

CYCLIC LIQUEFACTION POTENTIAL OF LACUSTRINE CARBONATE SILT FROM JULIAN ALPS

BOJAN ŽLENDER and STANISLAV LENART

About the authors

Bojan Žlender
University of Maribor,
Faculty of Civil Engineering
Smetanova ulica 17, 2000 Maribor, Slovenia
E-mail: bojan.zlender@uni-mb.si

Stanislav Lenart
Slovenian National Building and Civil Engineering,
Institute, Section for Earthquake Engineering
Smetanova ulica 17, 2000 Maribor, Slovenia
E-mail: stanislav.lenart@zag.si

Abstract

This paper presents the liquefaction studies of lacustrine carbonate silt from the Julian Alp landslide Stože. Geological conditions of the region and geomechanical characteristics of the ground were investigated. The research project was performed with the intention to determine the effects of cyclic loading on lacustrine carbonate silt. Investigation with 77 cyclic triaxial tests was performed on universal triaxial apparatus Wykeham Farrance UP 100 TR, in the Laboratory of soil mechanics, Faculty of Civil Engineering, University of Maribor. The essential equipment comprises cylindrical triaxial apparatus with a cell, a press with appurtenant electro-mechanic equipment, measuring equipment, and computer hardware and software equipment. Specimens with dimensions of height = 140 mm and diameter = 70 mm were saturated, then subjected to the arbitrary initial isotropic stress state and consolidated with vertical and radial draining, and then loaded with distortional loading of chosen dynamical axial force (frequency $f = 1$ Hz). A set of tests with a spectrum of different stress states and cyclic loading were performed. This research showed that lacustrine carbonate silt is a highly sensitive material. The resistance to liquefaction was somewhat higher than that of typical clean sand, but a perceivable excess pore pressure generation, which causes the decrease of strength parameters, was noticed. Test results of cyclic triaxial tests indicate dynamic characteristics of lacustrine carbonate silt and wide applicability of the test method.

Keywords

lacustrine carbonate silt, cyclic triaxial test, liquefaction potential, shear modulus, damping ratio, cyclic stress ratio, pore pressure ratio

1 INTRODUCTION

The paper presents the research project with 77 cyclic triaxial tests performed on reconstituted samples of lacustrine carbonate silt from the Julian Alps. The interpretation of the results is included. The aim of the research was to determine material's dynamic characteristics and to study its liquefaction potential in certain in situ conditions.

Liquefaction is defined as the transformation of soil from a solid to a liquid state. It happens in consequence of increased pore pressure and reduced effective stress. Liquefaction of soil is primarily associated with medium to fine grained saturated cohesionless soils. When such a saturated soil is subjected to ground vibrations due to earthquakes, cyclic loadings or blasts, it may tend to compact or dilate, depending on a soil state. Increased pore-water pressure is induced by the tendency of soil to compact when subjected to cyclic shear deformation.

Figure 1 [1] presents the mechanism of pore pressure generation due to cyclic loading in undrained conditions. Let A be the point on the compression curve that represents the initial void ratio and effective state of stress in saturated soil. Due to a certain number of cyclic loadings, the change of void ratio of the soil occurs if full drainage is allowed (point B).

$$e = e_0 - \Delta e \quad \sigma' = \sigma'_0 = \sigma_0 - u_0 \quad (1)$$

If drainage is prevented, the void ratio will remain as an initial void ratio and the effective stress will be reduced with an increase of pore pressure. Based on the effective stress principles

$$\sigma' = \sigma'_0 - \Delta u \quad e = e_0 \quad (2)$$

where

e	void ratio
e_0	initial void ratio
Δe	void ratio change
σ'	effective stress
σ'_0	initial effective stress
σ_0	initial stress
u_0	initial pore pressure
Δu	pore pressure change

The state of the soil after a certain number of cyclic loadings in undrained conditions can be represented by point C. If the number of cycles and the load level are large enough, the magnitude of Δu may become equal to σ'_0 and the soil will liquefy.

