PORE-PRESSURE PREDICTION DURING EARTHQUAKE LOADINGS

MEHMET A. SHERIF*, ISAO ISHIBASHI** and CHUZO TSUCHIYA***

ABSTRACT

The writers have previously proposed equations to predict pore-pressure rise under uniform and non-uniform dynamic shear stresses for a saturated Ottawa sand based on the results obtained from undrained cyclic shear experiments in the Torsional Simple Shear Device. In this paper, they extend those previous studies to earthquake-type loadings and to saturated Ottawa sand at different densities and propose equations that predict the pore-pressure rise during earthquake-type loadings.

The predicted and the measured pore pressures under earthquake-type shear stresses are compared, and a good correlation between the two is obtained.

A diagram and a table are presented from which the four material parameters, C_1 , C_2 , C_3 and α , used in the above pore-pressure prediction equations can be readily obtained for any given void ratio or soil density.

Key words : dynamic, earthquake, laboratory test, liquefaction, pore pressure, sand, saturation, torsion

IGC : D6/D7

INTRODUCTION

Since liquefaction occurs at the extreme stages of the gradual increase in pore pressure during dynamic loading, the prediction of pore-pressure buildup is essential in the determination of soil liquefaction.

Based on their extensive investigations on Ottawa sands (Ishibashi et al., 1977), the writers have presented equations for predicting the pore-pressure rise in a saturated sand under uniform cyclic excitations. These equations did not apply to predicting pore-pressure rise under earthquake-type loading. In this paper, the writers extend their previous findings on pore-pressure-rise prediction to different loading patterns, including earthquakes, and present data for the easy determination of the soil parameters that are used in the proposed pore-pressure-prediction equations. These new equations apply to the prediction of pore-pressure rise in saturated sands of differing densities excited under any kind of cyclic loading. A different procedure for predicting pore-pressure rise in saturated sands was proposed earlier by Martin et al. (1975) and Ishihara et al. (1976). The approaches taken by these researchers combine the stress-dilatancy relationship for a sand, including

* Professor of Civil Engineering and Adjunct Professor of Quaternary Research, University of Washington, Seattle, Washington 98195, U.S.A.

** Research Assistant Professor of Civil Engineering, University of Washington.

*** Engineer, Japan Highway Public Corporation. Formerly Graduate Student, University of Washington.

Written discussions on this paper should be submitted before October 1, 1979.

the effects of strain or stress history in drained conditions, and the volume-change/pore-pressure-rise relationships in undrained conditions.

PORE-PRESSURE PREDICTION DURING CYCLIC LOADING

Under Uniform Cyclic Loading

In their previous paper (Ishibashi et al., 1977), the authors analyzed the residual pore pressure at the end of each cycle based on liquefaction test results under uniform cyclic shear loading.

It was assumed that the pore-pressure rise was a product of the stress history H , number-of-cycle effects $\bar{N}$, and applied stress intensity function I , or

$$\Delta U^*_N = H \cdot \bar{N} \cdot I \quad (1)$$

where ΔU^* = the increment in normalized pore pressure during the N^{th} cycle (normalized pore pressure equals the actual pore pressure divided by the initial effective confining pressure). Based on the experimental evidence and boundary conditions, the above-mentioned functions H , $\bar{N}$, and I were determined as follows:

$$H = 1 - U^*_{N-1} \quad (2)$$

$$I = \left[\frac{\tau_N}{\sigma'_{N-1}} \right]^\alpha \quad (3)$$

$$\bar{N} = \frac{C_1 \cdot N}{N^{C_2} - C_3} \quad (4)$$

Consequently, Eq. (1) becomes Eq. (5), as proposed in the previous paper:

$$\Delta U^*_N = (1 - U^*_{N-1}) \left[\frac{C_1 \cdot N}{N^{C_2} - C_3} \right] \cdot \left[\frac{\tau_N}{\sigma'_{N-1}} \right]^\alpha \quad (5)$$

and

$$U^*_N = U^*_{N-1} + \Delta U^*_N$$

where U^*_{N-1} = the normalized residual pore pressure at the $(N-1)^{\text{th}}$ cycle; U^*_N = the normalized residual pore pressure at the N^{th} cycle; N = number of cycles; τ_N = the shear stress at the N^{th} cycle; σ'_{N-1} = the effective confining pressure at the end of the $(N-1)^{\text{th}}$ cycle; and C_1 , C_2 , C_3 and α are material parameters.

