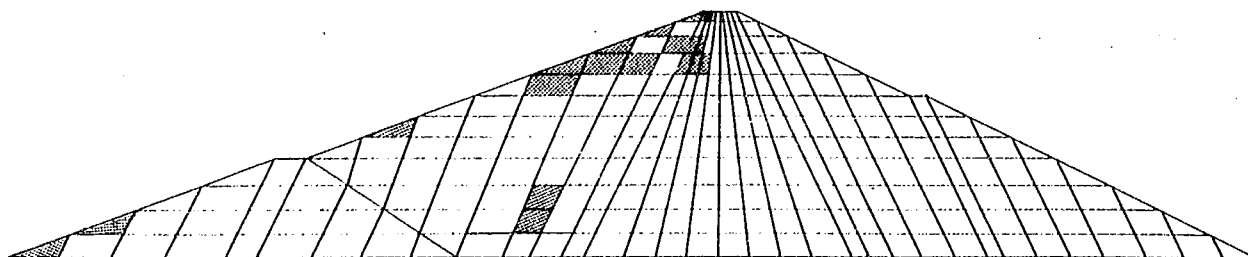


Fig.-A.3 Comparison of vertical stress (after reservoir filling analysis)

that triangular elements with a constant strain distribution inside cause stress concentration, unlike a quadrilateral element with a certain strain distribution.



((NOTENSION) reservoir full, Case 3



(ISBILD) reservoir full, Case 3

Fig.-A.4 Extensional stress of dam

Fig.-A.5 illustrates the deformation of a dam body after reservoir filling. A rise of 30cm upward at the core is recognized by two analysis programs. In two analyses, fill material is regarded as a continuous body, so an upward displacement results from a decrease of the load by buoyancy. However, such behavior seems not to be observed in an actual rockfill dam.

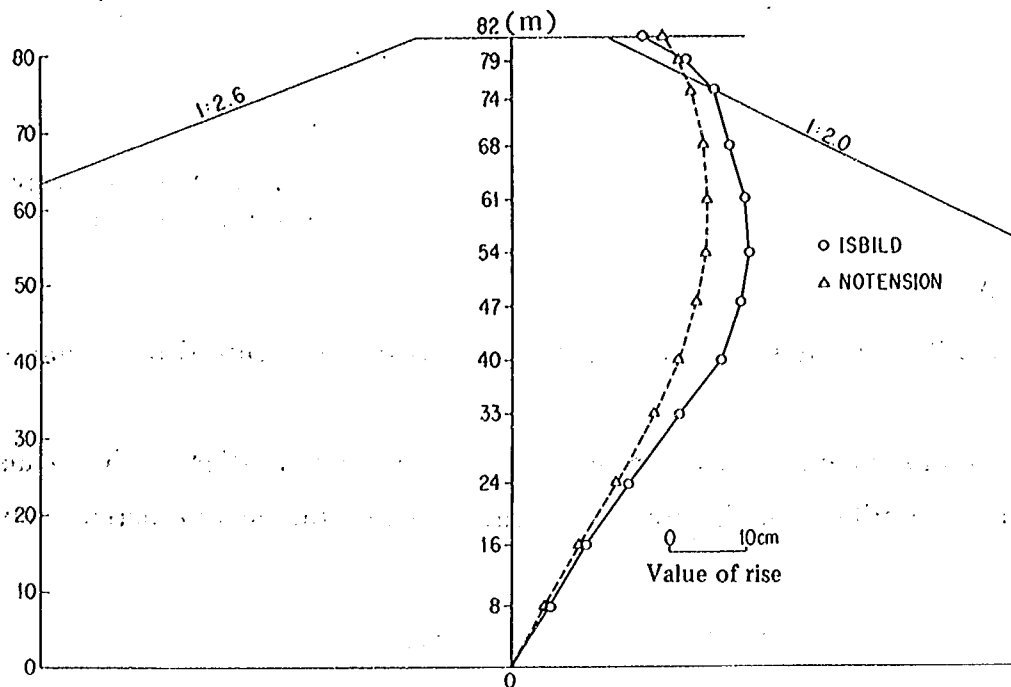
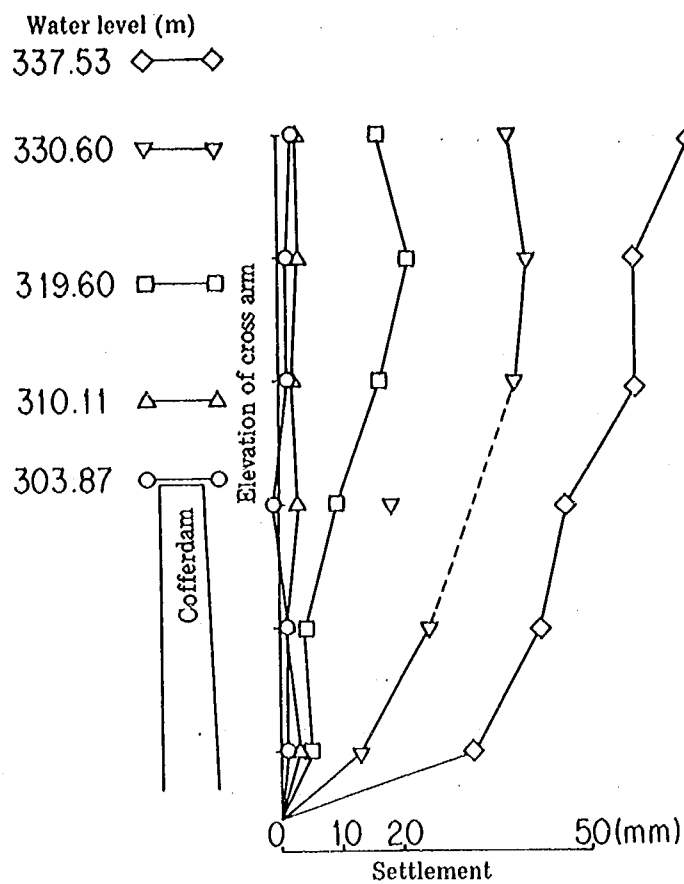
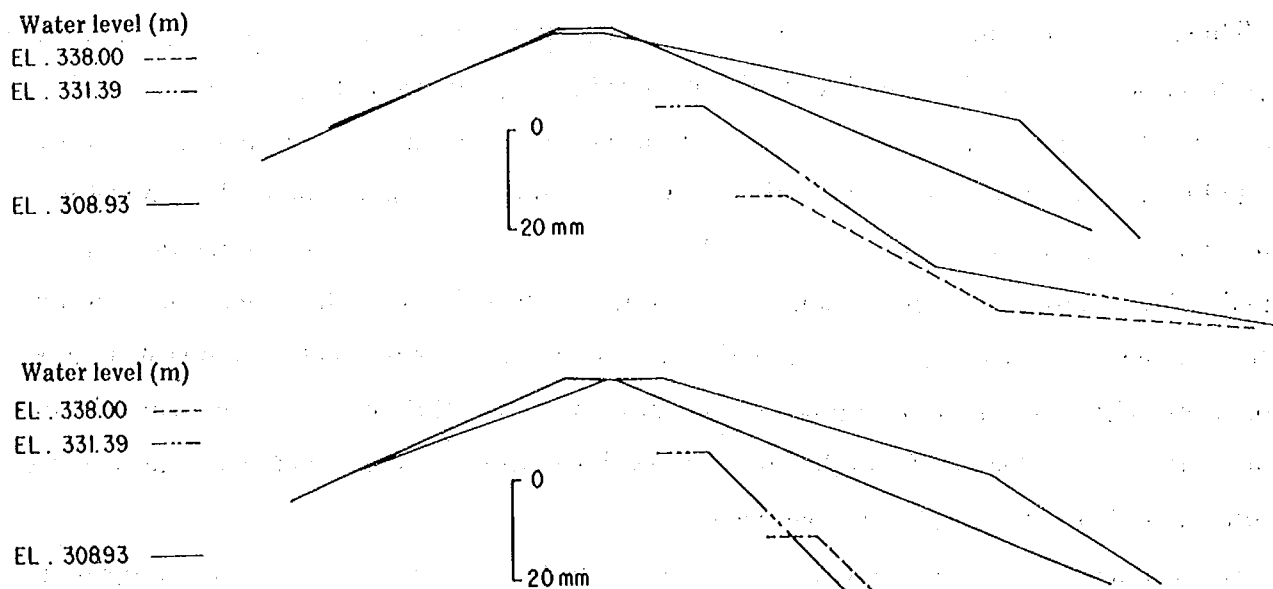


Fig.-A.5 Displacement after reservoir filling

Fig.-A.6 explains the settlement at each layer according to the reservoir filling at the Shirakawa Dam, except the deformation of the rock foundation. A rise with reservoir filling is not recognized, rather a maximum settlement of 7.0cm occurs. This displacement appears to contain the settlement due to consolidation and creep. Therefore it is hard to say that NOTENSION and ISBILD are available for the reservoir filling analysis from Fig.-A.6.



Settlement at core zone



External deformation of dam

Fig.-A.6 Behaviour during reservoir filling

4. Dynamic Analysis

4.1 Design Earthquake Motion

4.1.1 Definition

Earthquake waves are needed as input data to investigate the earthquake-proof status of a rockfill dam with dynamic analysis. Earthquake waves are influenced by the magnitude, the location, and the mechanism of the earthquake.

On the other hand, the behavior of a rockfill dam during an earthquake is affected by the maximum acceleration and frequency characteristic of an earthquake motion. The observed acceleration of a dam site generally becomes larger when the earthquake occurs in a neighboring area, however the response amplitude of a dam becomes smaller due to a high predominant frequency of an earthquake wave. On the contrary, the response amplitude of a dam becomes larger because the predominant frequency of an earthquake wave closes with the natural frequency of a dam body when the earthquake happens at a distant place.

Therefore, it is necessary to select several earthquake waves as input data for dynamic analysis except an effectless wave, the influence of which on a dam body has not been definitely evaluated.

There is a method for determining design earthquake waves from investigations and the specifications of a fault, which is considered to be the cause of an earthquake. Earthquakes are closely connected with faults, especially the oceanic trough in Japan. However it does not always follow that there is a relation between earthquakes and inland faults. Therefore, at the present time the magnitude and the location of a forecasted fault is decided on the basis of previously recorded historical earthquakes. Specifically, the concept is adopted that the occurrence of earthquakes with the same mechanism are expected in the future at a location where many earthquakes have happened in the past. The magnitude of an earthquake is decided from the maximum value in the past, considering an engineering judgment about the accuracy of records.

Several design earthquake waves are selected on the basis of the engineering judgment and the statistical product of the past records.

4.1.2 Selection of Design Earthquake Motion

The history of earthquakes at the dam site concerned is chosen from the list of

earthquakes, and then stereotyped and divided into several groups. The epicentral distance and magnitude are determined from the history of earthquakes and the engineering judgment in every group. It is necessary for design earthquake motion to include a neighboring earthquake and a major earthquake.

The design earthquake motion concerning an obvious active fault is decided on the basis of the location and magnitude of the fault.

It is difficult to decide on the design earthquake motion about the so-called blank area of earthquakes, so earthquake waves are judged from the past records, and the better product of earthquake engineering should be successively adopted.

Note: M ; Magnitude

Δ ; Epicentral Distance

4.1.3 Estimation of the Maximum Acceleration

The relation of the maximum acceleration and M , Δ is not clear as stated before, however the maximum acceleration is calculated by using publicly announced experimental equations with the engineering judgment. The maximum acceleration should contain an acceleration of which the probability of occurrence is once in more than 200 years.

The maximum acceleration of the vertical motion ranges from $1/4$ to $1/2$ that of the horizontal motion. In general, the longer the epicentral distance and the larger the magnitude becomes, the smaller the acceleration of the vertical motion becomes.

4.1.4 Frequency Characteristics

The frequency characteristics of earthquake motion are affected by M , Δ , the mechanism of the earthquake, and the geology of a dam site area. A rock foundation is in particular seriously influenced by the above-mentioned factors. It is not obvious to conclude the effect of M and Δ from recorded earthquakes in the past, however it is considered that the larger the magnitude (M) and the longer the epicentral distance (Δ) becomes, the longer the predominant period becomes. Definite predominant frequencies rarely appear, and frequencies usually range widely.

Therefore at present time predominant frequencies are estimated from an experimental equation by M and Δ , and further more it is proper to regard the peak value of the power spectrum with a spectrum window of 0.4Hz as the predominant frequencies.

As for the characteristics of the horizontal and vertical motions, vertical motions appear

to generally have components of higher frequencies than horizontal ones. However the above-mentioned relation cannot be said clearly from the past data, so the relation of real frequency characteristics of horizontal and vertical earthquake motions are used under the selection that they are similar to the expected frequency characteristics of design earthquake motion.

4.1.5 Duration of Motion

The duration of motion is calculated by means of some experimental equations with M and Δ , and it ranges from several seconds to several tens of seconds. The limit of the duration is at least 25 seconds, because a long duration entails much expense and labor for a computation. The vicinity of peak acceleration of main motion is important for the behavior of a dam body, and the other part of a motion is not so effective.

The criteria of duration at each magnitude is from observed records at the Public Works Research Institute, Ministry of construction, as follows.

Magnitude	Duration (seconds)
6.0	10.0
6.5	12.0
7.0	15.0
7.5	24.0
(8.0)	(More than 25 seconds is sometimes necessary)

4.1.6 Composition of Earthquake Motion

Several motions that have an influence on the behavior of a dam body are considered to be input earthquake waves.

The following methods are adopted as the composition of earthquake motion, and they are connected with the determination of the above-mentioned parameters.

- i) Determining response spectrum, input motions are composed to satisfy the given response spectrum.
 - a) Random phases are used.
 - b) Phases of real earthquake motions are used.
- ii) Input waves are composed to satisfy the amplitudes of the Fourier spectrum.
 - a) Random phases are used.
 - b) Phasis of real earthquake motions are used.

here, as for spectrum.

- a) Composing freely. (Envelope spectrum)
- b) Slightly changing the spectrum of a real earthquake wave.
- c) Raw spectrum of real earthquake wave is used.

Method i) is adopted in the field of constructing an atomic plant. It is difficult for a dam site to obtain response spectra because the number of earthquake records is limited at the present time. On the contrary, an earthquake motion that satisfies the envelope spectrum is not actual.

At the present time, taking a serious view of each real earthquake motion, it is proper to determine the input motion using the spectra of real earthquake motions. Real spectra do not usually give a required predominant frequency, so the frequency characteristics are adjusted by shifting the time scale, considering the magnitude and the duration of the earthquake motion. In this case it is desirable to fit obtained predominant frequencies with frequencies that are slightly over those of the peak value of the original power spectrum with the proper window.

As for phase, there are two methods of using random phases and the phases of a real earthquake wave. When random phases are used, the composed motion is not similar to a real earthquake motion, and many are composed from the same spectrum and a change of the response of a dam body is probable. Therefore the phasis of a real earthquake wave are used as a tentative measure.

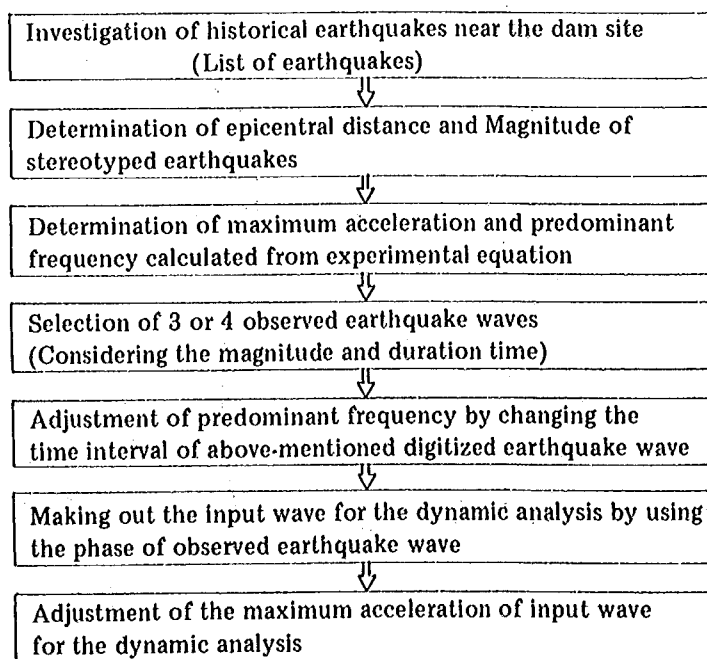


Fig.-4.1.1 Flowchart for making out the input wave

Hence, the following items must be taken care of composing input motion by modifying a real earthquake motion a little.

- ① It is desirable that the degree of adjustment of the maximum acceleration and predominant frequencies be smaller.
- ② Motions should not have a spike-shaped peak value. (Recently, the recorded peak value has an inclination to become larger according to the improvement of a seismograph's frequency characteristic, however the peak value is usually like a spike-shape and appears to have a small influence on the behavior of a dam body.)

The procedure of composing input motions is shown in Fig.-4.1.1.

4.2 Design Earthquake Motion in Analysis

Historical earthquakes at the dam site are investigated as shown in Fig.-4.2.1. Nineteen earthquakes occurred within a 150km radius. The locations where earthquakes occurred were classified into five zones, as shown in Table.-4.2.1. Those zones are Miyagi prefecture

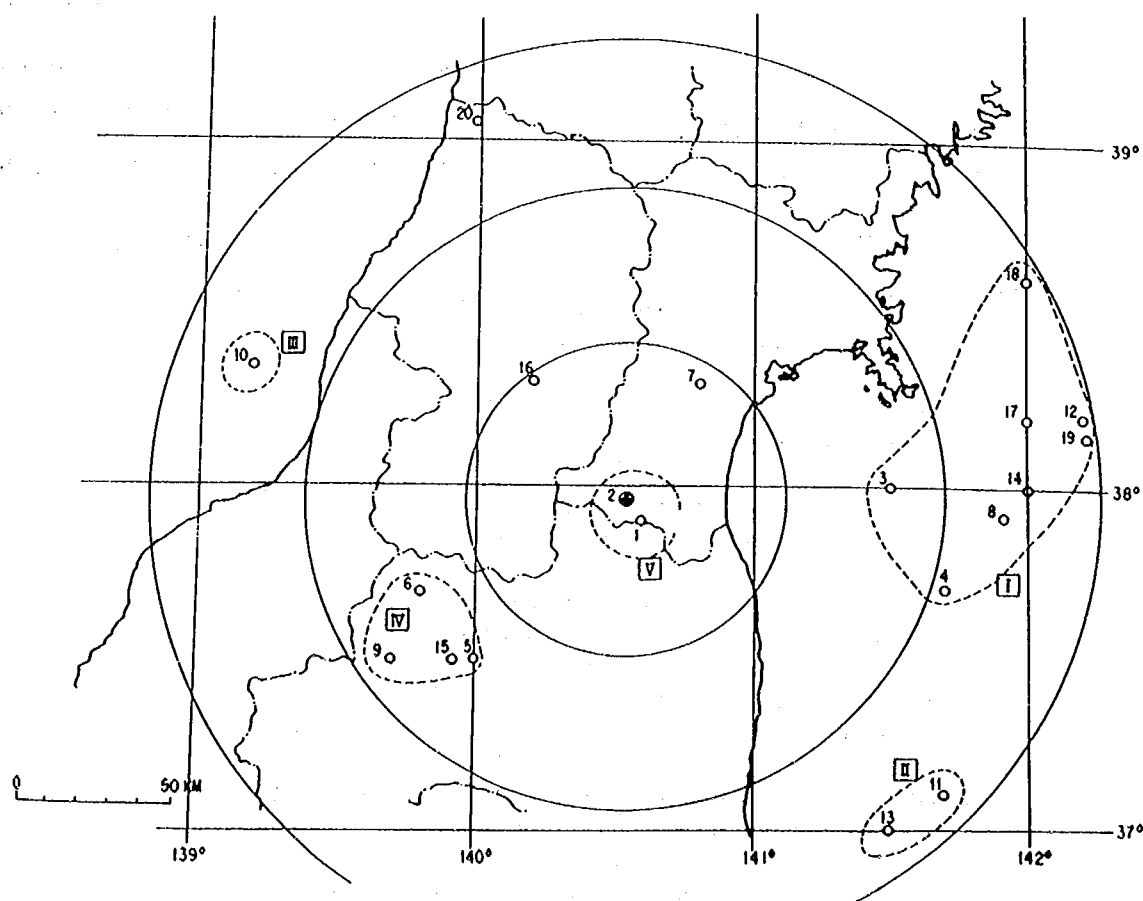


Fig.-4.2.1 Epicenters of historical earthquakes

off, Fukushima prefecture off, Niigata prefecture off, the vicinity of Lake Inawashiro at Fukushima prefecture, and the vicinity of the dam site. The maximum accelerations of stereotyped earthquakes are indicated in Fig.-4.2.2 and Table-4.2.3. Design earthquakes are determined on the basis of earthquake motions recorded at the existing dams in the past. Here the earthquake motions recorded at the Izu peninsula in 1980 are adopted as the motions anticipated to occur near the dam site.

The predominant frequencies of earthquake motions are decided from Seed's diagram, as shown in Fig.-4.2.2, with the magnitude and the epicentral distance of stereotyped earthquakes.

Table-4.2.1 The list of disastrous earthquake at the dam site concerned

No.	Date	M	Epicentral distance	after Okamoto, Tamura gal	after Iwasaki, et al. gal	after Kanai gal	after Cloud gal	Period (sec) (Seed)	Remark
1	1731.10. 7	6.6	9 km	261	106	16.8cm/s 351gal	415	0.3	
2	1956. 9.30	6.1	3	223	97	13.1 329	487	0.25	
3	1897.10. 2	7.2	86	126	58	2.8 39	105	0.45	
4	1646. 6. 9	7.6	108	124	61	3.5 40	81	0.55	
5	1555. 9.14	6.7	67	113	53	2.0 36	137	0.35	
6	1433.11. 7	6.7	69	111	52	1.9 34	133	0.35	20 days of aftershock
7	1736. 4.30	6.2	46	108	51	1.7 36	191	0.3	
8	1835. 7.20	7.6	121	106	56	2.9 33	70	0.55	
9	1611. 9.27	6.9	87	98	50	1.8 28	104	0.4	
10	1964. 6.16	7.5	125	96	53	2.5 29	67	0.55	Niigata earthquake V
11	1938.11. 5	7.7	141	91	53	2.7 28	57	0.6	East Fukushima prefecture off earthquake
12	1936.11. 3	7.7	150	82	51	2.5 24	53	0.65	
13	1677.11. 4	7.4	137	78	47	1.9 20	60	0.6	
14	1897. 2.20	7.3	130	78	47	1.7 19	64	0.55	V ~ VI
15	1710. 9.13	6.4	73	78	44	1.2 22	26	0.35	
16	1944.12. 7	5.8	48	71	42	0.9 23	85	0.25	Rumbling of the ground
17	1937. 7.27	7.2	133	69	44	1.5 17	62	0.55	
18	1770. 5.27	7.4	148	68	45	1.6 17	54	0.6	
19	1978. 6.12	7.4	151	66	44	1.6 17	52	0.6	Miyagi prefecture off earthquake
20	1804. 7.10	7.1	132	63	42	1.3 15	63	0.55	Niigata earthquake

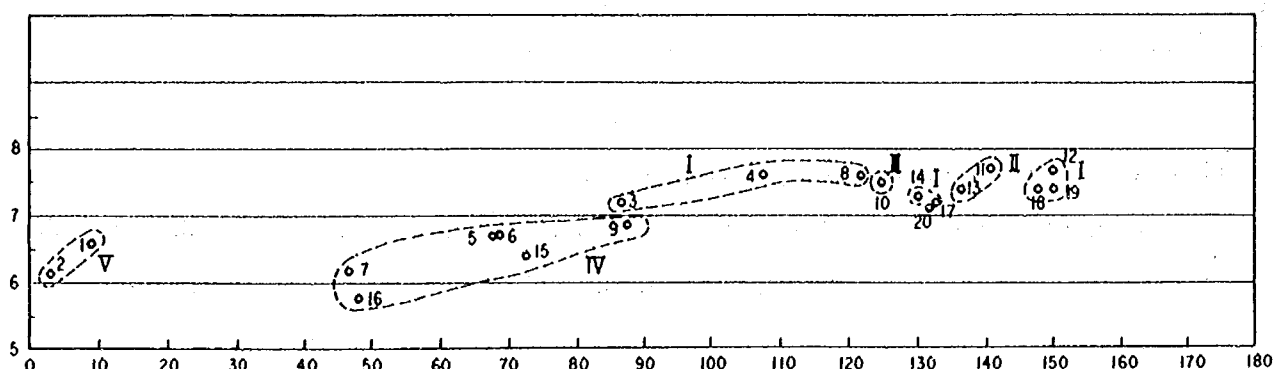


Fig.-4.2.2 The relation between epicentral distance of historical earthquakes and magnitude

Fig.-4.2.2 Design earthquake at each type

Type	Number of earthquake	M	Epicentral distance	Period (sec)	Remark
I	3, 4, 8 12, 14, 17 18, 19	7.5	120 km	0.55	Miyagi prefecture off earthquake system, V ~ VI of Japanese intensity scale in Shiroishi city
II	11, 13	7.5	140 km	0.6	East Fukushima prefecture off earthquake system, V of Japanese intensity scale in Shiroishi city
III	10	7.5	120 km	0.55	Niigata earthquake system, V of Japanese intensity scale in Shiroishi city
IV	5, 6, 16 9, 15, 7	6.7	75 km	0.35	Inland earthquake which distance is the same as that of earthquake occurred near the part of lake Inawashiro
V	1, 2	6¼	vicinity (6km)	0.25	Near-by earthquake

Table-4.2.3 The maximum acceleration and period corresponding to design earthquake

Type	M	Epicentral distance (km)	Principle period (Seed) sec	After Okamoto and Tamura (gal)	After Iwasaki, et al. (gal)	After Kanai		After Cloud (gal)	Maximum acceleration		Remark
						V (cm/s)	α (gal)		Horizontal	Vertical	
I	7.5	120	0.55	102	54	2.60	30	71	120 gal	40	$d_v = \frac{dH}{3}$
II	7.5	140	0.6	81	49	2.06	22	58	90	30	..
III	7.5	120	0.55	102	54	2.60	30	71	90	30	..
IV	6.7	75	0.35	117	54	1.69	30	141	70	27	..
V	6¼	6	0.25	254	97	13.12	330	449	250	125	$d_v = \frac{dH}{3}$

Table-4.2.4 General description of input earthquake motion

	Earthquake wave	Predominant frequency	Maximum acceleration (gal)	Duration time (sec)
Case-A	Yokoyama FY,FZ	4.2	250 (Hori.), 125 (Vert.)	8
Case-B	Tarumizu FY,FZ	1.9, 3.5	120, 40	18
Case-C	Ajiro EW,VD	3.7, 5.0	350, 333	12

Three design earthquake motions are used for the dynamic analysis, as shown in Fig.-4.2.4 to Fig.-4.2.6. The general description of motions are indicated in Table-4.2.4. The duration time is changed according to the main motion at each design earthquake.

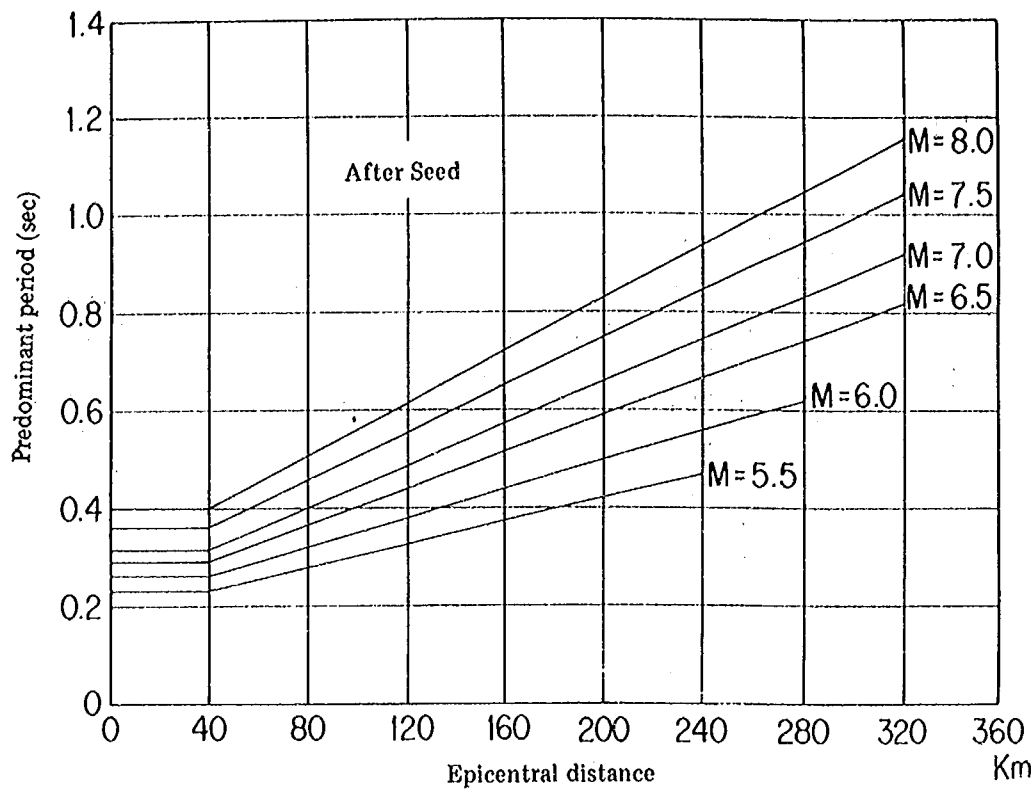
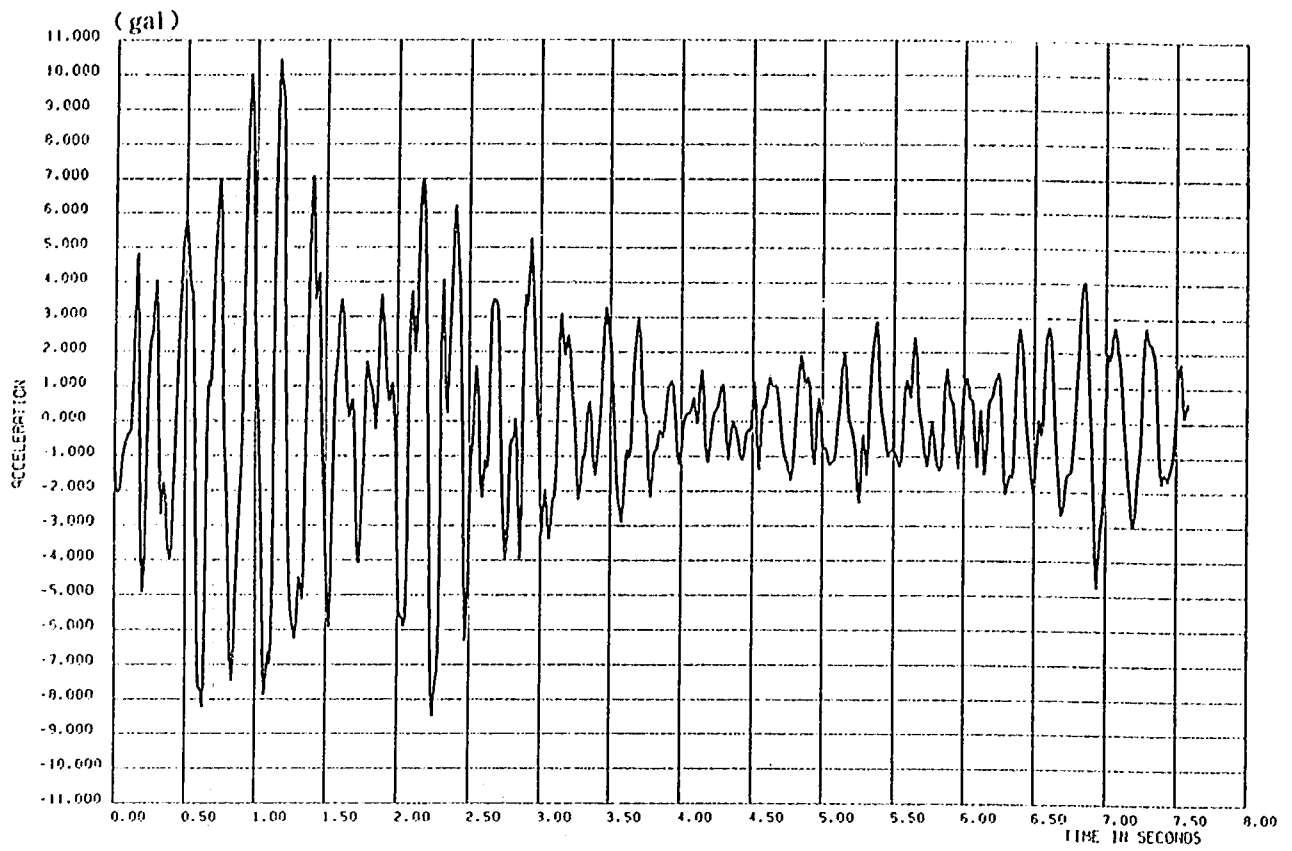
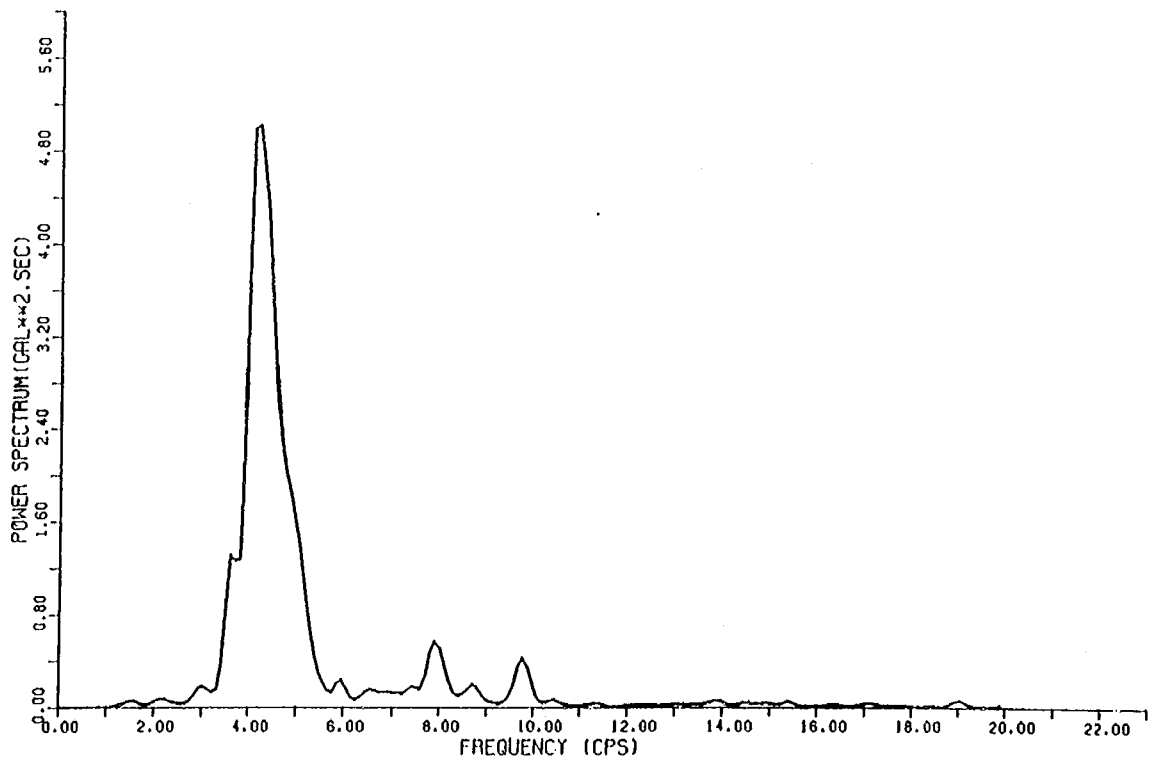