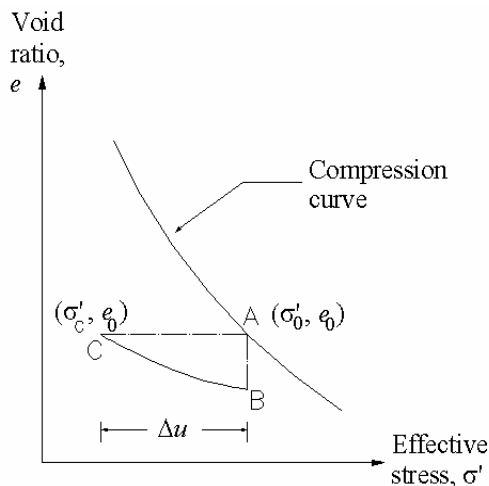


Figure 1. Mechanism of pore pressure generation due to cyclic loading in undrained conditions [1]; void ratio e vs. effective stress σ' .

2 LACUSTRINE CARBONATE SILT FROM JULIAN ALPS

Lacustrine soils can be found in different parts of Slovenia, but mostly in the region of the Julian Alps, which is located in the western part of the country and is seismically very active. During the strong earthquake which occurred in April 1998, with its epicenter in the Krn mountains and a magnitude of $M_{WA}=6.0$, an approximately 100 m long section of the shore of the 20 km distant Lake Bohinj collapsed due to liquefaction of fully-saturated lacustrine carbonate silt. At the same time the earthquake, in amplified form, caused serious damage to several buildings in the village of Mala vas near Bovec, which were founded on lacustrine soil [2]. These failure cases, which occurred during the seismic

event, drew our attention to the sensitive behavior of lacustrine carbonate silts.

Quite soon after the occurrences mentioned above, in November 2000, a very severe landslide occurred in the same region [3], in the area called Stože, which is located below Mt Mangart in the Julian Alps, at an altitude of 1340-1580 a.s.l. This area belongs to the region where glacial materials were deposited during ice melting in the Julian Alps. The width of the landslide was about 300 m, and it was 1.5 km long and up to 50 m thick. The event was a combination of two processes – sliding of the soil in the upper part, and debris flow in the lower part. Layers of lacustrine carbonate silt of relatively small thickness were observed in the material displaced during the landslide, and the question arose as to whether the presence of these layers was responsible for the landslide, and whether these layers could cause further landslides particularly in the case of later earthquakes.

The properties of lacustrine carbonate silts were investigated in view of the events described above. Deformation characteristics, such as the shear modulus and the damping ratio were studied at different strain levels using resonant column tests [4, 5]. Based on cyclic torsional tests performed on three different cyclic stress ratios, a liquefaction potential of material has been estimated. In spite of high fines content, lacustrine carbonate silts were recognized as a material for which liquefaction behavior can be expected. Compared to fine and medium sand tested by other researchers, lacustrine carbonate silt showed a little bit lower liquefaction potential. Figure 2 shows a number of load cycles causing liquefaction influenced by the cyclic stress ratio [5].

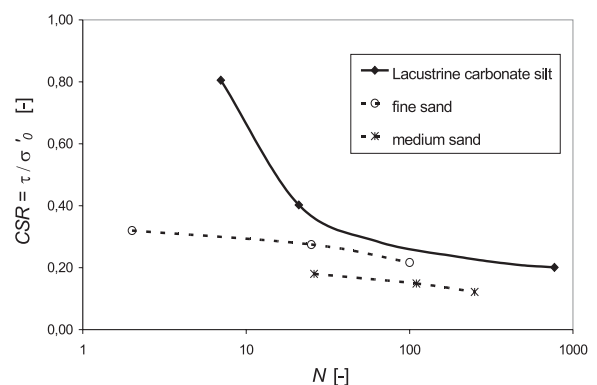


Figure 2. Number of load cycles N vs. initial load factor τ/σ'_0 [5].

