11th European Conference on Earthquake Engineering © 1998 Balkema, Rotterdam, ISBN 90 5410 982 3

Analytical relationships for earthquake induced pore pressure in sand, clay and silt

D.N. Egglezos & G.D. Bouckovalas
National Technical University of Athens, Athens, Greece

Keywords: earthquake, pore pressure, laboratory tests, sands, clays, silts

ABSTRACT: Excess pore pressure build up during seismic loading of saturated soils is a common phenomenon which degrades the strength and the deformability of soils. Acknowledging its practical importance, this article establishes a set of relationships for the preliminary prediction of earthquake-induced pore pressures in sands, clays and silts under stress or strain controlled cyclic loading. To achieve this aim, a statistical analysis is performed on data from 173 cyclic triaxial and cyclic simple shear (symmetric in the overwhelming majority) tests reported in the literature. For strain controlled cyclic loading, excess pore pressures depend upon the number of cycles, the cyclic shear strain and the effective consolidation stresses. For stress controlled loading, the cyclic shear strain is replaced by the normalized cyclic shear stress and the void ratio. The quantitative effect of these factors is different for sands and non-plastic silts as compared to clays and plastic silts, with less overall pore pressure build up in the later case.

1 INTRODUCTION

Excess pore pressure build up under seismic loading of saturated soils, is a common phenomenon with well understood effects on the strength and the deformability of soils. In the special case of saturated cohesionless soils this phenomenon leads to liquefaction, i.e. total loss of strength of the soil. Despite its widely acknowledged importance, earthquake induced pore pressures are not generally considered in the design of foundations and earth structures, unless liquefaction conditions are eminent. This is mainly because rigorous computations for 2-D and 3-D applications are involved in terms of solution algorithms and input soil properties. Hence there is a clear need for simplified solutions, derived on the basis of experimental data, which can be used for common applications.

Within this context, a set of general relationships are presented which can be used to estimate earthquake-induced pore pressures for:

- different types of soils (sands, silts, and clays),
- either triaxial or simple shear test conditions, and
- either constant cyclic stress or constant cyclic strain cyclic loading.

It should be acknowledged that these tests have been planned for the needs of various projects, and they have been performed in different laboratories, with different test equipment and various preparation methods. In addition, the performance of these tests covers a wide period in time (early 70's to late 90's) which reflects upon the accuracy of the results. All the above reasons may increase somewhat the scatter of data, relative to that of a uniform data base. However, the joint evaluation of different data bases helps to derive generalized conclusions which would not be otherwise identified from compilation of each separate data base. The experimental data base which was used to establish the relationships includes results from a total of 173 tests, 75 for triaxial (TX) and 98 for direct simple shear (DSS) loading. The majority of data are from tests on sand (57 TX and 60 DSS tests) and normally consolidated clay (12 TX and 32 DSS tests). Data from tests on silt are rather limited (6 TX and 6 DSS tests), but allow a gross outline of their response in correlation to the other two major soil types.

Table 1. Summary of natural soil properties for TX tests

Soil	No of tests	Void Ratio	Dr (%)	PI[1] (%)	C_u	Preparation method
Oosterschelde Sand	29	0.61-0.85	50-74	NP	1.4	pluviation, wet tamping
Nevada Sand	16	0.65-0.74	40-60	NP	1.5	dry pluviation, tamping
Banding Sand	4	0.73	40	NP	1.5	dry pluviation, tamping
Baskarp Sand	8	0.53	93-96	NP	(<5)	dry pluviation, tamping
Bonnie Silt	4	0.73-0.77	-	15	-	remoulding
Silica Flour	2	0.64	-	NP	-	hand tamping, shaking
Drammen Clay	7	1.02	-	35	-	undisturbed sampling
Nivaa Clay	5	0.80-0.86	-	20	-	remoulding

(1) NP=Non plastic

Table 2. Summary of natural properties for DSS tests

Soil	No of tests	Void Ratio	Dr (%)	PI[1] (%)	C_u	Preparation method
Oosterschelde Sand	21	0.68-0.73	56-66	NP	1.4	pluviation, wet tamping
Nevada Sand	12	0.65-0.73	41-63	NP	1.5	dry pluviation, tamping
Banding Sand	5	0.73	40	NP	1.5	dry pluviation
Frigg Field Sand	11	0.46-0.51	84-96	NP	(<5)	dry pluviation, tamping
Monterey Sand[2]	5	0.68	60	NP	1.5	dry pluviation
Baskarp Sand	9	0.53-0.54	91-94	NP	(<5)	dry pluviation, tamping
Bonnie Silt	8	0.68-0.70	-	15	-	remoulding
Aktio silt	3	0.82	-	NP	-	undisturbed sampling
Drammen Clay	7	1.02	-	35	-	undisturbed sampling
Boston Blue Clay	25	0.82	-	21	-	resedimentation