Fig.-4.2.3 The relation between epicentral distance and predominant period

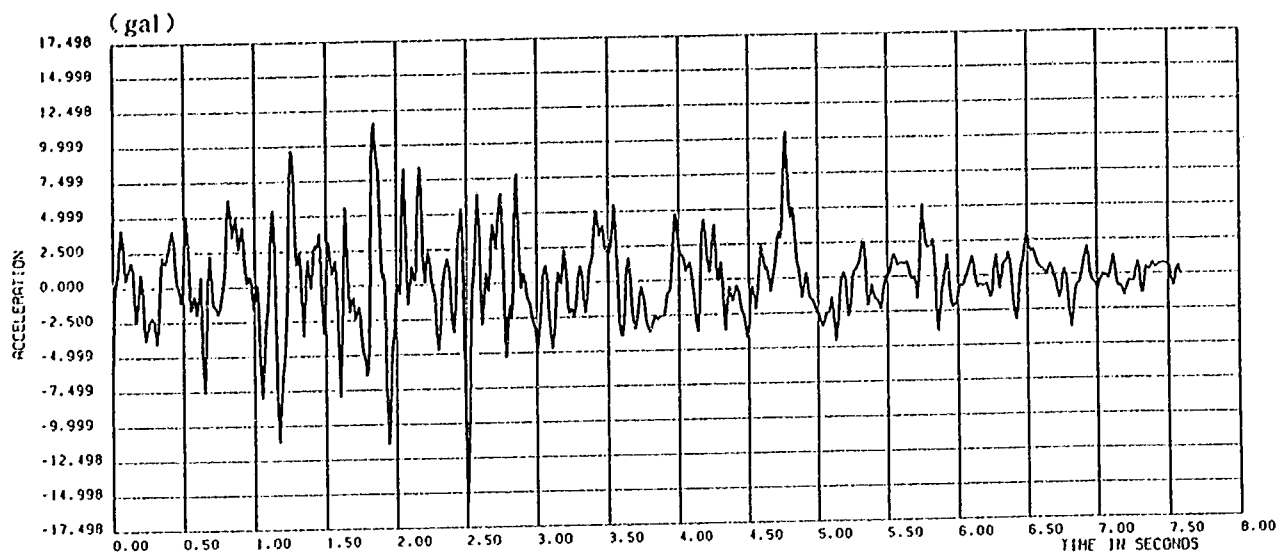


Time-history of acceleration

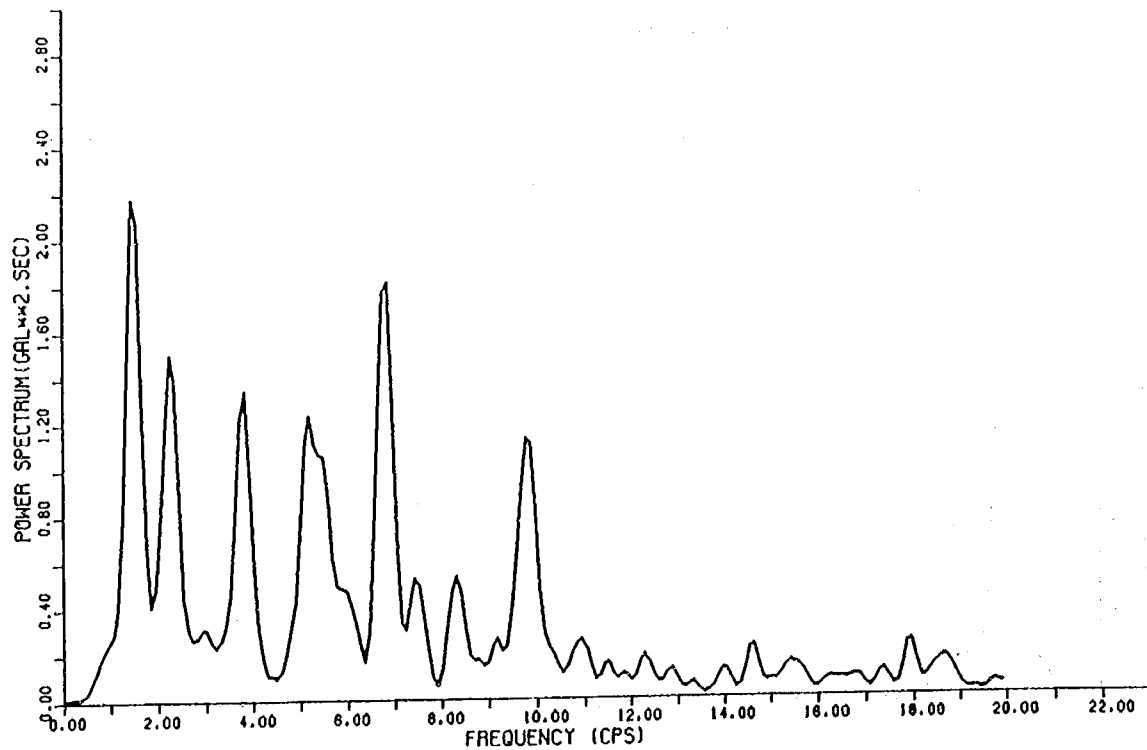


Power spectrum

Fig.-4.2.4(a) Case-A (Horizontal)

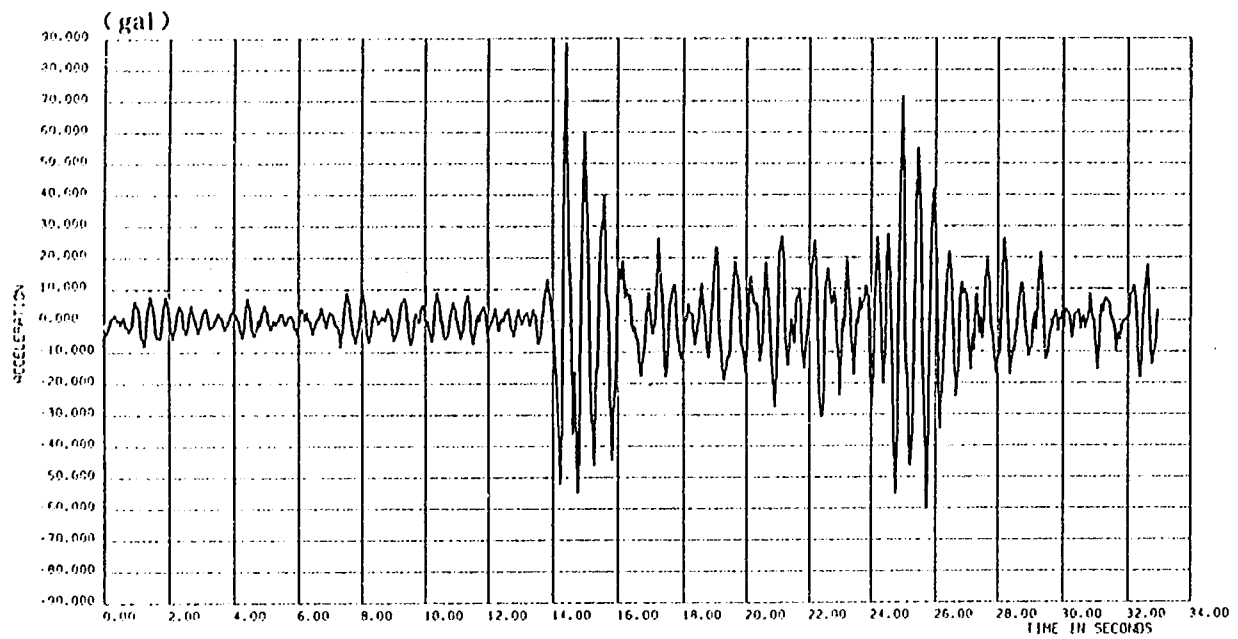


Time-history of acceleration

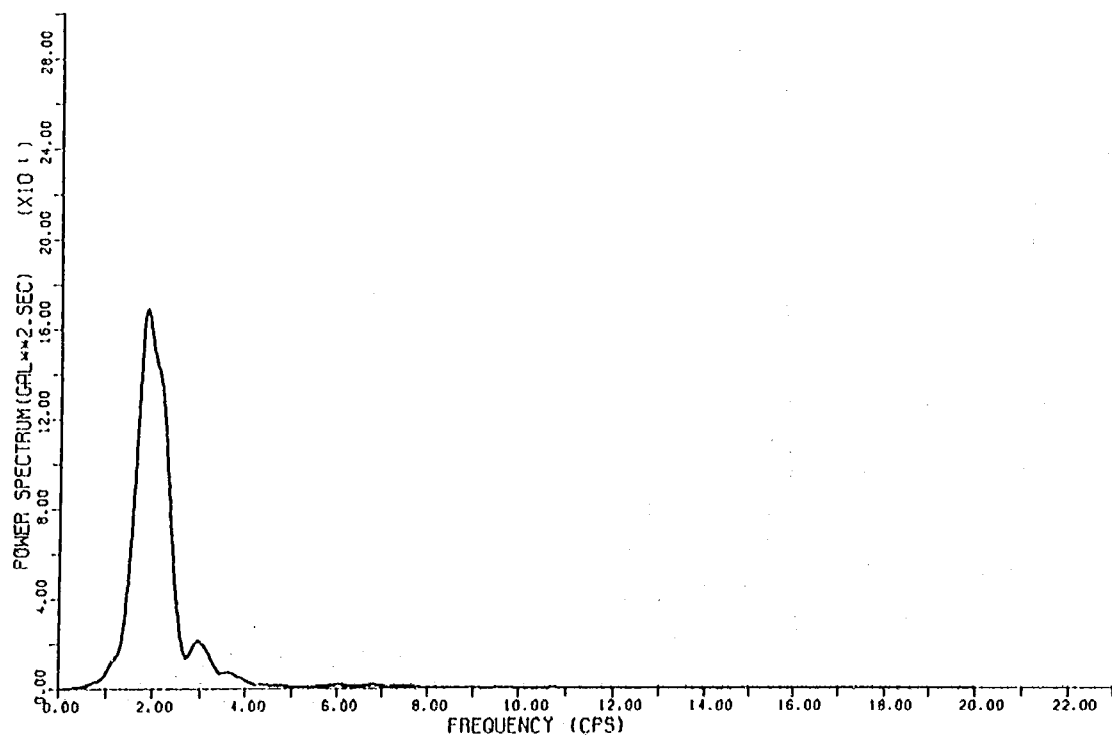


Power spectrum

Fig.4.2.4(b) Case-A (Vertical)

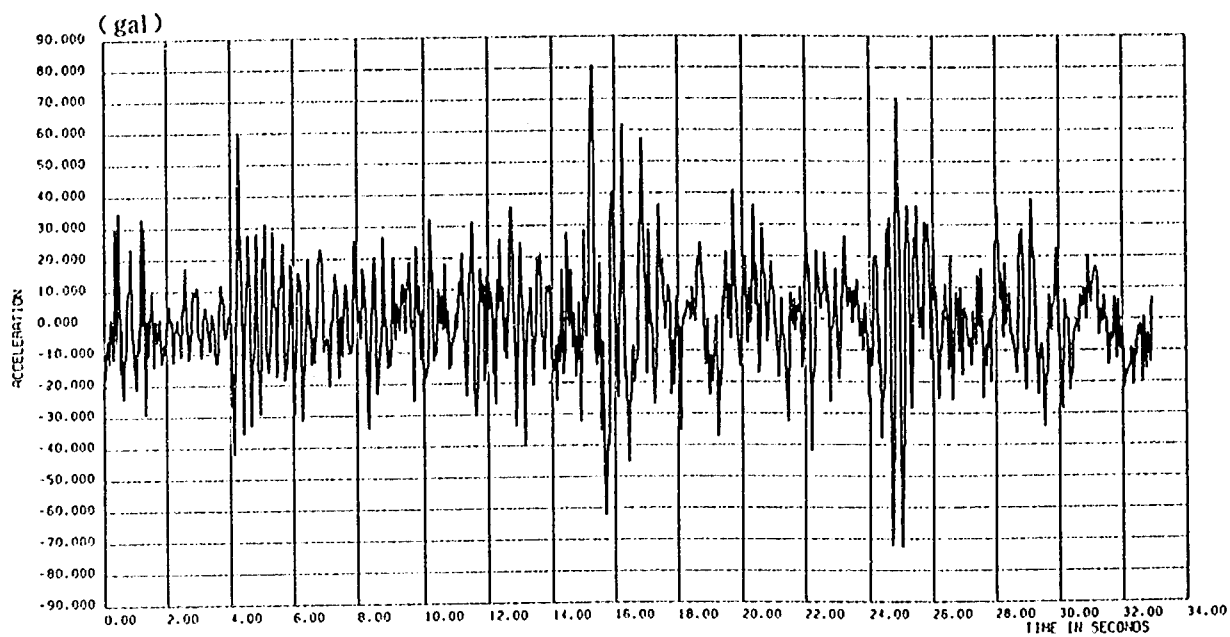


Time-history of acceleration

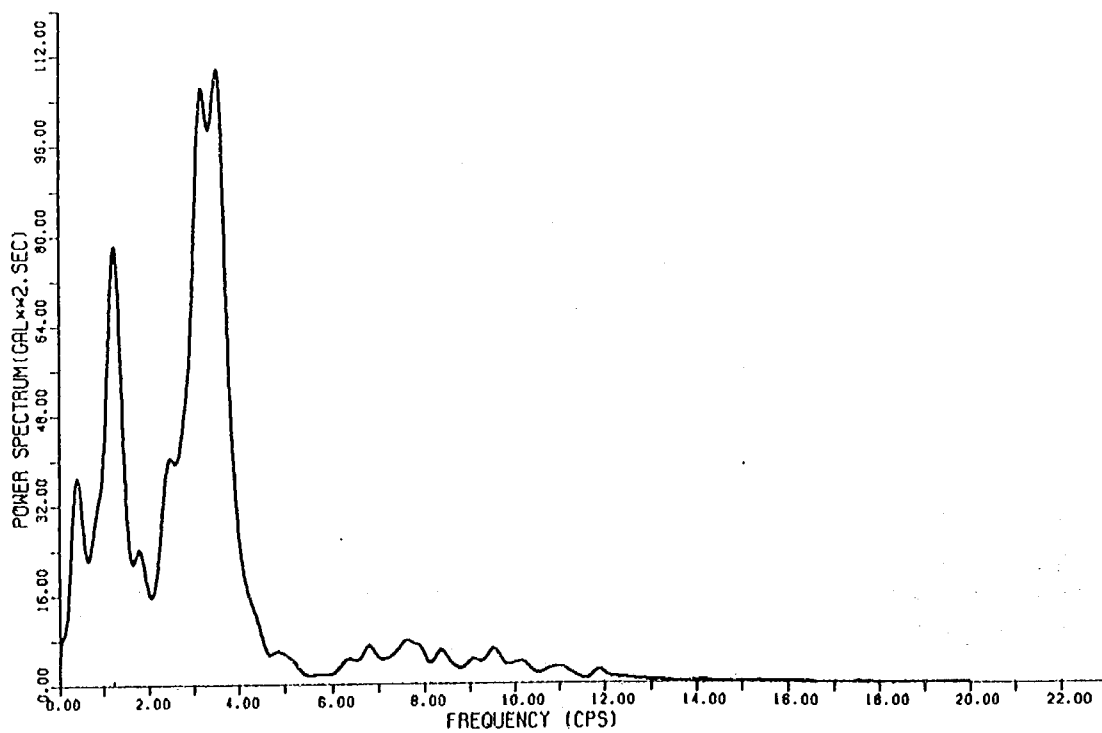


Power spectrum

Fig.-4.2.5(a) Case-B (Horizontal)

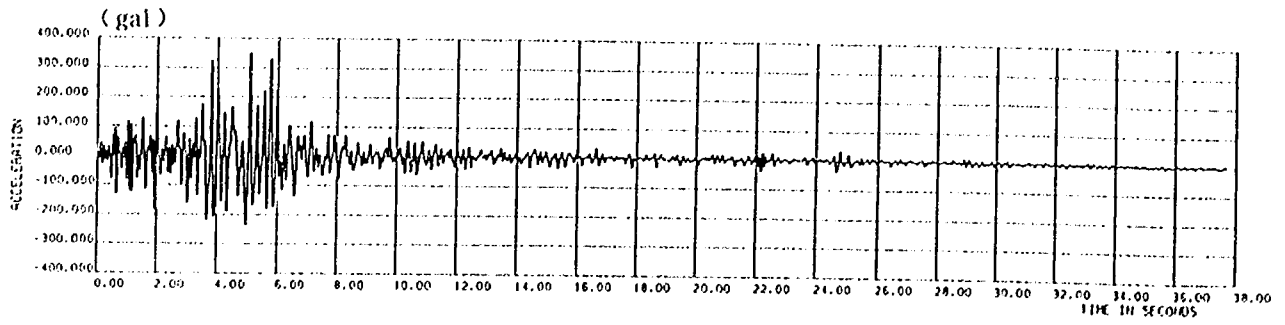


Time-history of acceleration

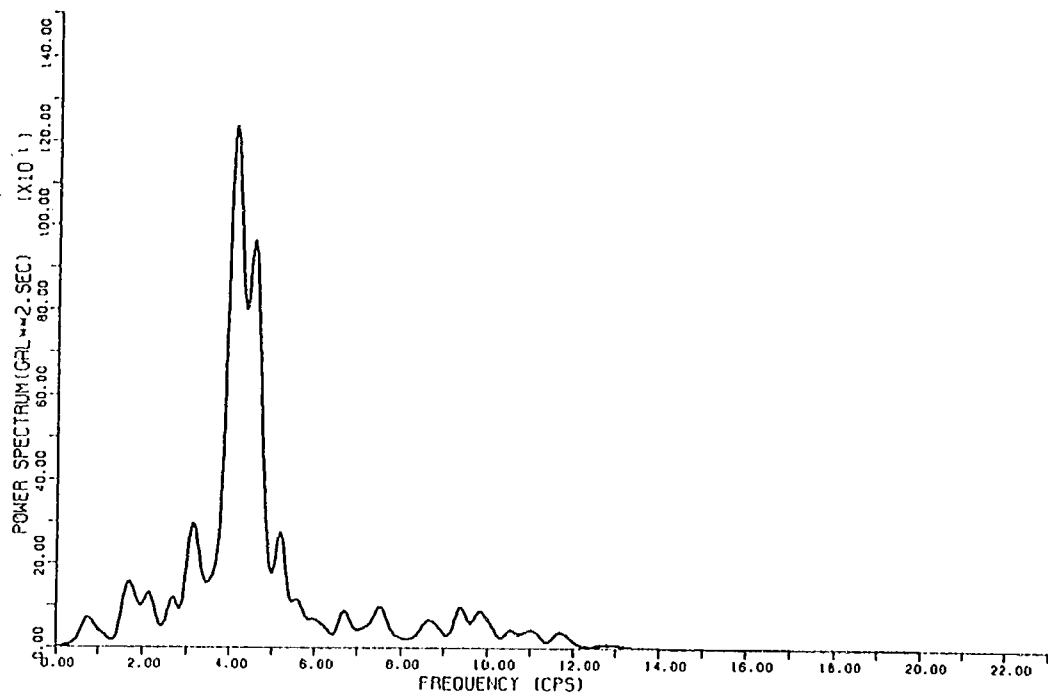


Power spectrum

Fig.-4.2.5(b) Case-B (Vertical)

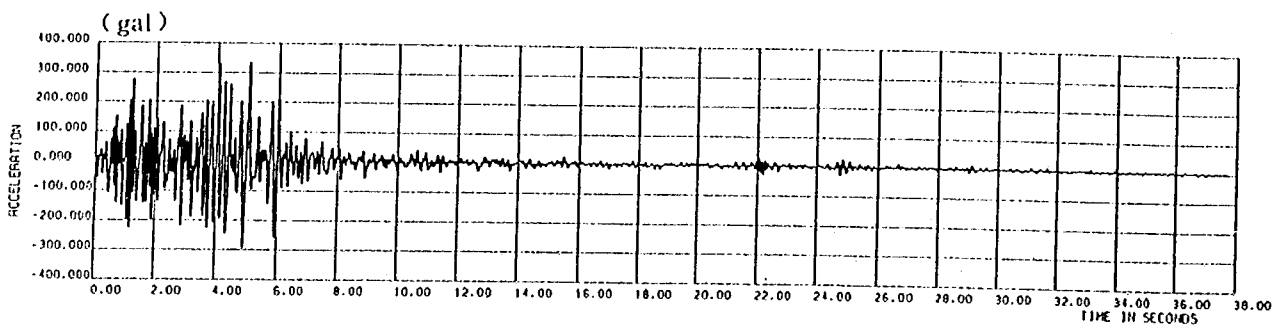


Time-history of acceleration



Power spectrum

Fig.-4.2.6(a) Case-C (Horizontal)



Time-history of acceleration

Fig.-4.2.6(b) Case-C (Vertical)

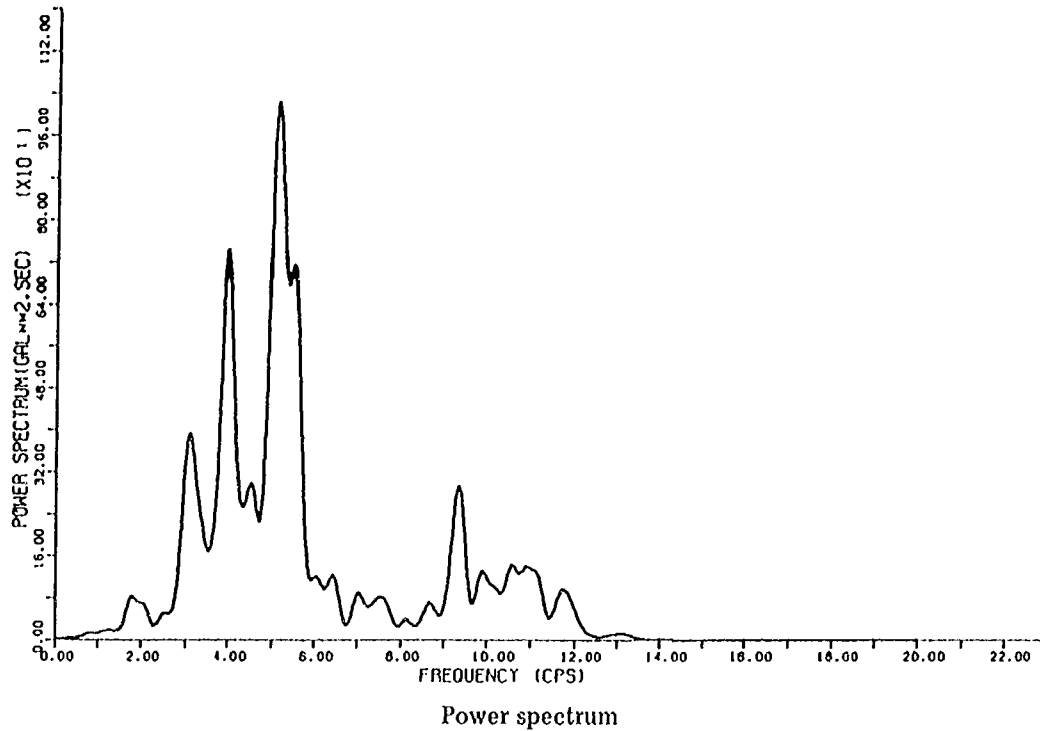


Fig.-4.2.6(b) Case-C (Vertical)

4.3 Properties of Rockfill Materials for Dynamic Analysis

4.3.1 Cyclic Triaxial Test in the Laboratory

The dynamic deformation characteristics of rock and filter materials are tested by a large-scale cyclic triaxial testing apparatus, transporting materials from the quarry site of a rockfill dam. The specimen used in the triaxial test is 30cm in diameter and 60cm high, and in a dry condition. The physical properties of the rock and filter are shown in Table-4.3.1. The rock material is single and andesite; river deposit is filled at the filter zone. The maximum particle size of the specimen is 38.1mm, which is less than 1/5 of the specimen size. The filter material is the so-called terrace deposit, composed of round gravel that is slightly weathered, easy to crush, and comparatively of many fine particles. The materials were placed in six layers, and each layer was compacted with a 33kg·f weight electric vibratory rammer. The test was determined from the compaction tests so that the relative density of the materials should be 85%. The gradation curve of the specimen is illustrated in Fig.-4.3.1.

The specimens were at first uniformly consolidated with a confining pressure of 1.0kgf/cm², and a cyclic deviatoric axial stress of 0.2Hz is applied to the axial stress. The tenth stress-strain loop of the cyclic loading was used to obtain the dynamic deformation

properties (Shear moduli and damping ratio) at each stress level. The cyclic deviatoric stress was increased gradually at the same confining pressure, and the specimen was subjected to the next confining pressure when the shear strain reached around 1×10^{-3} . Several levels of confining pressure were taken for a specimen. All specimens are in dry condition and drain condition through a consolidation and a cyclic loading.

The main features of performances are shown in Table-4.3.2, and Fig.-4.3.2 illustrates the apparatus.

Table-4.3.1 Physical properties of material

	Rock (Andesite)	Filter
	R-12	F-36
Maximum grain size (mm)	63.5	38.1
Minimum grain size (mm)	0.42	0.074
D_{10} (mm)	2.0	0.14
D_{15} (mm)	10	0.92
D_{30} (mm)	19	2.8
D_{60} (mm)	23	5.0
D_{95} (mm)	40	20
Uniformity coefficient	12	36
Coefficient of curvature	2.2	1.2
Specific gravity	2.575	2.571
Water absorption (%)	3.48	4.87
Water content (%)	2.3	1.2

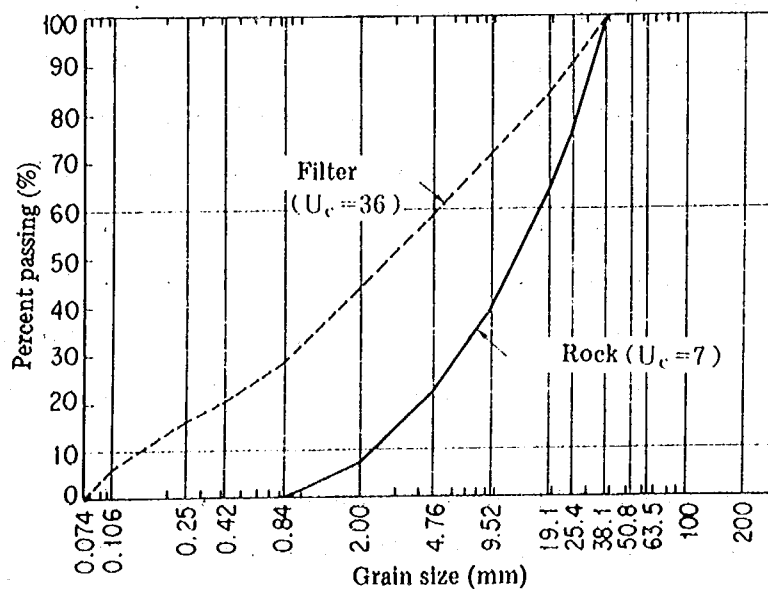


Fig.-4.3.1 Grain size distribution curve

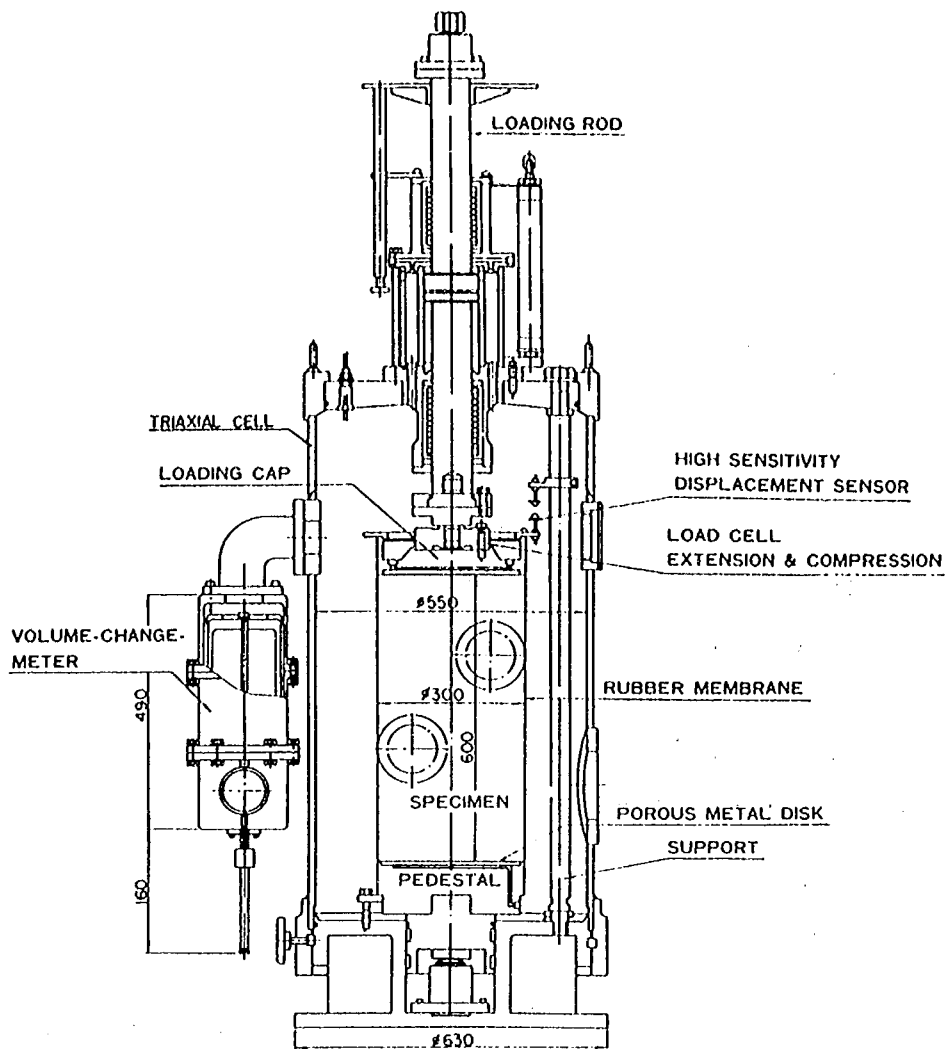


Fig-4.3.2 Cyclic triaxial testing apparatus

Table-4.3.2 Main features of performances

Specimen size	ϕ 30cm x h 60cm
Control system	Electro-Hydraulic-Servo
Wave form	Sinusoidal, triangular, and earthquake waves
Axial loading	Maximum loading capacity compression — 20 tf
	Maximum loading capacity extension — 20 tf
	Frequency range 0.2 — 10 Hz
Confining pressure	Max. 15 kgf/cm ²
	Frequency range 0.2 — 5 Hz

The shear moduli and damping ratios of the rock and filter materials were computed as follows.

$$\left. \begin{aligned} E_d &= \frac{\sigma_{ad}}{\epsilon_{ad}} \\ \nu &= \frac{1}{2} \left(1 - \frac{\epsilon_{vd}}{\epsilon_{ad}} \right) \\ G &= \frac{E_d}{2(1+\nu)} \\ \gamma &= (1+\nu) \cdot \epsilon_{ad} \end{aligned} \right\} \quad \dots (4.1)$$

where

E_d ; Dynamic elastic modulus

ϵ_{vd} ; Dynamic volumetric strain

σ_{ad} ; Dynamic deviatoric stress (Axial stress)

ν ; Poisson's ratio

ϵ_{ad} ; Dynamic axial strain

G ; Shear modulus

The damping ratio "h" was obtained as follows (see Fig.-4.3.3).

$$h = \frac{1}{2\pi} \cdot \frac{\Delta W}{W} \quad \dots (4.2)$$

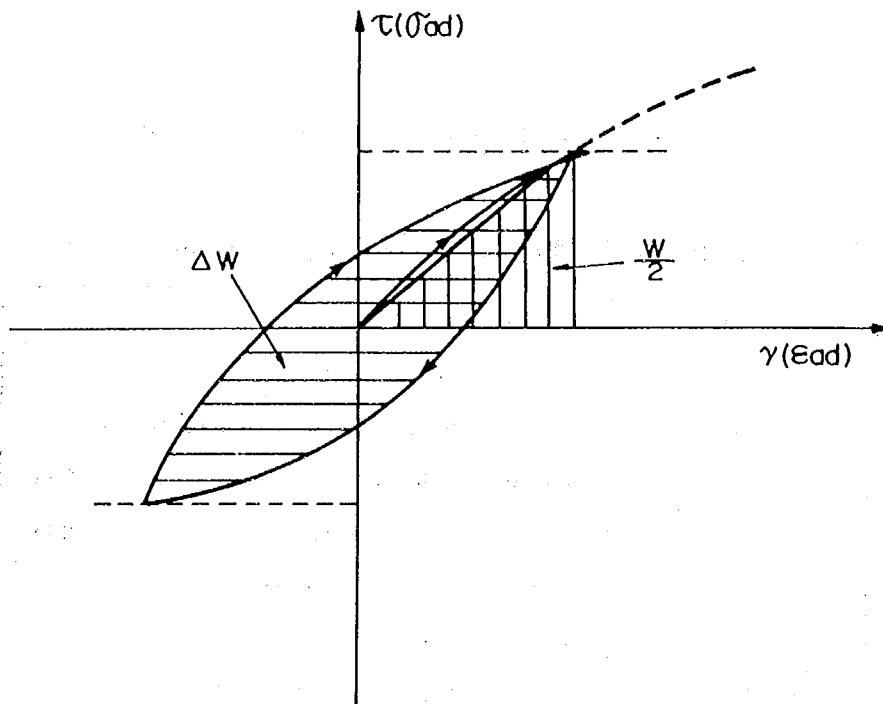


Fig.-4.3.3 Hysteresis loop

The five levels of confining pressure were taken for a specimen as 1,2,3,4 and 6kgf/cm². The test results are illustrated in Fig.-4.3.4 to Fig.-4.3.7. The shear moduli of the rock materials were larger than those of the filter materials at the same confining pressure.

The damping ratios of the filter materials were 2~5 percent larger than those of the rock materials.

The gradation distribution, void ratio, mean principal stress, and principal stress ratio have an influence on the shear moduli, and the effects of the void ratio and mean principal stress are described in the following.

Fig.-4.3.8 explains the relation between the shear moduli and void ratio. The shear moduli were divided by the mean principal stress $(\sigma_m)^m$ to exclude the effect of the confining pressure. An index number "m" is the gradient in the relation between the shear moduli and the mean principal stresses. G corresponds to the shear modulus at 1×10^{-5} of shear strain, and is a function of $(2.17-e)^2/(1+e)$ and coincident with the test result of coarse grained granular materials by Kokusho et al. In respect of the above-mentioned, the relation of the mean principal stress σ_m to $G \cdot (1+e)/(2.17-e)^2$ is indicated in Fig.-4.3.9. Consequently, the shear moduli of the rock and filter materials at shear strain $\gamma = 1 \times 10^{-5}$ is expressed by equation (4.3) as a function of the void ratio "e" and mean principal stress σ_m .

