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SYNOPSIS: The problems of selecting a reasonable generation model and a reasonable generation-dissipation model of pore water pressure and determining the ooefficient of permeability and the coefficient of volumetrio strain in the course of pore water pressure generation-dissipation are oruoial in the calculation of seismic pore water pressure. These problems are investigated theoretically and experimentally in this paper based on the oonoepts of transient pore pressure with connection to the practically possible calculating conditions, so as to improve the reliability of the current seismic effective stress analysing method.

INTRODUCTION

In analysing the seismic stability of an earth dam and a soil foundation the pore water pressure is generally determined according to its generating regulations obtained by some particular experiment under undrained condition with less consideration of the difference of soil density, consolidation stress state, seismic loading pattern and its intensity in practical condition. Meanwhile, the pore water pressure in its generation-dissipation condition is determined based on the assumption that a pore water pressure increment induced by seismic action under undrained condition in a short time interval is at first calculated and added to the residual pore water pressure at the beginning of this time interval, and then the dissipation caused by the seepage of void water occurs under this new pore water pressure at the end of the time interval, and finally the obtained pore pressure is considered the same as that in the generation-dissipation condition. In this way, the pore water pressure increment under the condition of generation-dissipation which takes place simultaneously in fact is represented by the summation of the generated pore pressure increment under undrained condition and the dissipated pore pressure increment under natural drained condition. In addition, no change in charactristics of permeability and volumetric strain of soil during earthquake is considered, i.e., the coefficient of permeability and the coefficient of volumetric strain are taken as a constant. It is obvious that to select a reasonable generation model of seismic pore water pressure under undrained condition and a reasonable generation-dissipation model of seismic pore water pressure under natural drained condition and to determine the coefficient of permeability and the coefficient of volumetric strain during earthquake are the key problems related to the reliability of the seismio effective stress analysis. In view of the particular importance of pore water pressure calculation in any effective stress analysis the experimental and theoretical analysis of these four problems are made in this paper.

REASONABLE SELECTION OF SEISMIC PORE WATER PRESSURE GENERATION MODEL

Basis of Model Selection

It has been for a long time that in research of the dynamic pore water pressure, more attention is paid only to the monotonously increasing property of average pore water pressure, and the pore water pressure generated at the end of every cycle of dynamic loading (at this time when dynamic stress is equal to zero) under the particular experimental condition is related to the number of cycle, shear strain, volumetric strain, consumed energy or other variables with increasing monotonously. The pore water pressure generation model is then established by imitating this kind of experimental relations with a mathematical equation. It can be seen from the regulations revealed by tests that the development of pore water pressure under the action of cyclic loading with constant stress amplitude has the different forms in Fig. 1.

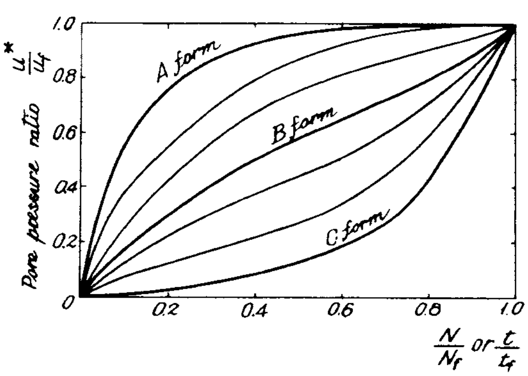


Fig. 1 The patterns of pore water pressure generation

Different forms of the curve group may be expressed by different pore water pressure generation models. If we name the upper, middle and lower parts of the curve group as forms A, B and C respectively, then the experiments show that which form of pore water pressure generation appears depends principally on the soil density, the consolidation stress and the intensity of dynamic loading. The form of pore pressure generation changes from A to B and to C when the soil density changes from the lower to the higher; or when the consolidation stress from isotropic consolidation to anisotropic consolidation; or when the intensity of dynamic loading from the higher to the lower while the other faotors remain constant in every above noted condition. It means that the form A often appears in the condition of lower density of soil, lower oonsolidation stress ratio and higher dynamic stress level. On the contrary, form C often appears in the oondition of higher density of soil, higher consolidation stress ratio and lower dynamic stress level. The forms of pore water pressure generation from A to C (close to form B) appear between the conditions form A and form C occur as mentioned above. The form of pore pressure generation under the action of cycle loading with constant strain amplitude is generally closed to the form A (Fig.2).

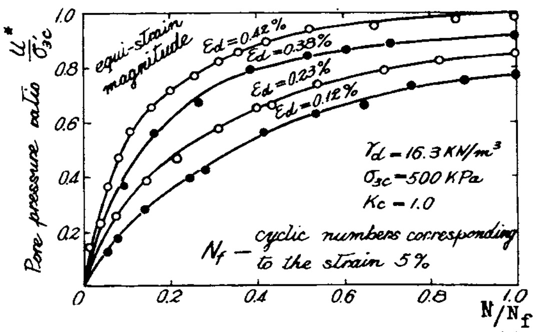


Fig. 2 The pattern of pore pressure generation under cyclic loading with constant-strain magnitude

As for the action of irregular dynamio loading, it can be divided into two patterns: the pattern of vibration and the pattern of impulsion (Ishihara et al, 1973). The pattern of impulsion is divided again into three patterns I, II and III according to the large amplitudes of a seismic wave appearing in the front, middle and back part of the earthquake process in time domain. In order to investigate the regulations of the pore pressure generation under irregular seismic loading, an irregular seismic loading generator is developed and used in dynamic triaxial test. Test results indicated that the development of pore pressure is not very obviously affected by the wave order of dynamic loading of vibration pattern. But under the dynamic loading of vibration pattern with higher stress amplitude and shorter vibration duration and under that with lower stress amplitude and longer vibration duration, the different patterns of seismic pore water pressure generation appear although these two kinds of dynamic loadings may be considered as the dynamic loading of equal intensity. The form of pore pressure generation transforms from the form A to B and to C as the dynamio stress amplitude changes from the higher to the lower while the vibration duration from the shorter to the longer. Moreover, the generating rate and developing regulation of seismic pore water pressure under the irregular dynamic loading of impulsion pattern are primarily depending on the amplitudes of large seismic waves and their position in the time array (Ishihara, et al, 1973; Chen, et al 1978; Xie, et al, 1987). Therefore, the three forms of pore pressure generation A, B and C can be used to imitate the pore pressure ohange process approximately in the conditions corresponding to that the position of large amplitude is located in the front, middle or back part of the seismio wave in time domain, respectively.

