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INTRODUCTION

We have already described (vol. 1, p.75) the laboratory triaxial test results about the mechanical behavior of cohesive soil under repeated loading. The results of undrained repeated loading test with a constant strain rate indicate the occurrence of nonrecoverable plastic strain (dediatoric and volumetric) within the state boundary surface. The stress paths migrates into the state boundary surface which was determined from monotonic loading tests. The excess pore water pressure is accumulated and eventually the stress condition reaches nonfailure equilibrium. On the monotonic loading after repeated shearing, the undrained strength of cohesive soil does not influenced by the preceding shear process, while the effective stress path seems to depend upon it in some degree.

These experimental results were compared with Pender model and it was found that the results was not satisfactory and the development of more general constitutive equation was required.

A NEW CONSTITUTIVE EQUATION

We have derived a new constitutive equation to interpret the mechanical behavior of clay under repeated loading. The proposed equation is based upon the non-associated flow rule and is similar to the Pender's kinematic hardening model. The idea of double yield loci for deviatoric (suffix s) and volumetric (suffix c) components is also adopted.

The theory of plasticity defines the following flow rule.

$$d\varepsilon_{ij}^{p} = \sum_{\alpha=1}^{n} C_{\alpha} h_{\alpha} \frac{\partial g_{\alpha}}{\partial \sigma_{ij}} df_{\alpha}$$

where f:yield function, g:plastic potential, h: hardening parameter.
The yielding within the state boundary surface is expressed by f_s , such as

$$f_s = A (\eta - \eta_i) = 0$$

where A = 1 for $d\eta>0$, -1 for $d\eta<0$.
Yielding occurs when the stress ratio η changes. Another yield function to define the normally consolidated region is :

$$f_c = \frac{p}{p_y} - \frac{M^2}{M^2 + \eta^2} = 0$$

A plastic potential is defined as the surface which is normal to the direction of a plastic increment vector. It was found that there is a linear relationship between η and plastic strain increment $dv^p/d\varepsilon^p$, which is used to get the plastic potential $g_s = 0$.

$$dv^p/d\varepsilon^p = M_m - \eta$$

From the η constant consolidation test, we get

$$dv^p/d\varepsilon^p = (M^2 - \eta^2)/\eta$$

Therefore the plastic potential will be derived as follows.

$$g_s = q - M_m p \ln (p_0/p) = 0$$

$$g_c = q - 2M^2 p^2 \ln (p_0/p) = 0$$

Hardening parameters h_s and h_c can be determined from the experimental results concerning the forms of equations f anf g. η constant consolidation will give the function h_c by using the relation

$$dv^p = \frac{\lambda-\kappa}{1+e} \frac{dp}{p}$$

$$h_c = \frac{\lambda-\kappa}{1+e} \frac{M^2 + \eta^2}{2M^2 p(M^2 - \eta^2)}$$

Determination of h_s inside the state boundary surface is not so easy. Here, in order to predict the change of loading direction (at a brake point) we define three parameters $\eta^*=|\eta - \eta_0|$, $M^* = |M - \eta_0|$, G^* (a value of $d\eta/d\varepsilon^p$ at a brake point) where η_0 is a η value at a brake point.

The results of usual repeated loading tests will five rise to the following equation.

$$d\varepsilon^p = \frac{M^*}{G^*(M^* - \eta^*)} d\eta$$

Therefore, h_s is given by

$$h_s = \frac{A M^*}{G^*(M^* - \eta^*)}$$

As a result, a constitutive law for cohesive soils under repeated loading is expressed by the following equations, where the influence of stress histories is involved in the parameters M_m and G^* which are determined empirically from laboratory results.

$$d\varepsilon^p = C_1 \frac{\lambda-\kappa}{1+e} \frac{\eta}{M^2 - \eta^2} \left(\frac{dp}{p} + \frac{2\eta d\eta}{M^2 + \eta^2}\right) + \frac{M^*}{G^*(M^* - \eta^*)} d\eta$$

$$dv^p = C_1 \frac{\lambda-\kappa}{1+e} \left(\frac{dp}{p} \quad \frac{2\eta d\eta}{M^2 - \eta^2}\right) + \frac{M^*(M_m - \eta)}{G^*(M^* - \eta^*)} d\eta$$

where $C_1 = 1$ for $f_c = 0$, $df_c > 0$
$C_1 = 0$ for $f_c = 0$, $df_c < 0$

COMPARISON WITH EXPERIMENTAL RESULTS

The results of theoretical prediction by the above constitutive equation was compared with the experimental results of repeated loading in undrained condition.

Fig. 1 is the case of η controlled test (η_{max} =0.75, η_{min}=0.0). A computed effective stree path agrees fairly well with the experimental result. A stress-strain relationship of the same case is shown in Fig. 2 where the amount of computed deviatoric strain is a little less than that of the experiment. For correct estimation of the clay behavior, more fundamental research on the evalation of parameters G^*, M_m must be carried out.