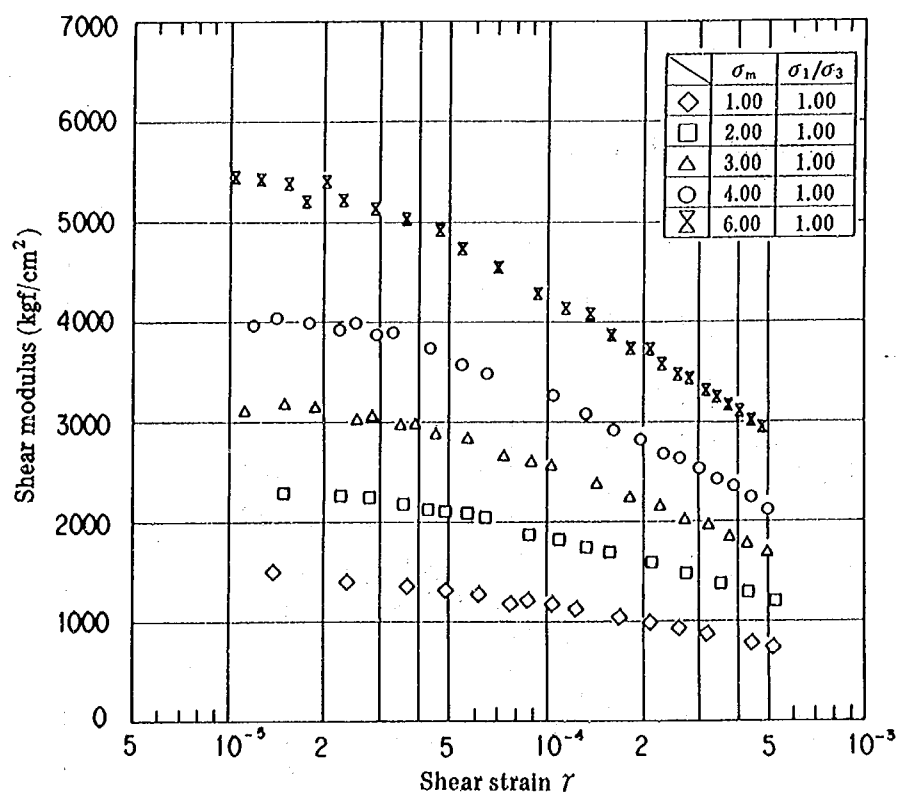


Fig.-4.3.4 Shear modulus ~ Shear strain (Rock)

$$\left. \begin{array}{l} \text{Rock } G = 930 \cdot \frac{(2.17 - e)^2}{1 + e} \cdot (\sigma_m)^{0.6} \\ \text{Filter } G = 600 \cdot \frac{(2.17 - e)^2}{1 + e} \cdot (\sigma_m)^{0.7} \end{array} \right\} \dots\dots\dots (4.3)$$

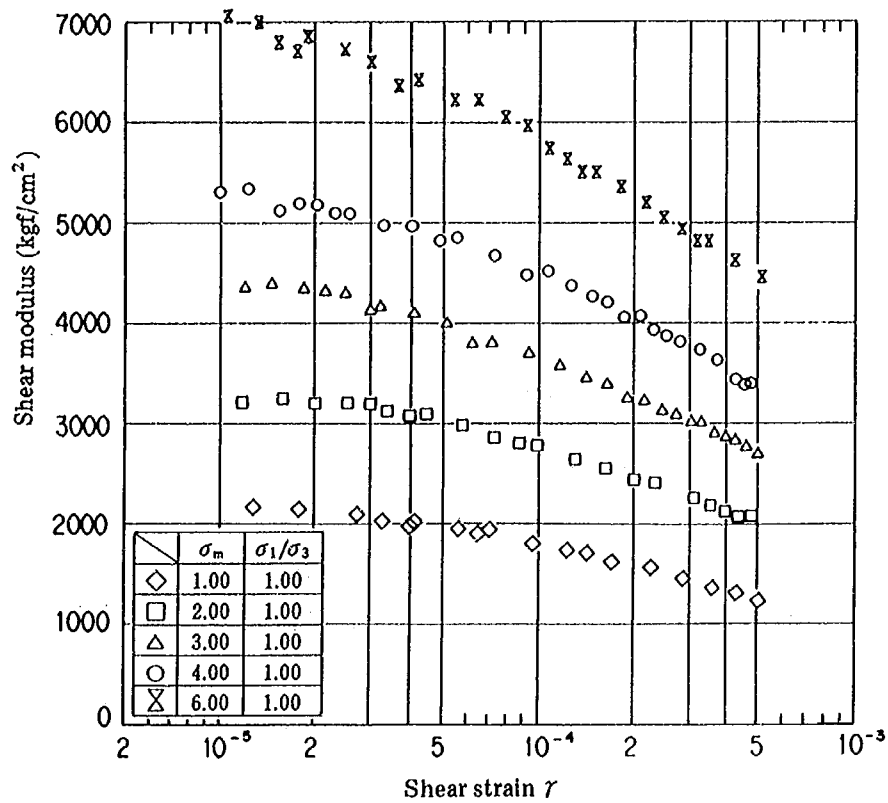


Fig.-4.3.5 Shear modulus ~ Shear strain (Filter)

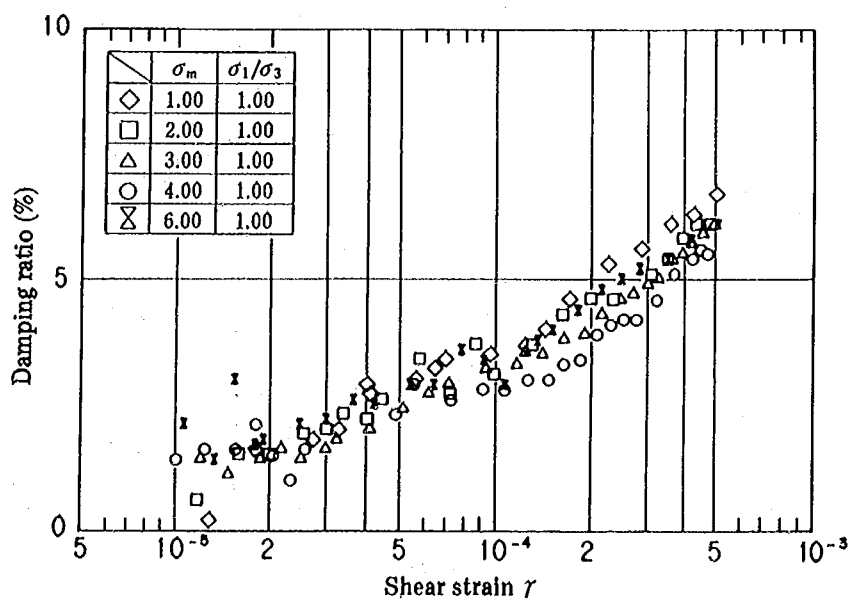


Fig.-4.3.6 Damping ratio ~ Shear strain (Rock)

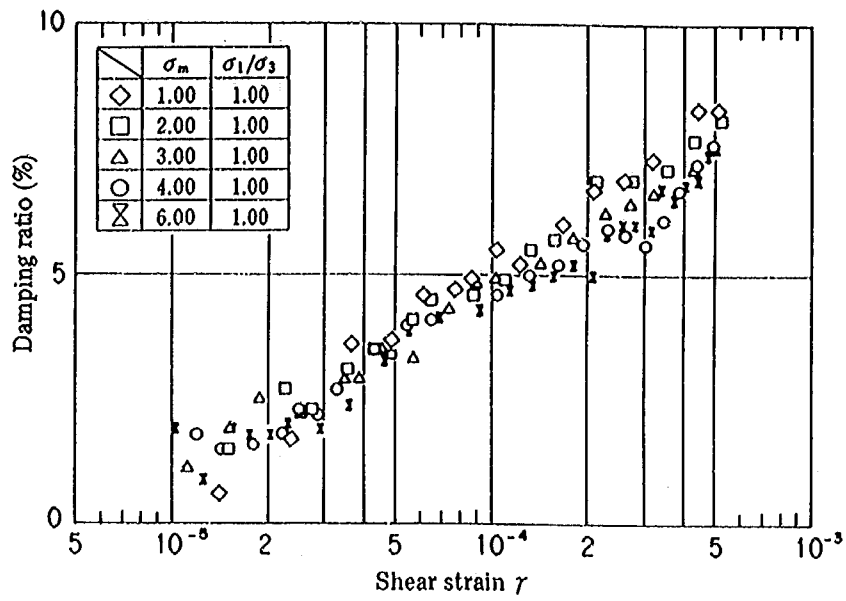


Fig.-4.3.7 Damping ratio $\sim$ Shear strain (Filter)

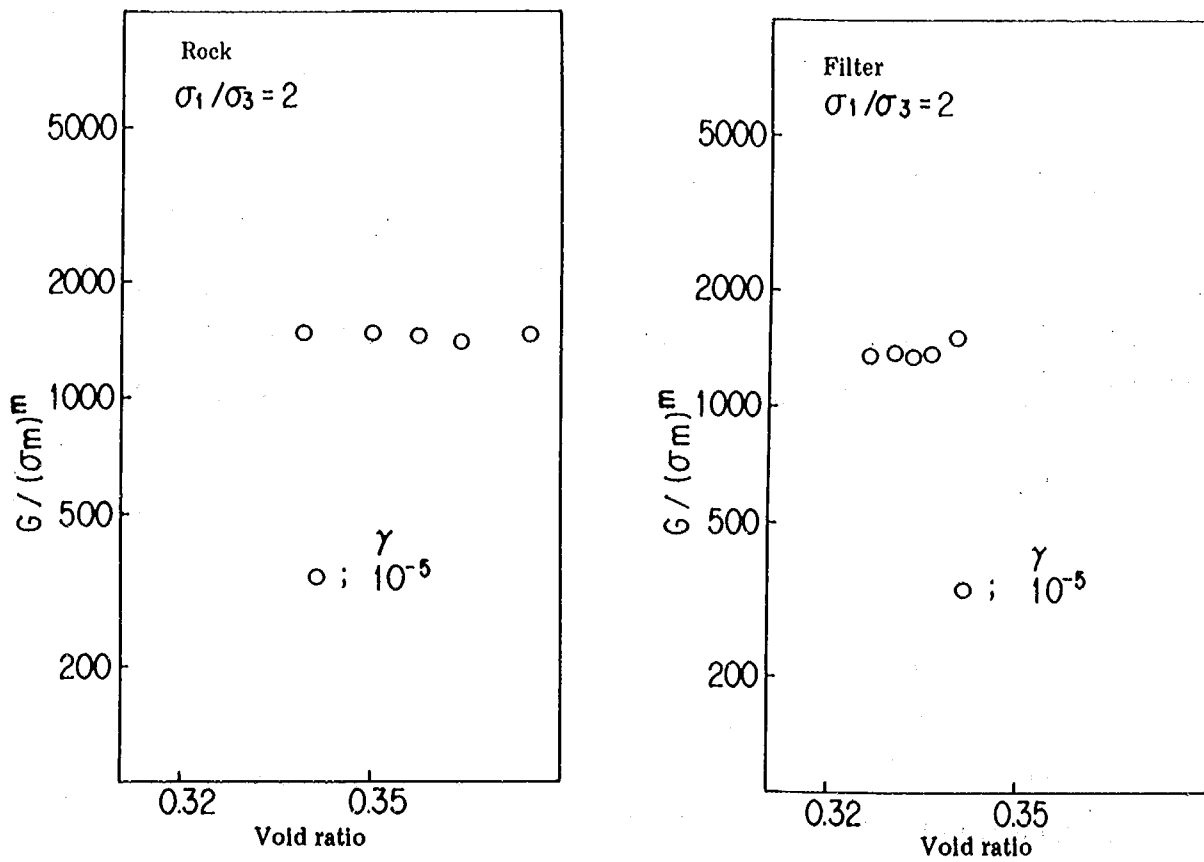


Fig.-4.3.8 $G/(\sigma_m)^m \sim$ Void ratio

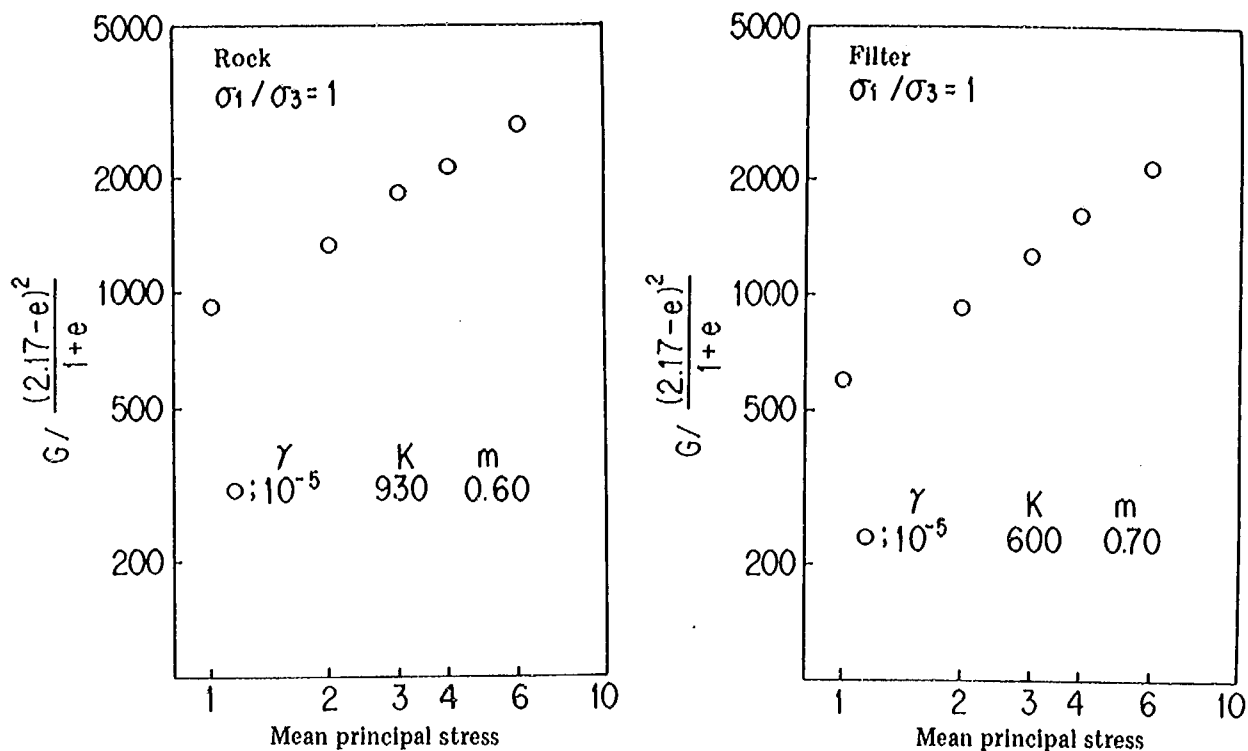


Fig.-4.3.9 $G / \frac{(2.17-e)^2}{1+e} \sim$ Mean principal stress

4.3.2 Initial Shear Modulus

It is said that soil materials show an elastic behavior at a shear strain range of 1×10^{-6} to 1×10^{-5} . The initial shear modulus is defined as the shear modulus at 1×10^{-5} of the strain. The mean effective confining pressure σ'_m is given as a function of the depth from the ground surface. The initial shear modulus G_0 is obtained from equation (4.3) as follows,

$$\sigma'_m = \frac{1+2K}{3} \cdot \gamma' \cdot D \quad \dots\dots\dots (4.4)$$

where

K : Principal stress ratio

(0.5 after the observation at Miho Dam)

D : Depth

γ' : Unit weight (Rock 1.88 t/m³)

Filter 1.81 t/m³

Core 1.85 t/m³)

for rock

$$G_0 = 930 \cdot \frac{(2.17-e)^2}{1+e} \cdot \left(\frac{1+2K}{3} \cdot \gamma' \cdot D \right)^{0.6} \quad \dots\dots\dots (4.5)$$

for filter

$$G_o = 600 \cdot \frac{(2.17 - e)^2}{1 + e} \cdot \left(\frac{1 + 2K}{3} \cdot \gamma' \cdot D \right)^{0.7} \quad \dots\dots\dots (4.6)$$

Converting the shear moduli into s-wave velocity, the relation of the s-wave velocity to the depth D is shown in Fig.-4.3.10 with the plotting of seismic prospecting at Miho Dam. The s-wave velocity of rock material is about 20 percent faster than that of filter material, and faster than the observed record at Miho Dam in a depth of 10 to 40 meters.

In following dynamic F.E.M. analysis, the initial shear modulus of each element is obtained so as to correspond to the depth between the slope surface and the center of gravity of an element. As for the core material, laboratory testing is not conducted, so the initial shear modulus is expressed by equation (4.6) for filter material.

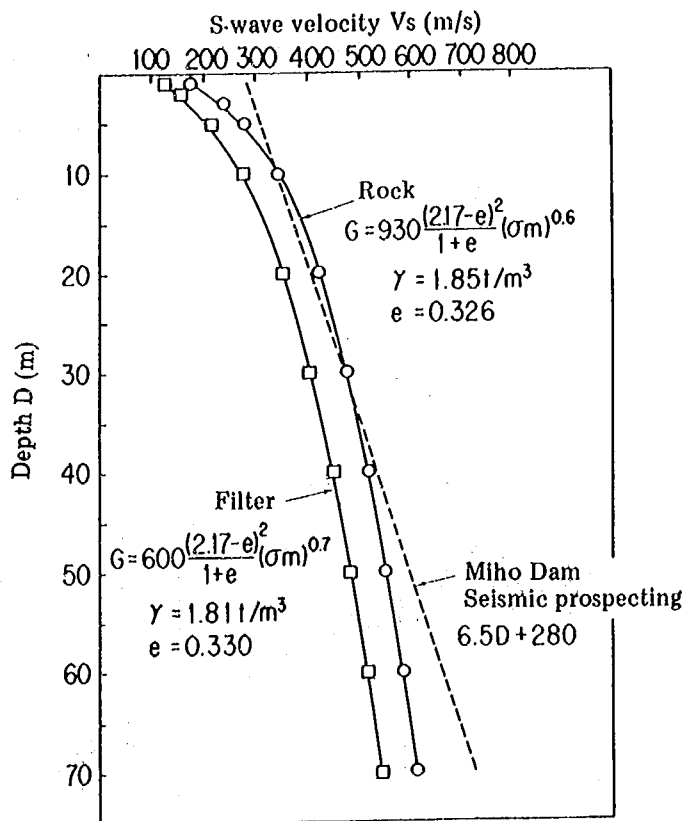


Fig.-4.3.10 S wave velocity distribution
(Shichigashuku Dam)

4.3.3 Strain Dependent Curve

Figs.-4.3.11 and 4.3.12 show the strain dependent curves of shear modulus normalized by $G\gamma_{10^5}$. The inclination of the decrease of normalized shear modulus according to the

increase in shear strain coincides with that of filter material, regardless of the level of confining pressure.

The decrease of normalized shear modulus begins at the smaller shear strain compared with filter material.

The strain dependent curves of shear moduli and damping ratios used in this analysis are illustrated in Fig.-4.3.14. As for the core material, triaxial testing is not done, so the results of the dynamic triaxial test by Obokata et al. is taken for the strain dependent curve of the core material. The shear moduli of the core coincide with those of the filter materials, and the damping ratio of the core is smallest among the core, filter, and rock materials.

The damping of the material itself is obtained from the hysteretic damping of the cyclic triaxial testing, and another dissipation damping at the boundary between the dam and foundation is added to this hysteretic damping. The dissipation damping is obtained by the theory of standing wave reflection in two layers. After Kanai, the dissipation damping between two layers is given by equation (4.7), when the excitation frequency is resonant.

$$h = \frac{1}{\pi} \cdot l_n \cdot \left| \frac{1 + \alpha}{1 - \alpha} \right| \quad \dots (4.7)$$

$$\alpha = \frac{\rho' \cdot G'}{\rho \cdot G} = \frac{\rho' \cdot \frac{1}{g} \cdot \rho' \cdot V_s'^2}{\rho \cdot \frac{1}{g} \cdot \rho \cdot V_s^2}$$

$$= \frac{\rho' \cdot V_s'}{\rho \cdot V_s}$$

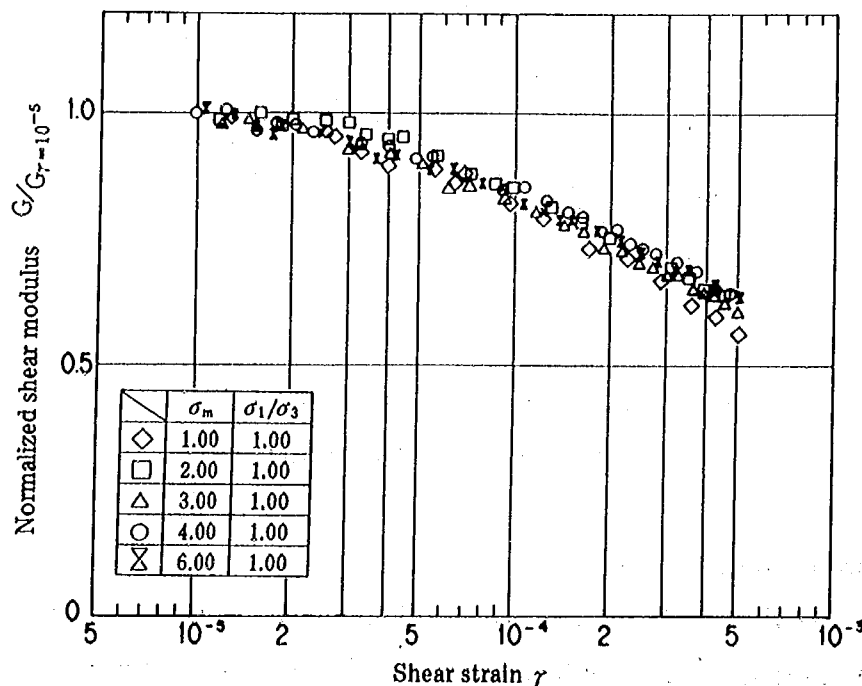


Fig.-4.3.11 Normalized shear modulus ~ Shear strain (Rock)

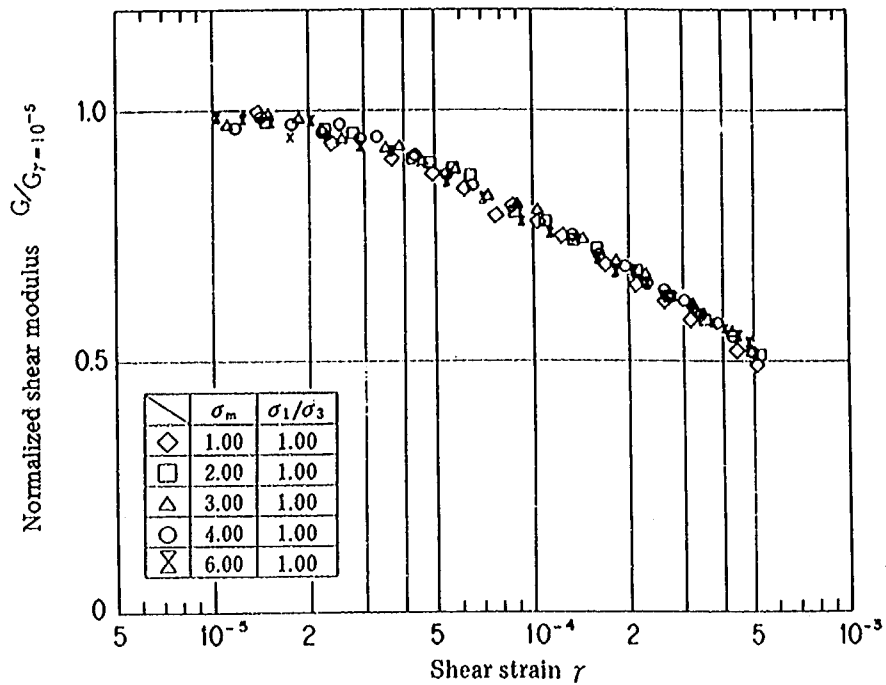


Fig. 4.3.12 Normalized shear modulus ~ Shear strain (Filter)

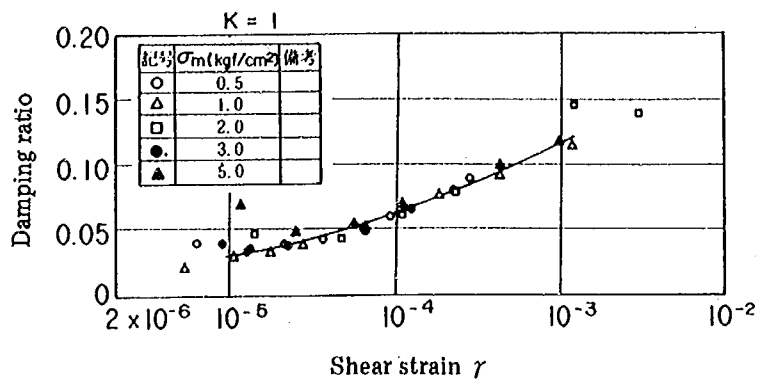


Fig. 4.3.13 Damping ratio ~ Shear strain (Core)

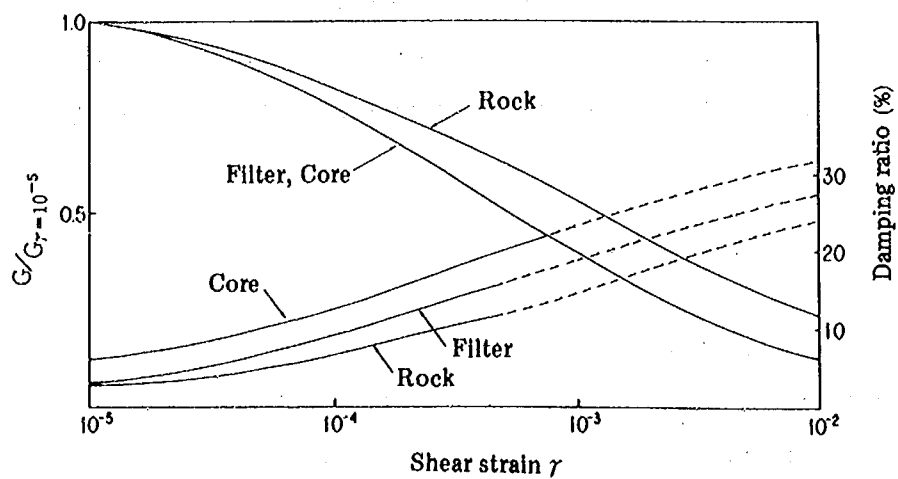


Fig. 4.3.14 Normalized shear modulus, Damping ratio ~ Shear strain

where

α : Impedance ratio

ρ' : Density of upper layer (gf/cm³)

G' : Shear modulus of upper layer (kgf/cm²):

ρ : Density of lower layer (gf/cm³)

G : Shear modulus of lower layer (kgf/cm²)

In the observation of a rockfill dam, the mean s-wave velocity of the dam and foundation are 0.5km/s and 2.0km/s, respectively, and the density of the dam and foundation are 2.3gf/cm³ and 2.5gf/cm³, respectively. Therefore, the impedance ratio α is 0.23, and the dissipation damping h becomes about 15 percent. As this dissipation damping at the boundary is not taken into account in the computer program, this 15 percent damping is added to the internal hysteretic damping.

4.4 Analysis Method

A two-dimensional dynamic response analysis was conducted using the computer program QUAD-4. The procedure of this program adopts the equivalent linear method for the nonlinear properties of fill materials, integrating the following equation directly;

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = -[M]\{\ddot{y}_0\} \quad \dots (4.8)$$

where

$[M]$: Mass matrix

$[C]$: Damping matrix

$[K]$: Stiffness matrix

$\{x\}$: Relative displacement vector of the dam

$\{y\}$: Displacement vector of the foundation

In equation (4.8) the response results are greatly influenced by the definition of the matrix $[C]$. The computer program QUAD-4 adopts Rayleigh damping for $[C]$ as follows,

$$[C] = \sum_q [c]_q$$

$$[c]_q = h_q \omega_1 [m]_q + \frac{h_q}{\omega_1} [k]_q \quad \dots (4.9)$$

where q denotes the element number, and h_q , $[c]_q$, $[m]_q$, and $[k]_q$ are the damping ratio, damping matrix, mass matrix, and stiffness matrix of the q element. Assuming that h_q is constant for all elements,

$$[C] = h\omega_1[M] + \frac{h}{\omega_1}[K] \quad \dots (4.10)$$

where

h : Damping ratio

ω_1 : Circular frequency of the first mode

Defining $[\phi]$ to be the modal matrix of the system, equation (4.10) can be transformed as follows.

$$[\phi]^T[C][\phi] = h\omega_1 \begin{bmatrix} 1 & 0 & \dots & 0 \\ 0 & 1 & & 0 \\ \vdots & & \ddots & \\ 0 & 0 & 0 & 1 \end{bmatrix} + \frac{h}{\omega_1} \begin{bmatrix} \omega_1^2 & 0 & \dots & 0 \\ 0 & \omega_2^2 & & \\ \vdots & & \ddots & \\ 0 & \dots & 0 & \omega_n^2 \end{bmatrix} \quad \dots (4.11)$$

where

$[\phi]^T$: Transformed matrix of $[\phi]$

Then, the equation of motion for the i -th mode in generalized coordinates is given by the following equation.

$$\ddot{X} + 2 \cdot h \cdot \frac{1}{2} \left(\frac{\omega_1}{\omega_i} + \frac{\omega_i}{\omega_1} \right) \omega_i \dot{X} + \omega_i^2 X = -\beta_i \ddot{y}_0 \quad \dots (4.12)$$

where

ω_i : Circular frequency of i -th mode

β_i : Participation factor of the i -th mode

X : Displacement in generalized coordinates

In equation (4.12), the damping ratio for the 1st mode is h , but it is $\frac{h}{2} \left(\frac{\omega_1}{\omega_i} + \frac{\omega_i}{\omega_1} \right)$ for the i -th mode, so that the damping ratio increases for a higher mode. No frequency dependency of the damping ratio is obtained for rockfill materials from the laboratory tests. The computer program FLUSH, for example, adopts non-viscous damping using complex numbers, and consequently in this program the damping does not depend on the frequency. However, the integration of the equation can be possible only in the frequency domain when the damping is expressed by complex numbers, so that non-viscous damping represented with complex numbers cannot be used for the computer program QUAD-4, in which integration is carried out in a time domain.

Thus, the damping defined by equation (4.10) is not suitable for the damping characteristics of rockfill materials, so that the equation was modified as follows:

$$[C] = (1 + \alpha) h \omega_1 [M] + (1 - \alpha) \frac{h}{\omega_1} [K] \quad \dots (4.13)$$

where, α is constant value ($-1.0 \leq \alpha \leq 1.0$)

In this case, the equation of motion for the i-th mode is

$$\ddot{X} + 2h \frac{1}{2} \left\{ \frac{\omega_1}{\omega_i} (1+\alpha) + \frac{\omega_i}{\omega_1} (1-\alpha) \right\} \omega_i \dot{X} + \omega_i^2 X = -\beta_i \ddot{y}_0 \quad \dots (4.14)$$

the rockfill material shows no frequency dependency for damping, so that the value 0.6 was chosen for α in the following computation.

4.5 Results of Analysis

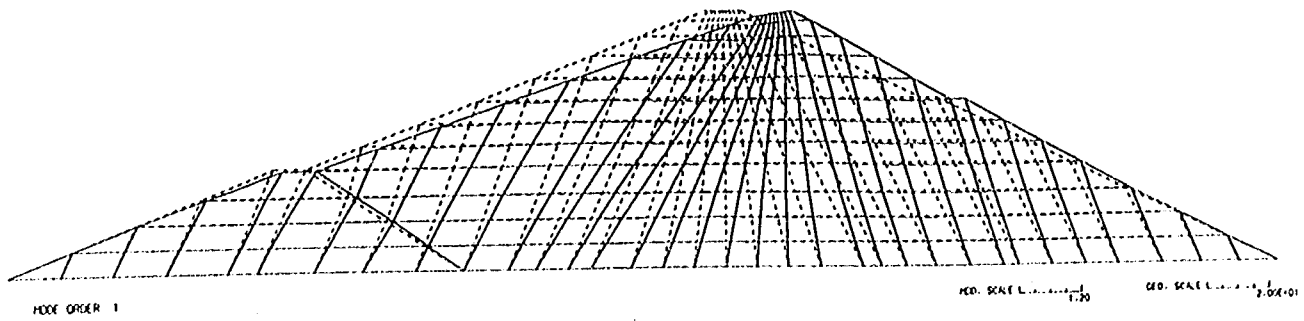
4.5.1 Frequency Response Characteristic

The vibration modes of a rockfill dam body are illustrated in Fig.-4.5.1. The first, second, and third natural frequencies are 1.66Hz, 2.54Hz, and 3.00Hz, respectively. The vibration of the dam body amplifies in the direction of the up-down stream at the first mode, and in the vertical direction at the second motion.

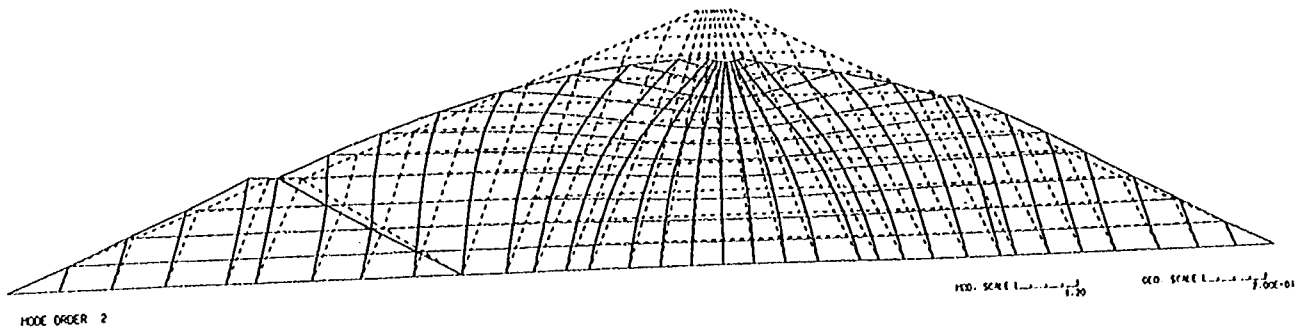
At the frequency response function of the crest, the peak value of the response amplitude at the first natural frequency is obvious for the horizontal vibration, but the value at the third natural frequency is not clear. However, the peak value of the response amplitude at the second natural frequency is recognized for the response function of the vertical direction, and it is 3 times and smaller than the horizontal one (see Fig.-4.5.2).