(1) NP=non plastic, (2) Cyclic torsional instead of DSS tests

Table 3. Test parameters of the triaxial data base

Soil	Consolidation[1]	Type of test	σ'_{oct} (kPa)	q_{cyc}/σ'_{oct}	γ_{cyc}(%),(N=1)
Oosterschelde Sand	CIU	stress controlled	50-687	0.036-0.429	0.021-0.423
Nevada Sand	CIU	stress controlled	39-157	0.154-0.343	0.045-0.435
Banding Sand	CIU	strain controlled	96	-	0.060-1.500
Baskarp Sand	CAU, CIU	stress controlled	155-250	0.407-1.614	0.100-2.400
Bonnie Silt	CIU	stress controlled	39-78	0.226-0.308	0.180-0.315
Silica Flour	CIU	strain controlled	286-400	-	1.500-3.000
Drammen Clay	CAU	stress controlled	230-160	0.066-0.200	0.034-0.300
Nivaa Clay	CAU	stress controlled	168-173	0.024-0.095	0.015-0.057

(1) CIU=isotropically consolidated undrained tests, CAU= anisotropically consolidated undrained tests

Table 4. Test parameters of the simple shear data base

Soil	Consolidation(1)	Type of test	σ'_v (kPa)	σ_{cyc}/σ'_v	γ_{cyc}(%),(N=1)
Oosterschelde Sand	CIU, CAU	stress controlled	344	0.070-0.430	0.022-0.285
Nevada Sand	CIU	stress controlled	80-160	0.070-0.300	0.310-2.730
Banding Sand	CIU	strain controlled	96	-	0.040-0.800
Frigg Field Sand	CIU	stress controlled	97-243	0.109-0.217	0.125-3.125
Monterey	CIU	strain controlled	96	-	0.070-1.150
Baskarp Sand	CAU, CIU	stress controlled	250-292	0.143-0.586	0.200-2.400
Bonnie Silt	CIU	stress controlled	76	0.210-0.220	1.010-1.300
Aktio Silt	CIU	strain controlled	200-207	0.100-0.300	0.160-0.920
Drammen Clay	CAU	stress controlled	320-392	0.072-0.187	0.140-0.900
Boston Blue Clay	CAU	stress controlled	400-800	0.041-0.174	0.110-1.380

(1) CIU=isotropically consolidated undrained tests, CAU= anisotropically consolidated undrained tests

The type and the natural properties of the soils included in the data base are summarized in Tables 1 and 2, for triaxial and simple shear tests respectively. The corresponding test conditions and the range of values of the main parameters considered in the statistical analysis are summarized in Tables 3 and 4.

2 GENERAL FORM OF ANALYTICAL RELATIONSHIPS

2.1 *Effect of number of cycles.*

Fig. 1 shows typical results for the accumulation of excess pore pressure with number of cycles obtained from two isotropically consolidated tests on sand, one with constant cyclic stress and the other with constant cyclic strain amplitude. These two types of cyclic loading, apply for TX as well as for DSS tests and will be briefly refered as “stress controlled” and “strain controlled” hereafter.

For strain controlled tests, the rate of pore pressure accumulation decreases gradually with number of cycles. This variation takes a linear form in the double logorithmic scale of Fig. 1, which is described analytically from the general relationship:

$$\Delta u(N) = \Delta u(1)\, N^{a_1} \tag{1}$$

where, $\Delta u(N)$ denotes the excess pore pressure after N cycles, and $\Delta u(1)$ denotes the excess pore pressure after the first cycle. For the test results examined here, the exponent a_1 varies between 0.30 and 0.80 with an average value 0.48 for cohesionless soils and 0.58 for cohesive soils.