The above mentioned cognition for the regularity of pore water pressure generation under the condition of the regular and irregular cyclic loading action may be used as the basis of selecting the pore water pressure generation model.

Suggested Model of Pore Pressure Generation

Based on the examination of the current 25 models of pore pressure generation (Zhang and Xie, 1990), it is found that the above indioated three forms of pore pressure generation A, B and C can be expressed by the following models

$$u^* = u_f\left(1 - e^{-\beta\frac{t}{t_f}}\right) \qquad (\text{A form})$$

$$u^* = \frac{2}{\pi} u_f \sin^{-1}\left(\frac{t}{t_f}\right)^{\frac{1}{2\alpha}} \qquad (\text{B form}) \qquad (1)$$

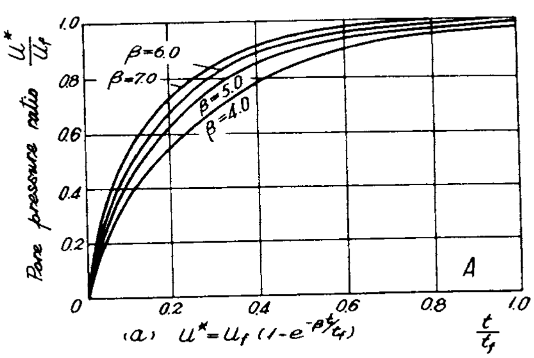
$$u^* = u_f\left[\frac{1}{2}\left(1 - \cos\pi\frac{t}{t_f}\right)\right]^b \qquad (\text{C form})$$

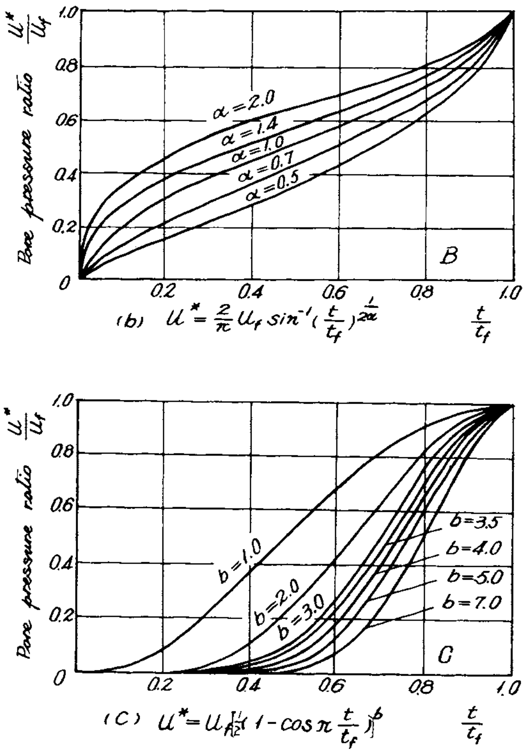
where β, α, b = the calculating parameters; u_f=the ultimate pore water pressure, i.e., the maximum value of pore pressure during earthquake; t_f=vibration duration oorresponding to the pore pressure u_f. In using these models it is necessary to judge preliminarily whioh model should be selected according to the possible seismic wave in practical condition, and then to verify the selected model and determine the calculating parameters in the light of the specific form of pore pressure generation measured by the test conducted under the possible seismic wave. This procedure is able to consider at a oertain extent the effect of the order of seismic wave on the pore pressure generation.

It should be pointed out that the formula (1) is different from the usually used expression $u^*/\sigma_0 - N/N_f$. The ratio u^*/σ_0' is replaced by the ratio u^*/u_f, and the ratio N/N_f is replaced by the ratio t/t_f. These replacements can consider, in addition to the effect of dynamic stress, the effect of more general factors, such as the different criteria of failure and the anisotropic consolidation condition.

Determination of Parameters in Selected Models

The values of the calculating parameters β, α and b in Eq.(1) depend on the pore pressure generation forms. They can be determined by imitating the measured prooess of pore pressure development from the tests conducted under the condition of the possible seismic loading action. The range of these parameters that may usually be taken is shown in Fig. 3.





$$\text{(b)}\quad u^{*} = \frac{2}{\pi} u_f \sin^{-1}\left(\frac{t}{t_f}\right)^{\frac{1}{2\alpha}} \qquad \frac{t}{t_f}$$

$$\text{(C)}\quad u^{*} = u_f\left[\frac{1}{2}\left(1-\cos\pi\frac{t}{t_f}\right)\right]^{b} \qquad \frac{t}{t_f}$$

Fig. 3 Theoretical curves of pore pressure generation models

The value of the ultimate pore water pressure u_f depends on the soil density, the practical stress-strain condition, the seismic stress intensity and the evaluation criterion. The maximum value of pore water pressure generation during earthquake is taken as u_f here. The two coefficients of m_s and m_L are drawn into the expression of u_f to consider the effect of stress-strain condition and the effect of seismic stress intensity respectively, i.e.

$$u_f = m_s m_L \sigma_o \tag{2}$$

where σ_o= the initial effective spherical stress, such as for dynamic triaxial test condition

$$\sigma_o = \frac{1}{3}\left(\sigma'_{1c} + 2\sigma'_{3c}\right) = \frac{1}{3}\left(K_c + 2\right)\sigma'_{3c} \tag{3}$$

and for dynamic simple shear test condition

$$\sigma_o = \frac{1}{3}\left(\sigma'_{vo} + 2\sigma'_{ho}\right) = \frac{1}{3}\left(1+2K_o\right)\sigma'_{vo} \tag{4}$$

where σ'_{1c} and σ'_{3c} are the maximum and minimum effective consolidation principal stresses respectively ; K_c = the consolidation stress ratio ($=\sigma'_{1c}/\sigma'_{3c}$); and K_o = the coefficient of earth pressure at rest.