Lacustrine carbonate silt also exhibited very sensitive behavior during increasing pore pressure. Figure 3 shows the increasing of pore pressure and decreasing of shear stress magnitude during cyclic loading at stress-controlled tests [5].

To make conclusions about the likelihood of liquefaction triggering in investigated material as well as to get more information about possible landslide remediation, more detailed liquefaction studies were needed. Therefore a series of cyclic triaxial tests were performed in the Laboratory for soil mechanics (LMT) at University of Maribor.

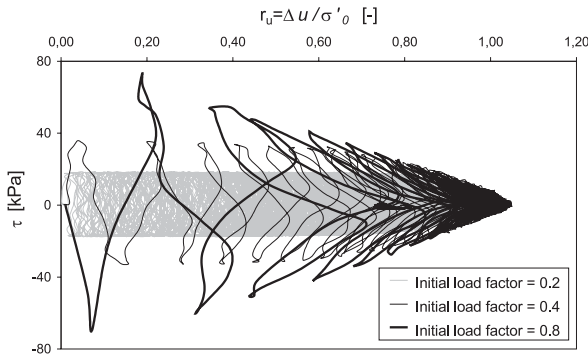


Figure 3: Pore pressure ratio $r_u = \Delta u / \sigma'_0$ vs. shear stress τ [5].

2.1 PHYSICAL PROPERTIES OF LACUSTRAINE CARBONATE SILT

It has been shown [6] that the fine sliding material, which carries big pieces of rocks, wood, etc. in a landslide, is a critical importance for the general behavior of the landslide. For this reason it was decided to take samples of the lacustrine carbonate silt, with some clay and moraine fines added, as a representative material from the investigated landslide site. Taking into account the general mixing of soils in the landslide, materials from different boreholes were mixed to prepare the sample material. Some physical properties of the sample material and the reference in situ landslide material are compared in Table 1.

Table 1. Comparison between the physical properties of the sample material and the reference in situ material [5].

Parameter	Symbol	Unit	In situ material	Sample material
Dry unit weight	γ_d	[kN/m ³]	19.9	18.0–20.0
Water content	w	[%]	13.3	7.0–13.0
Plastic limit	w_p	[%]	15.3	15.6
Unit weight of solids	γ_s	[kN/m ³]	27–28.5	28.3
Liquid limit	w_l	[%]	21.4	19.9
Plasticity index	I_p	[%]	6.0	4.3
Strength parameters	ϕ'	[°]	44	33.7
	c'	[kPa]	0	0

The grain size distribution curves of the sample material and reference landslide material are presented in Figure 4.

The gradation curve of the reference material without grains above 4 mm is almost identical to the gradation curve of the sample material.

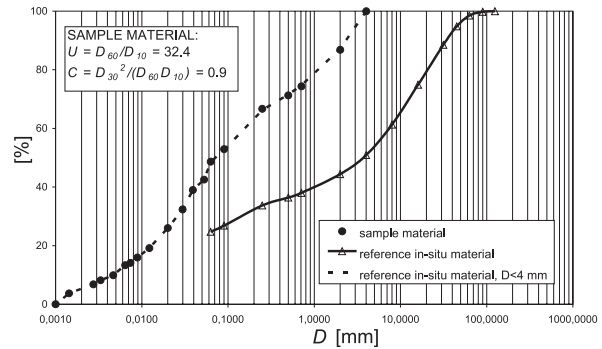


Figure 4. Gradation curve of the sample material, grain size D vs. percent finer [4].