4.5.2 Distributions of Shear Modulus and Shear Strain in a Dam Body

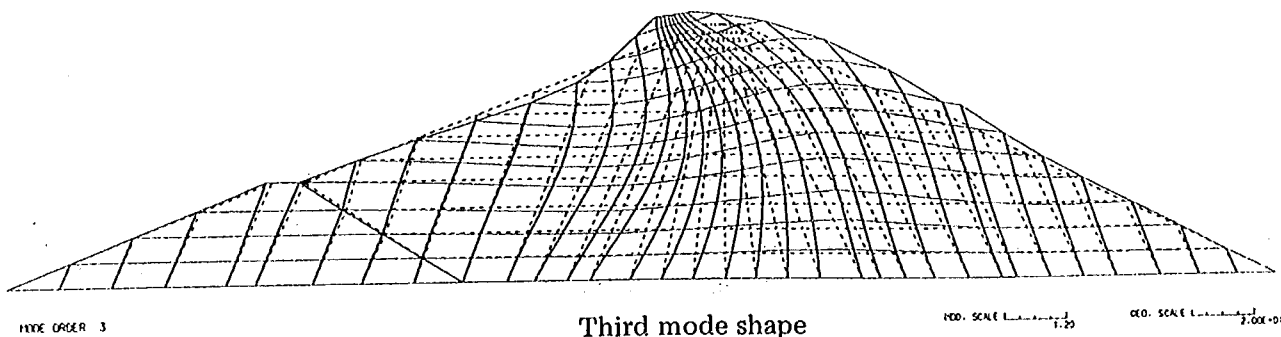
The initial shear modulus at the central core zone is smaller than that of the rock zone, as shown in Fig.-4.5.3 (1). The shear modulus at the final iteration of the computation decreases strongly for Case-C and becomes half the initial value. However the maximum response acceleration of Case-A is two times larger than that of Case-C, and the decrease of the shear modulus of Case-A is smaller than that of Case-B because of the difference of the predominant frequencies of input earthquake motion to each other. Eventually a predominant frequency of 1.9Hz for Case-B is very close to the natural frequency of the dam body, but 4.2Hz for Case-A is far from a natural frequency.



First mode shape



Second mode shape



Third mode shape

Fig.-4.5.1 Mode shape

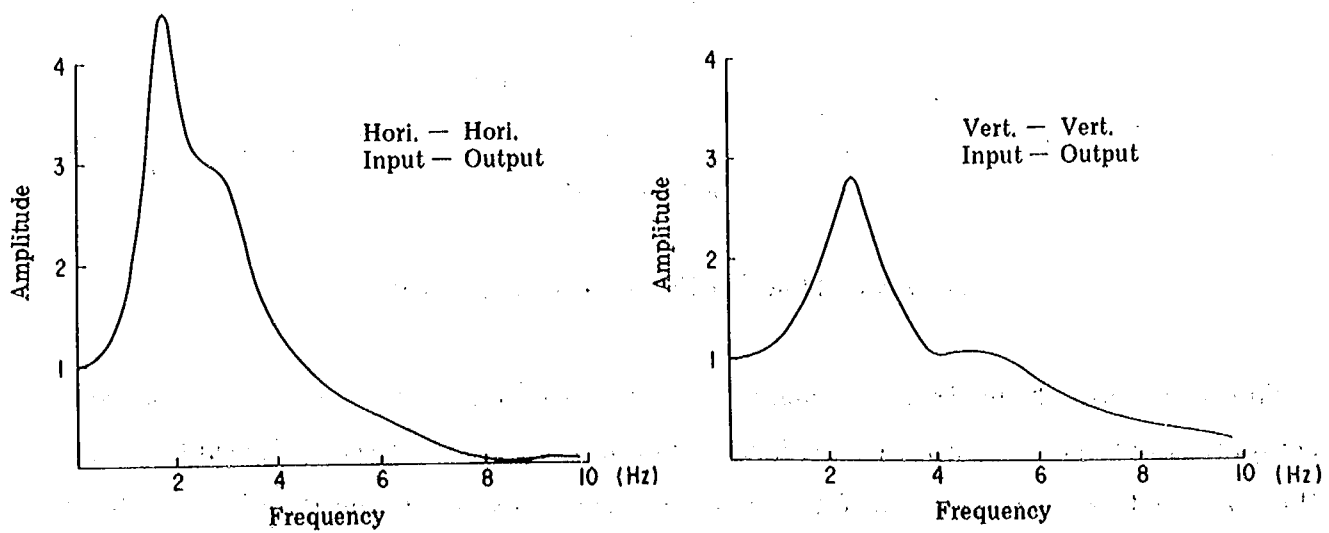


Fig.-4.5.2 Frequency response function at crest

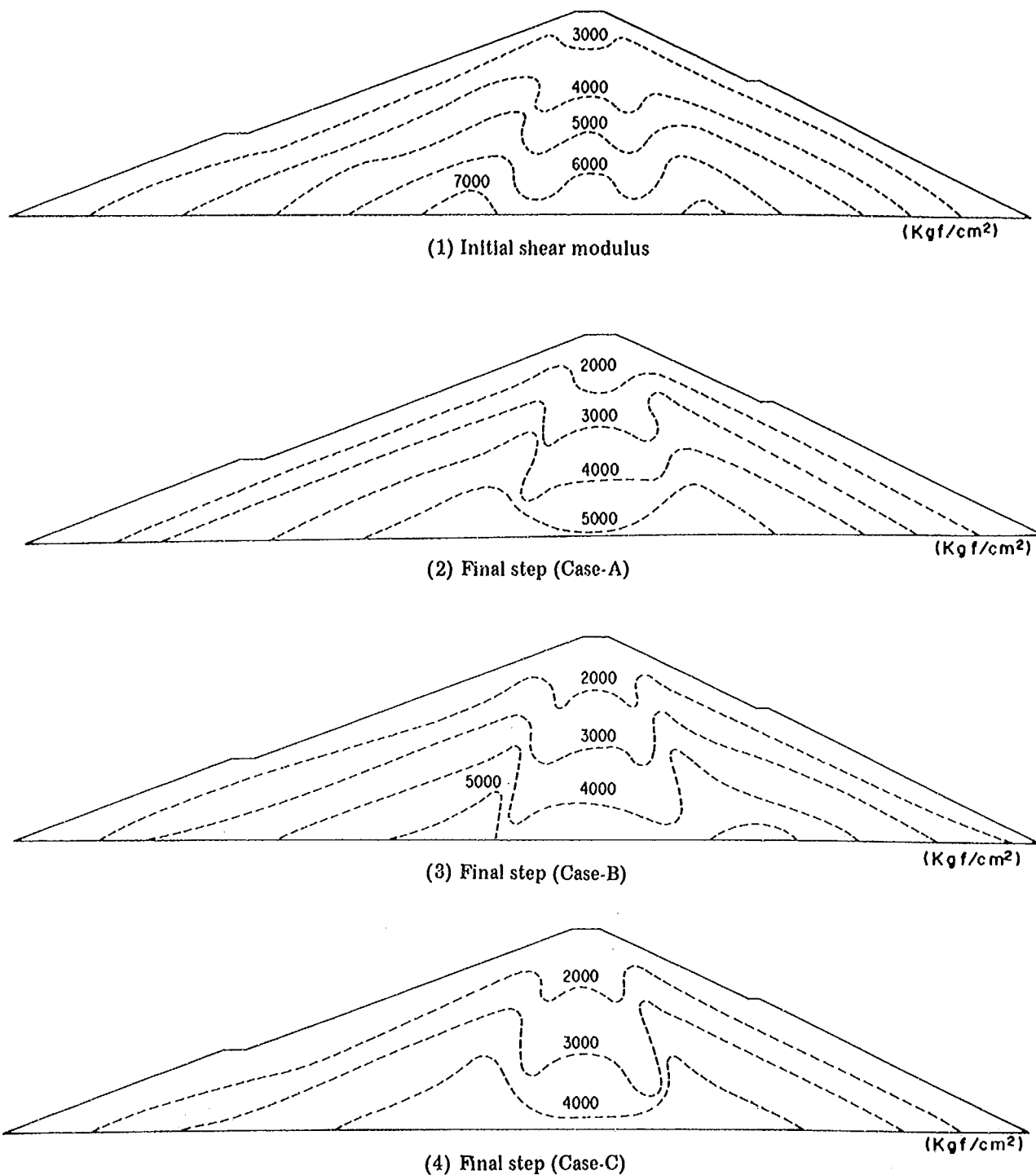


Fig.-4.5.3 Distribution of shear modulus

The maximum shear strain of each element ranges from 1×10^{-4} to 3.5×10^{-4} for each computational case. A comparatively larger value of shear strain is recognized at the crest, the slope surface, and the circumference in Case-A and Case-B. On the other hand, a shear strain of 3×10^{-4} to 3.5×10^{-4} occurs from the upper level to the lower level of the central part

of the dam body for Case-C, and especially in the circumference of the filter zone. (see Fig.-4.5.4)

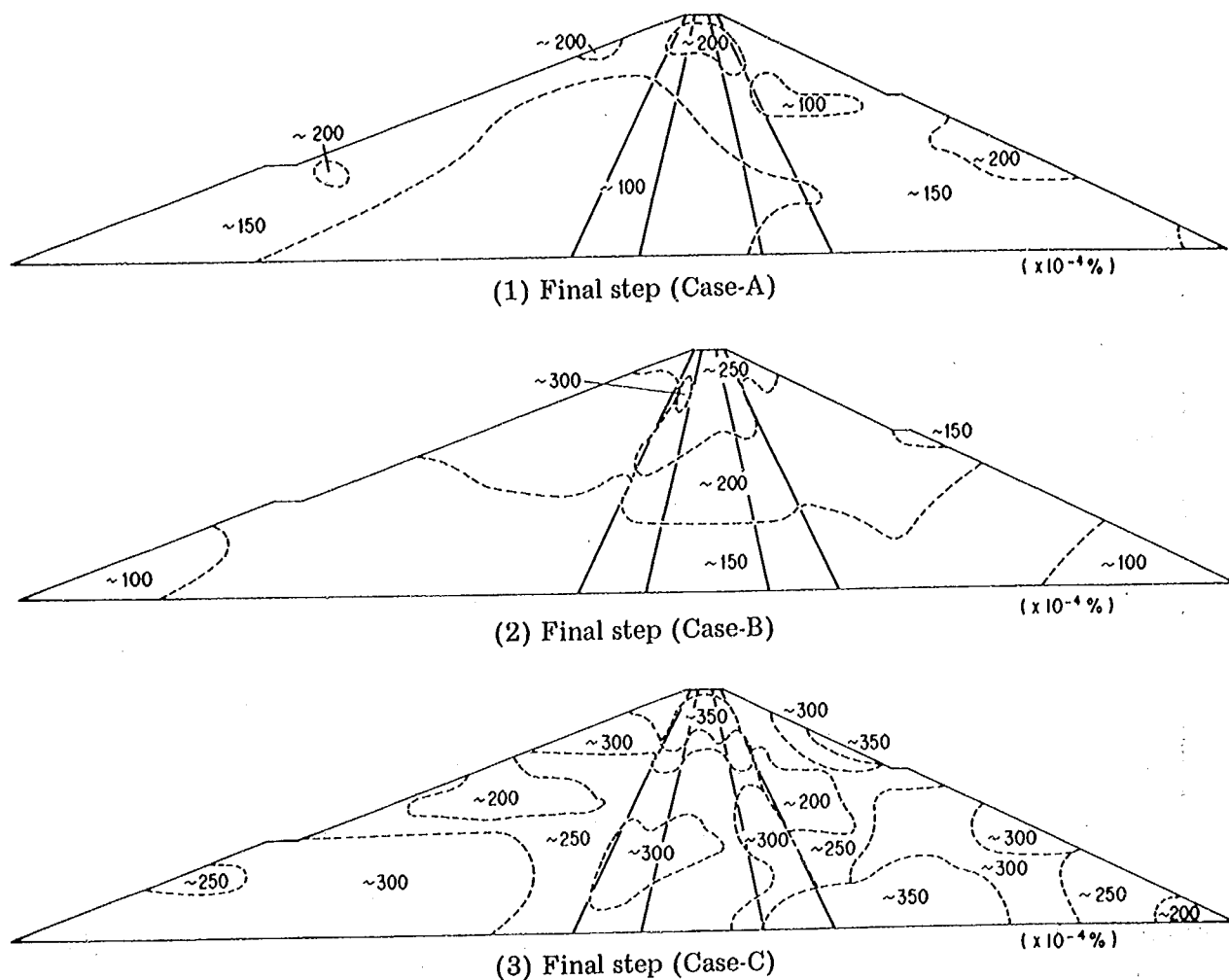


Fig.-4.5.4 Maximum shear strain

4.5.3 Response Accelerations and Response Displacements

Fig.-4.5.5 shows the nodal points of the center of a dam for response acceleration and displacement. From Table-4.5.1, the distribution of the maximum shear strain and horizontal response accelerations for Case-A are the same as Case-B. However, the vertical response acceleration of Case-A is half that of Case-B. The maximum response acceleration in the horizontal direction for Case-C is about 500 gals, and the response amplitude is 1.43 because the input maximum acceleration is 350 gals in the horizontal direction. Fig.-4.5.6 illustrates the distribution of the maximum acceleration for each computational case. The maximum acceleration occurs on the slope of both the up and down stream below the

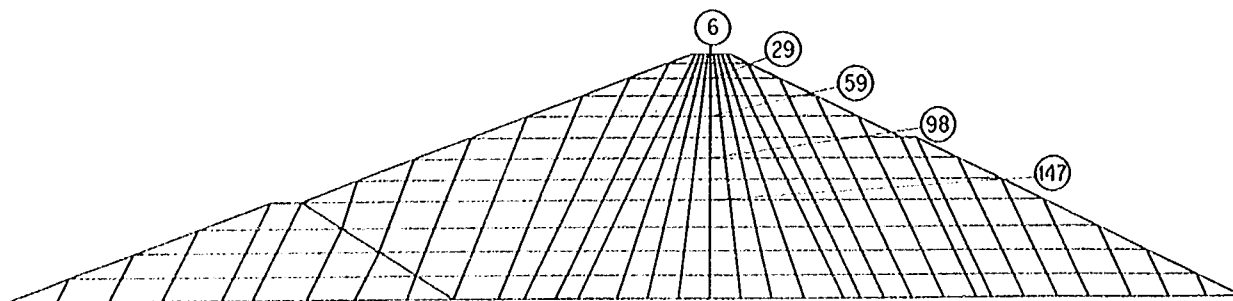


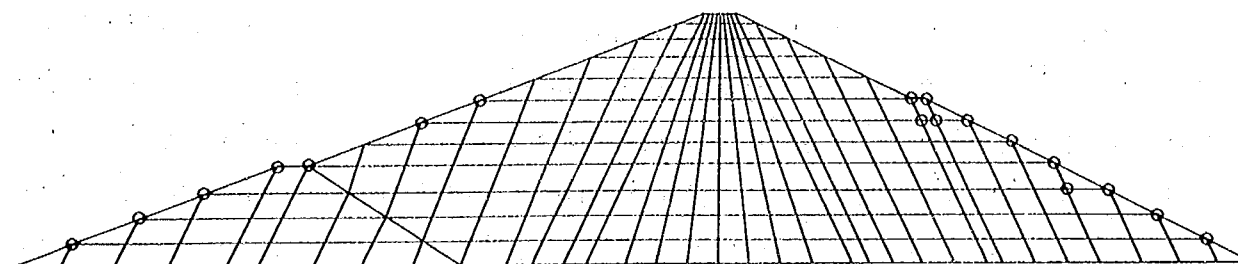
Fig.-4.5.5 Nodal point number of F.E.M. mesh

Table-4.5.1 Response acceleration and displacement

Nodal point			6		29		59		98		147	
			Up stream	Down stream	Up stream	Down stream	Up stream	Down stream	Up stream	Down stream	Up stream	Down stream
Case			Up	Down	Up	Down	Up	Down	Up	Down	Up	Down
Case-A	Acc.	Hori.	276.6 gal	-226.4	219.5	-191.5	125.6	-81.4	133.6	-115.8	171.3	-175.7
		Vert.	127.9	-79.7	94.9	-85.2	76.2	-72.2	75.1	-64.5	65.3	-62.2
	Disp.	Hori.	0.73 cm	-0.68	0.70	-0.63	0.70	-0.58	0.62	-0.55	0.47	-0.44
		Vert.	0.29	-0.29	0.26	-0.27	0.22	-0.22	0.16	-0.17	0.10	-0.16
Case-B	Acc.	Hori.	228.6	-179.5	211.1	-168.5	157.4	-115.9	124.8	-91.8	121.6	-73.6
		Vert.	52.9	-58.9	48.8	-52.5	42.3	-44.0	43.3	-41.4	44.5	-41.0
	Disp.	Hori.	1.30	-1.77	1.25	-1.71	1.05	-1.39	0.80	-1.03	0.55	-0.69
		Vert.	0.21	-0.21	0.20	-0.19	0.16	-0.15	0.12	-0.11	0.08	-0.08
Case-C	Acc.	Hori.	351.5	-488.5	287.3	-412.2	190.6	-263.4	194.8	-222.3	280.7	-317.0
		Vert.	382.5	-267.1	353.0	-245.6	309.9	-233.7	299.8	-245.1	303.0	-255.8
	Disp.	Hori.	3.06	-2.08	2.90	-2.0	2.36	-1.63	1.86	-1.32	1.33	-0.96
		Vert.	0.87	-1.28	0.82	-1.21	0.72	-1.01	0.62	-0.83	0.49	-0.63

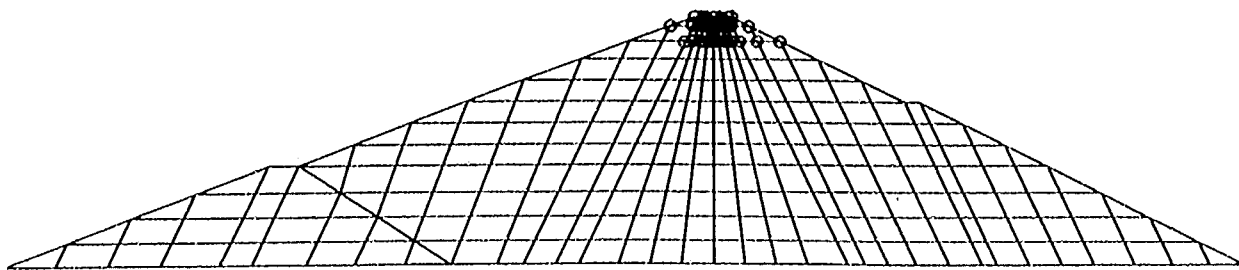
middle level for Case-A, on the circumference of the crest for Case-B, and inside the berms of both the up and down stream surface for Case-C.

The response displacements of a dam body tend to become larger at the dam crest and circumference, and are eventually small, as the maximum displacement of Case-A, Case-B, and Case-C is less than 1.0cm, 2.0cm, and 3.0cm, respectively.

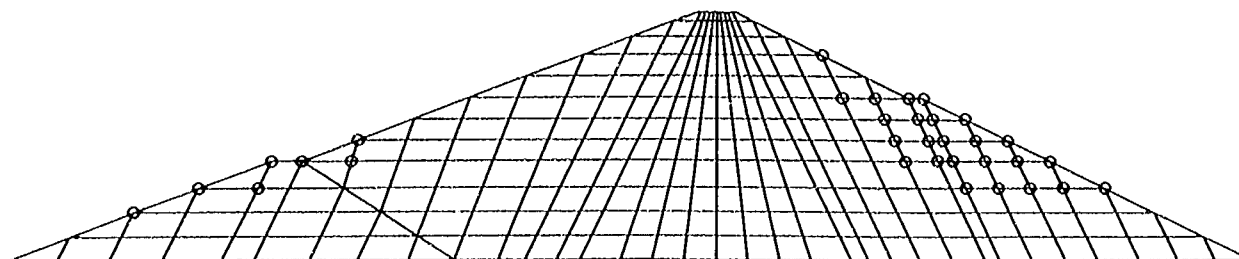


(1) 300 ~ 350gal (Case-A)

Fig.-4.5.6 Distribution of maximum response acceleration



(2) 250 ~ 300gal (Case-B)



(3) 500 ~ 550gal (Case-C)

Fig.-4.5.6 Distribution of maximum response acceleration

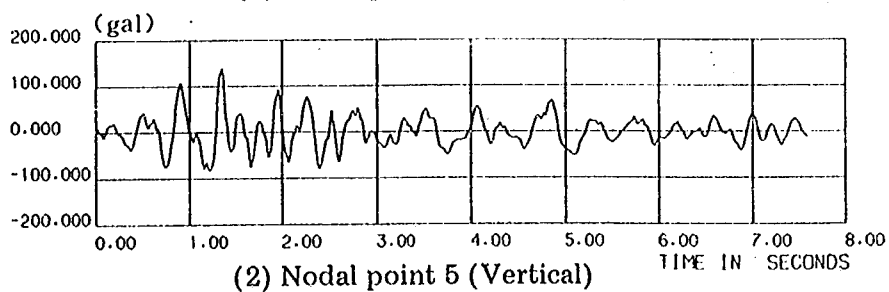
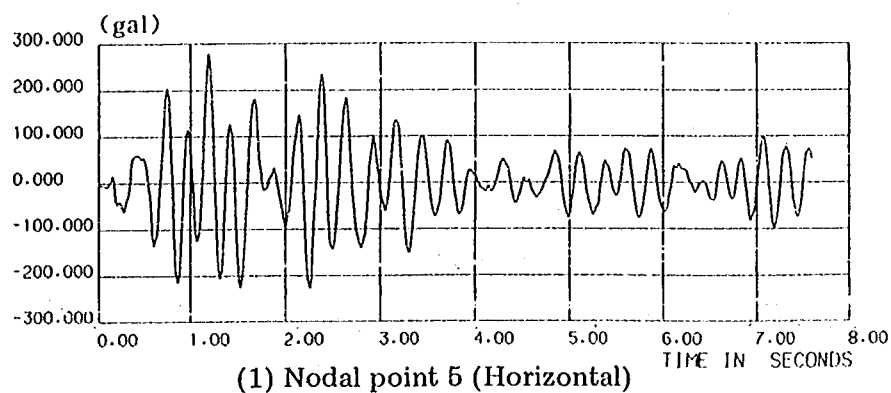


Fig.-4.5.7(a) Time-history of response acceleration at crest (Case-A)

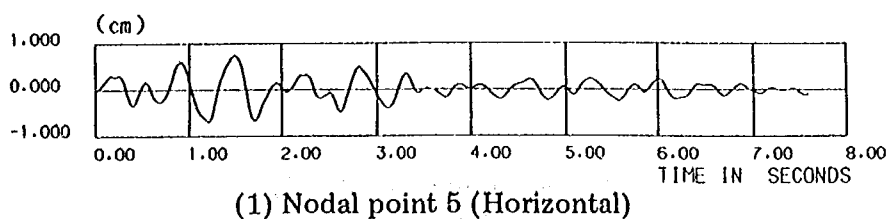


Fig.-4.5.7(b) Time-history of response displacement at crest (Case-A)

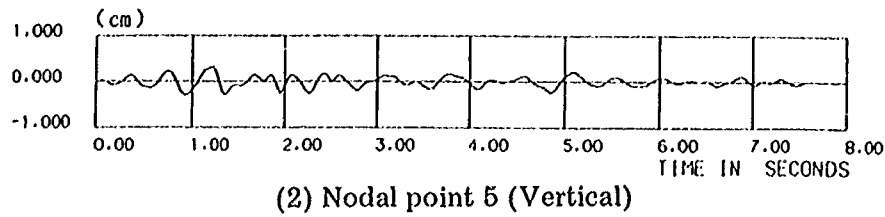


Fig.-4.5.7(b) Time-history of response displacement at crest (Case-A)

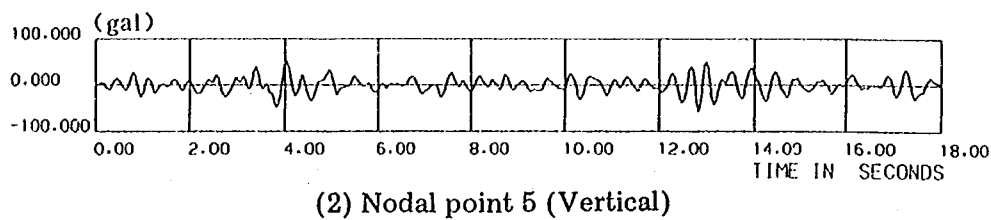
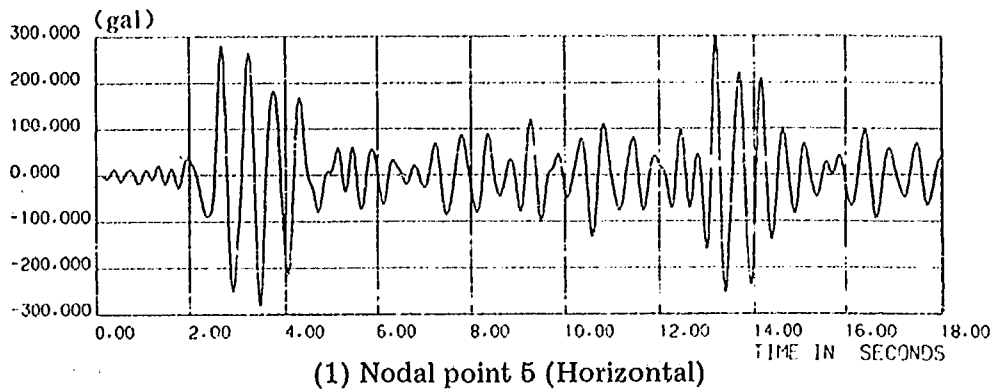


Fig.-4.5.8(a) Time-history of response acceleration at crest (Case-B)

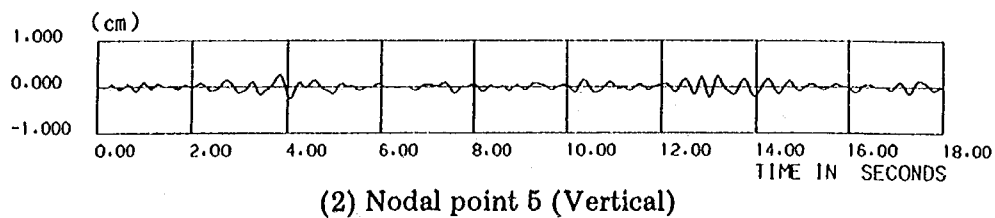
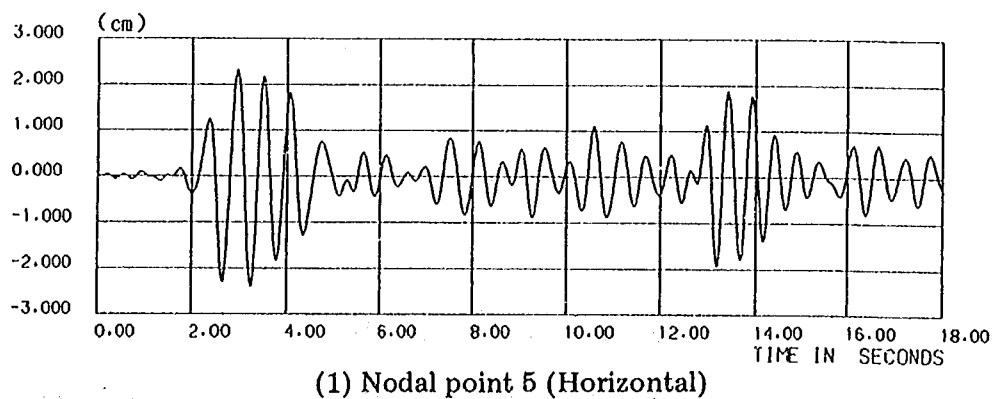


Fig.-4.5.8(b) Time-history of response displacement at crest (Case-B)

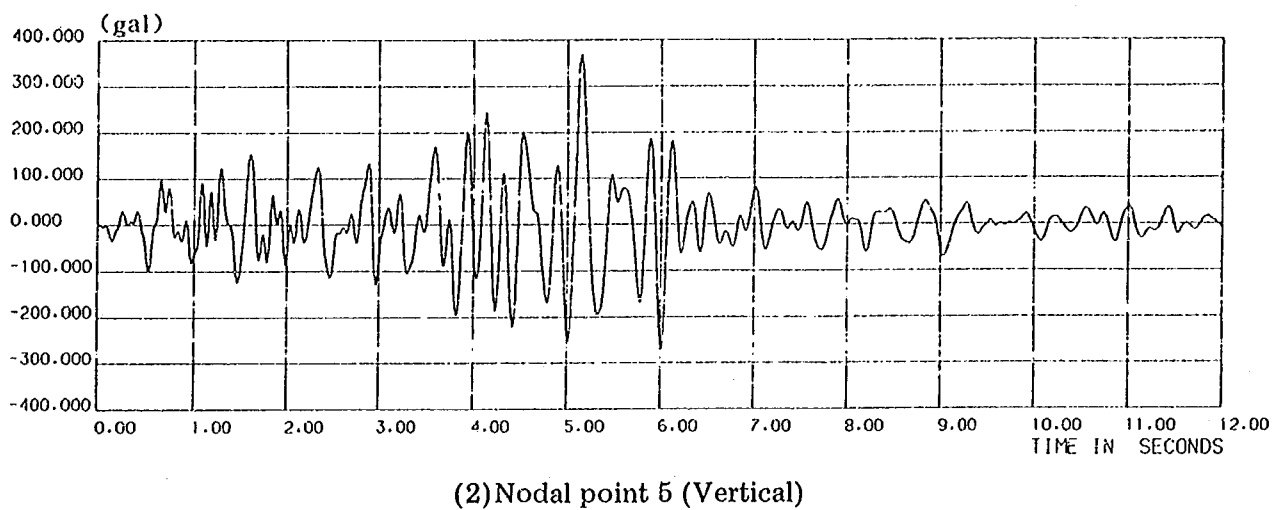
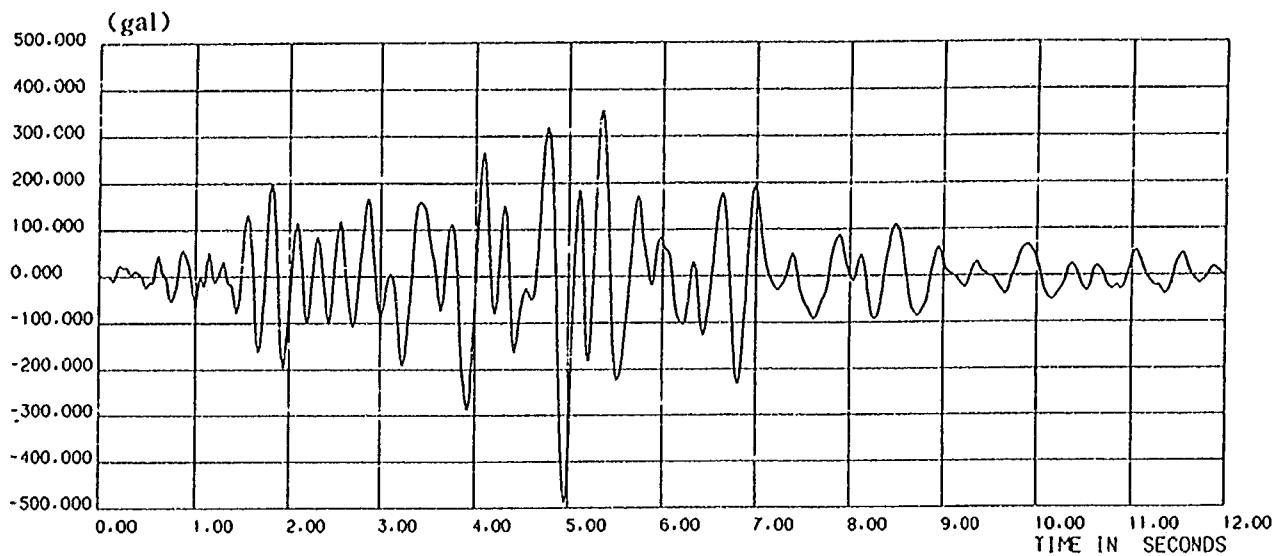


Fig.-4.5.9(a) Time-history of response acceleration at crest (Case-C)

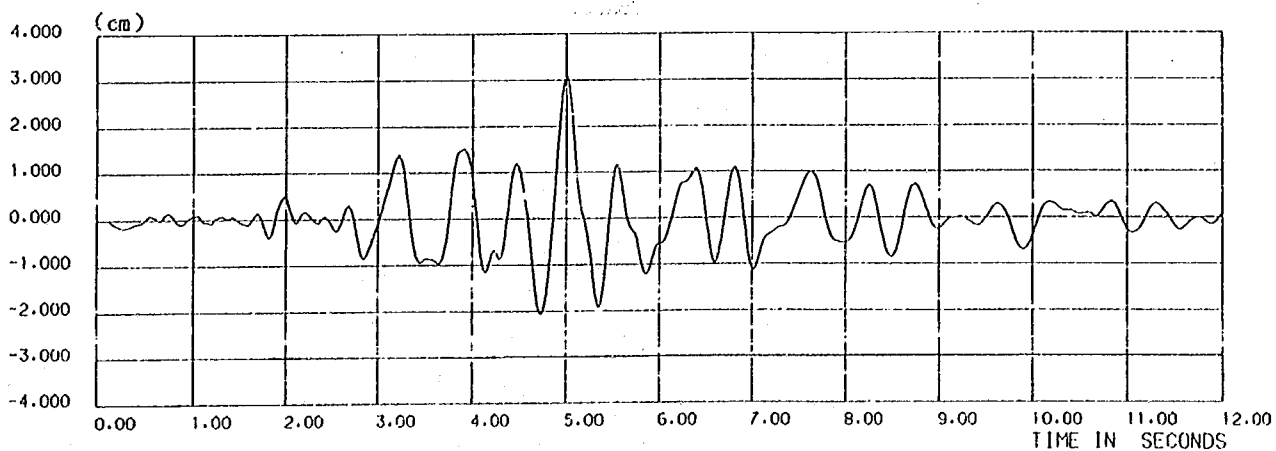


Fig.-4.5.9(b) Time-history of response displacement at crest (Case-C)

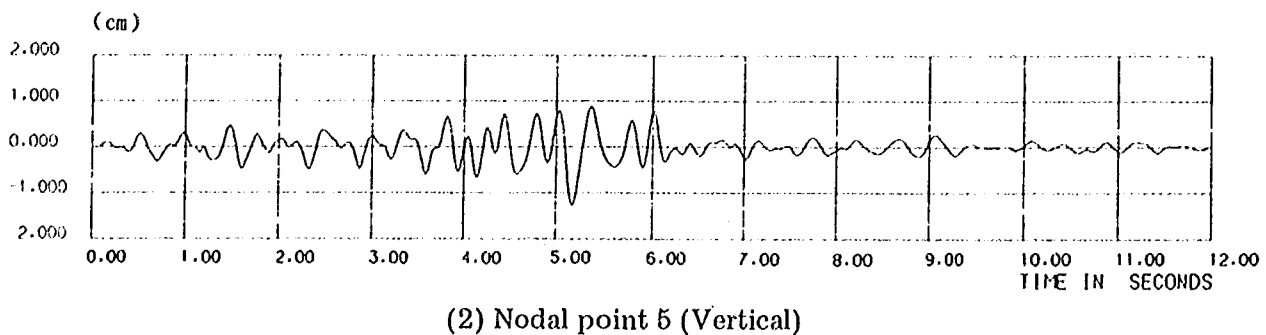


Fig.-4.5.9(b) Time-history of response displacement at crest (Case-C)

Time histories of the response accelerations and displacements at the dam crest are shown in Figs.-4.5.7 to 4.5.9. High frequency components, which are recognized at the input earthquake motion, are seemed to be faded away by the effect of characteristics of computational program QUAD-4.

4.5.4 Stresses of a Dam Body

Fig.-4.5.10 indicates the investigated elements of a dam body. The maximum dynamic stresses that occurred at each element during the computation are listed in Table-4.5.2. The maximum and minimum dynamic stresses of each element for Case-A and Case-B are less than 1.0 kgf/cm^2 , so there is no possibility of extensional stress, considering the static stress of an element. The horizontal dynamic stresses of some elements in Case-C have an inclination to exceed the static stresses slightly, however the maximum dynamic stress in the vertical direction is greatly less than the static stresses, so there is no problem about the following local stability.