(1). Coefficient of stress-strain condition m_s

The stress-strain condition refers to the static and dynamic stress condition acting on the soil specimen and the strain condition of soil specimen under the action of the static and dynamic stress.

In analysing from the stress condition it is seen from the numerously tested results that the saturated sand can liquefy at last in isotropically consolidated condition ($\sigma'_{1c}=\sigma'_{3c}$) as long as the dynamic loading action intensity is large enough. At the moment when sand specimen is in a state of liquefaction, the effective stress and the static and dynamic shear stress are equal to zero, and the ultimate pore pressure u_f reaches its maximum value equal to the confined shperical stress in liquefaction σ_{Lo} in magnitude and also equal to the initial effective consolidation stress σ_o. i.e., $u_f=\sigma_{Lo}=\sigma_o$ in isotropically consolidated condition. The coefficient of stress-strain condition m_s=1 in this case; Otherwise, whether or not u_f could reach its maximum value σ_{Lo} depends on the dynamic loading intensity and its acting path. If the dynamic triaxial test is done in anisotropically consolidated condition ($\sigma'_{1c}\neq\sigma'_{3c}$), then the soil specimen will liquefy and u_f is equal to σ_{Lo} ($=\sigma'_{3c}$) when dynamic stress $\sigma_d \geq (\sigma'_{1c}-\sigma'_{3c})$, but won't liquefy when $\sigma_d < (\sigma'_{1c}-\sigma'_{3c})$ and u_f develops to its final level u_f/σ'_{3c} that decreases as the consolidation stress ratio K_c increases (Fig.4). The relation between u_f and K_c follows approximately the expression

$$u_f = \left[1-\eta\left(K_c-1\right)\right]\cdot\sigma'_{3c} \tag{5}$$

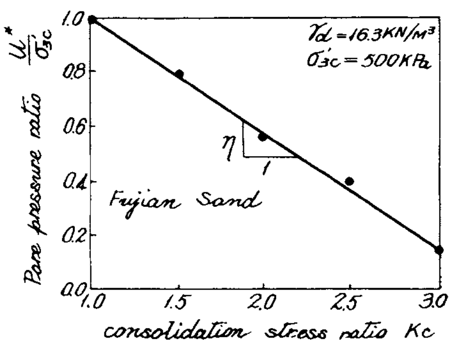


Fig. 4 Effect of K_c on pore pressure generation

In analysing from the strain condition, it can be seen from the dynamic triaxial test that the static maximum principal stress σ_1 decreases from σ'_{1c} to the initial static minimum principal stress σ'_{3c} as long as the dynamic stress intensity is large enouth to make the saturated sand sample liquefy because the soil sample is allowed to deform laterally. The consolidation stress ratio K_c decreases to 1, and $u_f=\sigma_{Lo}=\sigma'_{3c}$ in this case. But in the dynamic simple shear test condition, the static minimum principal stress σ_h increases from σ'_{ho} to the initial static maximum principal stress σ'_{vo} because the soil sample is completely restricted to deform laterally. $K_o=K_o$=1 and $u_f=\sigma_{Lo}=\sigma'_{vo}$ in liquefied state. It can be reckoned that under the condition of the partially limited lateral deformation, the static minimum principal stress σ_3 increases in company with the increasing of the static maximum principal stress σ_1 as the pore pressure level increases, thus both of them approach to the confined spherical stress in liquefaction σ_{Lo}. This indicates that the value of σ_{Lo} changes in the range between the initial static minimum and maximum principal stresses depending on the degree to what the lateral strain of sand sample is restricted.

It is known from the above mentioned analysis that the ultimate pore pressure u_f may be equal to or less than the confined spherical stress in soil liquefied condition σ_{Lo} depending on the stress-strain condition. The coefficient of stress-strain condition m_s in dynamic triaxial test can be obtained from formulae (2), (3) and (5) as follows

$$m_s = \begin{cases} \frac{3}{K_c+2} & (\sigma_d \geqslant \sigma'_{1c} - \sigma'_{3c}) \\ [1-\eta(K_c-1)]\frac{3}{K_c+2} & (\sigma_d < \sigma'_{1c} - \sigma'_{3c}) \end{cases} \qquad (6)$$

and that in dynamic simple shear test can be obtained from formulae (2) and (4)

$$m_s = \frac{3}{1+2K_o} \qquad (7)$$

It is worth doing further research on how to determine the coefficient m_s in condition between the above-mentioned two tests, i.e., the condition of partially limited lateral deformation of sand sample.