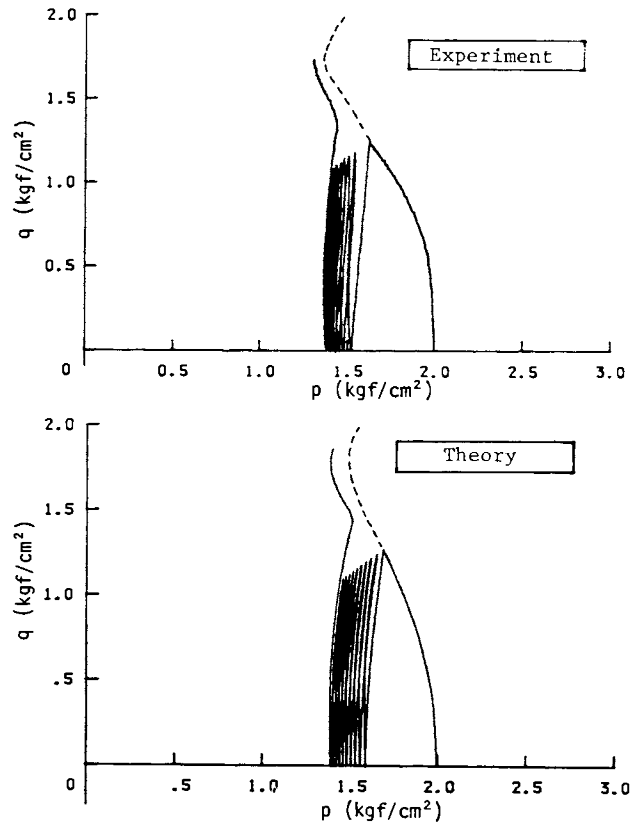

Fig. 1 Effective Stress Path (η-controlled)

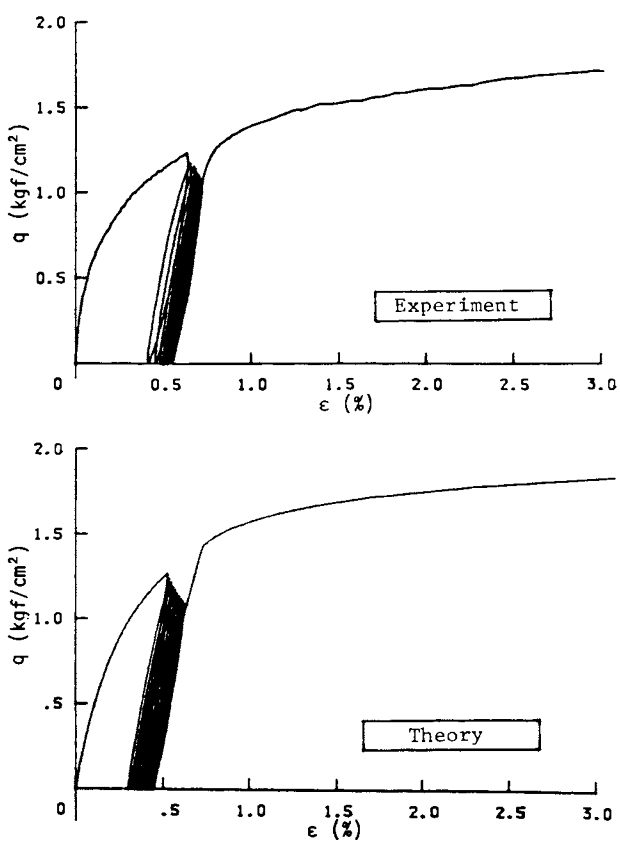

Fig. 2 Stress-Strain Relationship (η-controlled)

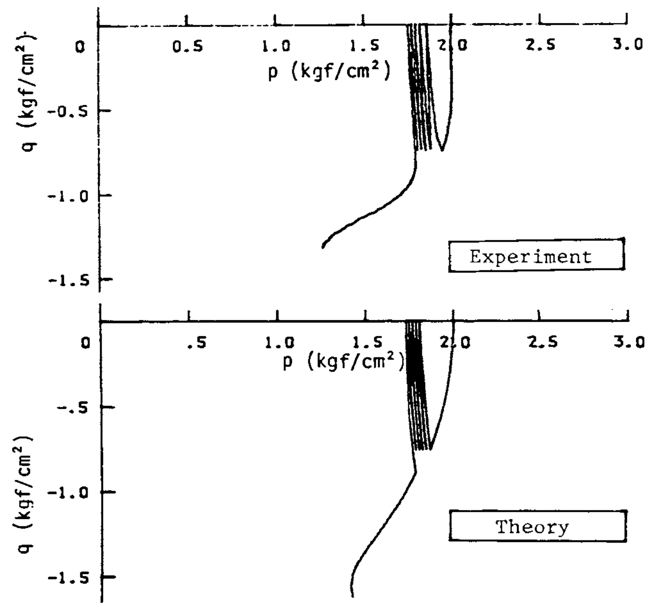

Fig. 3 Effective Stress Path (Extension Side)