3 TEST DEVICE

The cyclic triaxial system Wykeham Farrance applies cyclic loading to the soil specimen. The essential equipment comprises:

- load frame capacity 100 kN
- triaxial pressure cell
- hydraulic press with electro mechanical equipment
- automatic hydraulic equipment and connections for cyclic loading
- measuring and recording equipment for
 - cell pressure
 - pore pressure
 - back pressure
 - axial force (stress)
 - displacement (axial strain)
 - volume change
- control and data acquisition system
- computer hardware and software
- de-air watering apparatus
- reservoir for de-aired water
- compressor
- air-dryer

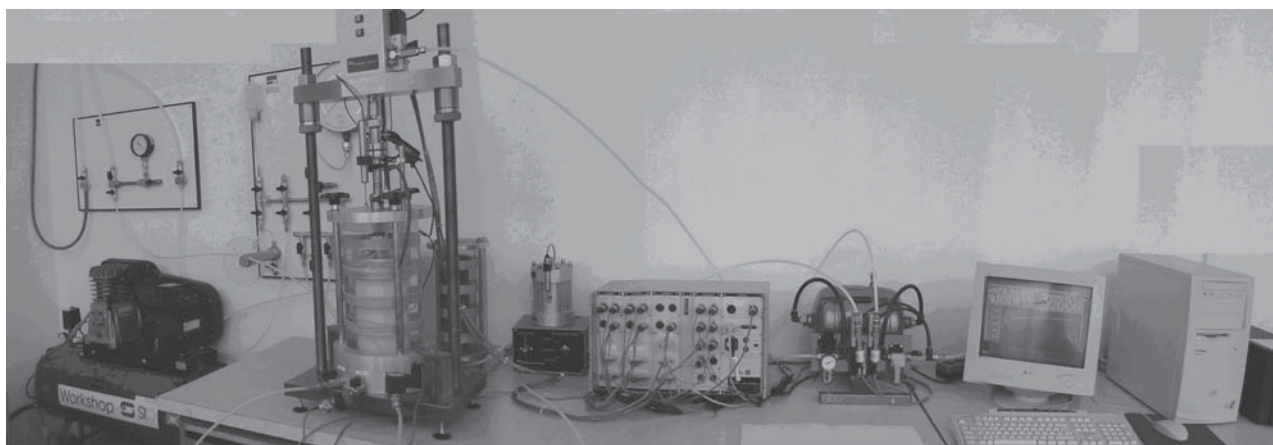


Figure 5. Universal triaxial apparatus Wykeham Farrance UP 100 TR.

4 THE PREPARATION OF TEST SPECIMENS AND TESTING PROCEDURES

Reconstituted samples were used in the tests. They were all prepared by means of wet tamping with the objective of achieving moisture contents and densities similar to those occurring naturally.

Table 2. Initial properties of specimens.

Parameter	Symbol	Unit	Value
Height	H	[mm]	140
Diameter	D	[mm]	70
Volume	V	[cm ³]	583.51
Void ratio	e_c	[-]	0.49
Degree of saturation	S_r	[-]	0.76
Water content	w_0	[%]	13.11
Unit weight	γ_0	[kN/m ³]	21.49
Unit weight of solids	γ_s	[kN/m ³]	28.30
Dry unit weight	γ_d	[kN/m ³]	19.00

The test procedure was done in the following steps: preparing of specimens, saturation and consolidation of specimens, undrained stress controlled cyclic loadings of specimens, test performance and interpretation of results.

In saturation phase, the saturation check was done according to the criteria of ratio $B = u/\sigma > 0.96$.

After the saturation of the specimen was completed, the sample was consolidated with radial draining at selected effective isotropic consolidation stress σ'_0 , expressed as difference of cell pressure σ_0 and back pressure u_b .

The research was performed with a set of tests, where conditions varied as shown in Table 3.

Table 3. Conditions of tests.

Parameter	Symbol	Unit	Value
Initial effective stress	σ'_0	[kPa]	50, 100, 150, 200
Void ratio	e_c	[-]	0,32 ÷ 0,49
Cyclic stress ratio	CSR	[-]	0,15 ÷ 0,30

Cyclic loading was performed with the selected frequency $f = 1$ Hz and the proper cyclic load amplitude σ_a . It is supposed that frequency did not essentially influence on the results.