(2). Coefficient of seismic stress intensity m_L

The intensity of dynamic stress action is measured by the three essential factors: amplitude, frequency, and duration in general. In order to investigate the regulations of pore pressure change under the **different** intensity of dynamic stress action, it is a better way to change the amplitude while fixing the frequency and the duration for regular dynamic loading condition, and to enlarge or reduce the amplitude proportionally along time coordinate axis while fixing the duration and the order of seismic wave for irregular dynamic loading condition. It is found from the tests that the final level of seismic pore water pressure decrease as the decreasing of dynamic stress intensity, so the coefficient of dynamic stress intensity m_L also decreases and $0 \leqslant m_L \leqslant 1$. The relation between the ultimate pore water pressure level u_f/σ'_{3c} and the maximum dynamic stress ratio $\sigma_{dmax}/\sigma'_{3c}$ measured by the triaxial test under irregular loading (the duration of seismic action is seven seconds)(Fig. 5) may be expressed by a hyperbolic equation

$$\frac{\sigma_{d.max}}{\sigma'_{3c}} = \frac{\frac{u_f}{\sigma'_{3c}}}{a+b(\frac{u_f}{\sigma'_{3c}})} \qquad (8)$$

This equation can be transformed into

$$u_f = \frac{\sigma_{d.max}}{C_1 - C_2\,\sigma_{d.max}}\,\sigma'_{3c} \qquad (9)$$

Then the coefficient of seismic stress intensity is obtained from formulae (2) and (6)

$$m_L = \frac{\sigma_{d.max}}{C_1 - C_2\,\sigma_{d.max}} \qquad (10)$$

where $C_1 = \sigma'_{3c}/a$, $C_2 = \frac{b}{a}$, they are the test parameters. Their values depends on the soil density, the consolidation stress, the pattern of seismic wave and the duration of seismic action.

For the condition of horizontal saturated sand layer (the one-dimensional foundation), the seismic shear

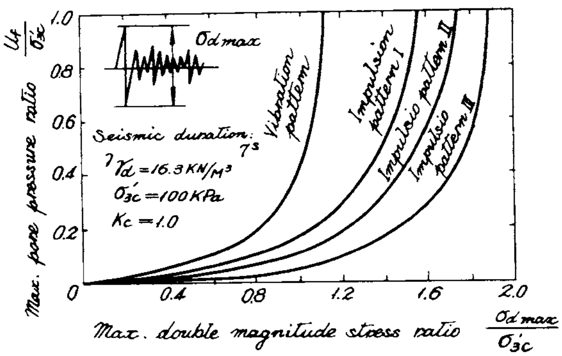


Fig. 5 Relation between u_f and dynamic loading intensity

stress on the horizontal plane may be imitated by the shear stress on the 45° plane of the specimen in dynamic triaxial test, and $\tau_d = \frac{1}{2}\sigma_d$. So Eq.(10) can be written as

$$m_L = \frac{2\tau_{d.max}}{C_1 - 2C_2\tau_{d.max}} = \frac{\tau_{d.max}}{C'_1 - C_2 \cdot \tau_{d.max}} \qquad (11)$$

where $C'_1 = \frac{1}{2}C_1$. In this condition, the ultimate seismic pore pressure u_f can be determined based on Eq.(2), Eq.(7) and Eq.(11) as

$$u_f = \frac{\tau_{d.max}}{C'_1 - C_2\tau_{d.max}} \cdot \frac{3}{1+2K_o} \cdot \sigma_o \qquad (12)$$

In addition, the effective range of the formula (10): $0 \leqslant \sigma_{d.max} \leqslant \frac{1}{b}\sigma'_{3c}$ on the basis of considering the property of the function in Eq.(8), and $m_L=1$ when $\sigma_{d,max} > \frac{\sigma'_{3c}}{b}$. Similarly, the effective range of Eq. (11) can also be determined.

MODEL OF SEISMIC PORE PRESSURE GENERATION-DISSIPATION AND VERIFICATION OF ITS REASONABLENESS

Theoretical Analysis of Generation-Dissipation Model

Let's take a differential element with the coordinates of its geometric center (x,y,z). Supposing that the soil is fully saturated, the compressibility of void water and soil particles is neglectable, and the seepage of void water obeys the Darcy's Law, then equation of liquid continuity can be established

$$\frac{\partial}{\partial x}\left(\frac{K_x}{\gamma_w} \cdot \frac{\partial u}{\partial x}\right) + \frac{\partial}{\partial y}\left(\frac{K_y}{\gamma_w} \cdot \frac{\partial u}{\partial y}\right) + \frac{\partial}{\partial z}\left(\frac{K_z}{\gamma_w} \cdot \frac{\partial u}{\partial z}\right) = \frac{\partial \varepsilon_v}{\partial t} \qquad (13)$$

where K_x, K_y, and K_z = the coefficients of permeability of soil in the directions of x,y,z, respectively; γ_w = unit weight of water; ε_v = the volumetric strain that is positive when the volume of soil decreases; $\frac{\partial u}{\partial x}$, $\frac{\partial u}{\partial y}$, and $\frac{\partial u}{\partial z}$ = the pore water pressure gradient in the directions of x,y, and z, respectively.

Considering the mechanism of transient change of pore water pressure (Xie and Zhang, 1990), the pore water pressure in saturated sand under dynamic loading can be divided into the stress pore pressure (u_σ), the structure pore pressure (u_c), and the transmission pore pressure (u_T). Their definitions can briefly be mentioned as follows. u_σ is a recoverable pore water pressure corresponding to a certain elastic volumetric strain potential that the skeleton of saturated soil possesses under undrained condition; u_c is an irreversible pore water pressure conrresponding to a certain non-elastic volumetric strain potential that the skeleton of saturated soil possesses under undrained condition and its magnitude depends mainly upon the degree to what the structure of soil skeleton has been destructed; u_T is the pore water pressure caused by the variation of soil skeleton volumetric strain potential as a result of the void water seepage (diffusion or dissipation) under natural drained condition and its magnitude depends mainly upon the pore water pressure gradient, the drainge condition, and the drainage duration. In general, the pore water pressure u^* generating in saturated sand under the seismic loading action and the fully undrained condition at a certain moment must be the result coupling the stress pore pressure u_σ with structure pore pressure u_c at the corresponding moment. Its incremental expression in a time interval δt is

$$\delta u^* = \delta u_\sigma + \delta u_c \tag{14}$$

The pore water pressure increment δu in saturated sand under the seismic loading action and the partially drained condition must be the result coupling δu^* with the transmission pore pressure increment δu_T, i.e.

$$\delta u = \delta u^* + \delta u_T = \delta u_\sigma + \delta u_c + \delta u_T$$

or

$$\delta u_T = \delta u - \delta u^* \tag{15}$$

Assumption that the change of total stress in the saturated soil element are neglectable, then only the transmission pore pressure δu_T in δu can cause the volumetric strain $\delta \varepsilon_v$ under the partially drained condition, i.e.

$$\delta \varepsilon_v = m_v \delta u_T = m_v (\delta u - \delta u^*) \tag{16}$$

where: m = the coefficient of volumetric strain of soil, and it means that the volumetric strain change caused by the unit change of transmission pore pressure.