In stress controlled tests, the rate of pore pressure accumulation is initially similar to that for strain controlled tests. However, at the final stages of the test, it increases abruptly until the pore pressure becomes essentially equal to the isotropic consolidation stress (liquefaction). Seed and Booker (1977) proposed the following expression to describe this form of pore pressure accumulation:

$$\Delta u(N) = \left(\frac{2}{\pi}\right)\sigma'_{oct}\, \arcsin\left(\left(\frac{N}{N_l}\right)^{(1/2b_1)}\right) \tag{2}$$

where N_l denotes the number of cycles with constant stress amplitude required for liquefaction and σ'_{oct} is the effective consolidation stress. The exponent b_1 ranges from 0.4 to 2.5, with a suggested average value of b_1=0.7. The test results examined here suggest that b_1=1.03 provides the best fit on sand and that the same more or less value applies for silt and clay.

Application of Eq. 2 for N=1 shows that the number of cycles N_l required for liquefaction can be expressed in terms of the excess pore pressure after the first load cycle $\Delta u(1)$:

$$N_l = \left(\sin\left(\frac{\pi}{2} \frac{\Delta u(1)}{\sigma'_{oct}} \right) \right)^{(-2b_1)} \tag{3}$$

Hence, Eq. 2 may be alternatively written as:

$$\Delta u(N) = \left(\frac{2}{\pi}\right) \sigma'_{oct} \arcsin\left(N^{(1/2b_1)} \sin\left(\frac{\pi}{2} \frac{\Delta u(1)}{\sigma'_{oct}} \right) \right) \tag{4}$$

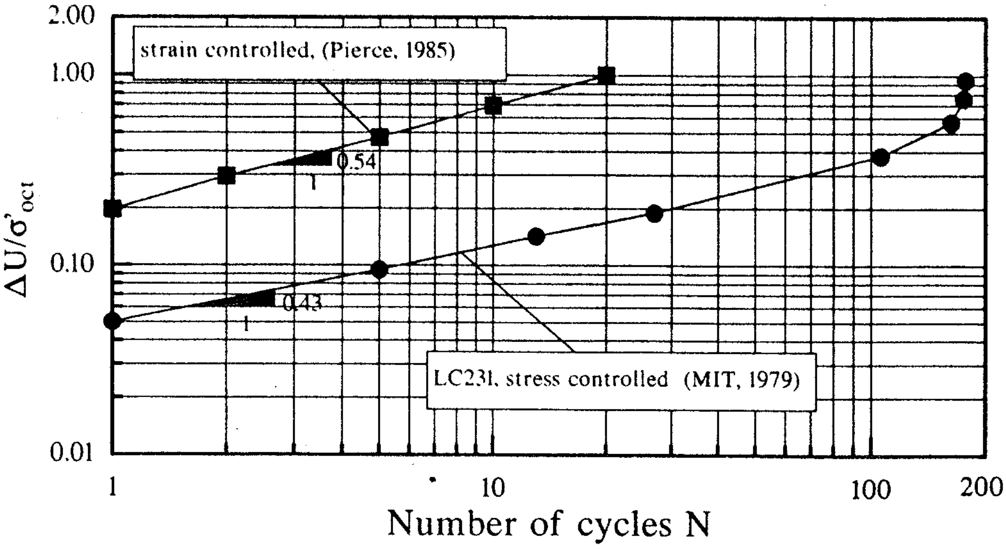

Figure 1. Effect of number of cycles: Typical test results from stress and strain controlled undrained cyclic TX tests

2.2 *Excess pore pressure build up after the first cycle.*

From Eqs. 1 and 4 it becomes evident that the key parameter for the pore pressure prediction, regardless of loading conditions, is the excess pore pressure after the first cycle $\Delta u(1)$. Depending upon the available data, this parameter can be correlated either to the cyclic strain or to the cyclic stress amplitude. In the former case, $\Delta u(1)$ is expressed as:

$$\Delta u(1) = A\, P_a \left(\frac{\sigma'}{P_a} \right)^{a_2} \gamma_{cyc}^{\ a_3}\, e^{a_4} \tag{5}$$

where, γ_{cyc} (%) is the double amplitude of cyclic shear strain in the first cycle, σ' is the vertical or the octahedral effective consolidation stress, e is the void ratio, P_a is the atmospheric pressure (=98.1 kPa) and A, a_2, a_3 and a_4 are material and test dependent constants. Alternatively, in terms of the stress amplitude, $\Delta u(1)$ is expressed as:

$$\Delta u(1) = B\, P_a \left(\frac{\sigma'}{P_a}\right)^{b_2} \left(\frac{\tau_{cyc}}{\sigma'}\right)^{b_3} e^{b_4} \tag{6}$$

where, τ_{cyc} is the amplitude of cyclic shear stress in DSS tests or the half amplitude of dynamic axial stress in TX tests, e is the void ratio and B, b_2, b_3, b_4 are material and test dependent constants. The choice of stress, strain and volume parameters in Eqs. 5 and 6 was based on evidence from a number of published experimental studies, mainly dealing with the cyclic strength of saturated cohesionless soils (e.g. Ishihara 1984, Dobry et al. 1981, De Alba et al. 1976).

It is worth to stress that the choice between Eq. 5 and Eq. 6 depends solely on whether γ_{cyc} or τ_{cyc} is known during the first load cycle, and not whether loading is stress or strain controlled. In other words both equations can be used to express $\Delta u(1)$ in either Eq. 1 or Eq. 4.

3 STATISTICAL ANALYSIS

The constants in Eqs. 5 and 6 have been determined with the aid of a multi-variable statistical analysis, materialized with the software package STATISTICA (StatSoft, Inc., 1995). For this purpose the “dependent variable”, here the excess pore pressure after the first cycle, has been related to the various independent variables. According to Eq. 5 the idependent variables for strain controlled conditions are the cyclic shear strain in the first cycle, the initial effective (vertical or octahedral) consolidation stress and the void ratio. Similarly, according to Eq. 6 the independent variables for stress controlled conditions are the cyclic stress ratio (τ_{cyc}/σ'), the initial effective (vertical or octahedral) consolidation stress and the void ratio. The statistical analysis is performed with the “quasi-Newton” algorithm, on the decimal logarithmic values of the dependent and independent variables.

As shown in Fig. 1, the initial variation of excess pore presure with number of cycles is linear in a double logarithmic (log-log) scale. However, in some of the tests, this log-log linear variation is attained after some loading cycles while there is an “irregular” behavior during the first few cycles. This phenomenon is attributed to errors in the experimental procedure, as well as to errors in pore pressure recording during the first cycles where pore pressures are relatively low. In these tests, a corrected excess pore pressure was estimated for the first cycle from extrapolation to N=1 of the log-log linear part of the data set.

The data to be analyzed according to Eqs. 5 and 6 are divided in groups, according to test type and soil type. The limited number of tests on silt does not permit an independant analysis for this soil type. For this reason, plastic (PL) silt was grouped together with clay, while non plastic (NP) silt was grouped together with sand. Hence the folllowing four (4) groups of data were finally analyzed:

- Triaxial tests on sand
- Triaxial tests on clay and PL silt
- Simple shear tests on sand and NP silt
- Simple shear tests on clay and PL silt

Each of the above groups includes only data from tests with $\gamma_{cyc}\leq 1.3\%$ and $\Delta u(1)/\sigma'_{oct}\leq 0.50$. These restrictions are necessary in order to make sure that the analysis concerns the cyclic behavior far from the liquefaction.

The constants in Eqs. 5 and 6, as well as the correlation coefficient R, are determined separately for each one of the above groups, while the results are summarized in Tables 5 and 6 respectively. The relative effects of the independent variables are presented in detail in Figs. 2 to 5 for strain controlled conditions and in Figs. 6 to 9 for stress controlled conditions. The effect of void ratio has been evaluated only in triaxial tests on sand. In the other cases the void ratio could not be evaluated statistically with reliability, because of its limited range and has been omitted as an independent variable in Eqs. 5 and 6.

Table 5. Constants for the estimation of excess pore pressure after the first cycle, in terms of cyclic shear strain amplitude

	TX		DSS	
Parameters	Sand	Clay & PL Silt	Sand & NP Silt	Clay & PL Silt
No. of tests	36	14	44	30
A	1.21	0.40	0.45	0.16
a_2	0.88	0.79	0.63	0.71
a_3	0.90	1.44	0.75	1.29
a_4	0.95	-	-	-
R	0.93	0.98	0.82	0.91

Table 6. Constants for the estimation of excess pore pressure after the first cycle, in terms of cyclic shear stress amplitude