Time histories of the stresses at elements 29, 31, 34, 36 and 38 are illustrated in Figs.-4.5.11 to 4.5.13 for reference, excluding the static stresses. Finally, element stresses should be expressed as follows.

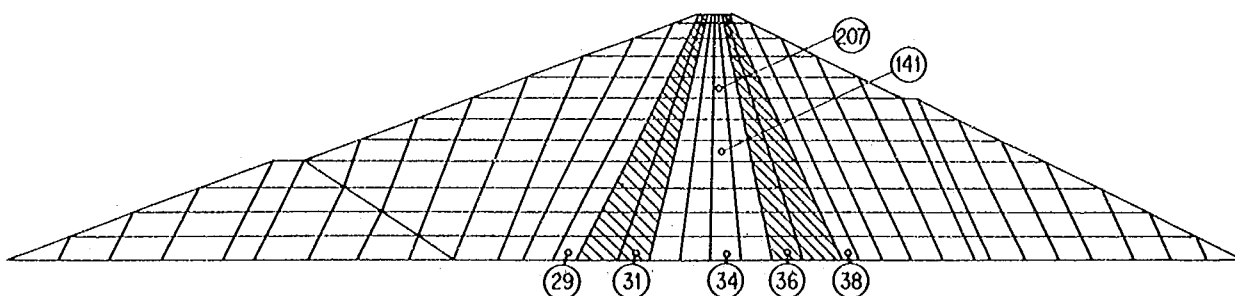


Fig.-4.5.10 Element number of F.E.M. mesh

$$\left. \begin{aligned} \sigma_x &= \sigma_{xs} + \sigma_{xd} \\ \sigma_y &= \sigma_{ys} + \sigma_{yd} \\ \tau_{xy} &= \tau_{xys} + \tau_{xyd} \end{aligned} \right\} \dots (4.15)$$

where

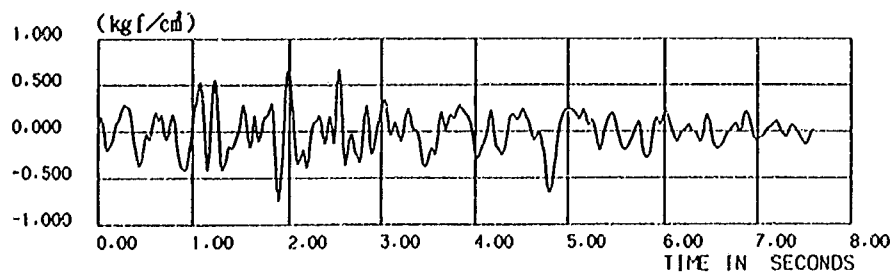
$\sigma_{xs}, \sigma_{ys}, \tau_{xys}$: Static stress

$\sigma_{xd}, \sigma_{yd}, \tau_{xyd}$: Dynamic stress

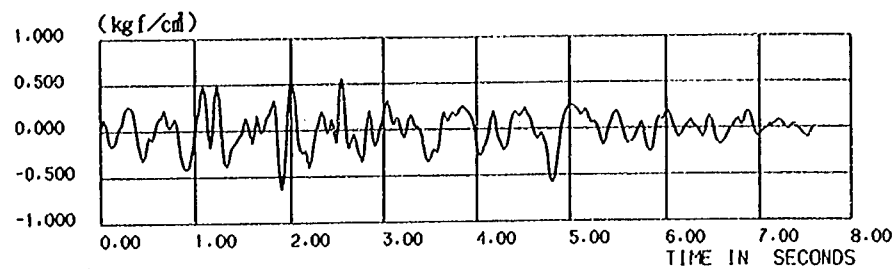
The stability element is reviewed in the following chapter by a local safety factor using the Mohr-Coulomb failure law.

Table-4.5.2 Maximum and minimum principal stress of element (Three components)

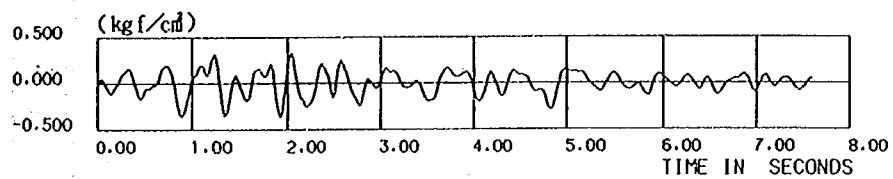
		(kgf/cm ²)						
Element number		29	31	34	36	38	141	207
Static stress	σ_{xs}	3.26	2.68	3.97	4.72	5.89	2.25	1.85
	σ_{ys}	9.37	6.93	7.39	11.63	14.67	4.07	2.92
	τ_{xys}	1.73	0.73	-1.07	-2.49	-4.71	-0.17	-0.22
Case-A	σ_{xd}	0.66	0.54	0.42	0.33	0.36	0.43	0.27
		-0.74	-0.64	-0.46	-0.36	-0.41	-0.41	-0.23
	σ_{yd}	0.93	0.81	0.76	0.73	0.78	0.44	0.29
		-1.02	-0.92	-0.82	-0.81	-0.86	-0.54	-0.33
	τ_{xyd}	0.87	0.70	0.67	0.73	1.03	0.55	0.39
		-0.98	-0.77	-0.76	-0.73	-0.96	-0.48	-0.40
Case-B	σ_{xd}	0.52	0.43	0.31	0.29	0.39	0.33	0.19
		-0.58	-0.44	-0.34	-0.30	-0.37	-0.33	-0.17
	σ_{yd}	0.78	0.59	0.56	0.59	0.81	0.34	0.20
		-0.78	-0.60	-0.59	-0.58	-0.79	-0.35	-0.20
	τ_{xyd}	0.11	0.96	0.10	0.93	0.11	0.50	0.36
		-0.11	-0.92	-0.99	-0.89	-0.10	-0.71	-0.54
Case-C	σ_{xd}	3.06	2.61	1.96	1.42	1.60	1.65	0.91
		-3.19	-2.97	-2.47	-1.98	-2.29	-2.19	-1.10
	σ_{yd}	4.24	3.68	3.26	2.66	3.40	1.84	0.98
		-4.56	-4.22	-4.14	-3.71	-5.08	-2.42	-1.33
	τ_{xyd}	2.18	1.47	1.45	1.42	2.04	1.05	0.75
		-1.44	-1.15	-1.11	-1.12	-1.63	-0.80	-0.65



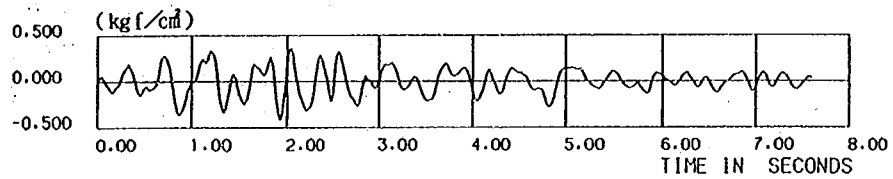
(1) Element 29 (Horizontal stress)



(2) Element 31 (Horizontal stress)



(3) Element 36 (Horizontal stress)



(4) Element 38 (Horizontal stress)

Fig.-4.5.11(a) Time-history of element stress (Case-A, Horizontal stress)

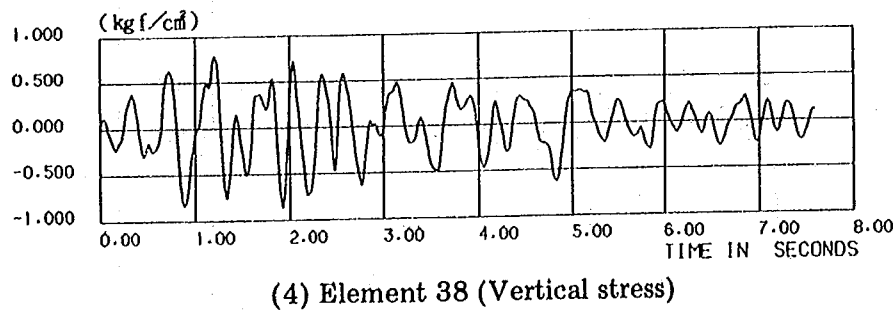
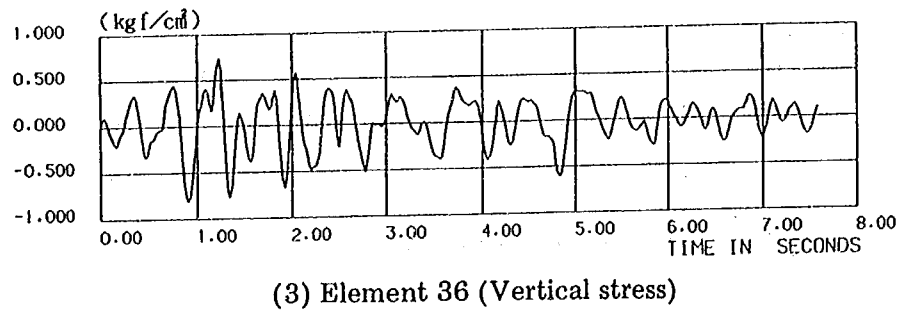
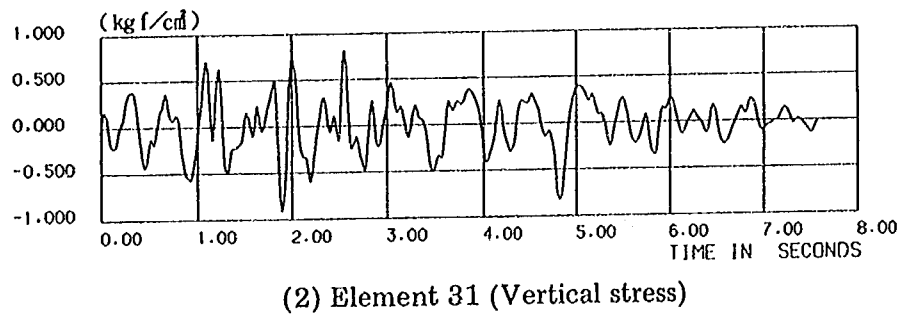
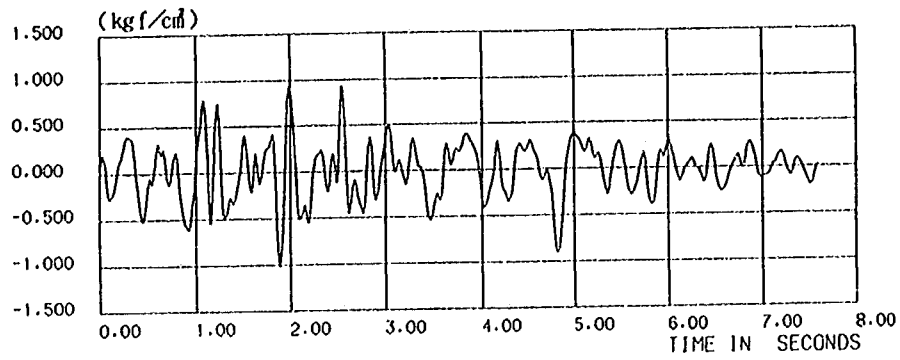
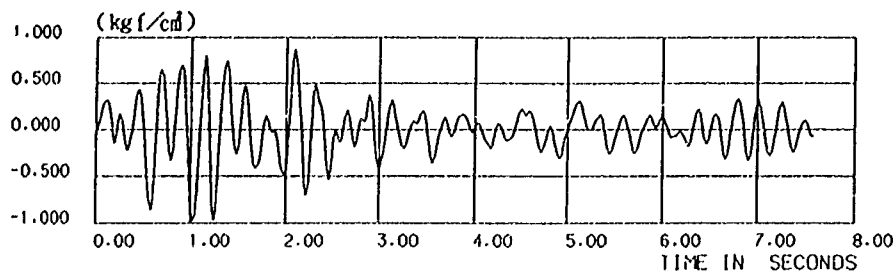
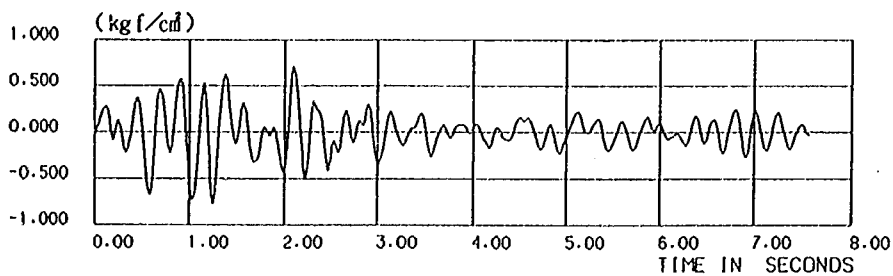


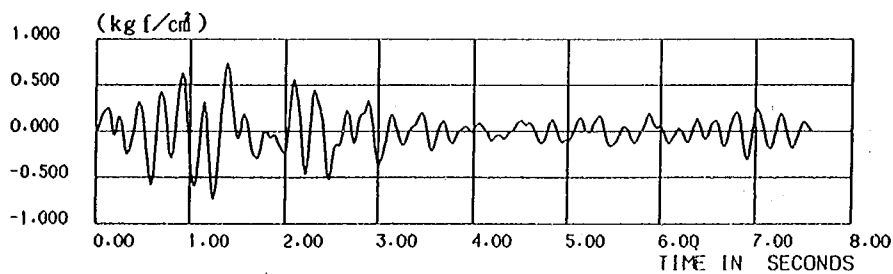
Fig.-4.5.11(b) Time-history of element stress (Case-A, Vertical stress)



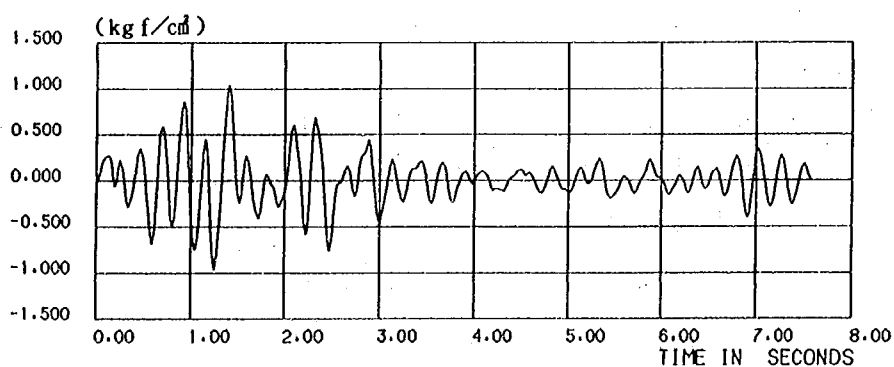
(1) Element 29 (Shear stress)



(2) Element 31 (Shear stress)



(3) Element 36 (Shear stress)



(4) Element 38 (Shear stress)

Fig.-4.5.11(c) Time-history of element stress (Case-A, Shear stress)

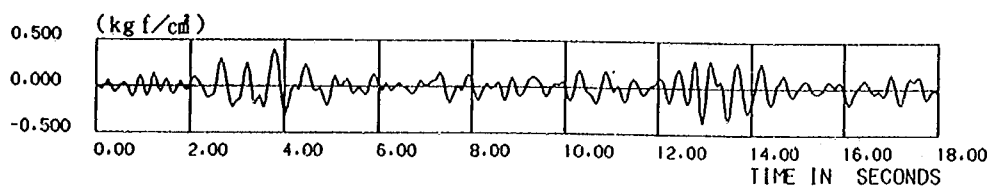
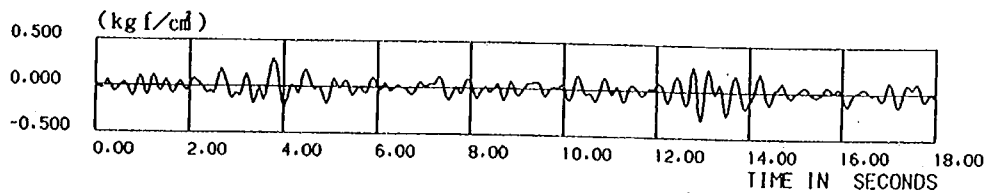
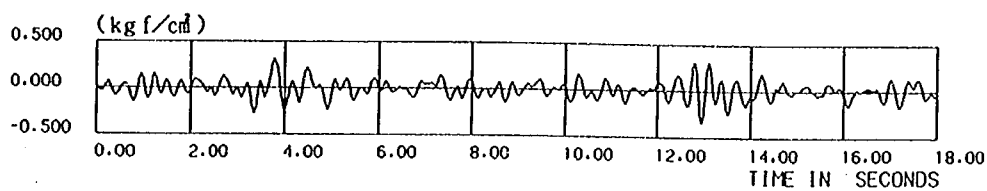
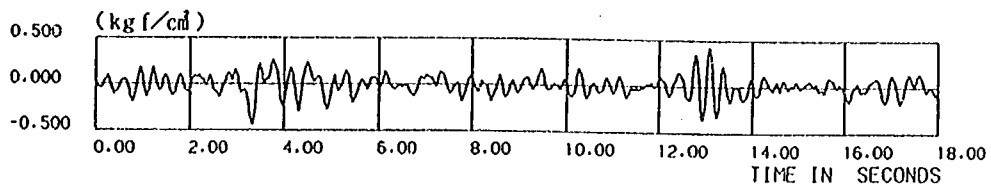
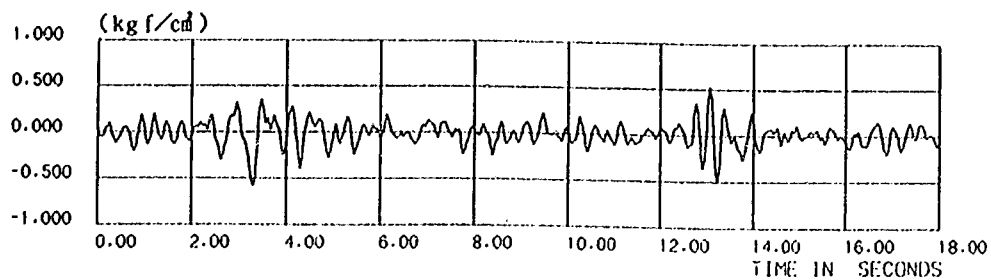
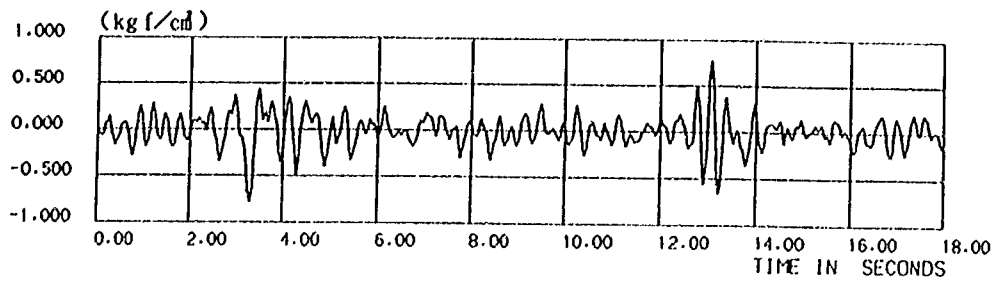
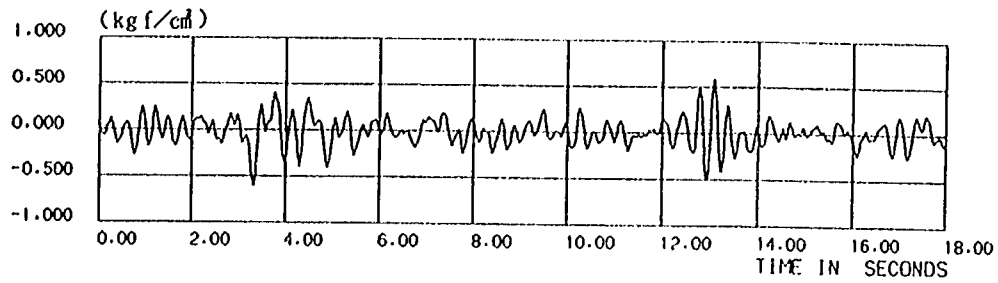


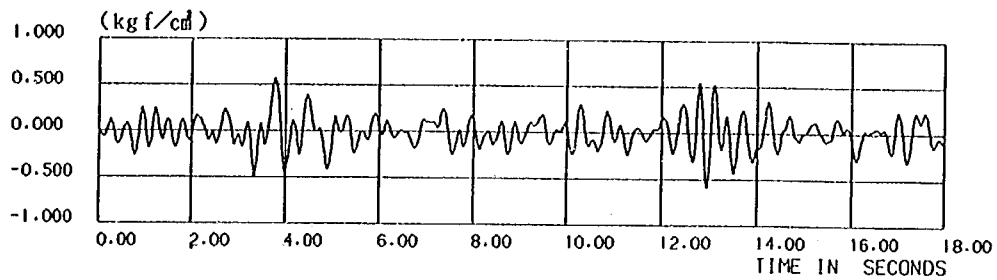
Fig.-4.5.12(a) Time-history of element stress (Case-B, Horizontal stress)



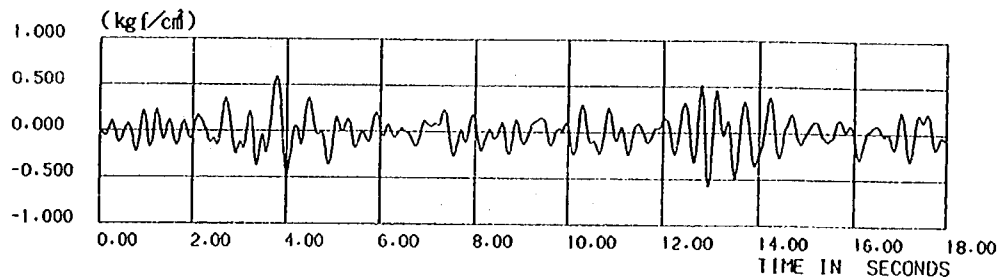
(1) Element 29 (Vertical stress)



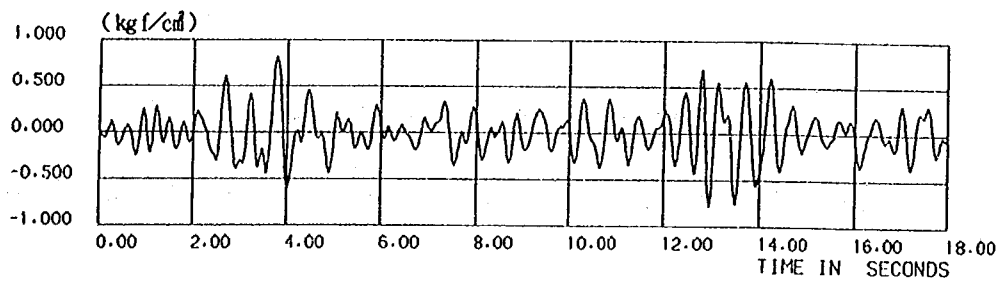
(2) Element 31 (Vertical stress)



(3) Element 34 (Vertical stress)



(4) Element 36 (Vertical stress)



(5) Element 38 (Vertical stress)

Fig.-4.5.12(b) Time-history of element stress (Case-B, Vertical stress)

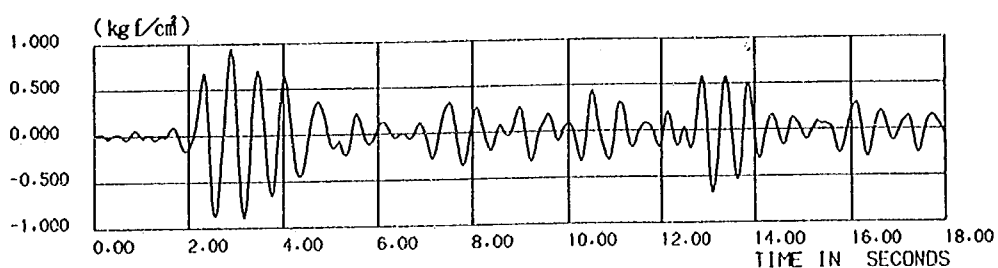
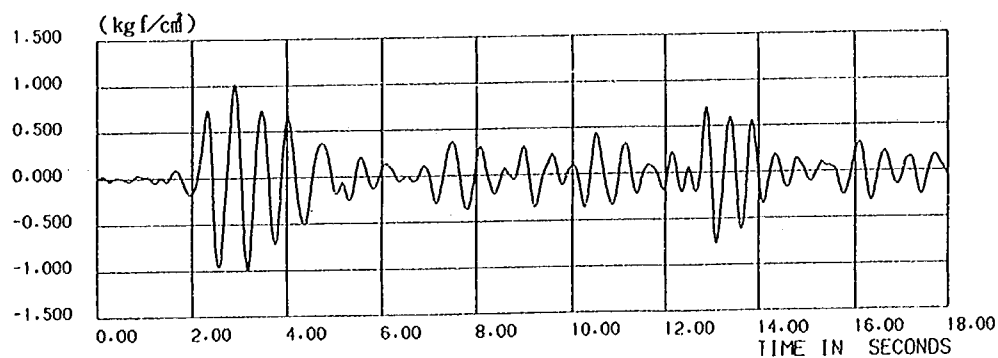
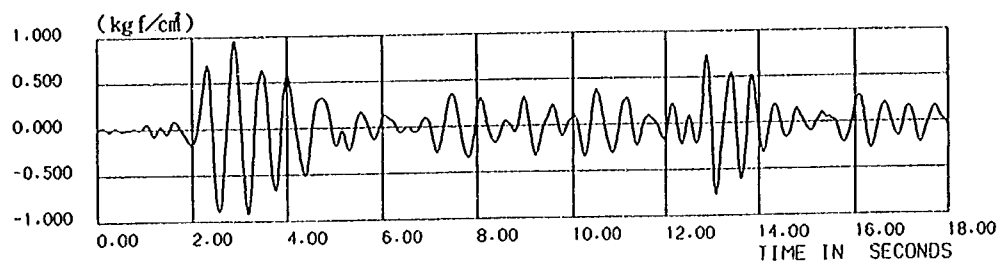
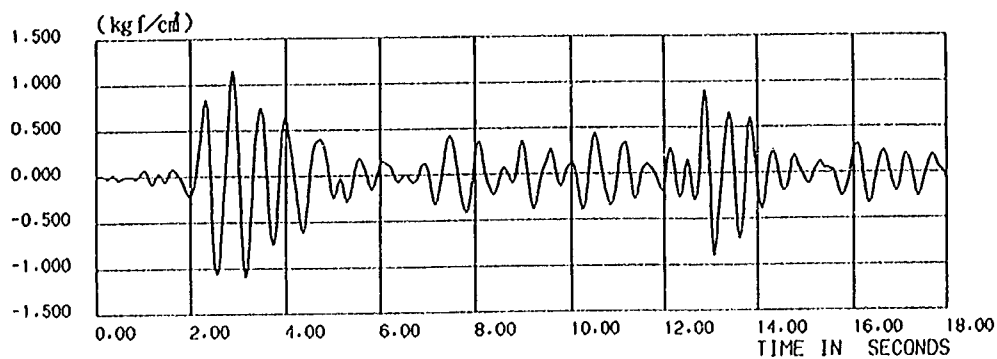
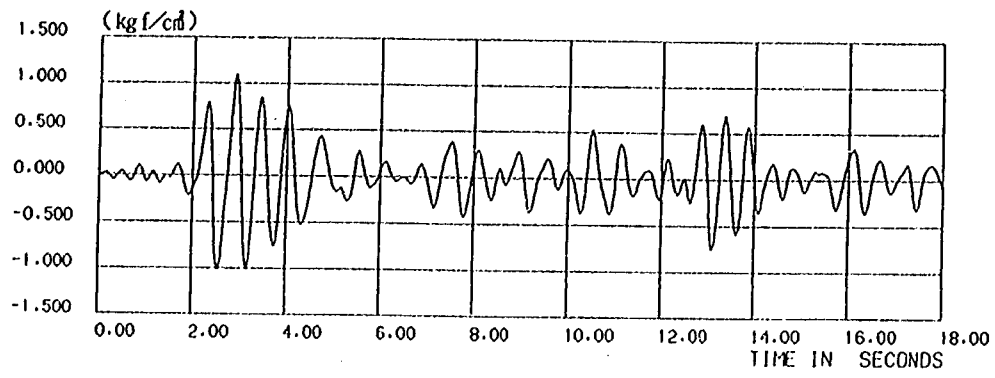
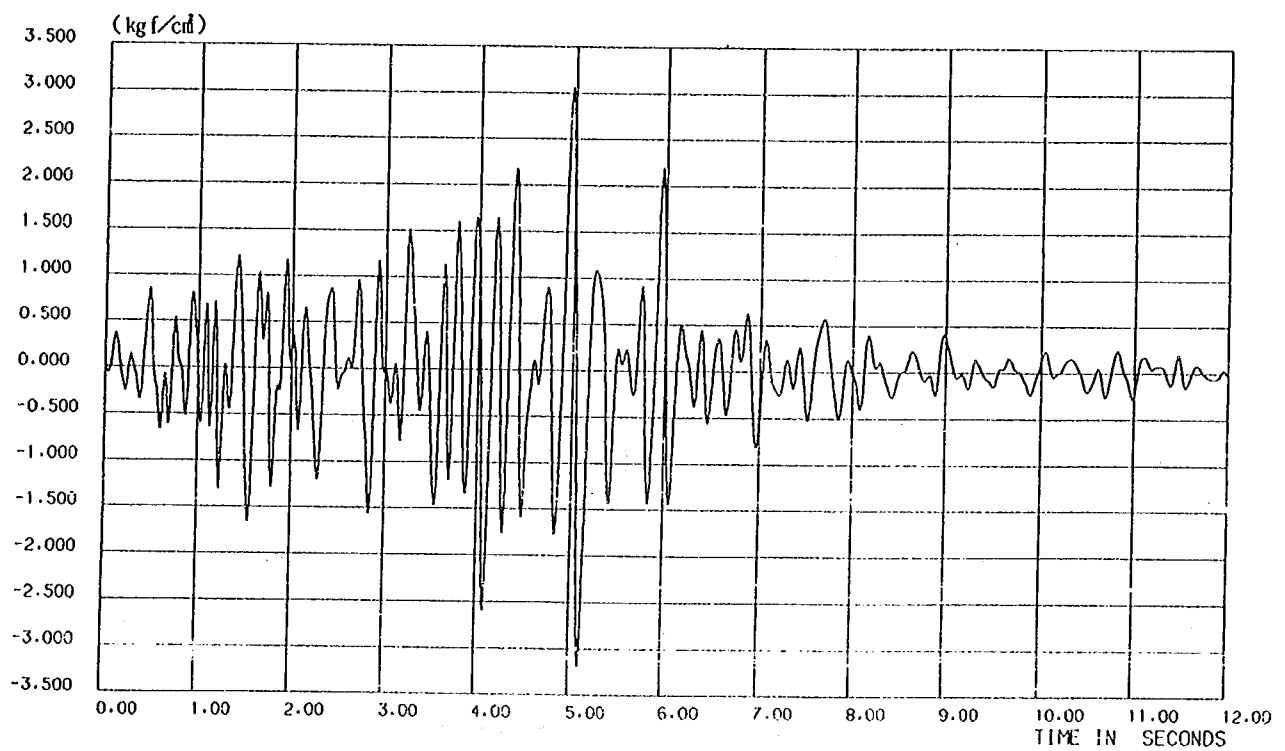


Fig.-4.5.12(c) Time-history of element stress (Case-B, Shear stress)



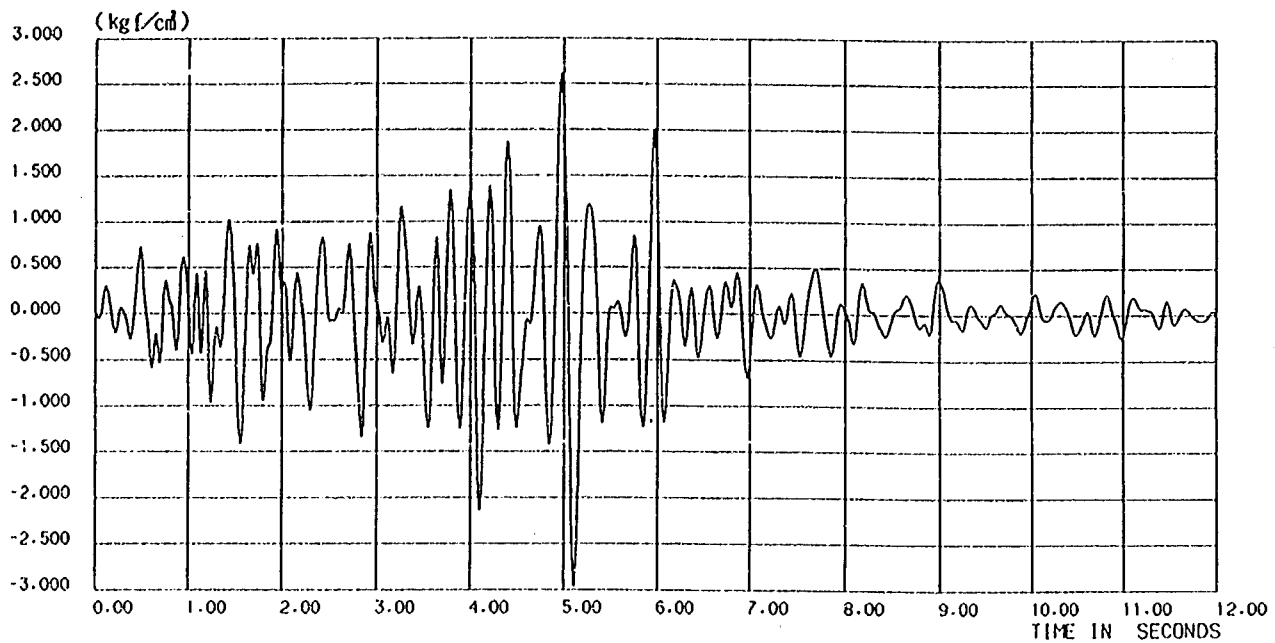
(5) Element 38 (Shear stress)

Fig.-4.5.12(c) Time-history of element stress (Case-B, Shear stress)

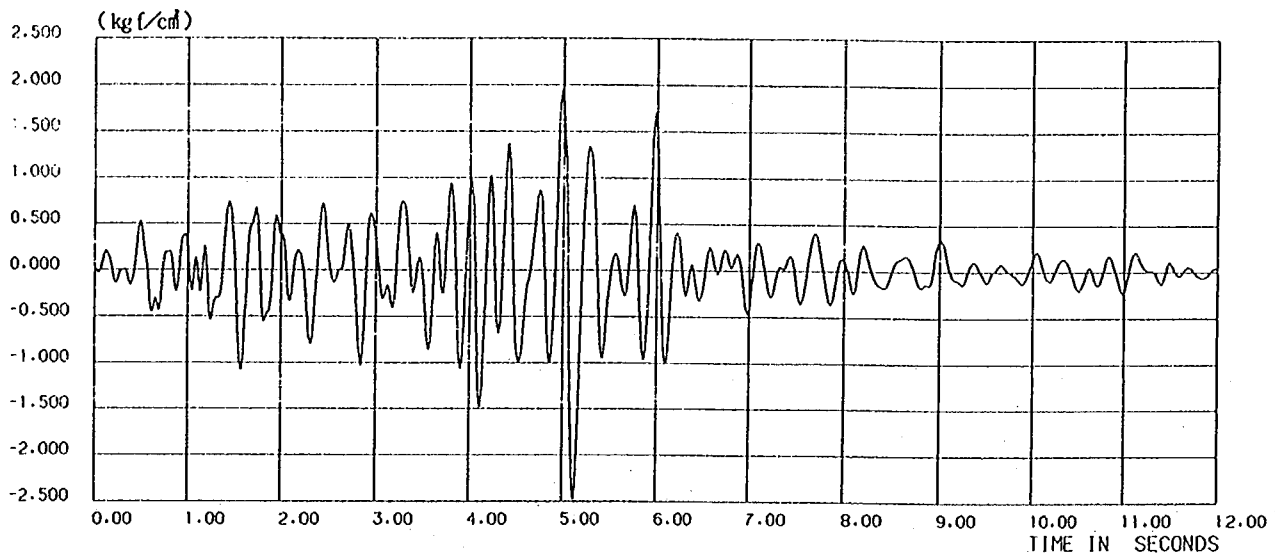


(1) Element 29 (Horizontal stress)