Taking the partial differential with time to Eq.(16), we can obtain

$$\frac{\partial \varepsilon_v}{\partial t} = m_v \frac{\partial u_T}{\partial t} = m_v \left(\frac{\partial u}{\partial t} - \frac{\partial u^*}{\partial t} \right) \tag{17}$$

Substituting Eq. (17) into Eq.(13), we have

$$\frac{\partial}{\partial x} \left(\frac{K_x}{\gamma_w} \cdot \frac{\partial u}{\partial x} \right) + \frac{\partial}{\partial y} \left(\frac{K_y}{\gamma_w} \cdot \frac{\partial u}{\partial y} \right) + \frac{\partial}{\partial z} \left(\frac{K_z}{\gamma_w} \cdot \frac{\partial u}{\partial z} \right) = m_v \left(\frac{\partial u}{\partial t} - \frac{\partial u^*}{\partial t} \right) \tag{18}$$

Eq. (18) can be used to calculate the seismic pore pressure generation and dissipation in saturated sand. This equation is essentially the Terzaghi's consolidation theory coupling with the seismic pore pressure generation model under the undrained condition. If this equation is only used to calculate the residual pore pressure change, i.e., the average change of the accumulated structure pore pressure, then the suggested model (1) can be selected as formula (14).

Experimental Verification of Pore Pressure Generation -Dissipation Model

Six programs of dynamic triaxial tests were conducted with saturated Fujian standard sand to investigate the effect of the different patterns and condition of pore pressure generation-dissipation on the pore pressure. These programs planed by considering the practically possible combination of pore pressures of different formations are (a). the program of pore pressure dissipation in condition of consolidation; (b). the program of pore pressure diffusion and dissipation in condition without structure pore pressure; (c). the program of pore pressure dissipation in condition of different structure pore pressure levels; (d). the program of pore pressure diffusion in condition of different structure pore pressure levels; (e). the program of pore pressure dissipation in condition of different initial structural pore pressure levels with addition of different transmission pore pressure increment (diffusion) and (f). the program of pore pressure dissipation in condition of different initial transmission pore pressure levels with addition of the structure pore pressure to make soil liquefied. In these different programs of tests the structure pore pressure is formed by static stress (initial consolidation) or cyclic loading (vibration); the transmission pore pressure is formed by increasing the back pressure (diffusion) or decreasing the back pressure (dissipation). The relations between pore pressure change (increasing and decreasing) and the change of the corresponding volumetric strain for all the these programs can be determined and plotted on the coordinate plane with the zero pore pressure as the origin of coordinate and the increasing of transmission pore pressure and the corresponding increasing of volumetric strain as the positive direction of two coordinate axes respectively. In this way, the volumetric strain corresponding to a certain pore pressure level u/σ'_{3c} represents the volumetric strain potential of saturated soil skeleton under this pore pressure level. It can be called as the soil skeleton volumetric strain potential (notes as $\bar{\varepsilon}_v$). The change curves of u/σ'_{3c} with respect to $\bar{\varepsilon}_v$ in their accumulated values are shown in Fig. 6-12. Comparing and analysing the experimental curves in these figures, we can draw the following conclusions:

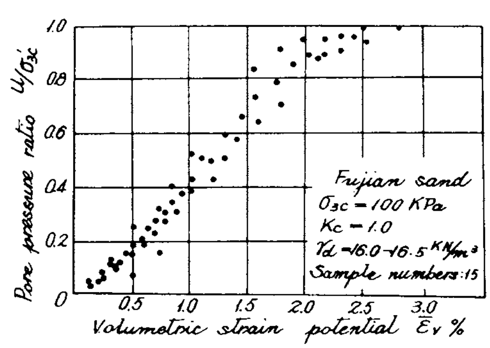


Fig. 6 Relation between $\frac{u}{\sigma_{3c}}$ and $\bar{\varepsilon}_v$ in consolidation process

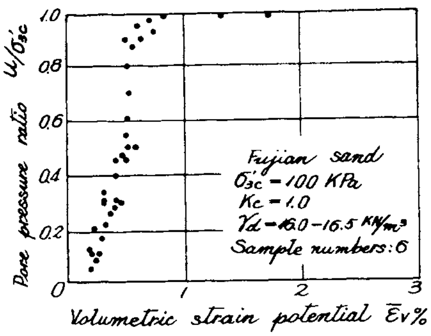


Fig. 7 Relation between $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in diffusion process without the structure pore pressure

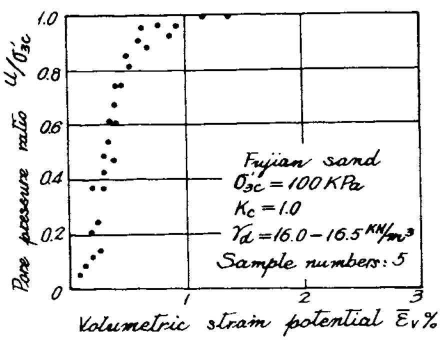


Fig. 8 Relation between $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in dissipation process without the structure pore pressure