Eq. (5) provides for pore-pressure generation U^*_N by a step-by-step method under a given stress, τ_N , and initial boundary conditions (i.e., $U^*_0 = 0$ and σ'_0 = initial effective confining pressure $\bar{\sigma}_c$).

Under Non-uniform Cyclic Loading

For non-uniform cyclic loading, in which the cyclic shear stress amplitudes are randomly variable but symmetrical with respect to the zero-stress axis, the term "equivalent number of cycles" is introduced to evaluate all the numbers of cycles of different stress amplitudes prior to the cycle at which the pore pressure is to be predicted. This is done by replacing the term N in Eq. (5) by the equivalent number of cycles N_{eq} , which is defined as

$$N_{\text{eq}} = \sum_{i=1}^N \left[\frac{\tau_i}{\tau_N} \right]^\alpha \quad (6)$$

where τ_i = cyclic shear stress at the i^{th} cycle ($1 \leq i \leq N$); τ_N = cyclic shear stress at the N^{th} cycle; and α is the same as in Eq. (5).

When the shear stress amplitude τ_N during the N^{th} cycle is quite large in comparison with the preceding ones (i.e. $\tau_N \gg \tau_i$ at $i=1$ to $N-1$), the value of the equivalent number of cycles N_{eq} at the N^{th} cycle becomes close to, but always greater than, 1.0. This implies that the previous smaller values of cyclic shear stress τ_i do not have much cyclic effect

on the pore-pressure rise at the N^{th} cycle. It is also seen from Eq. (6) that if only uniform cyclic shear stresses are applied during the test (i.e. $\tau_i = \tau_N$), the equivalent number of cycles N_{eq} in Eq. (6) becomes equal to the actual uniform number of stress cycles, N .

When N in Eq. (5) is replaced by N_{eq} in Eq. (6), the equation for predicting the pore-pressure rise in saturated loose sands under non-uniform cyclic loading is expressed as :

$$\left. \begin{aligned} \Delta U^*_N &= (1 - U^*_{N-1}) \cdot \frac{C_1 N_{\text{eq}}}{N_{\text{eq}}^{C_2 - C_3}} \cdot \left[\frac{\tau_N}{\sigma'_{N-1}} \right]^\alpha \\ \text{and} \quad U^*_N &= U^*_{N-1} + \Delta U^*_N \end{aligned} \right\} \quad (7)$$

EXPERIMENTS AND SOIL TYPE

All of the tests in this investigation were conducted in the Torsional Simple Shear Device, in which the sample is hollow with 4 in. (102 mm) in outside and 2 in. (51 mm) in inside diameters, and 1 in. (25 mm) in outside and 1/2 in. (13 mm) in inside heights. For more information on the above device, the reader is referred to Ishibashi and Sherif (1974), Ishibashi et al. (1977), and Sherif and Ishibashi (1976).

Ottawa sand of ASTM designation C-109 (at loose, medium dense and dense states) was used for all the liquefaction tests during this investigation. This is a relatively uniform natural silica sand, the physical properties of which are shown in Table 1.

The dry sample was first weighed and submerged in water overnight. The inside and outside membranes were fixed to the base of the shear device and confined vertically by specimen molds. Before placing the soil between the inner and outer molds, the sample-receiving cavity was filled with de-aired water, and the upper porous stone was kept submerged in the water. Vacuum and compressed de-aired water were then applied consecutively through the lines leading to the bottom and top of the specimen to remove any entrapped air in the porous stones and saturation lines. Thereafter, the previously de-aired specimen was slowly and evenly poured between the specimen molds with a spoon. The top plate with the porous stone was placed on the specimen and then the membranes were clamped to it. After applying vacuum to the specimen through the line leading to the top of the specimen, the outside specimen mold was removed, while the inner mold collapsed after removal of the rod which sustained it. The torque pickup and the deformation-measurement plates were set on top of the specimen.