	TX		DSS	
Parameters	Sand	Clay & PL Silt	Sand & NP Silt	Clay & PL Silt
No. of tests	31	14	30	31
B	4.73	0.55	1.50	5.19
b_2	1.04	1.03	1.17	0.93
b_3	1.61	1.87	1.46	2.19
b_4	4.22	-	-	-
R	0.90	0.93	0.81	0.86

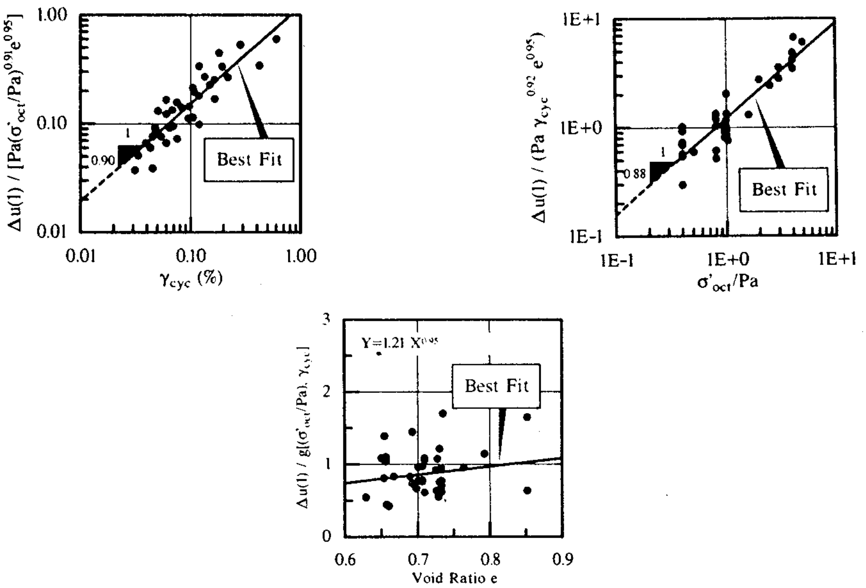

Figure 2. Parametric analysis of excess pore pressure Δu(1) in terms of strain: TX tests on sand

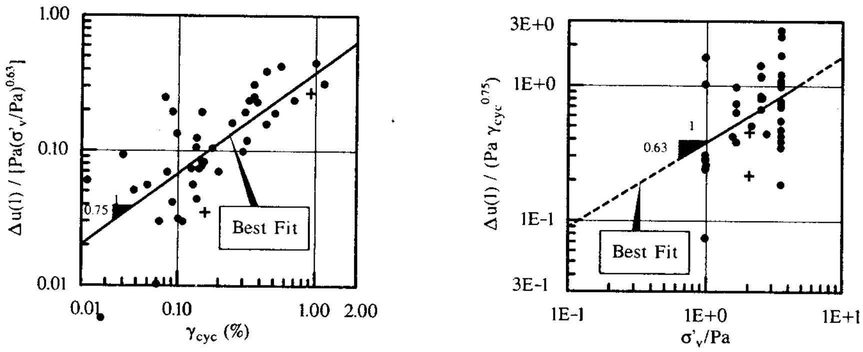

Figure 3. Parametric analysis of excess pore pressure $\Delta u(1)$ in strain terms: DSS tests on sand and NP silt (symbols: ● clay, + NP silt)

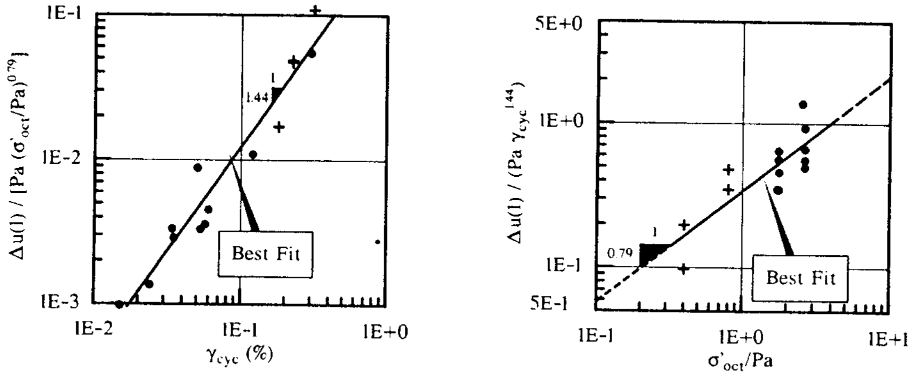

Figure 4. Parametric analysis of excess pore pressure $\Delta u(1)$ in strain terms: TX tests on clay and PL silt (symbols: ● clay, + PL silt)