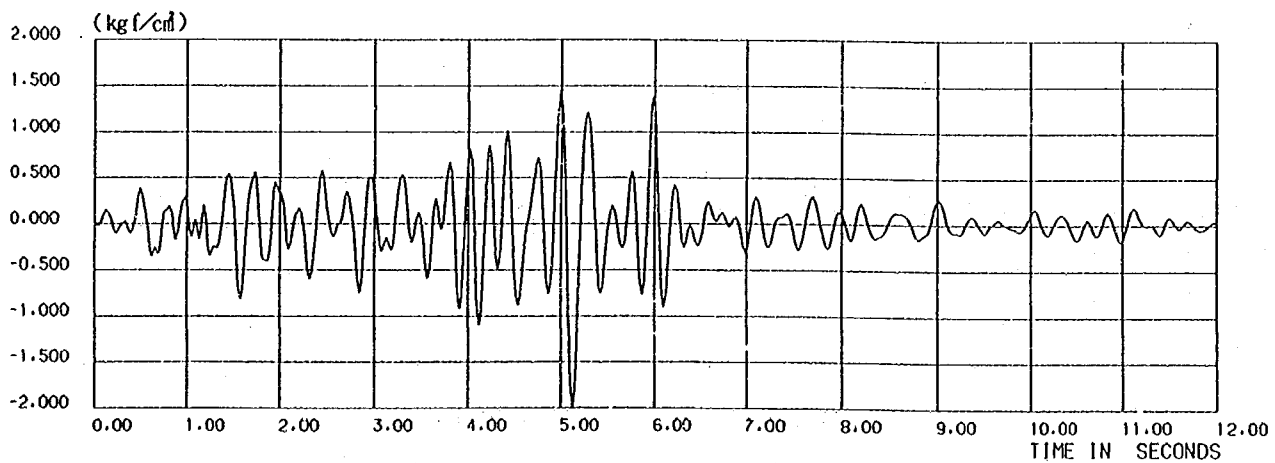
Fig.-4.5.13(a) Time-history of element stress (Case-C, Horizontal stress)



(2) Element 31 (Horizontal stress)



(3) Element 34 (Horizontal stress)



(4) Element 36 (Horizontal stress)

Fig.-4.5.13(a) Time-history of element stress (Case-C, Horizontal stress)

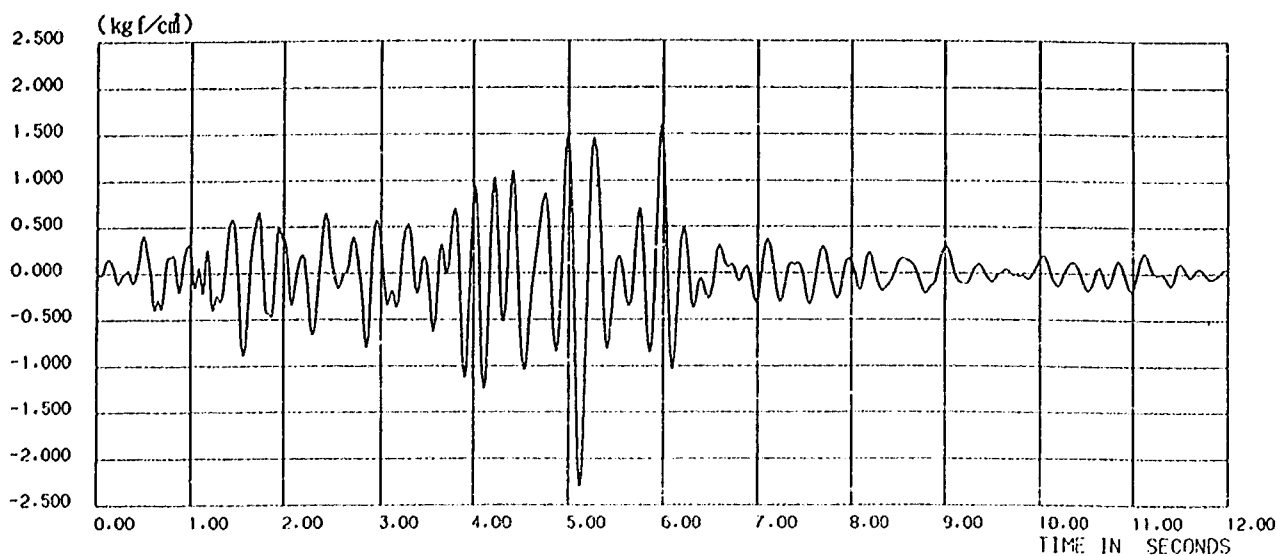


Fig.-4.5.13(a) Time-history of element stress (Case-C, Horizontal stress)

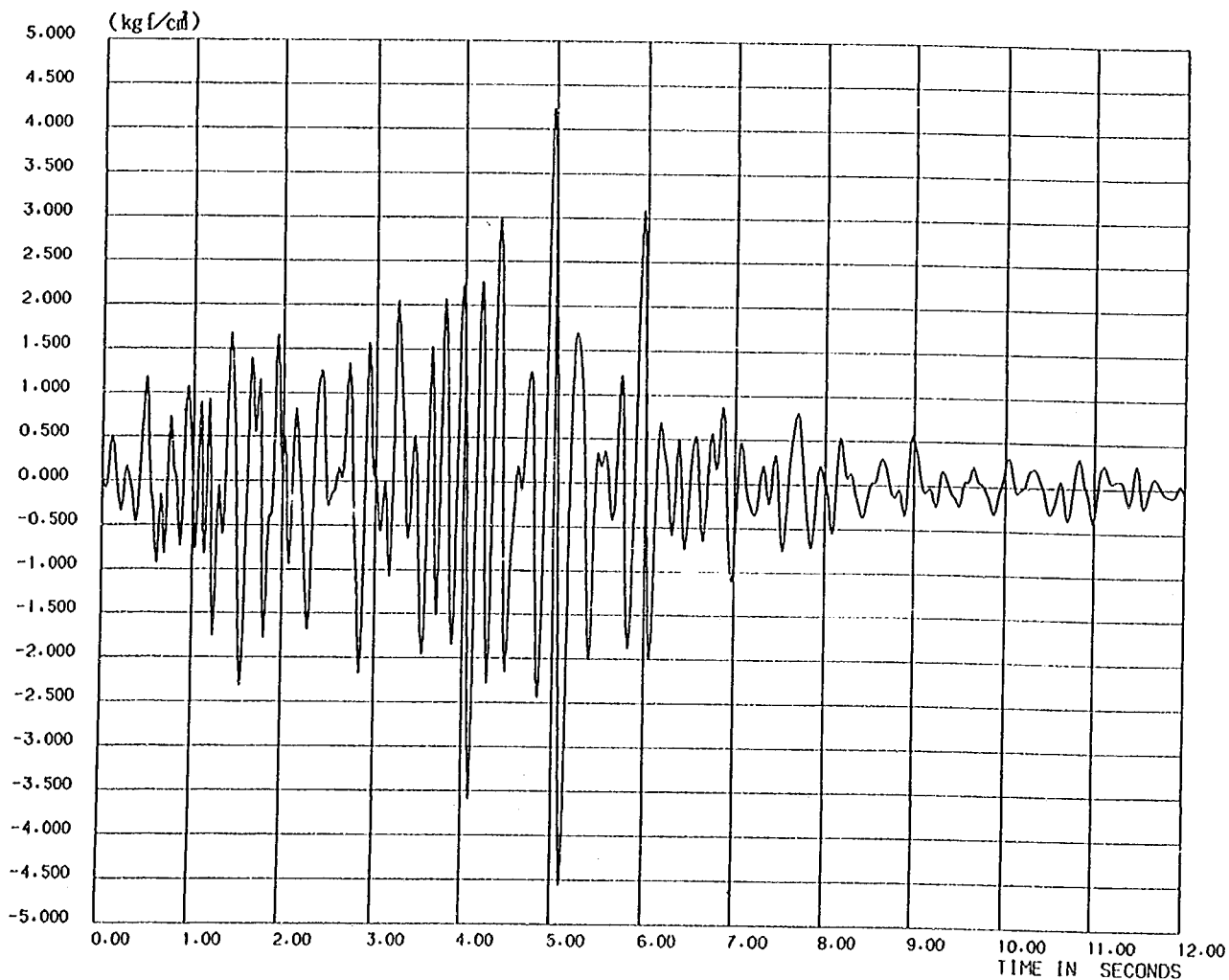
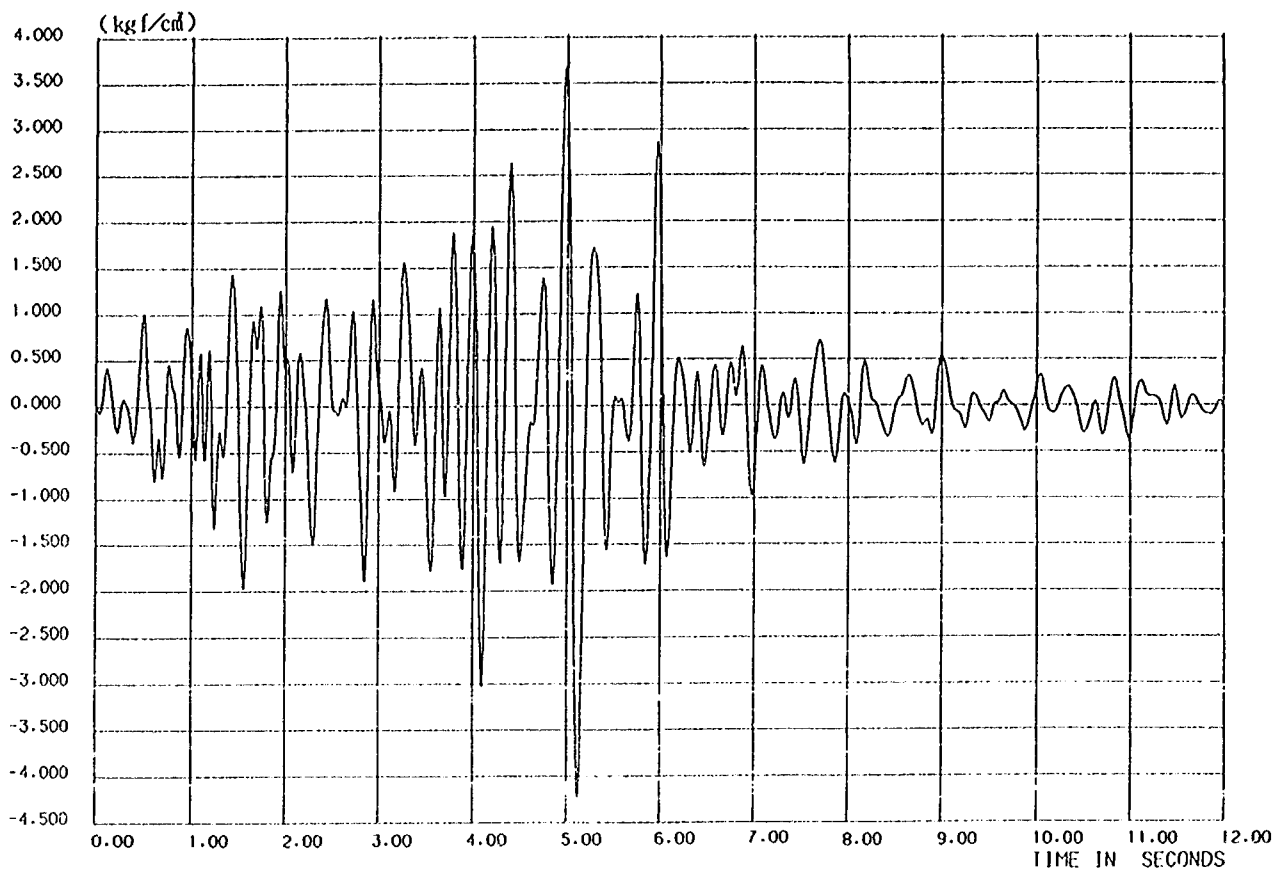
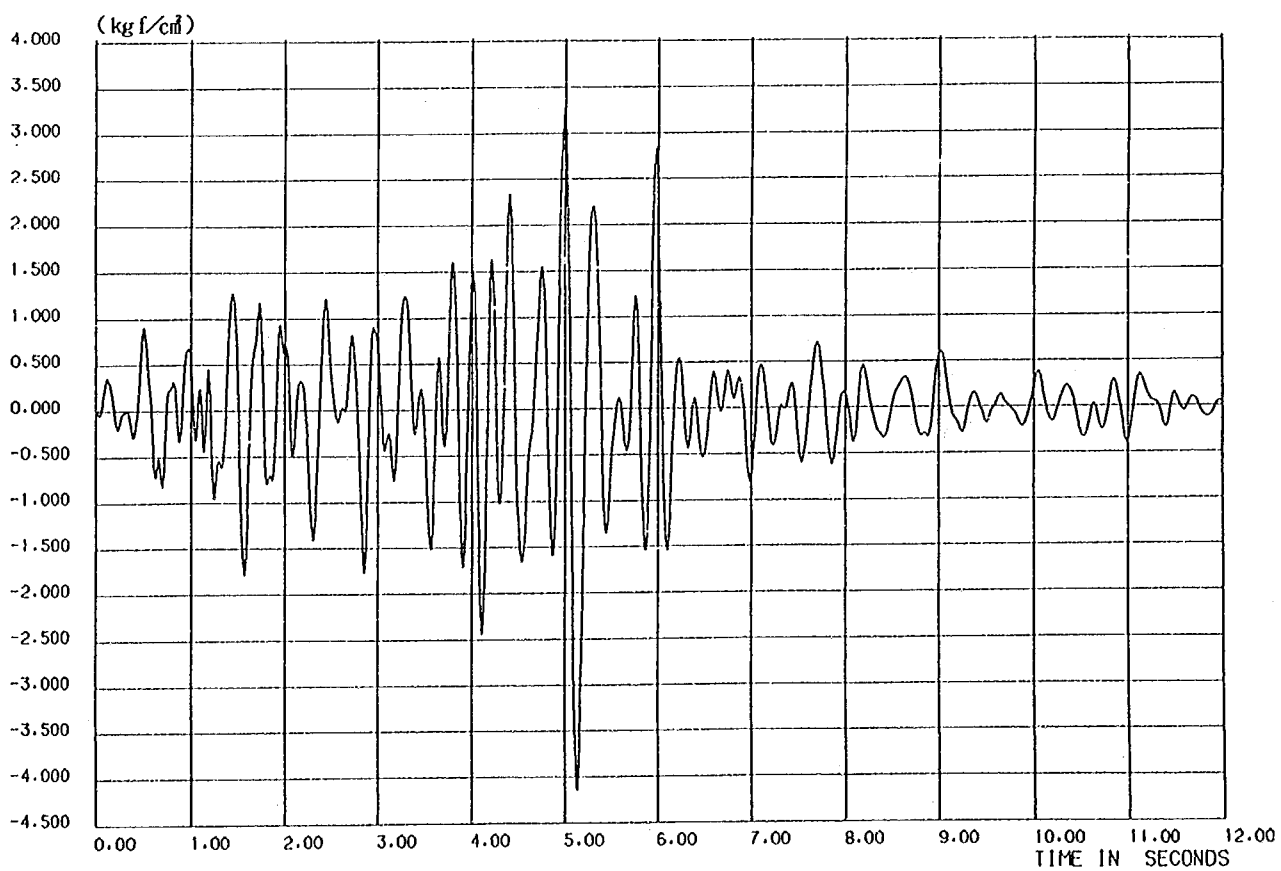


Fig.-4.5.13(b) Time-history of element stress (Case-C, Vertical stress)

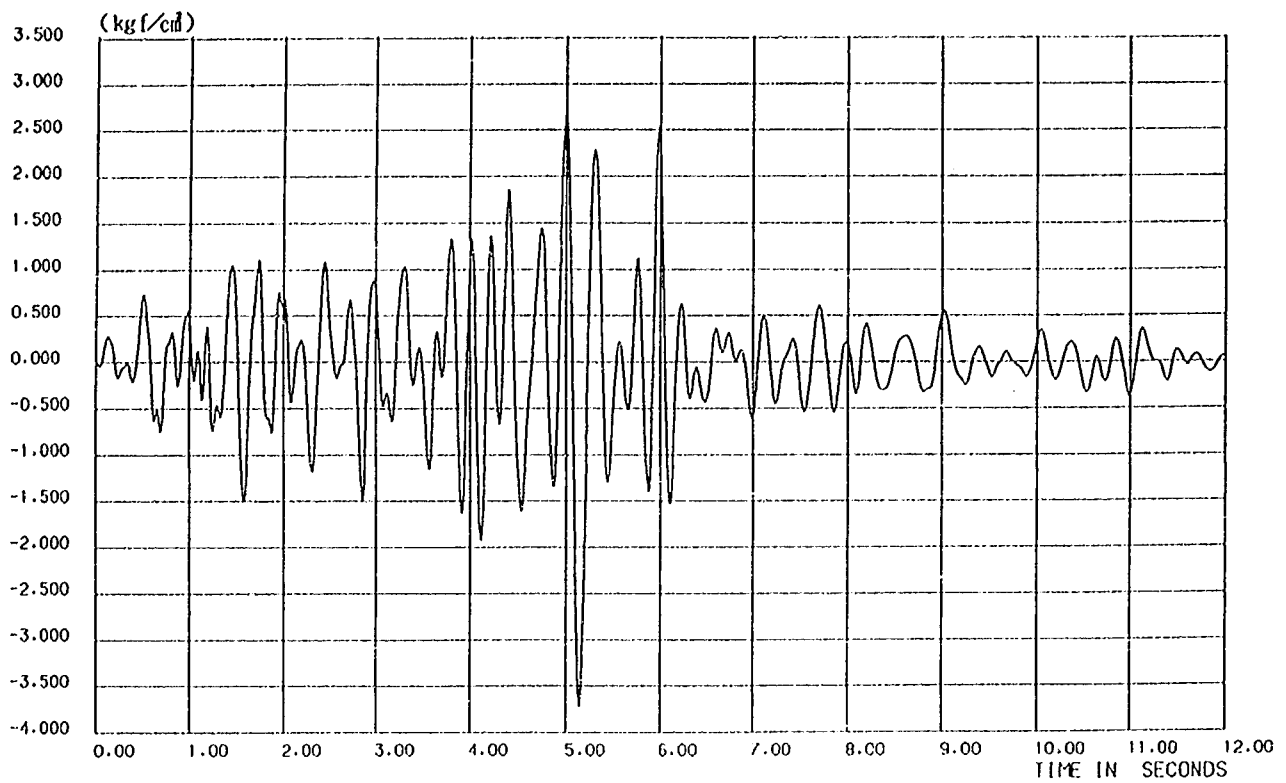


(2) Element 31 (Vertical stress)

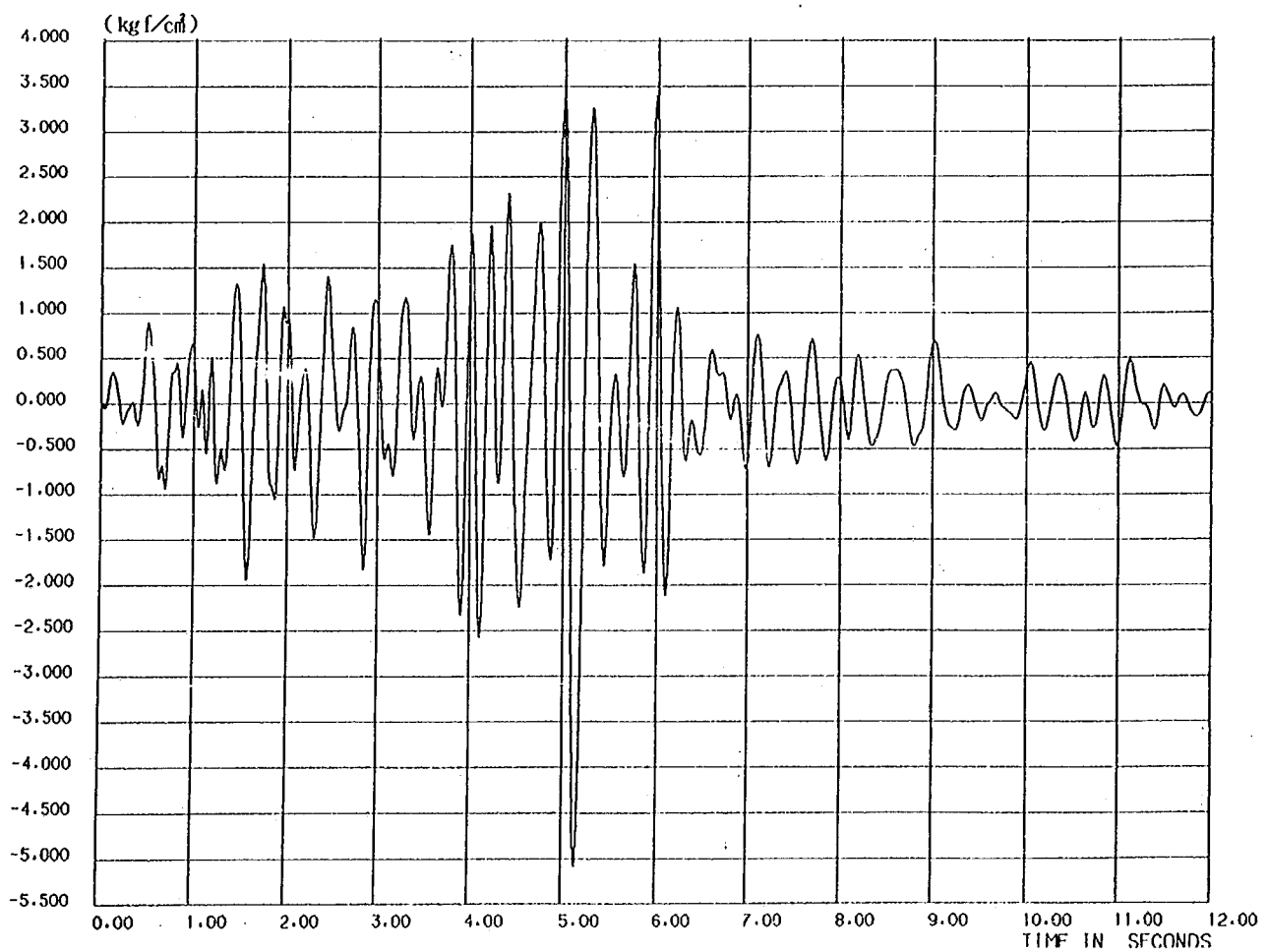


(3) Element 34 (Vertical stress)

Fig.4.5.13(b) Time-history of element stress (Case-C, Vertical stress)



(4) Element 36 (Vertical stress)



(5) Element 38 (Vertical stress)

Fig-4.5.13(b) Time-history of element stress (Case-C, Vertical stress)

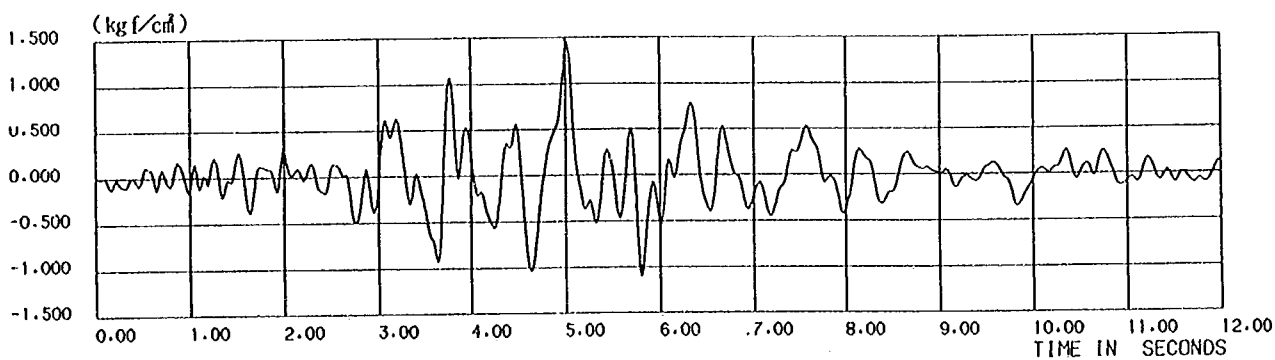
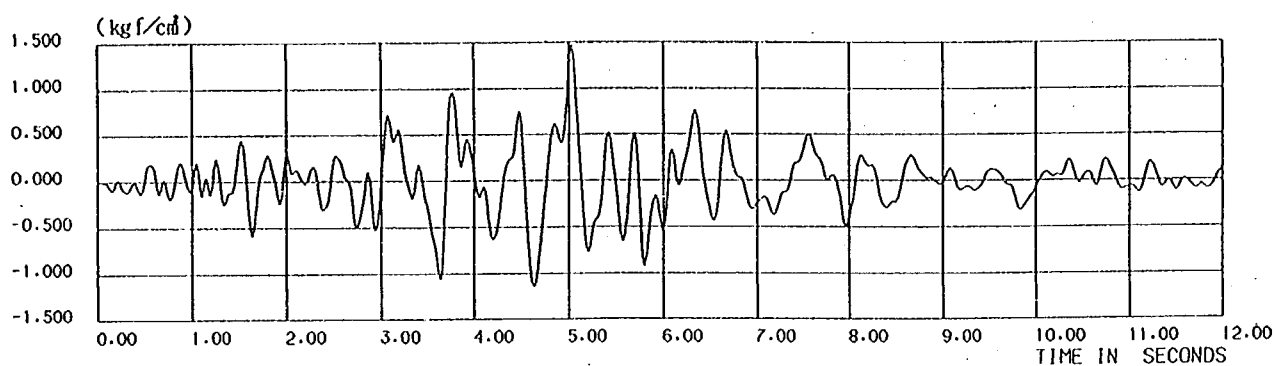
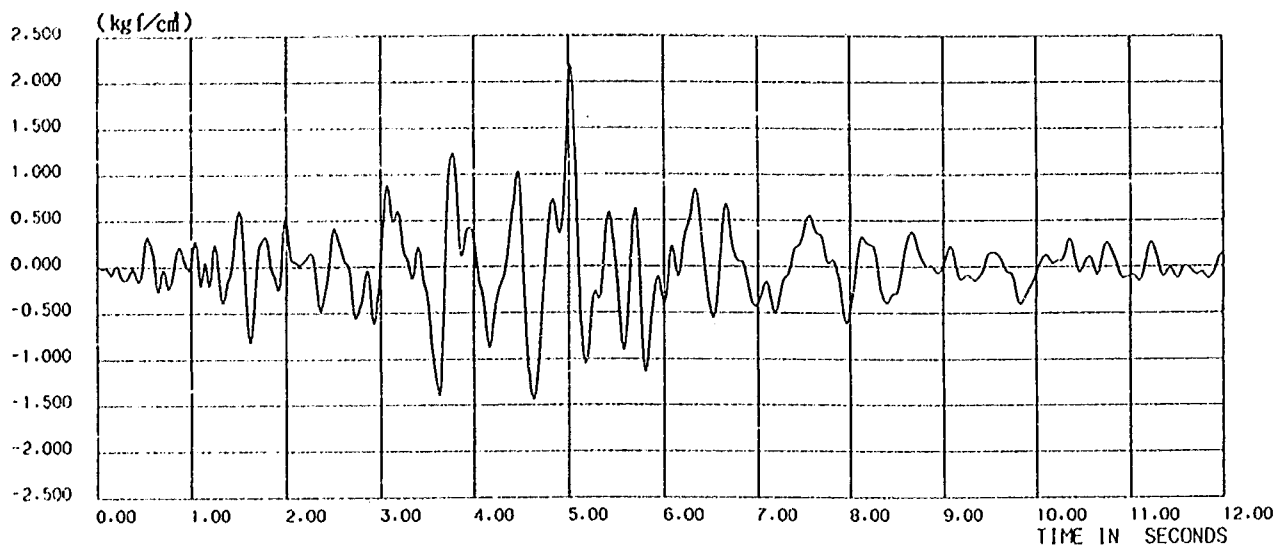


Fig.-4.5.13(c) Time-history of element stress (Case-C, Shear stress)

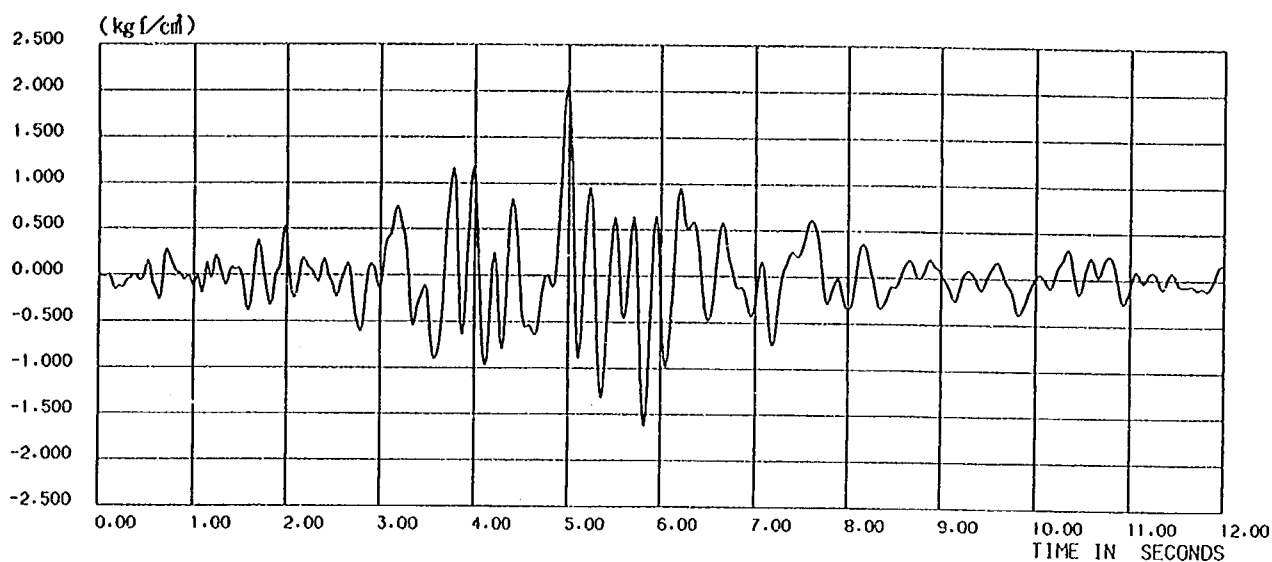
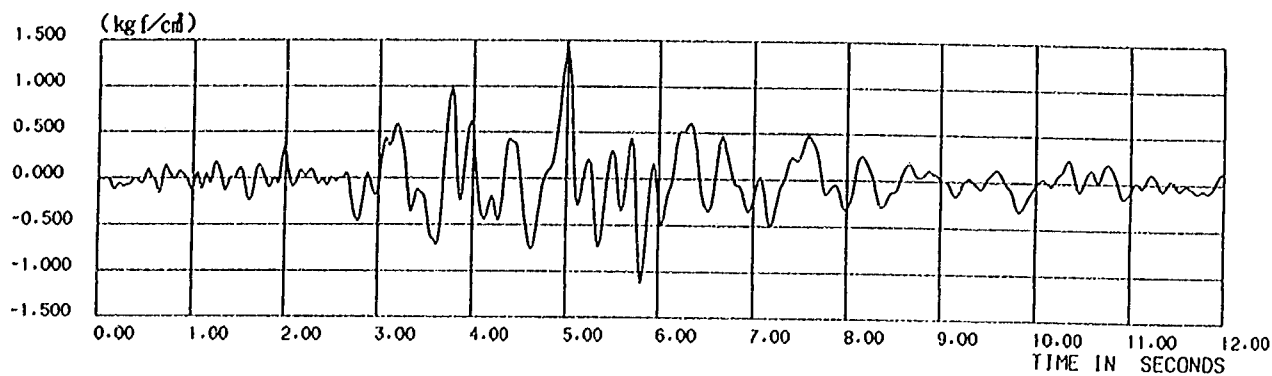


Fig.-4.5.13(c) Time-history of element stress (Case-C, Shear stress)

5. Evaluation of Dam Safety

In this chapter, the point safety factor of an element and the safety against sliding are investigated.

5.1 Point Safety Factor of a Dam Body

The safety of an element is investigated by the results of static and dynamic stresses with the Mohr-Coulomb failure law.

The point safety factor “ n ” is obtained as follows.

$$\left. \begin{aligned} n &= \frac{2 \cdot c \cdot \cos \phi + (\sigma_1 + \sigma_3 - 2 \cdot U_d) \sin \phi}{\sigma_1 - \sigma_3} \\ U_d &= \frac{(1+\nu)(\sigma_{1d} + \sigma_{3d})}{3} \end{aligned} \right\} \quad \dots (5.1)$$

where

c : cohesion

ϕ : Angle of internal friction

σ_1, σ_3 : Principal stress

(Results of static and dynamic stresses)

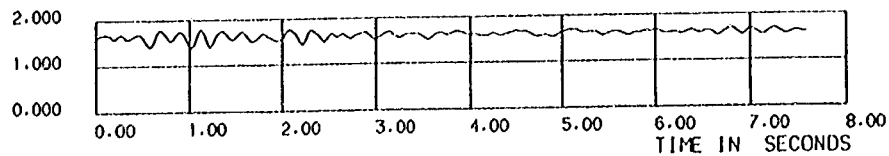
U_d : Dynamic pore water pressure

σ_{1d}, σ_{3d} : Dynamic principal stress

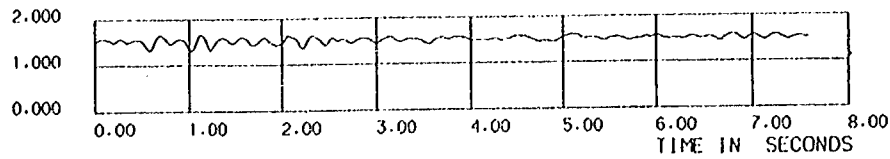
ν : Poisson's ratio

The elements at the base of a body and the core zone are studied as shown in Fig.-4.5.10. Figs.-5.1.1 to 5.1.3 show time histories of the safety factor. The angle of internal friction and cohesion of the rockfill material are the results of laboratory testing.