Three different sample densities, namely loose, medium dense and dense, were tested during this investigation. These densities were obtained by using the following procedures:

(a) Loose samples

Loose samples were obtained by gently pouring the soil into the mold as described pre-

Table 1. Physical properties of Ottawa sand

Ottawa sand (ASTM C-109)	
Grain size	$D_{100}=0.60 \text{ mm}$, $D_{60}=0.42 \text{ mm}$, $D_{50}=0.40 \text{ mm}$ $D_{10}=0.2 \text{ mm}$, $D_0=0.15 \text{ mm}$
Uniformity coefficient	2.1
Specific gravity	2.67
Maximum void ratio, e_{max}	0.76*
Minimum void ratio, e_{min}	0.50

* $e_{\text{max}}=0.82$ was reported in Ref. 1 for the same sand.

vously, without compaction. The samples obtained by this method had void ratios ranging from 0.690 to 0.709 (with a mean value of 0.700).

(b) Medium dense samples

Before clamping the membranes, the specimen in the mold was placed on the vibrating table and vibrated at 3600 cycles per minute for 5 seconds. During this vibration process a counterweight of 400 grams was placed on top of the specimen to protect it from the pounding of the cap and from undergoing uneven settlement.

The samples obtained by this method had void ratios ranging from 0.629 to 0.657, with a mean value of 0.644.

(c) Dense samples

After placing the sample between the inside and outside molds, a kneading-type compaction was applied, using a cone-shaped edge with a 45-degree slope. The cone was pressed 60 times with equal force on the surface of the specimen. After compaction, the sample surface was smoothed, the top cap was put in place and the sample was placed on the vibrating table for 20 seconds. The samples obtained by this method had void ratios ranging from 0.574 to 0.614, with a mean value of 0.595.

A qualitative understanding of the degree of soil saturation was obtained by measuring the pore-pressure parameter $\bar{B}$ (Skempton, 1954). During all these experiments, the saturation of the sample was ensured by making sure that the pore-pressure parameter $\bar{B}$ values were over 0.90 prior to dynamic testing. Table 2 shows the summary of the test program during this investigation.

Table 2. Test program for saturated samples

Cyclic stress pattern	Uniform cyclic loading	Non-uniform cyclic loading
Sample type	Ottawa sand (ASTM C-109)	
$\bar{B}$	>0.90	
Back pressure	45 psi	
Wave shape	Sine wave	
Average void ratio	Loose ($e_{avg}=0.700$), medium dense ($e_{avg}=0.644$) and dense ($e_{avg}=0.595$)	
Frequency	2 cps	
Initial effective vertical and horizontal stresses	10, 20, 30 psi	20 psi
(1 psi=6.9 kN/m ²)		

**PORE-PRESSURE PREDICTION IN MEDIUM DENSE AND DENSE SANDS
UNDER UNIFORM AND NON-UNIFORM LOADINGS**

The previously developed equations {Eqs. (5), (6) and (7)} to predict pore-pressure rise in saturated loose sand can be extended to predict pore-pressure rise in medium dense and dense sands.

The experimental data in Figs. 1 and 2 show the relationships between the modified pore-pressure rise and the stress ratio for medium dense and dense sands respectively. It is seen from these figures that the pore-pressure rise in medium dense and dense sands is similar to the one in loose sands, as previously reported in Ishibashi et al. (1977) and shown here in Fig. 3. The slopes of the lines α in Figs. 1 and 2 are unique for each soil density, which implies that parameter α is density-dependent. The other three soil parameters, C_1 , C_2 and C_3 , in Eqs. (5) and (7) can be determined by curve fitting into the experi-

Fig. 1. Incremental rise of pore-pressure in each cycle (Medium dense sand)

Fig. 2. Incremental rise of pore-pressure in each cycle (Dense sand)

mental data, as shown in Figs. 4 and 5 for medium dense and dense sands. For example, for medium dense sand the values of C_1 , C_2 and C_3 are respectively equal to 2.4, 1.82 and 0.30. For dense sand they are 2.09, 2.03 and 0.09. For the benefit of the reader the above four parameters are shown as a function of void ratio in Fig. 6 and in Table 3.

The fact that loose samples show higher values of C_1 and C_3 than dense samples implies that these two parameters mainly influence the magnitudes of pore-pressure rise at the initial stages of loading, such as from 1 to 10 cycles, where the pore-pressure rise is normally high. The value of C_3 becomes almost zero for the dense sample, which means that the incremental rise in pore-pressure decreases linearly with the increase in the number of cycles for dense soils (i.e., void ratio $e < 0.57$). The parameter C_2 , on the other hand, affects the slope of the straight portion of the lines in Figs. 4 and 5. The fact that the value of C_2 decreases with increasing soil density implies that the incremental rise in pore pressure in dense soils is less as compared with the loose soils. The slope α is a measure of the sensitivity of pore-pressure rise to the change in applied stresses, and the lower value of α for loose samples means that the pore-pressure buildup in loose samples is more sensitive to stress changes than in dense ones.