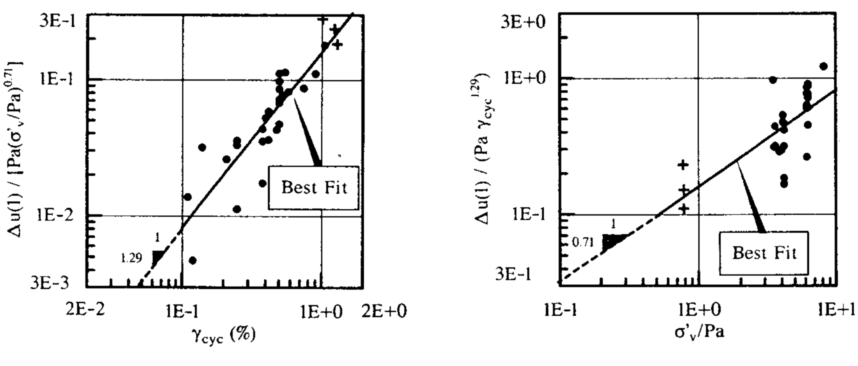

Figure 5. Parametric analysis of excess pore pressure $\Delta u(1)$ in strain terms: DSS tests on clay and PL silt (symbols: ● clay, + PL silt)

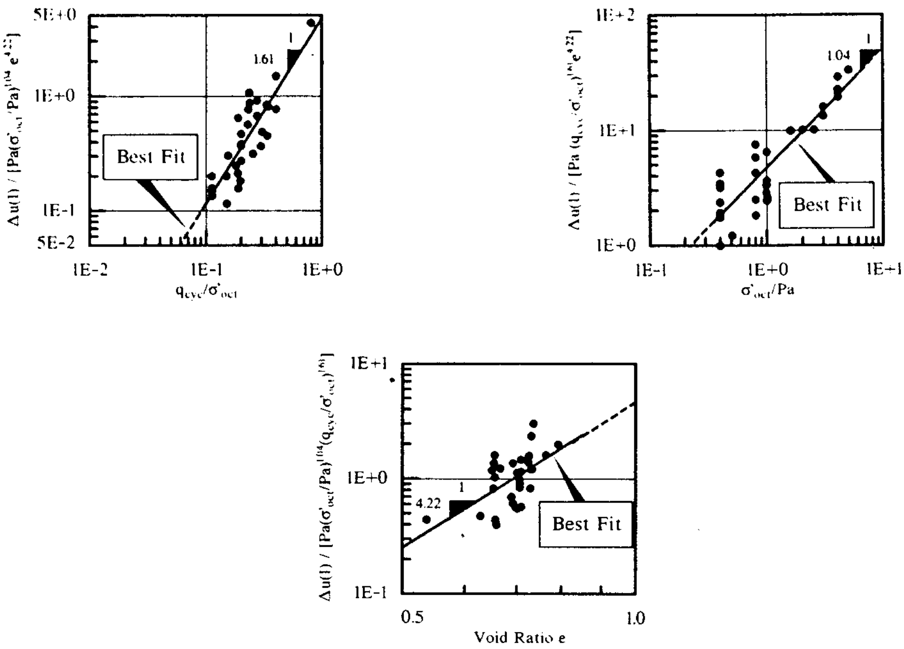

Figure 6. Parametric analysis of excess pore pressure Δu(1) in terms of stress: TX tests on sand.

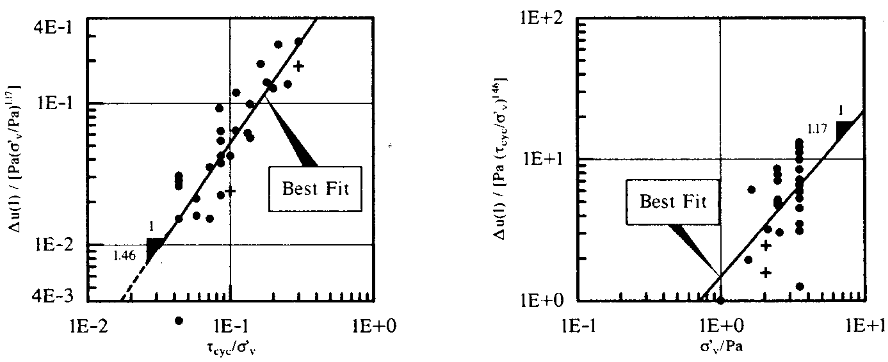

Figure 7. Parametric analysis of excess pore pressure Δu(1) in terms of stress: DSS tests on sand and NP silt (symbols: ● sand, + NP silt)