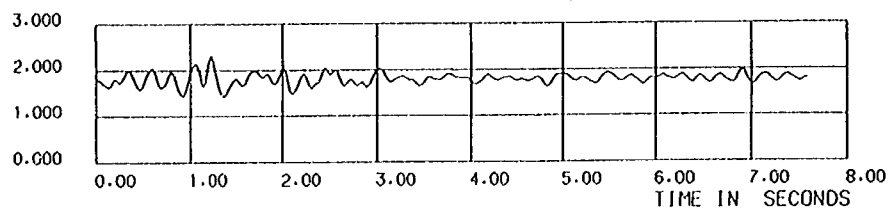
The point safety factors of Case-A and Case-B are not less than 1.0. The point safety factor of the dam base is frequently under 1.0, however the safety factors of elements 141 and 207 at the upper level of the core zone are never less than 1.0. The frequency, that the safety factor of element 34 at the core zone is below 1.0, is lower compared with other elements at the dam base. Assuming that a decrease of the point safety factor at an element suggests the possible occurrence of cracks, a decrease of the safety factor especially at the core zone is problem, because infiltration failure of the core damages the stability of a dam. However, the point safety factor of an element over the middle level is not less than 1.0, as in the above-mentioned, and the safety factor at the base is under 1.0 in a moment. Consequently, it seems that stability of the core zone during an earthquake is preserved. Cohesion of the core material is disregarded in this computation, but core material has a certain value of cohesion, so larger safety factors are really expected.



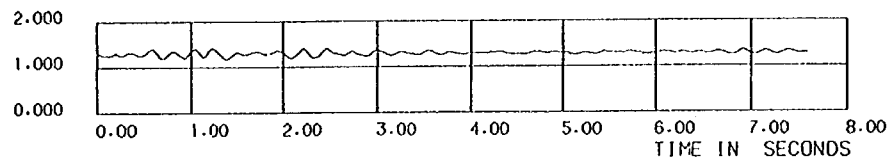
(1) Element 29



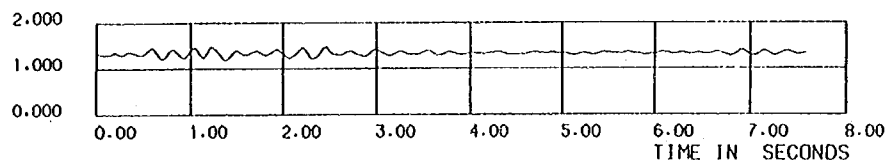
(2) Element 31



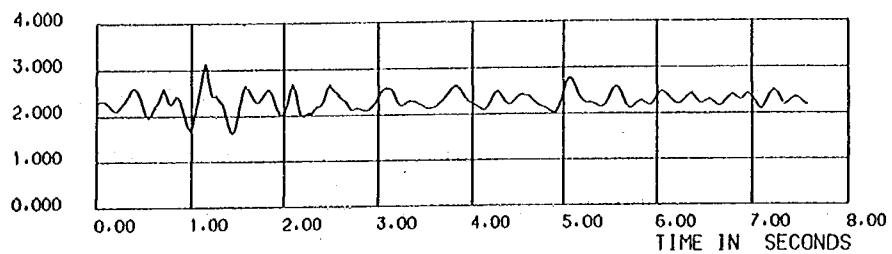
(3) Element 34



(4) Element 36

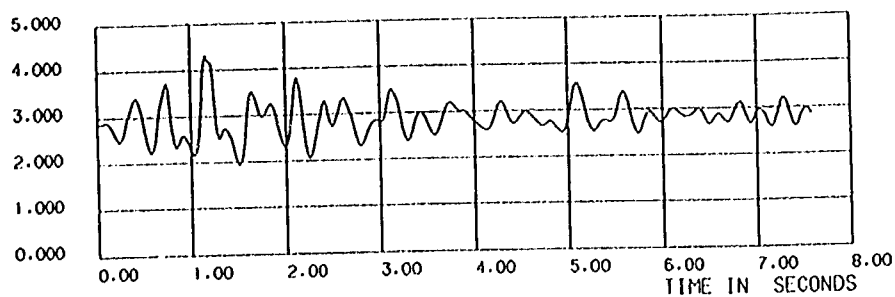


(5) Element 38



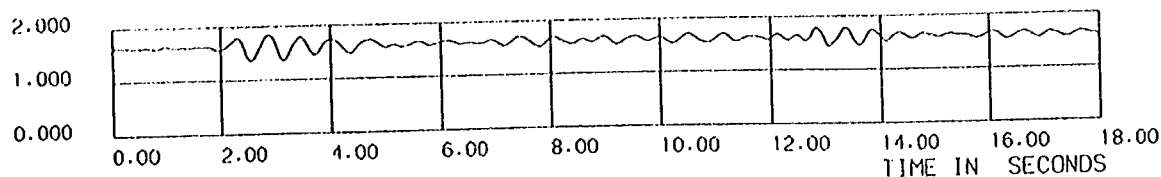
(6) Element 141

Fig.-5.1.1 Time-history of safety factor at element (Case-A)

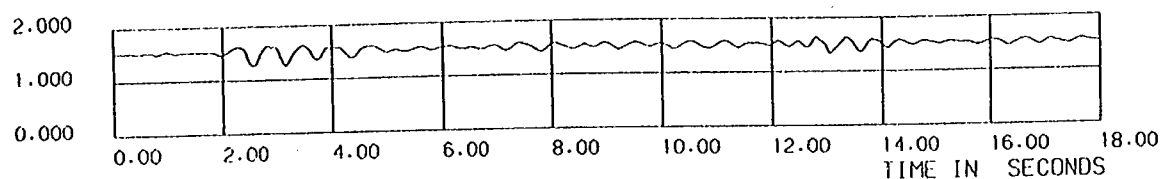


(7) Element 207

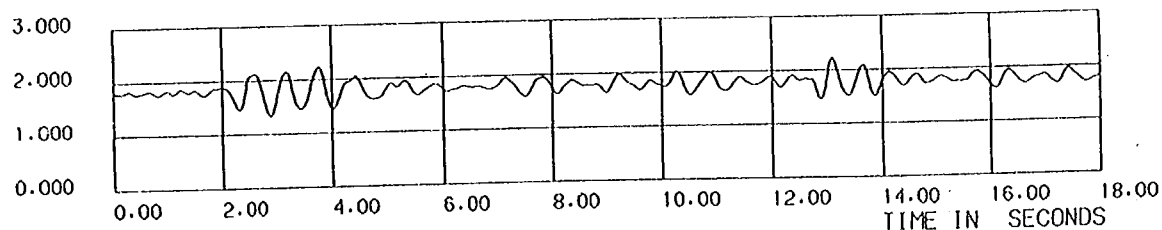
Fig.-5.1.1 Time-history of safety factor at element (Case-A)



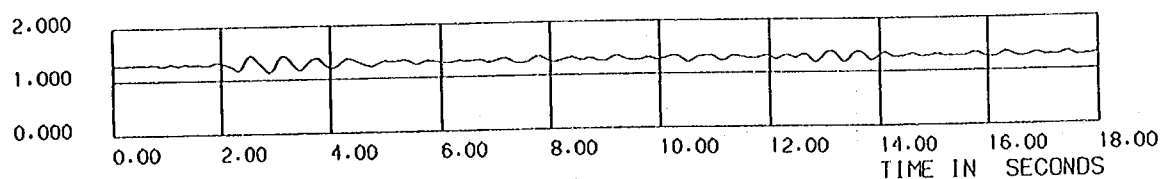
(1) Element 29



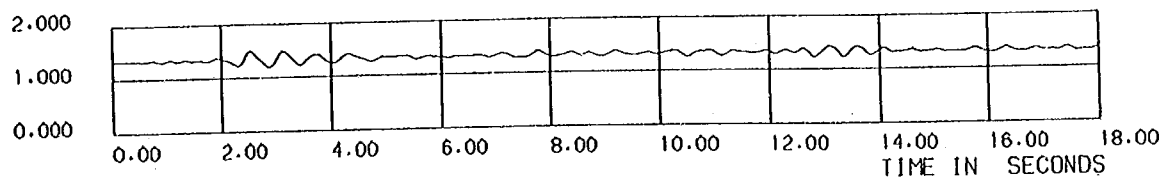
(2) Element 31



(3) Element 34

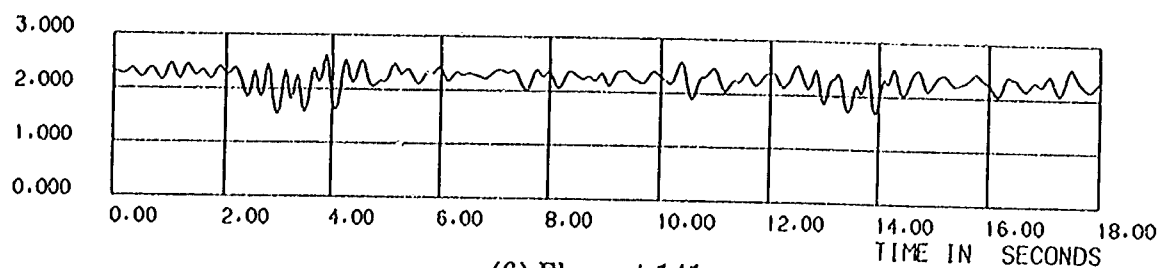


(4) Element 36

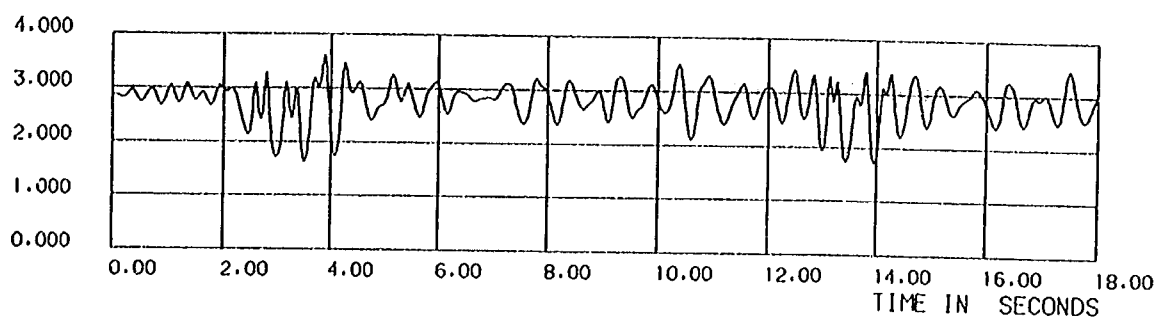


(5) Element 38

Fig.-5.1.2 Time-history of safety factor at element (Case-B)

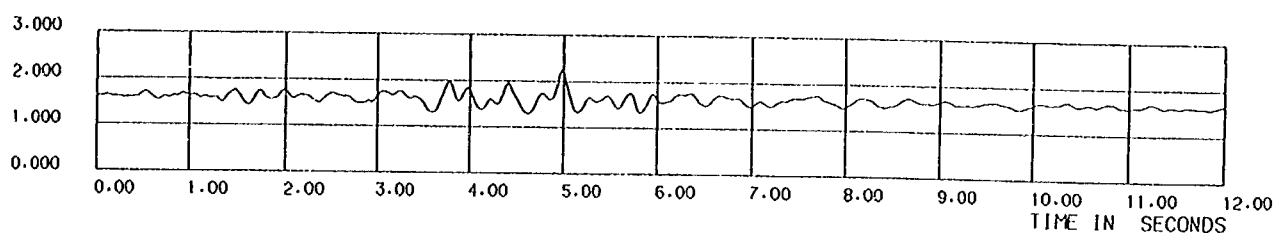


(6) Element 141

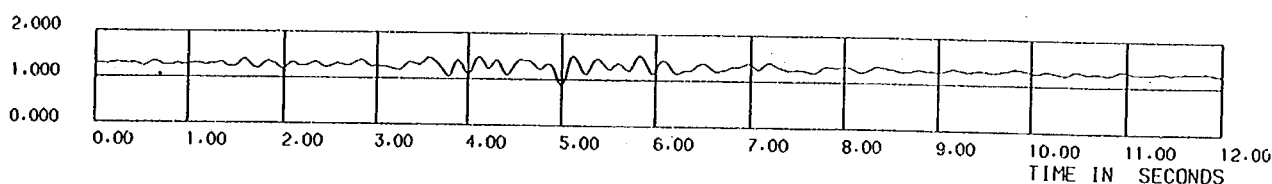


(7) Element 207

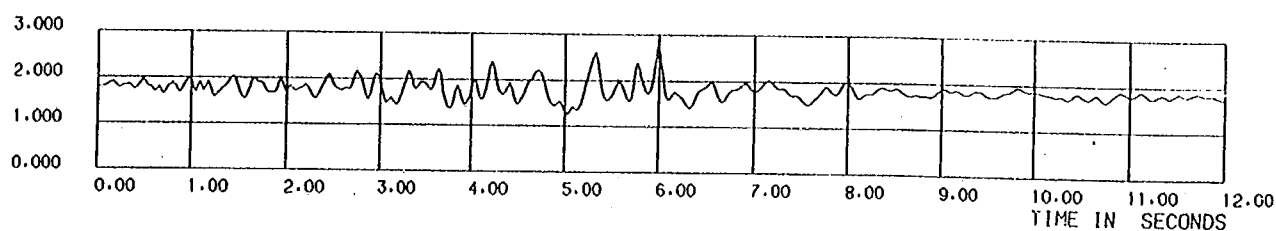
Fig.-5.1.2 Time-history of safety factor at element (Case-B)



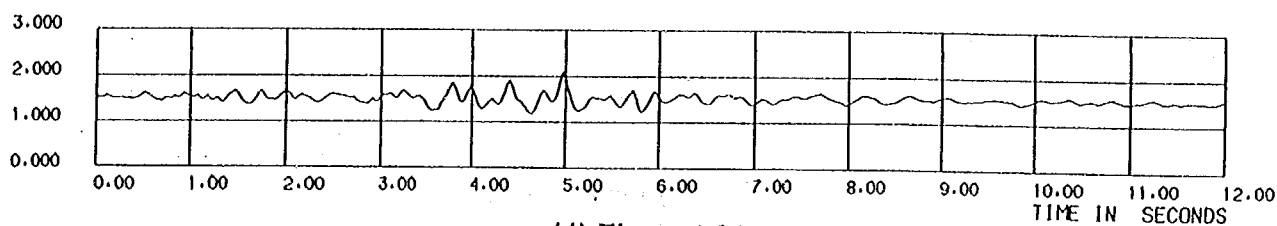
(1) Element 29



(2) Element 31

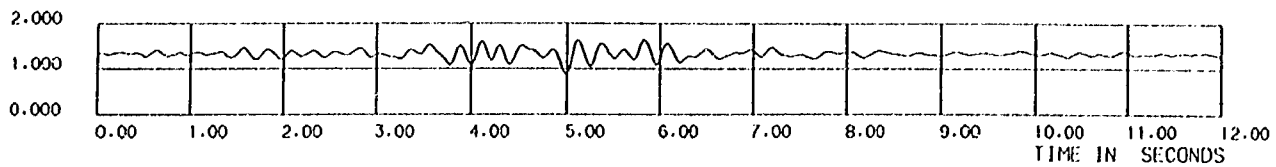


(3) Element 34

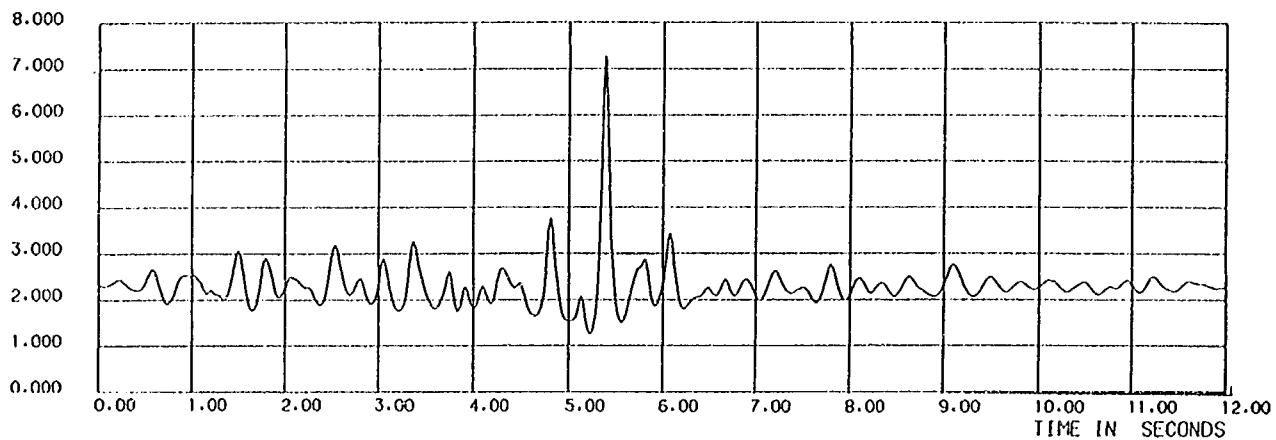


(4) Element 36

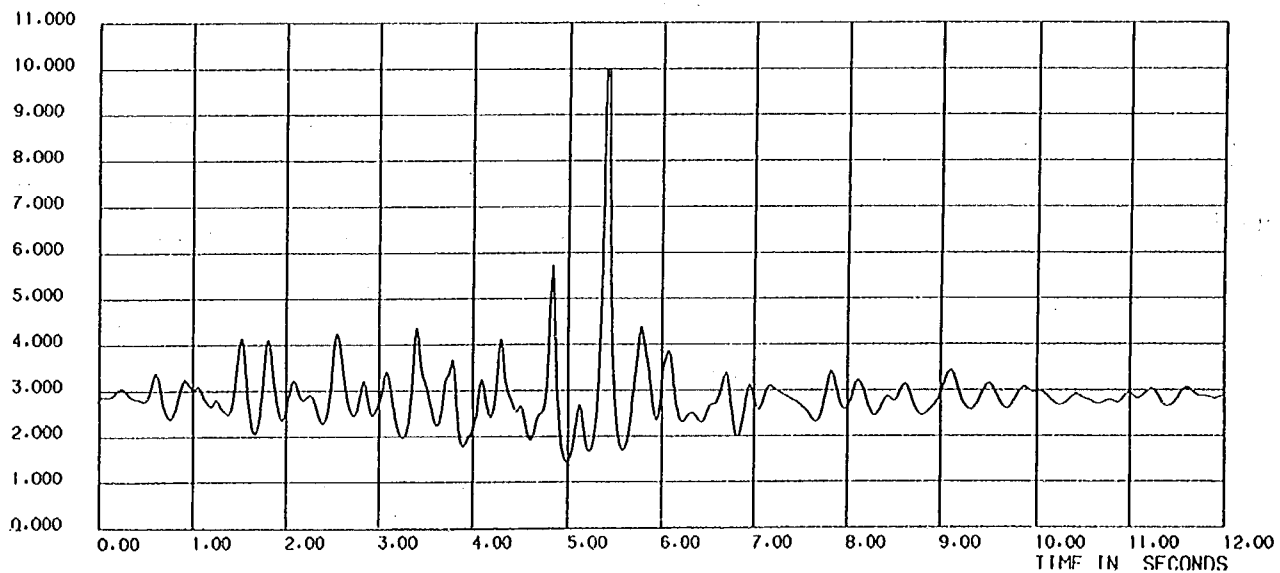
Fig.-5.1.3 Time-history of safety factor at element (Case-C)



(5) Element 38



(6) Element 141



(7) Element 207

Fig.-5.1.3 Time-history of safety factor at element (Case-C)

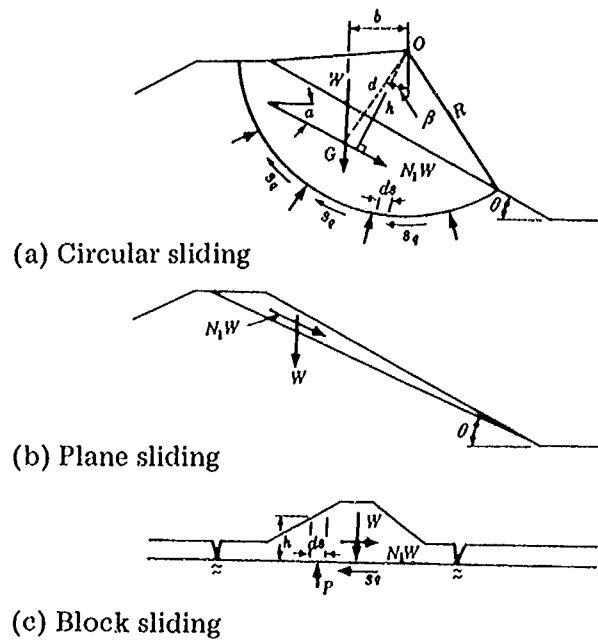


Fig.-5.2.1 Forces on slide block

5.2 Stability against Sliding

After Newmark, the failure of slope during an earthquake is classified into three types, circular sliding, plane sliding, and block sliding. Investigations on the circular sliding and plane sliding surface are carried out in this report (see Fig.-5.2.1).

As for the circular sliding surface, the movement of a slip circle is calculated using the mean response acceleration of the circle. The mean response acceleration of the slip circle is given as the following. The mean acceleration is the weighted average of elements that hold nodal points inside the slip circle in common. The weighted mean response acceleration $\bar{\alpha}_i$ that results from the acceleration α_i of nodal point i is shown in Fig.-5.2.2.

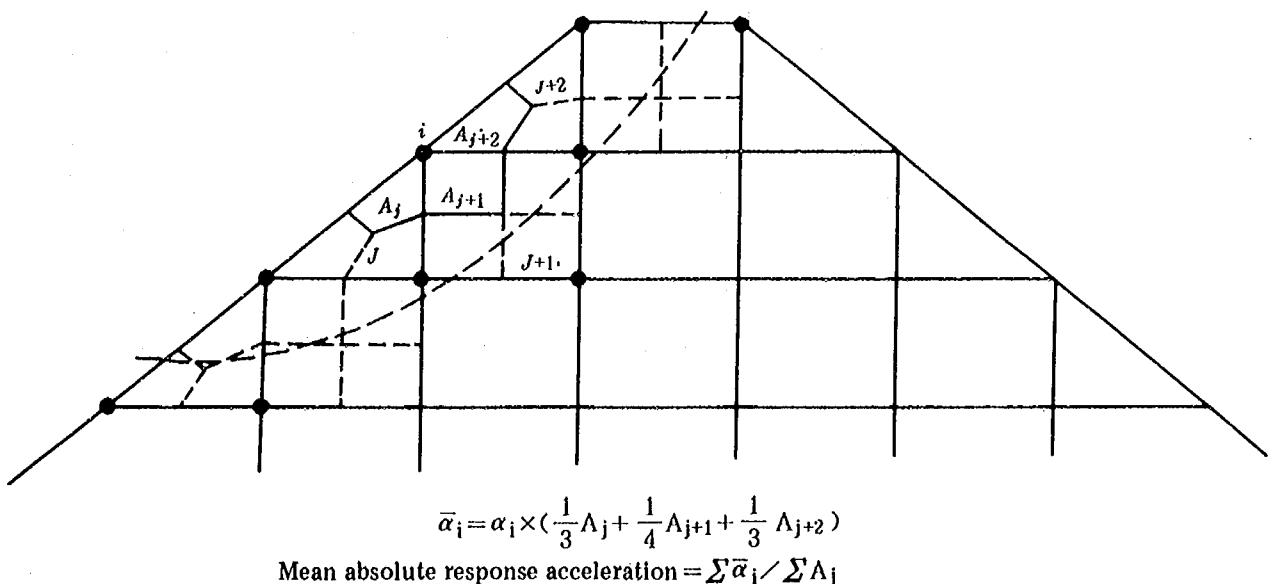


Fig.-5.2.2 Mean response acceleration of slip circle

$$\bar{\alpha}_i = \alpha_i \times \left(\frac{1}{3} A_i + \frac{1}{4} A_{i+1} + \frac{1}{3} A_{i+2} \right) \quad \dots (5.2)$$

The mean response acceleration of slip circle A_{mean} is obtained as follows.

$$A_{mean} = \frac{\sum \bar{\alpha}_i}{\sum A_i} \quad \dots (5.3)$$

Newmark's sliding block method can be applied to estimate the movement along sliding surfaces due to earthquakes (see Fig-5.2.3). A_{cri} is defined as the critical acceleration at which a slide mass starts moving, and the movement of the slide mass D is calculated by equation (5.4). The calculation for sliding movement starts when A_{mean} exceeds A_{cri} as shown in Fig-5.2.4.

$$D = \int_{t_0}^{t_1} (A_{mean} - A_{cri}) \cdot dt \cdot dt \quad \dots (5.4)$$

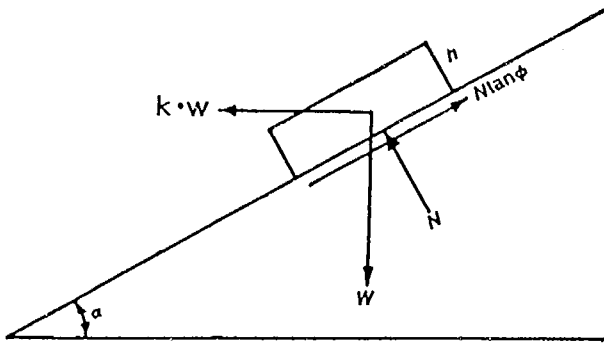


Fig.-5.2.3 Slide block

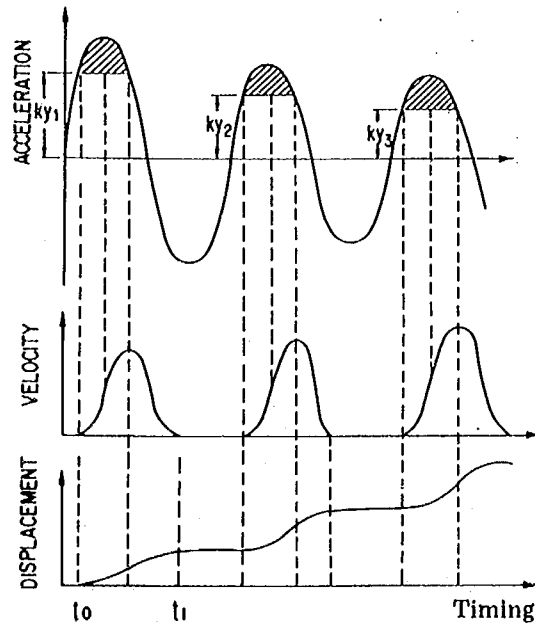


Fig.-5.2.4 Integration of effective time-history to determine velocities and displacements

The calculation is continued until t_1 when the velocity, which is the integration of $(A_{mean} - A_{cri})$, becomes zero. The slide mass is only permitted to move downward.

The potential circular sliding surfaces are obtained by connecting the potential sliding plane of an element, resulted from a Mobilized Plane*) after dynamic analysis. The critical acceleration of a slip circle is calculated by the modified Swedish-method considering the horizontal seismic coefficient, and it is the acceleration with which pseudo-static analysis gives a safety factor of unity to the potential slip circle.

Newmark's sliding block method is also applied to estimate the displacement along the plane sliding surfaces. The total displacement caused by an earthquake can be calculated by integrating the response acceleration of the nodal point on the slope. The critical acceleration A_{crit} is given by both the gradient of the slope and the angle of shearing resistance of the fill material.

* Mobilized Plane: The conjugate failure plane is determined from the angle of shearing resistance and the dynamic principal stresses of each element. The potential circular sliding surfaces are assumed to advance a rupture along this mobilized plane.

5.2.1 Stability of the Circular Sliding Surface

Fig.-5.2.5 explains the reviewed circular sliding surfaces. Time histories of the mean

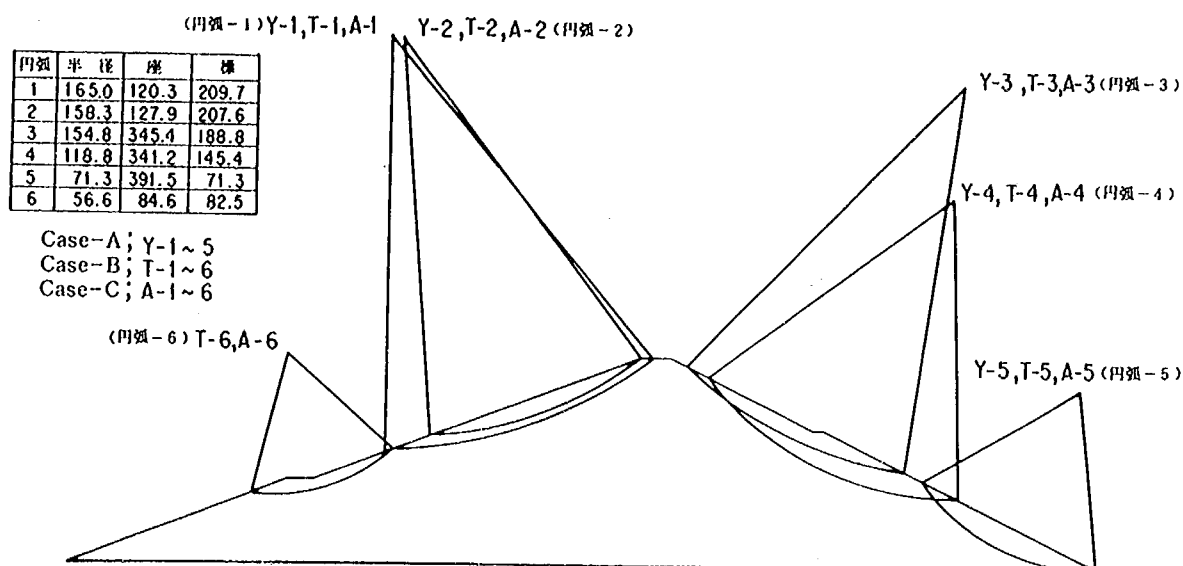


Fig.-5.2.5 Slip circle to calculate mean response acceleration

Table-5.2.1 Mean Response Acceleration of Slip Circle

Slip circle	Radius (m)	Critical acceleration gal (seismic coefficient)	Maximum acceleration		
			Case-A	Case-B	Case-C
1	165	323.4 (0.33)	145.2	184.4	229.7
2	160	323.4 (0.33)	167.5	198.0	226.3
3	155	313.6 (0.32)	230.9	158.4	255.2
4	120	333.2 (0.34)	236.1	125.7	281.0
5	70	323.4 (0.33)	255.0	85.1	301.6
6	55	509.6 (0.52)		178.3	534.1

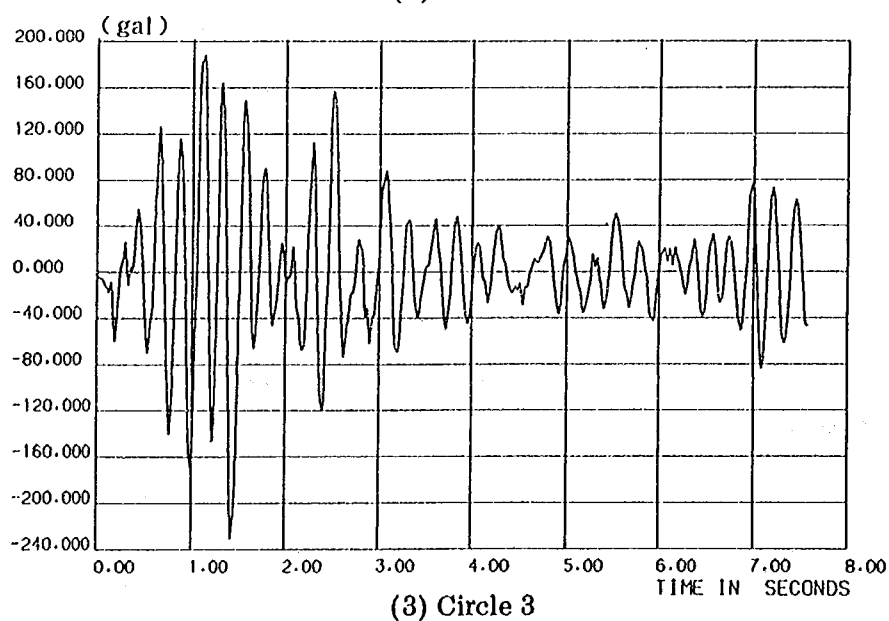
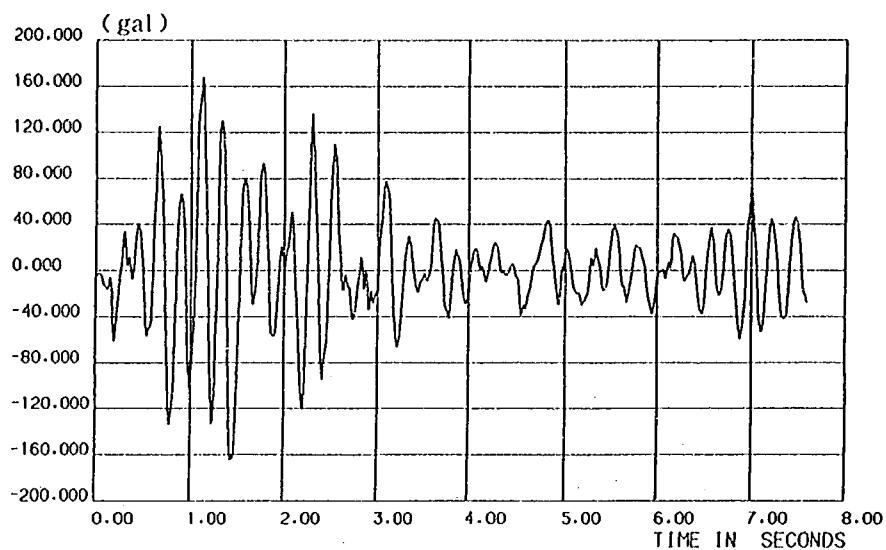
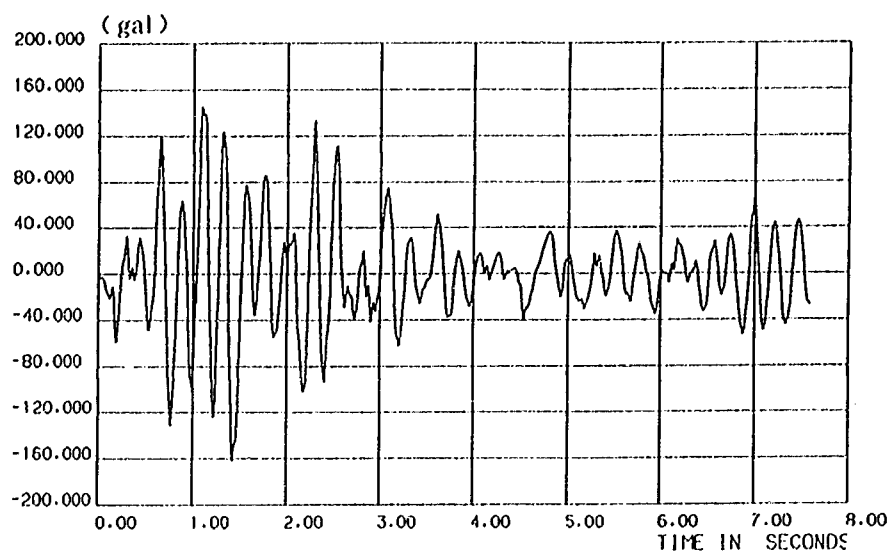
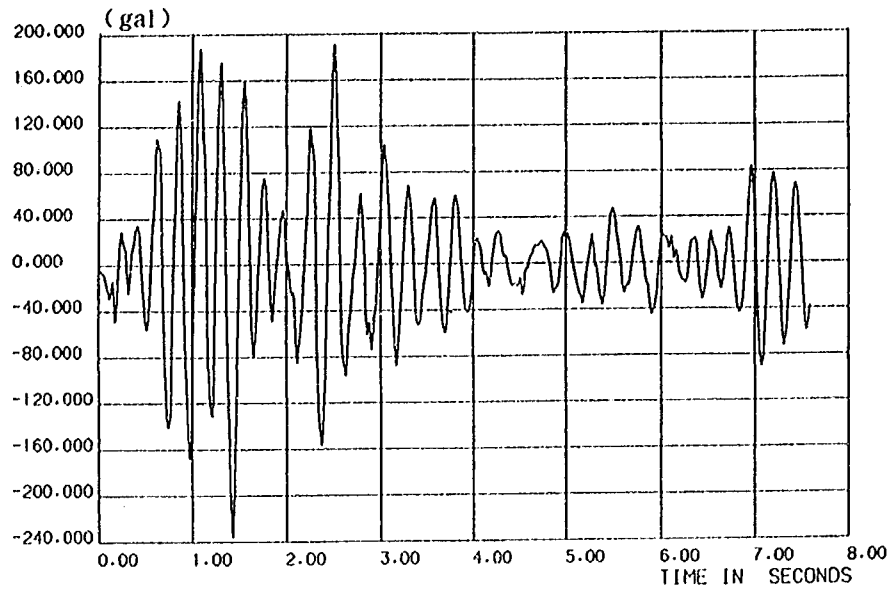
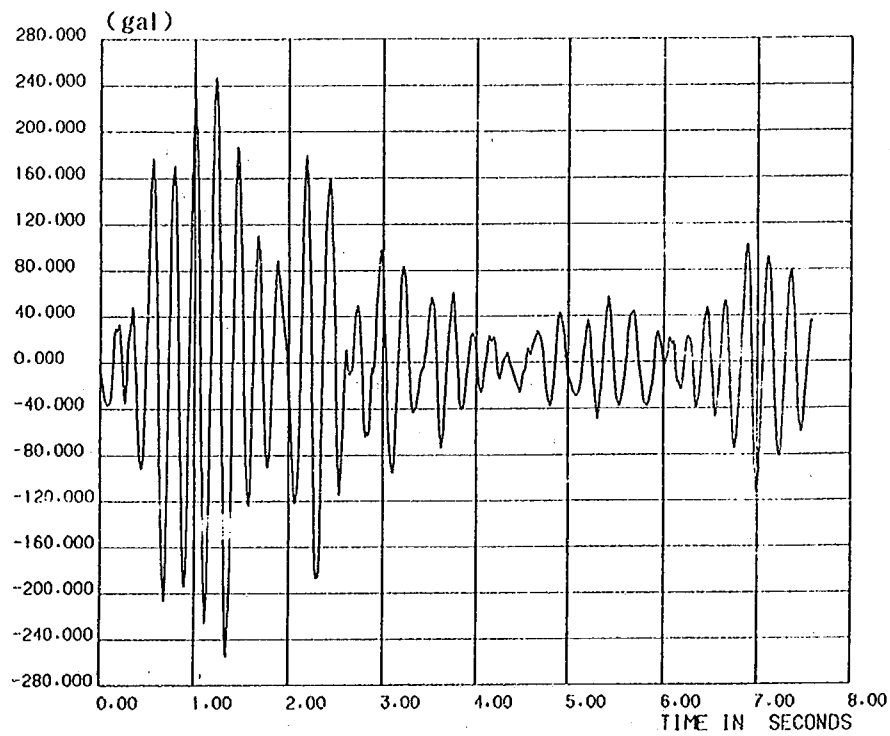