When the appropriate values of the parameters C_1 , C_2 , C_3 and α are inserted into Eq. (5), then Eq. (5) also predicts the incremental pore-pressure-rise values in medium dense and dense saturated sands under uniform cyclic loading. Using the density-dependent C_1 , C_2 , C_3 and α values and the concept of equivalent number of cycles N_{eq} expressed in Eq. (6), the writers predicted the pore-pressure buildup in saturated sands of various densities when

Fig. 3. Incremental rise of pore pressure in each cycle (Loose sand) (Ishibashi et al., 1977)

Fig. 4. Relationship between incremental rise of pore pressure and number of cycles (Medium dense sand)

Table 3. Parameters C_1 , C_2 , C_3 and α for different densities of Ottawa sand

Densities	Parameters			
	C_1	C_2	C_3	α
Loose ($e_{avg}=0.700$)	6.13	1.77	0.46	2.40
Medium dense ($e_{avg}=0.644$)	2.40	1.82	0.30	2.17
Dense ($e_{avg}=0.595$)	2.09	2.03	0.09	2.00

subjected to any non-uniform cyclic excitations, as shown in Figs. 7(a) and 7(b). It is seen in Fig. 7 that a good agreement exists between the predicted and the measured pore-pressure values for both medium dense and dense samples. It can therefore be concluded that Eq. (7) provides a valid basis for the prediction of pore-pressure rise under non-uniform cyclic loading.

PREDICTION OF PORE-PRESSURE RISE DURING ACTUAL EARTHQUAKE-TYPE LOADING

In the previous section, the proposed Eqs. (5) and (7) were shown to be applicable for predicting pore-pressure rise in soils under both uniform and non-uniform cyclic loadings. Before applying these equations to the prediction of pore-pressure rise in saturated sands

Fig. 5. Relationship between incremental rise of pore pressure and number of cycles (Dense sand)

Fig. 6. Variation of parameters C_1, C_2, C_3 and α with void ratio

Fig. 7(a). Comparison between predicted and measured pore-pressure rise under non-uniform cyclic loading (Medium dense sand)

Fig. 7(b). Comparison between predicted and measured pore-pressure rise under non-uniform cyclic loading (Dense sand)

during earthquake-like loadings, it is necessary to consider the fact that during an earthquake, the shear stress amplitudes vary erratically from one cycle to another. This random variation in shear stresses has the following characteristics: (a) abrupt change in the amplitude of shear stress during the time history, (b) asymmetry in the peak values of shear stress during one cycle, (c) more than one peak stress may exist during a half cycle (above or below the zero-shear-stress line), and (d) stress frequency changes randomly from one cycle to another.

It should be realized that an accurate prediction of pore-pressure rise can only be accomplished by considering every half cycle because of the asymmetry in the peak stress values during each cycle. Supposing that half of the total residual pore-pressure rise per cycle results at the end of the half cycle under uniform cyclic loading, as shown in Fig. 8(a); the pore-pressure rise during the N^{th} cycle can then be expressed as

$$\Delta U^*_N = \frac{1}{2} [(\Delta U^*_N)_p + (\Delta U^*_N)_n] \quad (8)$$

where ΔU^*_N = normalized pore-pressure rise during the N^{th} cycle; $(\Delta U^*_N)_p$ = normalized pore-pressure rise in the positive region under cyclic shear stress τ_{Np} during N^{th} cycle; and $(\Delta U^*_N)_n$ = normalized pore-pressure rise in the negative region under cyclic shear stress τ_{Nn} during the N^{th} cycle.

Fig. 8(a). Illustration of pore-pressure rise during half-cycle (Symmetrical case)

Fig. 8(b). Illustration of pore-pressure rise during half-cycle (Asymmetrical case)

Eq. (8), above, also applies to the case where the amplitudes of shear stresses in the positive and negative regions of the cycle are not equal, which condition is shown in Fig. 8(b). It is seen from Fig. 8(b) that the pore-pressure rise due to the peak shear stress τ_{Np} is expressed by the line AB, while the pore-pressure rise due to peak shear stress in the negative region τ_{Nn} is expressed by the line BC.