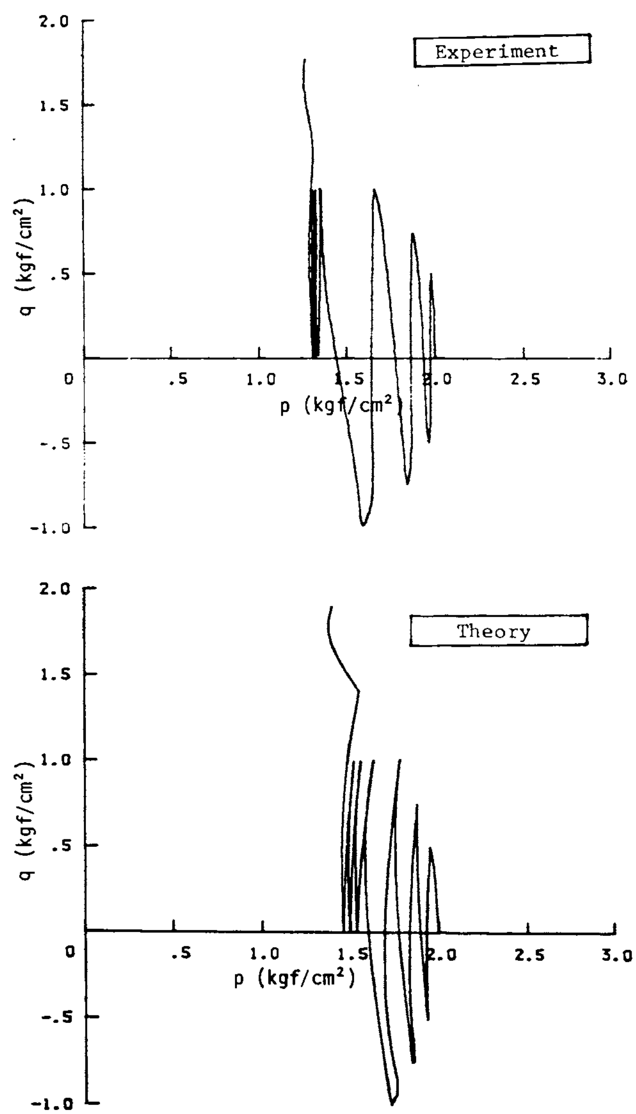

Fig. 4 Effective Stress Path (Complicated)

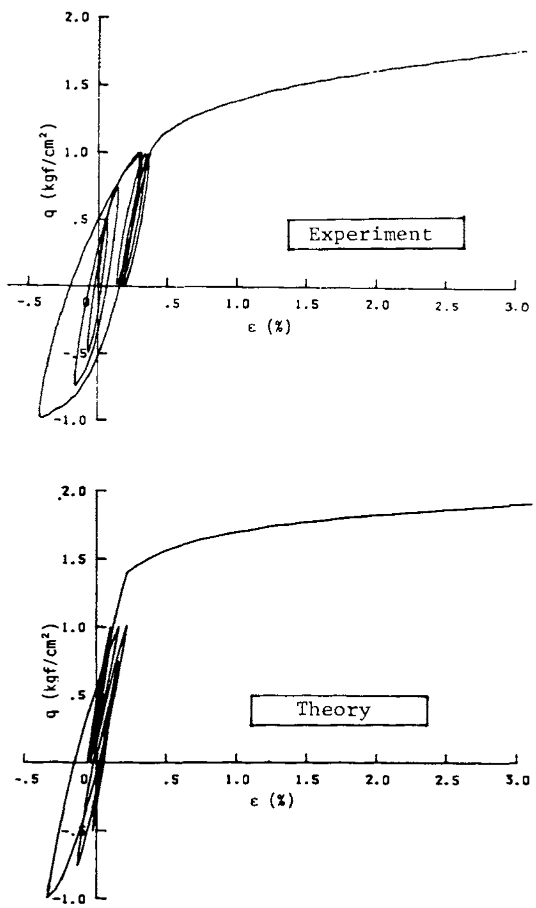

Fig. 5 Stress-Strain Relationship (Complicated Loading Pattern)

A result of the stress amplitude controlled test (extension side, q_{min} =-o.75, q_{max} =0 kgf/cm^2) is shown in Fig. 3 for the effective stress space. The computed result predicts the behavior of clay fairly well, but the monotonic loading after repeated one can not be said good estimate, probably because of unknown factors of clay behavior in a overconsolidated region.

Figs. 4 and 5 show the case in which a complicated loading is applied to the specimen to examine the effect of stress history. Generally speaking, the prediction by the proposed theory is satisfactory, except in the region of overconsolidation after migration into the state boundary surface.

It seems that accurate prediction of the mechanical behavior of cohesive soil under repeated loading is very difficult. Although our new constitutive law can play an important role, the accumulation of experimental data will strongly be requested.