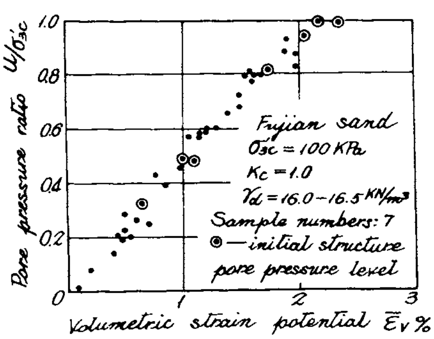


Fig. 9 Relation bwtween $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in dissipation process under condition of different initial structure pore pressure

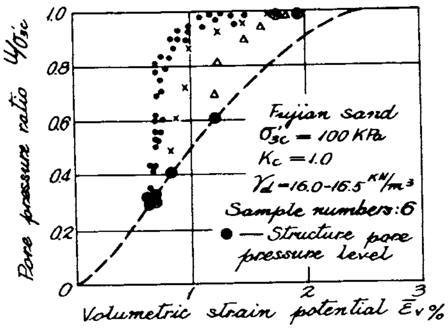


Fig. 10 Relation between $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in diffusion process under condition of different intitial structure pore pressure

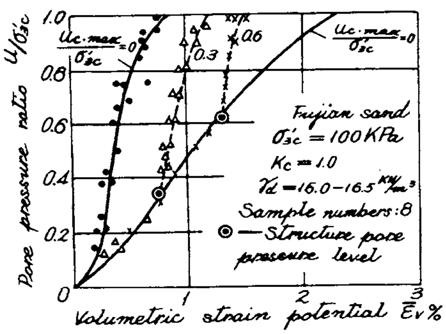


Fig. 11 Relation between $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in dissipation process after diffusion

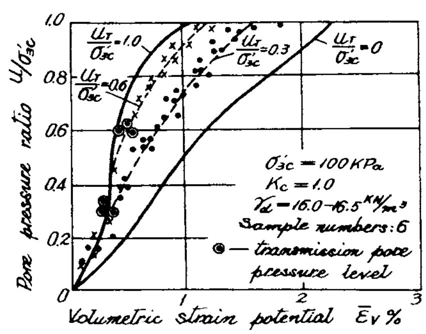


Fig. 12 Relation between $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in dissipation process after liquefaction formed by vibration under different levels of u_T/σ'_{3c}

(1). The dissipation curve (i.e., pore pressure-volumetric strain curve) of structure pore pressure due to consolidation (Fig.6) coincides approximately with that due to vibration (Fig.9). This indicates that the dissipation curve of structure pore pressure is independent of the causes of structure pore pressure formation;

(2). The dissipation curves (Fig.9) in condition of different structure pore pressure levels coincides approxi mately. This indicates that the dissipation regulations are not affected by the structure pore pressure level from which the dissipation begins;

(3). The dissipation curves in condition of different initial structure pore pressures with addition of a certain transmission pore pressure increment run along the same path (Fig. 11) as that without the additional transmission pore pressure increment (Fig. 9) after they dissipate and reach the initial structure pore pressure level. This indicates that the diffusion in the course of dissipation does not affect the original regulation of dissipation;

(4). The dissipation curve (Fig. 8) and the diffusion curve (Fig. 7) in the condition without structure pore pressure coincide approximately and the volumetric strain in this condition is much less than that in the condition with structure pore pressure (Fig. 9 and Fig. 10). This indicates that the dissipation and diffusion appears only a lower elastic property when there is no further structure destruction of soil skeleton;

(5). The diffusion curve and the dissipation curve after diffusion under the condition of different initial structure pore pressure levels have the approximately parallel charaterisitics each other (Fig.10 and Fig. 11). This indicates that the initial structure pore pressure level and the initial soil density have less effect on the pore pressure change in this case;

(6). The dissipation curves in condition of different transmission pore pressure level with addition of structure pore pressure due to vibration (Fig. 12)are approximately parallel. This indicates that the existence of transmission pore pressure does not affect the dissipation property of the structure pore pressure.

It can be seen from the above-mentioned that the dissipation of the structure pore pressure and the dissipation and diffusion of the transmission pore pressure have their own independent regulations that do not affect approximately each other. Therefore, it is tenable that the value of pore pressure increment at the end of a small time interval is not affected by the dissipation path in this time interval. That is, the conventional physical model of pore pressure generation-dissipation is available.

DETERMINATION OF COEFFICIENT OF VOLUMETRIC STRAIN UNDER DYNAMIC LOADING CONDITION

The coefficient of volumetric strain in the Eq. (18)is charaterized by the volumetric strain caused by the unit transmission pore pressure, i.e.

$$m_v = \frac{d\,\varepsilon_v}{du_T} \qquad (19)$$

The value of m_v can be determined from the $\frac{u}{\sigma'_{3c}} - \bar{\varepsilon}_v$ relation (Fig. 6 - 12) obtained in the process of dissipation and diffusion under the condition of combinations of pore pressures with different cause of formation. It depends on the changing rate of ε_v with respect to u/σ'_{3c}, i.e., the reciprocal of the slope of $u/\sigma'_{3c} - \bar{\varepsilon}_v$ relation curve.

In virture of that the $\frac{u}{\sigma'_{3c}} - \bar{\varepsilon}_v$ relation curves in the process of dissipation and diffusion are quantitatively close or different each other depending on the regulations of combination of the pore pressures with different causes of formation, the method for determining m_v should be selected based on the specific condition of calculation. The calculating conditions that must often run into in the practical calculations of seismic pore water pressure generation-dissipation are the pore pressure dissipation caused by consolidation prior to the earthquake; the dissipation and diffusion of structure pore pressure both during and after the earthquakes. The determination of m_v in these indicated conditions are discussed as follows.