The values of $(\Delta U_N^*)_p$ and $(\Delta U_N^*)_n$ in Eq. (8) can be computed by Eq. (7). The only difference between Eqs. (7) and (8) lies in the computation of the equivalent number of cycles N_{eq} . Since shear stresses are now considered independently in the positive and negative regions, the equivalent number of cycles should also be evaluated independently in each region. That is, at the N^{th} cycle, shear stress amplitudes from the 1st cycle to the N^{th} cycle in the positive region give the equivalent number of cycles in that region, and the same is true for the negative region. Hence, $(\Delta U_N^*)_p$ can be expressed by the following equation:

In the positive region,

$$(\Delta U_N^*)_p = (1 - U_{N-1}^*) \cdot \frac{C_1 (N_{eq})_p}{(N_{eq})_p^{C_2} - C_3} \cdot \left[\frac{\tau_{Np}}{\sigma'_{N-1}} \right]^\alpha$$

and

$$(N_{eq})_p = \sum_{i=1}^N \left[\frac{\tau_{ip}}{\tau_{Np}} \right]^\alpha$$

(9)

where τ_{ip} , τ_{Np} = the amplitude of shear stress in the positive region at the i^{th} and N^{th} cycles; and $(N_{eq})_p$ = equivalent number of cycles in the positive region at the N^{th} cycle.

Eq. (9) also applies in the negative region by simply changing the subscript from p to n . It is clear in Eqs. (8) and (9) that if shear stresses in the positive and negative regions are equal [that is, $(\Delta U_N^*)_p$ equals $(\Delta U_N^*)_n$], then the same value of ΔU_N^* is obtained as in the case of uniform cyclic loading.

Previous studies (Peacock and Seed, 1968; Yoshimi and Oh-oka, 1975) have shown that

the difference in the frequency of cyclic shear stresses does not have a highly significant effect on the pore-pressure rise and the liquefaction potential of sands. Furthermore, it is considered that the peak stress value within the half cycle will have the most significant influence on pore-pressure buildup; that is to say, for all practical purposes the magnitudes of the pore pressures are determined solely by the value of the maximum shear stress in the half cycle.

In spite of the randomness of the time history of shear stress variation during earthquakes, Eqs. (8) and (9) do provide an appropriate basis for the prediction of pore-pressure rise during earthquakes.

A liquefaction test was conducted for a saturated medium dense Ottawa sand ($e=0.645$) by applying an earthquake-type shear stress, which is shown in the upper part of Fig. 9. The lower part of Fig. 9 shows the comparison between actual pore pressures measured during the above test and the pore-pressure values predicted based on Eqs. (8) and (9). From the data shown in Fig. 9, it is evident that Eqs. (8) and (9) do indeed adequately predict the pore-pressure rise under earthquake-type loadings and that the assumptions made in the derivation of the above equations (that is, disregarding the effect of frequency change and the secondary peak shear stresses in half a cycle) appear to be reasonable for all practical purposes.

Fig. 10 shows the results of the computations of pore-pressure rise based on Eqs. (8) and (9) during an actual earthquake loading (Seed et al., 1975) corresponding to the one shown in the upper part of Fig. 10. Pore-pressure rise under the above earthquake loading was computed for three different densities of saturated Ottawa sand, the void ratios of which were 0.700, 0.644 and 0.594. It can be observed from Fig. 10 that when the deposit is loose, it liquefies after 5 seconds, while the same deposit with medium density

Fig. 9. Comparison between predicted and measured pore-pressure rise under earthquake-type loading (1 psi=6.9 kN/m²)

Fig. 10. Predicted pore-pressure rise during actual time history of an earthquake ($1 \text{ psf} = 47.9 \text{ N/m}^2$; $1 \text{ ft.} = 0.305 \text{ m}$)

liquefies after 10 seconds. On the other hand, if the above soil were dense, it would not liquefy at all during this particular earthquake. It is also interesting to note in Fig. 10 that only the large peak shear stresses contribute significantly to pore-pressure rise.