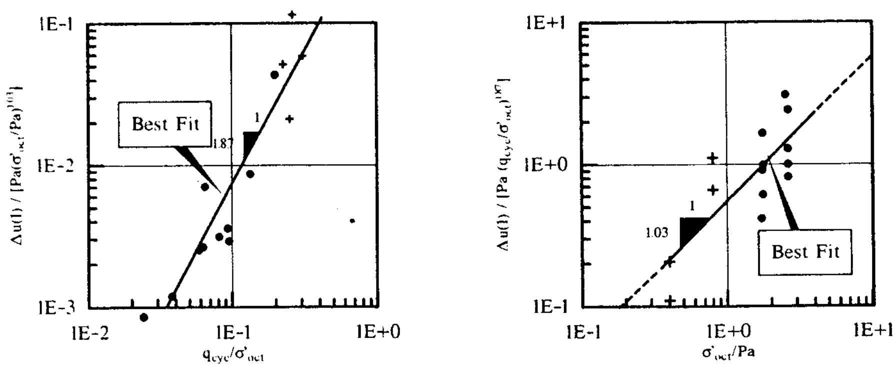

Figure 8. Parametric analysis of excess pore pressure Δu(1) in terms of stress: TX tests on clay and PL silt (symbols: ● clay, + PL silt)

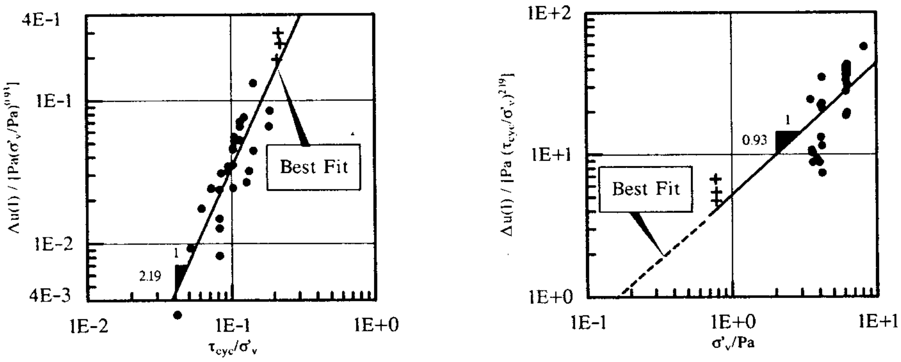

Figure 9. Parametric analysis of excess pore pressure Δu(1) in stress terms: DSS tests on clay and PL silt. (symbols: ● clay, + PL silt)

4 DISCUSSION

The empirical relationships, obtained from the preceding statistical analysis, reveal some interesting aspects with regard to the factors affecting excess pore pressure build up. In brief, it is worth to focus upon the following main findings:

a) The general form of the relationships for Δu(N) and Δu(1) are the same for cohesive and cohesionless soils, as well as, for TX and DSS tests. However, in quantitative terms, the above factors influence excess pore pressures in a systematic way. To show this, Table 7 summarizes the sand versus clay and the TX versu DSS tests ratios of excess pore pressures Δu(1), for stress and strain controlled cyclic loading. In this way, it is observed that sands develop 2.50 to 7.70 times higher excess pore pressure than clays, regardless of loading conditions. On the other hand, TX tests develop about 1.50 times higher excess pore pressure than DSS tests under the same cyclic shear strain, and less than half of the excess pore pressure developed in DSS tests under the same cyclic shear stress.

b) In order of importance, first comes the effect of cyclic shear stress or strain amplitudes, followed by the effect of effective confining stress. The effect of void ratio is of primary importance when Δu(1) is expressed in terms of the cyclic stress amplitude but it is practically insignificant when Δu(1) is expressed in terms of the cyclic strain amplitude.