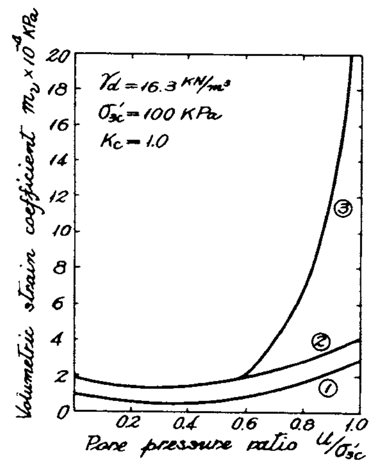


Fig. 13 Relation between m_v and $\frac{u}{\sigma_{3c}}$ under the different calculation condition

1. In considering the pore pressure diffusion, the pore pressure - volumetric strain relations for any of these conditions coincide with the diffusion curve in condition without structure pore pressure (Fig. 7).The $m_v - u/\sigma'_{3c}$ relation (curve ① in Fig. 13) can be obtained by transforming the Fig. 7. It shows that the value range of $m_v = 3.3\times10^{-5} - 5.0\times1.0^{-5}KP^{-1}$ that is close to constant;

2. The value of m_v can be equally adopted as above for both the pore pressure dissipation and the pore pressure diffusion in the condition without structure pore pressure because the pore pressure-volumetric strain relations under this two conditions are coincident (Fig. 7 and Fig. 8);

3. The $m_v - u/\sigma'_{3c}$ relation curves (Curve ③ in Fig.3) of structure pore pressure dissipation caused by the consolidation stress and the cyclic loading can be obtained by transforming the Fig. 9 (for cyclic loading) which coincides with the pore pressure-volumetric strain curve in Fig. 6 (for consolidation stress). It has to be noted that the dissipation tests in different structure pore pressure levels are conducted with the procedures: (a). soil specimen consolidation; (b). cyclic loading to cause the structure pore pressure with different stress levels in undrained condition; and (c). dissipation under the static stress (consolidation stress) and measurement of the corresponding volumetric strain in process of dissipation. Therefore,the value of m_v obtained in these tests can be used in calculating the dissipation of structure pore pressure after the earthquake;

4. The $m_v - u/\sigma'_{3c}$ relation can be obtained by transforming the $u/\sigma'_{3c} - \bar{\varepsilon}_v$ relation (Fig. 14) in the

test conducted with the same procedure as that in the third point above described but the pore pressure dissipation and the corresponding volumetric strain measurement are in progress in company with the continuous vibration action. The measured volumetric strain in this kind of test is much higher than that in static dissipation (comparing Fig. 9 with Fig.14). The $m_v - u/\sigma'_{3c}$ relation of dynamic dissipation (curve (3) in Fig. 13) shows that the change range of m_v is $1.7\times10^{-4} - 2.0\times10^{-4}$ KPa^{-1} that is essentially equal to that of static dissipation when the pore pressure level $u/\sigma'_{3c} < 60\%$, but it is clearly higher than that of static dissipation when the pore pressure level $u/\sigma'_{3c} > 60\%$.

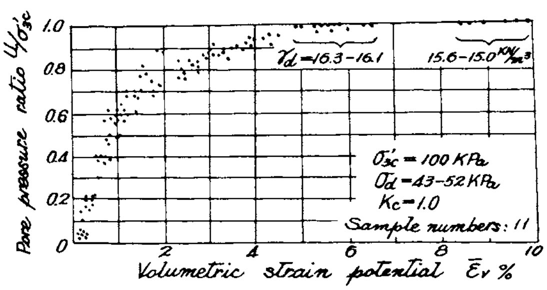


Fig. 14 Relation between $\frac{u}{\sigma'_{3c}}$ and $\bar{\varepsilon}_v$ in dynamic dissipation process

If we take the mean value of the $m_v - u/\sigma'_{3c}$ curve (close to a straight line) when u/σ'_{3c} is less than 60% and define it as a coefficient of initial volumetric strain m_{vo}, then the coefficient in pore pressure diffusion or dissipation without structure pore pressure is equal to 4.15×10^{-5} kPa^{-1} for the Fujian saturated sand under any calculating condition and the coefficient in pore pressure dissipation during and after earthquake or under consolidation stress is equal to 1.85×10^{-4} KPa.

In order to consider the effect of soil density on the value of m_v, the research results (Seed, et al 1976) may be made use of. Their study shows that the soil density has less effect on m_v in the lower pore pressure level, but it has obvious effect when pore pressure level is higher than 60%. A mathematical expression denoting the increasing property of m_v with the pore pressure level u/σ'_{3c} and soil density is suggested

$$m_v = \frac{m_{vo} \, e^{-A\left(u/\sigma'_{3c}\right)^B}}{1+A\left(\frac{u}{\sigma'_{3c}}\right)^B + \frac{1}{2}A^2\left(\frac{u}{\sigma'_{3c}}\right)^{2B}} \qquad (20)$$

where m_{vo} = the coefficient of initial volumetric strain, i.e., the m_v in the lower pore pressure level. A and B = the calculating parameters, $A=5(1.5 - D_r)$, $B=3-2\,D_r$.

The experimental data of different calculating conditions and soil density on the value of m_v are very limited, so the further study needs doing.

In addition, it is necessary to select the approciate test apparatus and the reasonable experimental method so as to have the stress-strain condition as close to the practical condition as possible because the $u/\sigma'_{3c} - \bar{\varepsilon}_v$ relations are quantitatively different under the different stress-strain condition.