CONCLUSION

(1) Based on the pore-pressure data shown in Figs. 7(a) and 7(b), the writers conclude that the proposed equations {Eqs. (5) and (7)} do adequately predict the pore-pressure rise in saturated Ottawa sands of varying densities when the soil is subjected to non-uniform cyclic loading.

(2) Eqs. (5) and (7) have been extended to predict pore-pressure rise under actual earthquake-type loadings as expressed by Eqs. (8) and (9). The good correlation between the predicted and the measured pore-pressure values indicated in Fig. 9 shows that Eqs. (8) and (9) do indeed predict pore-pressure rise under actual earthquake-type excitation.

(3) The data in Fig. 10 show that the major peak earthquake stress levels are the primary ones that cause pore-pressure buildup during earthquakes.

ACKNOWLEDGMENT

The writers gratefully acknowledge the financial assistance of the National Science

Foundation, which made this investigation possible.

NOTATION

$\bar{B}$ = pore-pressure parameter

C_1, C_2, C_3 = material parameters

$D_0, D_{10}, D_{50},$

D_{60}, D_{100} = soil diameters of which 0, 10, 50, 60 and 100% of soil weights are finer, respectively

e = void ratio

e_{avg} = average void ratio

e_{max} = maximum void ratio

e_{min} = minimum void ratio

N = number of cycles

N_{eq} = equivalent number of cycles

$(N_{eq})_p$ = equivalent number of cycles in positive region

$(N_{eq})_n$ = equivalent number of cycles in negative region

U^*_N = normalized residual pore pressure at the end of N^{th} cycle

ΔU^*_N = normalized pore-pressure rise during N^{th} cycle

$(\Delta U^*_N)_p$ = normalized pore-pressure rise during N^{th} cycle due to stress in positive region

$(\Delta U^*_N)_n$ = normalized pore-pressure rise during N^{th} cycle due to stress in negative region

α = material parameter

$\bar{\sigma}_c$ = initial effective confining pressure

σ'_N = effective confining pressure at the end of N^{th} cycle

τ_N = cyclic shear stress at N^{th} cycle

τ_{Np} = cyclic shear stress at N^{th} cycle in positive region

τ_{Nn} = cyclic shear stress at N^{th} cycle in negative region

REFERENCES

- 1) Finn, W.D.L., Pickering, D.J. and Bransby, P.L. (1971): "Sand liquefaction in triaxial and simple shear tests," Proc. of ASCE, Vol.97, SM 4, pp.639-659.
- 2) Ishibashi, I. and Sherif, M.A. (1974): "Soil liquefaction by torsional simple shear device," Proc. of ASCE, Vol.100, GT 8, pp.871-881.
- 3) Ishibashi, I., Sherif, M.A. and Tsuchiya, C. (1977): "Pore-pressure rise mechanism and soil liquefaction," Soils and Foundations, Vol.17, No.2, pp.17-27.
- 4) Ishihara, K., Lysmer, J., Yasuda, S. and Hirao, H. (1976): "Prediction of liquefaction in sand deposits during earthquakes," Soils and Foundations, Vol.16, No.1, pp.1-16.
- 5) Martin, G.R., Finn, W.D.L. and Seed, H.B. (1975): "Fundamentals of liquefaction under cyclic loading," Proc. of ASCE, Vol.101, GT 5, pp.423-438.
- 6) Peacock, W.H. and Seed, H.B. (1968): "Sand liquefaction under cyclic loading simple shear conditions," Proc. of ASCE, Vol.94, SM 3, pp.689-708.
- 7) Seed, H.B., Idriss, I.M., Makdisi, F. and Banerjee, N. (1975): "Representation of irregular stress time history by equivalent uniform stress series in liquefaction analyses," Report No. EERC 75-29, Earthquake Engineering Research Center, University of California at Berkeley.
- 8) Sherif, M.A. and Ishibashi, I. (1976): "Dynamic shear moduli for dry sands," proc. of ASCE, Vol.102, GT 11, pp.1171-1184.
- 9) Skempton, A.W. (1954): "The pore pressure coefficients A and B," Geotechnique, Vol.4, Dec., pp.143-147.
- 10) Yoshimi, Y. and Oh-oka, H. (1975): "Influence of degree of shear stress reversal on the liquefaction potential of saturated sands," Soils and Foundations, Vol.15, No.3, pp.27-40.

(Received January 23, 1978)