Fig.-5.2.6 Time-history of mean response acceleration of slip circle (Case-A, Horizontal)

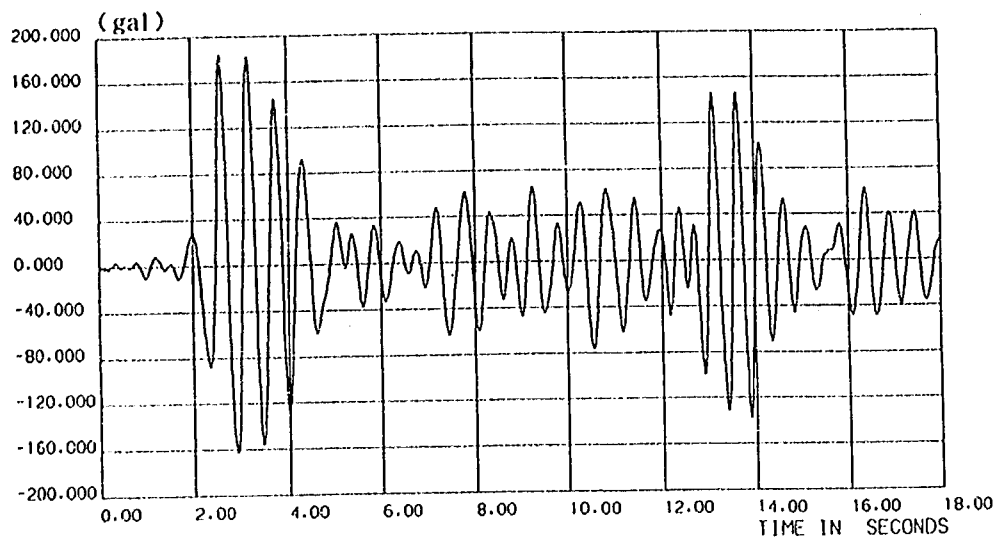


(4) Circle 4

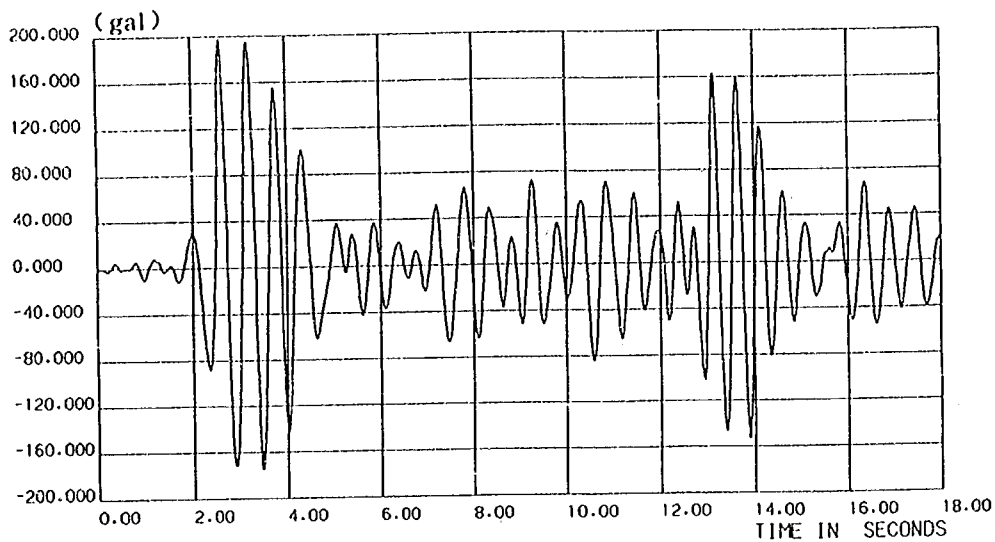


(5) Circle 5

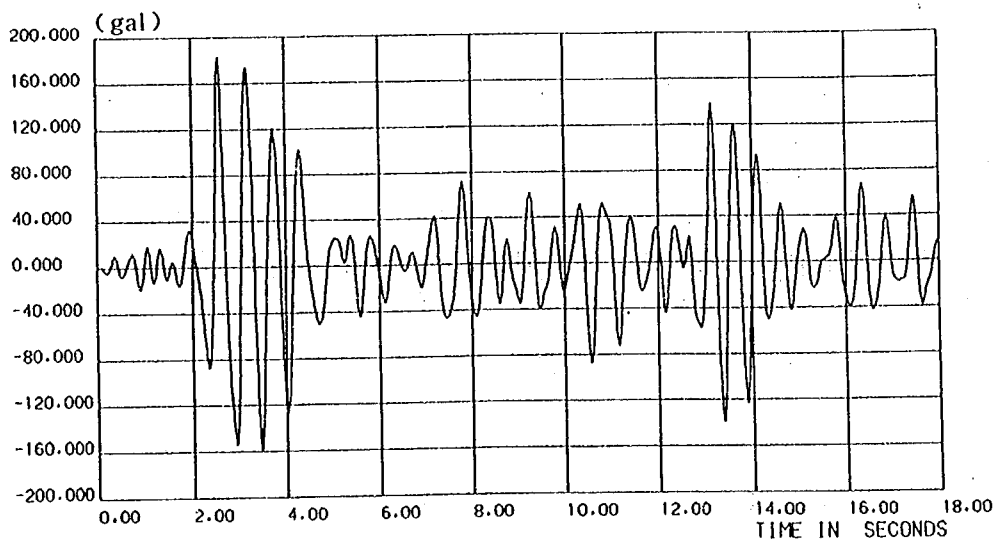
Fig.-5.2.6 Time-history of mean response acceleration of slip circle (Case-A, Horizontal)



(1) Circle 1

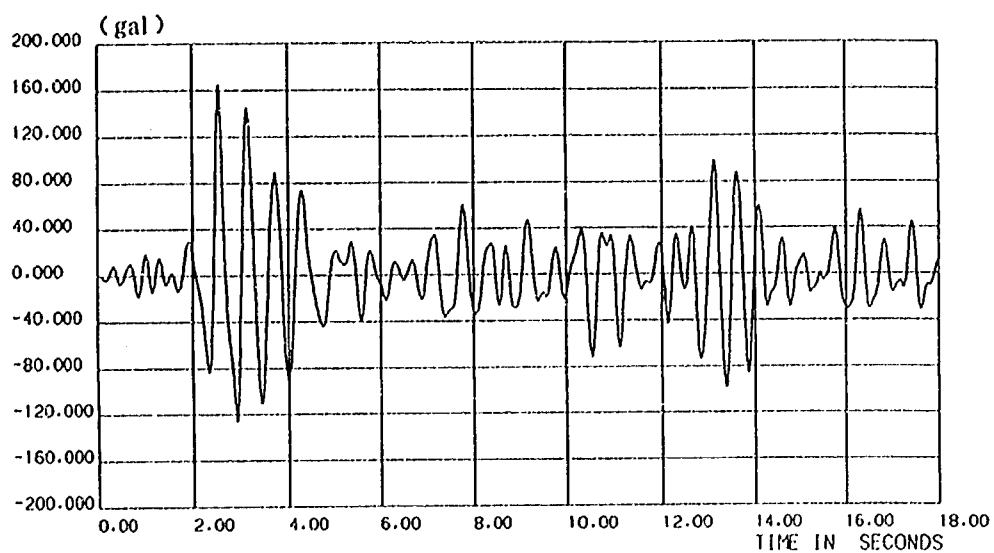


(2) Circle 2

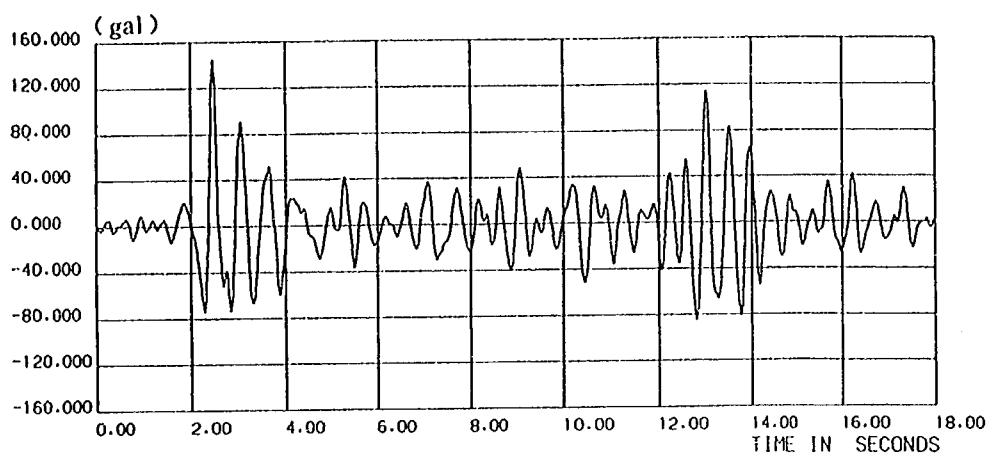


(3) Circle 3

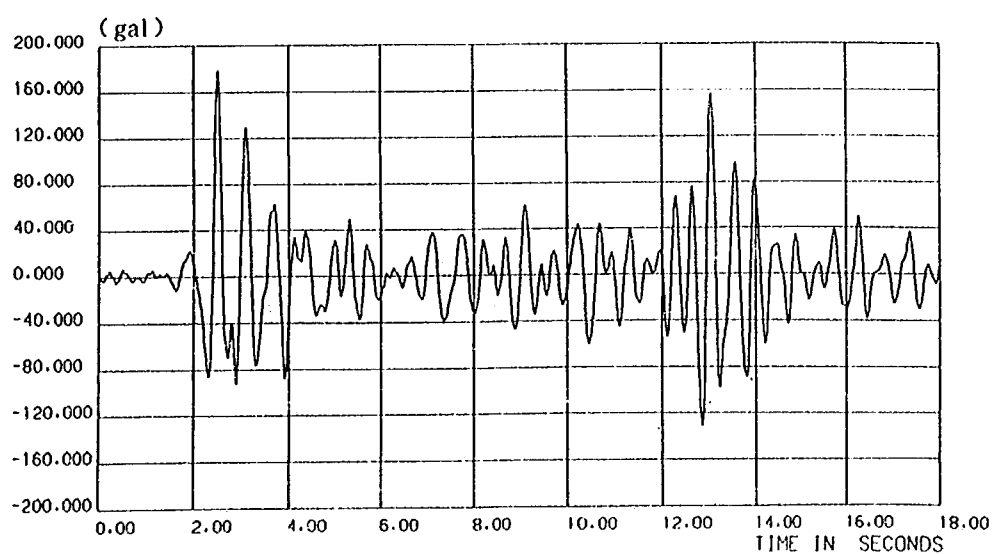
Fig.-5.2.7 Time-history of mean response acceleration of slip circle (Case-B, Horizontal)



(4) Circle 4

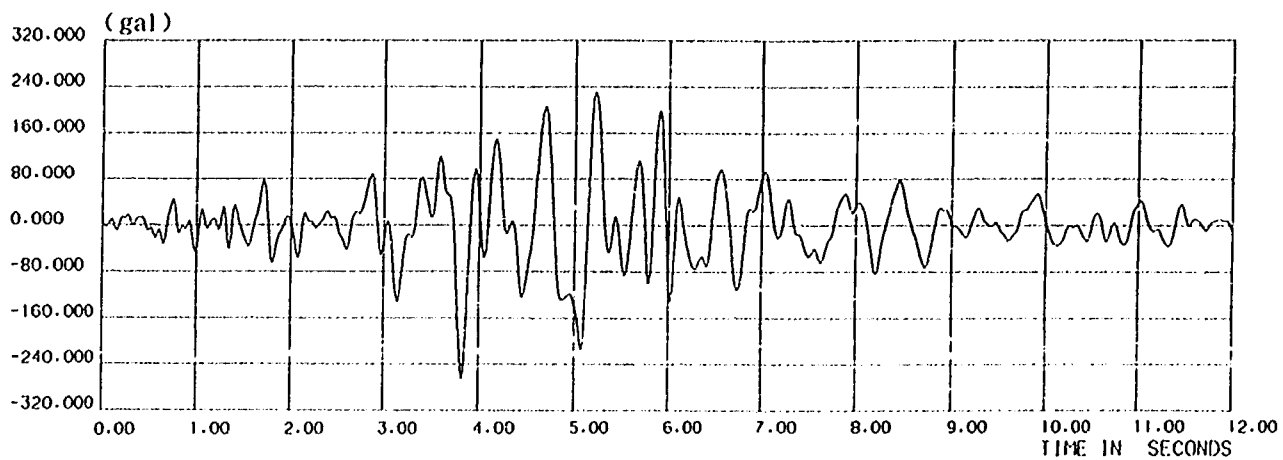


(5) Circle 5

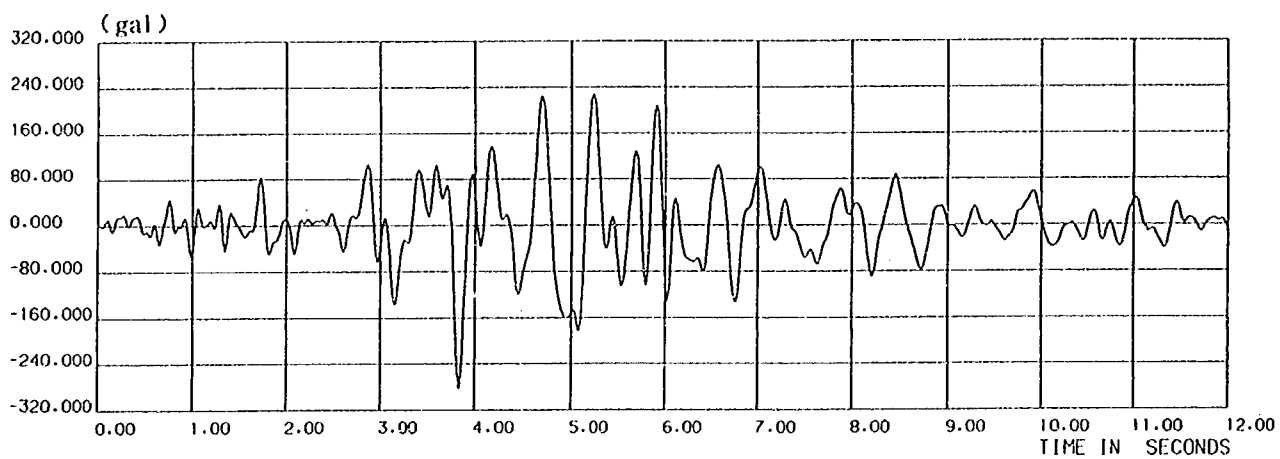


(6) Circle 6

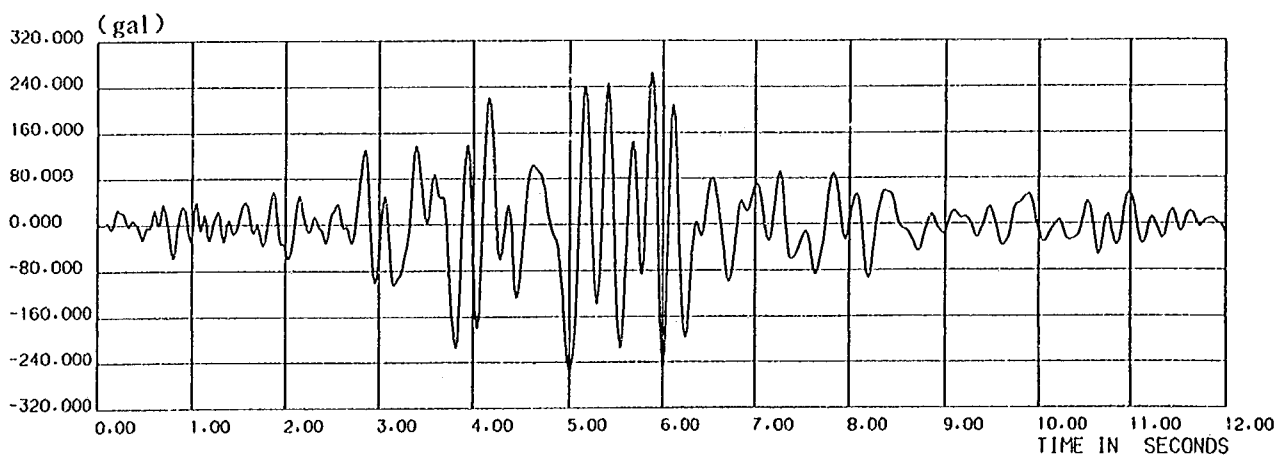
Fig.5.2.7 Time-history of mean response acceleration of slip circle (Case-B, Horizontal)



(1) Circle 1

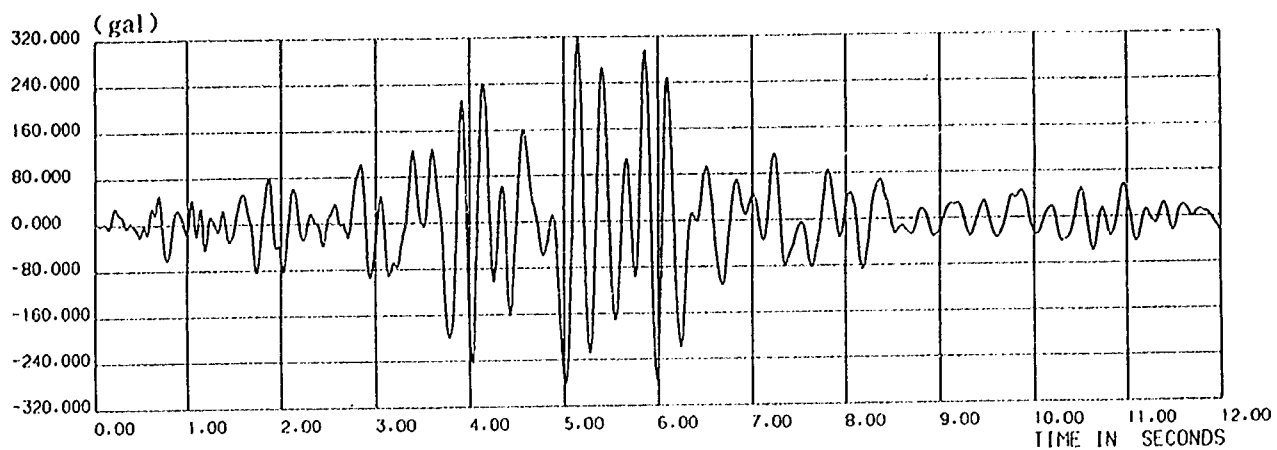


(2) Circle 2

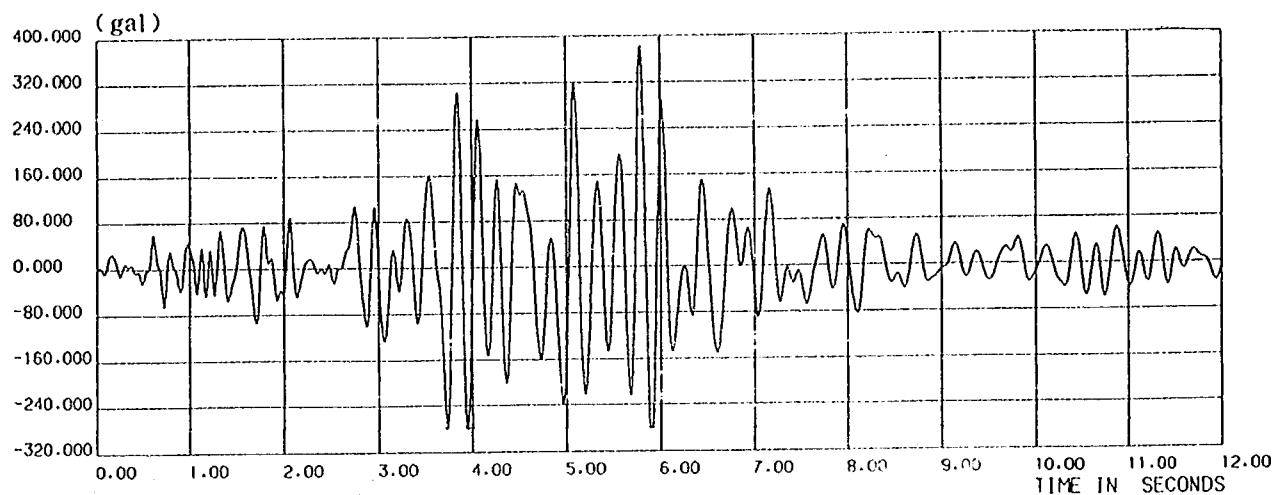


(3) Circle 3

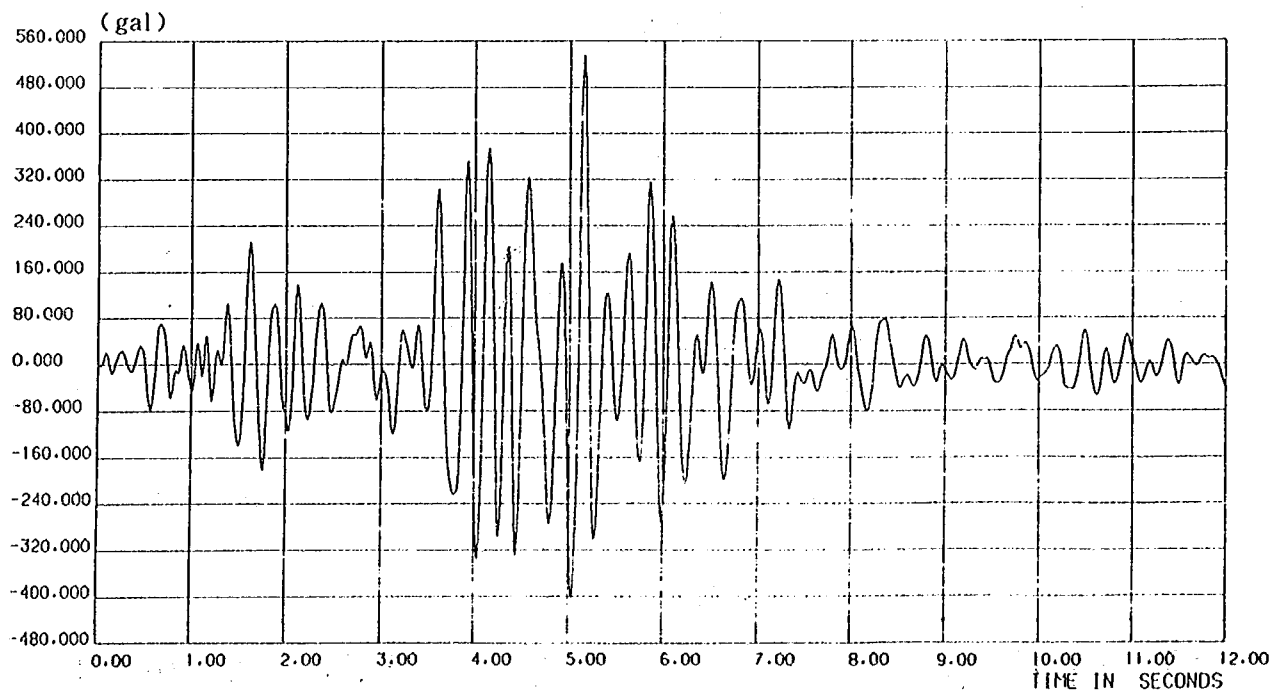
Fig.-5.2.8 Time-history of mean response acceleration of slip circle (Case-C, Horizontal)



(4) Circle 4



(5) Circle 5



(6) Circle 6

Fig.-5.2.8 Time-history of mean response acceleration of slip circle (Case-C, Horizontal)

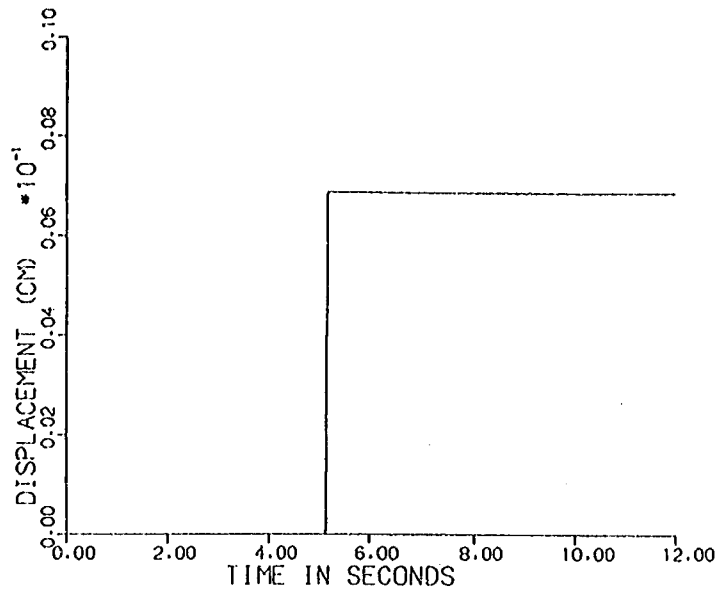


Fig.-5.2.9 Integration of movement of slip circle (Case-C, Circle-6)

response accelerations of a slip circle at each computational case are illustrated in Figs.- 5.2.6 to 5.2.8, however only the maximum mean acceleration of slip circle -6 for Case-C surpasses the critical acceleration, as illustrated in Table-5.2.1. The movement of the sliding circle is obtained in Fig.-5.2.9, which shows the relation of the displacement value to time. Sliding starts at 5.3 seconds, and then the total value is 0.007cm, as shown in Fig.5.2.9.

5.2.2 Stability of the Plane Sliding Surface

The total movements of the plane sliding surfaces are shown in Fig.-5.2.11. Eleven nodal points on the slope are totally studied, including points near the berms and the crest. The maximum value of movement for Case-C is 6cm, occurring at nodal point 134, and movement of less than 3cm is recognized at the other nodal points. Nodal point 134 is included in the above-mentioned circular sliding surface 6, the movement of which was

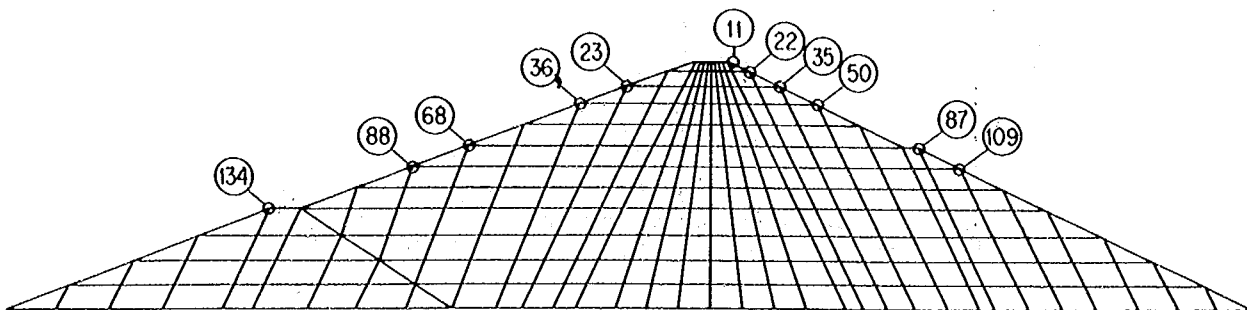
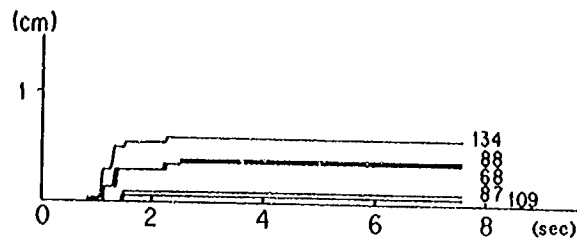
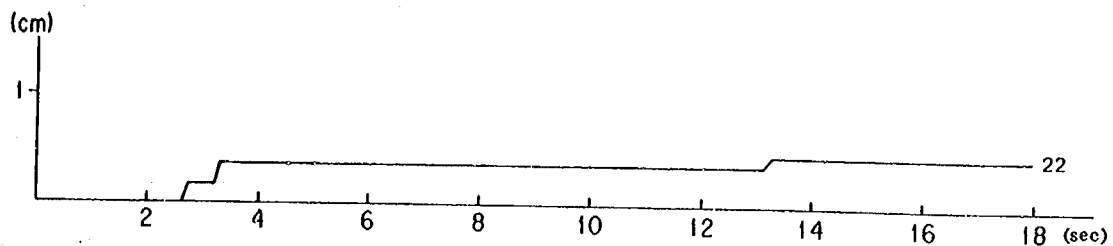


Fig.-5.2.10 Calculation of movement of surface sliding

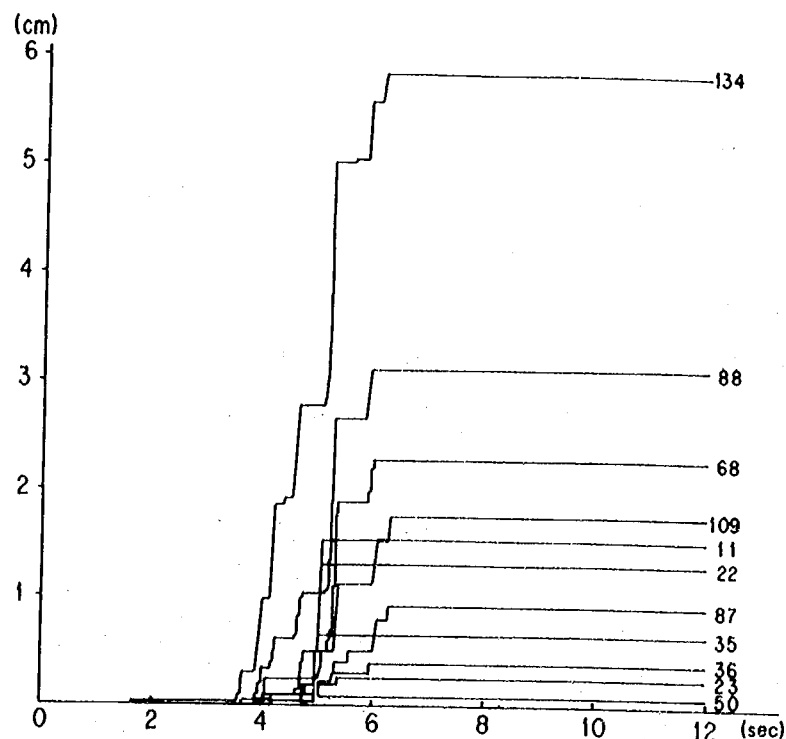
zero. Therefore, it is concluded that the movement at the berm of the upstream slope cannot extend deep. As for Case-A, the maximum movement also occurs at nodal point 134, however there is no problem regarding stability because the value is 5cm and very small.



(Case-A)



(Case-B)



(Case-C)

Fig-5.2.11 Movement of surface sliding

6. Conclusions

1. The maximum settlement of a dam body after embankment analysis ranges from 64.7cm to 82.8cm at each computational case.
2. A maximum principal stress of 12kgf/cm² is recognized at the base of the core and rock zone, and a minimum one of 6kgf/cm² occurs at the core base.
3. Vertical stress at the upstream base of the rock and filter zone decreases and becomes 70 to 80% after the reservoir filling, however no extensional stress occurs in the dam body.

Three design earthquakes were considered for the dynamic analysis from the survey of historical earthquakes at the dam site.

4. The first and second natural frequencies of the dam are 1.66Hz and 2.54Hz, respectively.
5. The maximum response acceleration tends to occur at the crest, up and downstream slope, and the response amplitude to the maximum input acceleration ranges from 1.2 to 2.5.
6. A maximum displacement of 3cm occurs at the crest in Case-C.
7. The absolute value of dynamic stress of an element during an earthquake is less than that of the static stress, so extensional stress seems not to occur.

The point safety factor of an element and the safety against sliding are investigated.

8. The point safety factor at the core base is slightly less than 1.0, but the safety factor of unity seems to be preserved, considering the cohesion of the core material.
9. The potential circular sliding surfaces are almost impossible to move.
10. As for the plane sliding surface, a maximum displacement of 6cm is recognized for Case-C, however it is out of the question for the stability of a dam body.

As mentioned above, the present cross-section of a model rockfill dam is stable against anticipated earthquake motions at the vicinity of the damsite.

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Key Words: Rockfill Dam, Fill Material, Shear Strength,
Shear Modulus, Triaxial Test, Dynamic Analysis

This model dam is a 93.0-m-high rockfill dam with a total storage capacity of 109 million m³ and a central core zone type. In this report an investigation on the stability of a rock-fill dam during earthquakes was carried out using dynamic analysis.