DETERMINATION OF COEFFICIENT OF SOIL PERMEABILITY UNDER DYNAMIC LOADING

The coefficient of permeability K of saturated sand in condition of steady seepage (in which the relative slide of soil particles doesn't occur and the volumetric strain potential of saturated soil skeleton is equal to zero) depends principally upon the void ratio or the porosity of soil and the temperature. The following expression is often suggusted in general textbook of soil mechanics

$$K = C\cdot\frac{e^3}{1+e}\cdot\frac{\gamma_w}{\eta}\cdot d_{50}^2 = C\cdot\frac{n^3}{(1-n)^2}\cdot\frac{\gamma_w}{\eta}\cdot d_{50}^2 \qquad (21)$$

or

$$K = C\cdot f(e)\cdot\frac{\gamma_w}{\eta}\cdot d_{50}^2 = C\cdot f(n)\cdot\frac{\gamma_w}{\eta}\cdot d_{50}^2$$

where K = the coefficient of permeability; e and n = the void ratio and the porosity of soil respectively, f(e) or f(n) = the functions of void ratio or porosity. It can be determined from tests; γ_w = the unit weight of water; η = the coefficient of dynamic viscosity of water, it can be taken from the related tables; d_{50} = the mean diameter of soil particles ; C = the coefficient of mean diameter determined from tests.

In the condition of dynamic loading action, the coefficient of permeability is not a constant because the seepage goes in company with the relatively sliding of soil partiles and the changing of soil skeleton volumetric strain potential. Assumption that the relation between the coefficient of permeability K and the porosity n of soil in condition of without structure pore pressure during the increasing and decreasing of transmission pore pressure accompanied simultaneously by the dilation and compression of soil volumetric strain still satisfy the formula(21), then the coefficient of permeability K in condition with structure pore pressure can be determined by the following expression

$$K = Cd\cdot C\cdot\frac{(n_o-\delta n)^3}{(1-n_o+\delta n)^2}\cdot\frac{\gamma_w}{\eta}\cdot d_{50}^2$$

$$= Cd\cdot C\cdot\frac{(n_o-\varepsilon_v)^3}{(1-n_o+\varepsilon_v)^2}\cdot\frac{\gamma_w}{\eta}\cdot d_{50}^2 \qquad (22)$$

where n_o = the initial porosity of sand; Cd = the coefficient denoting the effect of dynamic loading on seepage. The value of Cd depends on the corresponding calculating condition and the pore pressure level u/σ'_{3c} (or the corresponding volumetric strain potential $\bar{\varepsilon}_v$). Owing to that the coefficient Cd is equal to 1 in the condition of pore pressure diffusion of any deformation causes and in the condition of pore pressure dissipation without structure pore pressure. The coefficient Cd in the condition of dynamic dissipation and static dissipation with structure pore pressure can be obtained by the curve (1) and the curve (2) in Fig. 15 respectively for a certain pore pressure level u/σ'_{3c} (or volumetric strain potential). ε_v = the accumulated volumetric strain of soil which can be obtained by determining the volumetric strain increment $\delta\varepsilon_v$ of every calculating interval divided in all the considered time domain and then accumulating them step by step, i.e., $\varepsilon_v = \Sigma\delta\varepsilon_v$. In addition, $\delta\varepsilon_v < o$ for diffusion and $\delta\varepsilon_v > o$ for dissipation.

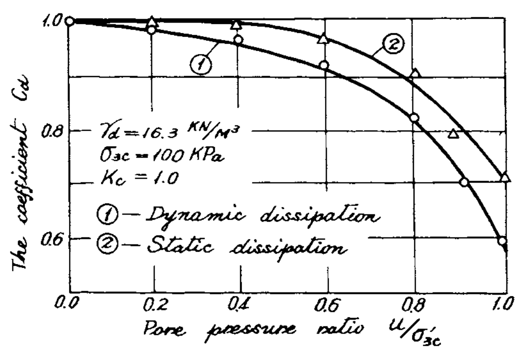


Fig. 15 Variation of Cd with $\frac{u}{\sigma_{3c}}$

In order to obtain the relations shown in Fig. 15,the dissipation curve (Fig. 8) in condition of without structure pore pressure is at first used to calculate the regulation of coefficient of permeability versus the transmission pore pressure by Eq.(22)with Cd=1; Then the Cd - u/σ'_{3c} relations (the curve (1)and the curve (2) in Fig. 15) are obtained by solving Eq.(22) on the basis of using the dissipation curves in condition of static dissipation (Fig. 9) and dynamic dissipation (Fig. 14) of structure pore pressure and taking the K - u_T relation obtained just in the prior step as the foundation. Here the Cd- u/σ'_{3c} relations in Fig. 15 are corresponding to the Eq.(22) in which f(e) is used in calculation with the first power of void ratio e because the K - e curve of saturated Fujian standard sand adopted in the tests appears in a straight line (Fig. 16).

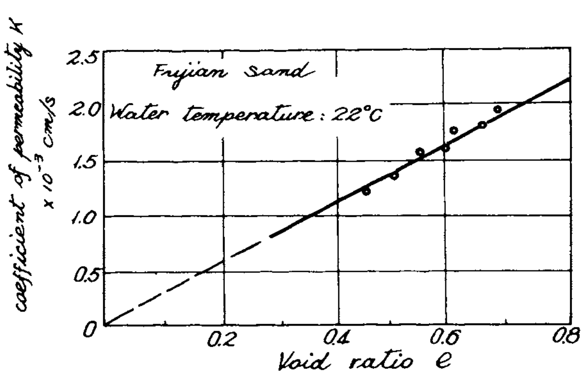


Fig. 16 Relation between K and e

It is obvious that the further work needs to be done so that the changing regulation and practical range of Cd value can be summarized in accordance with the different calculating condition and soil properties.

END WORDS

In this paper some key problems in calculating the seismic pore pressure generation-dissipation are more deeply investigated theoretically and experimentally. The approach of solving these problems and methods of calculation are suggested and also used in calculating the seismic pore pressure generation-dissipation of a saturated sand layer (Zhang and Xie, 1991). It can be noticed that the further complement of the suggested method will improve the reliability of the calculation by the current seismic effective stress analysing method.

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