Table 7. Comparison of pore pressure development relating to either soil or test type (σ'_{oct}=100 kPa, e=0.90, γ_{cyc}=0.05÷0.50%, τ_{cyc}/σ'=0.05÷0.50)

	Test or soil type	Test coditions	Median Value	Min. Value	Max. Value
	TX	strain	4.38	3.14	10.89
Δu(1), sand	TX	stress	7.69	6.57	11.94
Δu(1), clay	DSS	strain	4.78	3.46	12.00
	DSS	stress	2.52	1.95	5.14
	sand	strain	1.34	0.94	1.52
Δu(1), TX	sand	stress	0.44	0.34	0.49
Δu(1), DSS	clay+PL silt	strain	1.50	1.16	1.64
	clay+PL silt	stress	0.28	0.23	0.51

c) The correlations for $\Delta u(1)$ are systematically better for TX than for DSS tests. This conclusion is based on the reported values of R, as well as, on the scatter of the data, in the relevant figures. It is possible, that the higher scatter in DSS tests data is attributed to the well known objective difficulties related to the materialization of the more complex boundary conditions required for this type of testing.

5 CONCLUSIONS

A set of empirical relationships has been proposed for the simplified computations of excess pore pressures in saturated soils during earthquakes. The relationships are based mostly on experimental data from cyclic loading of sands and clays. The available data on silts were too limited for an independent evaluation; however, it was observed that the response of non plastic silts is consistent with that of sands while the response of plastic silts conform with that of clays.

Apart from the effect of soil type, the proposed relarionships distinguish between constant cyclic stress and constant cyclic shear strain loading, as well as, between triaxial and simple shear test conditions. In practical applications, these choices depend upon engineering judgement, combined with thorough understanding of the prevailing loading and boundary conditions.

As a final remark, it should be noted that the data base used in the present study is neither uniform nor complete. Hence, it is reasonable to expect that the the accuracy of the proposed relationships may benefit from the evaluation of additional data. In doing so priority should be given to experiments on other soil types (e.g. silt, gravel) or soil mixtures, as well as on non-isotropic initial stress conditions.

REFERENCES

Andersen, K.H., Kleven, A. & Ronglien B., 1986. Results from cyclic laboratory tests on Drammen clay. *NGI, Report 40013-15*.

Andersen, K.H., 1988. Bearing capacity of gravity platform foundations. *NGI, Report 52422-5*.

Bouckovalas, G., Gazetas, G. 1996. Seismic bearing capacity of shallow foundations. *Final Report, Prenovative research in support of EC8 (CEC-Human Capital and Mobility, nr. ER8 CHRXCT92-011).*

DGI, 1991. Tension Leg Platform, Model tests. *Report 16006 359*.

Lambe, T.W., 1979. Cyclic triaxial tests on Oosterschelde sand. *MIT Research Report R79-24, Soils Publication No. 646.*

Malek A.M., 1987. Cyclic behavior of clay in undrained simple shearing and application to offshore Tension Piles. *Thesis submitted to the Department of Civil Engineering in partial fulfillment of the requirements of the degree of Doctor of Science in Civil Engineering*, MIT, Cambridge MA, USA.

NGI. 1973. Cyclic loading simple shear tests on sand. *Report 51505-1*.

NGI. 1977. Oosterschelde caissons. Undrained cyclic simple shear tests on Oosterschelde sand. *Report 77302-3*.

NGI. 1993. Cyclic DSS testing, Aktio-Preveza crossing. *Report 922540*.

Pierce, W. G., 1983. Constitutive relation of saturared sand under undrained loading. *Ph. D. Dissertation, Dept. of Civil Engineering*, Rensselaer Polytechnic Institute, Troy, New York.

Sangseom, J. 1988. The behavior of silt under triaxial loading. *Thesis submitted in partial satisfaction of the requirements for the degree of Master of Science in Engineering*, Davis, California, USA.

Seed, H.B. & Booker, J.R. 1977. Stabilization of potentially liquefiable sand deposits using gravel drains. *Journal of the Geotechnical Engineering Division, ASCE*, vol. 103 (7): 757-768.

StatSoft, Inc. 1995. STATISTICA for windows [*computer program*]. Tulsa.

The Earth Technology Corporation. 1992. VELACS: Verification of analyses by centrifuge studies. Laboratory testing program. Soil data report. *Earth Technology Project No. 90-0562*.