During the cyclic test the following parameters were measured:

- $\sigma_0(t)$ cell pressure
- $u_b(t)$ back pressure
- $u(t)$ pore pressure
- $\sigma_c(t)$ peak cyclic stress in compression
- $\sigma_e(t)$ peak cyclic stress in extension

From measured stresses, the cyclic stress ratio CSR and the average cyclic stress ratio CSR_{ave} , for $n=1$ to m cycles are calculated.

$$CSR = \frac{\sigma_a}{2 \cdot \sigma'_0} \quad (3)$$

$$CSR_{ave} = \left[\frac{1}{m} \cdot \sum_{n=1}^m CSR_n \right] \quad (4)$$

where

$$\sigma_a = \frac{\sigma_c + \sigma_e}{2} \quad (5)$$

The cyclic pore pressure ratios r_u is expressed as

$$r_u = \frac{\Delta u}{\sigma'_0} \quad (6)$$

Due to ground shaking during an earthquake, a cyclic shear stress is imposed on the soil element. A laboratory test to study the liquefaction problem must be designed so as to simulate the condition of a constant normal stress and a cyclic shear stress on the plane of a soil specimen. As the actual triaxial tests can be conducted by applying a cyclic load in the axial direction only, the corrected cyclic pore pressure ratio $r_{u,corr}$ is used.

From the corrected cyclic pore pressure

$$\Delta u_{corr} = \Delta u - \frac{\sigma_a}{2} \quad (7)$$

the corrected cyclic pore pressure ratio $r_{u,corr}$ is expressed as

$$r_{u,corr} = \frac{\Delta u - \frac{\sigma_a}{2}}{\sigma'_0} \quad (8a)$$

$$r_{u,corr} = r_u - CSR \quad (8b)$$

Axial deformations in compression $\epsilon_c(t)$ and extension $\epsilon_e(t)$ are measured. The double amplitude axial strain $\epsilon_{da}(t)$ is calculated. Because the test is performed in undrained conditions, the volumetric strain is $\epsilon_v(t)=0$ and Poisson ratio is assumed to be $\nu=0.5$.

The calculation of dynamic strength parameters, the Young modulus E , the shear modulus G and the damping ratio is performed for a given hysteresis loop.

$$E = \frac{L_{da} \cdot H_s}{S_{da} \cdot A_s} \quad (9)$$

$$G = \frac{\tau}{\gamma} = \frac{E}{2 \cdot (1 + \nu)} \quad (10)$$

$$\zeta = \frac{A_L}{4\pi A_T} \quad (11)$$

where

L_{da}	cyclic load in compression or extension (kN)
S_{da}	cyclic deformation in compression or extension (mm)
H_s	height of specimen after consolidation (mm)
A_s	area of specimen after consolidation (mm ²)
	Poisson ratio (-)
A_L	area of the hysteresis loop
A_T	area of hysteresis triangle

If results are to be used for practical purposes, the axial strain ϵ in the triaxial mode should be converted to the shear strain γ in the simple shear mode according to the relation $\gamma=1.5 \cdot \epsilon$ [7].

Two kinds of criteria were used to determine liquefaction triggering during cyclic triaxial tests: pore pressure becomes equal to the effective confining pressure (the required number of cycles was marked as N_L), and axial strain reaches a 5% double amplitude (the required number of cycles was marked as N_{Le})

The set of results was obtained varying afore mentioned conditions. The interpretation of results ($N_L, N_{Le}, \epsilon_{da}, E, G, \zeta, r_u$) is given versus N, σ'_0, CSR and e_c .

5 CONSOLIDATION PARAMETERS AND PERMEABILITY OF TESTED MATERIAL

Further, within a research, physical characteristics of lacustrine carbonate silt were determined, and strength tests were performed on static loading. The results of these tests are briefly summarized in [2] and supplemented in [5] and [8]. Triaxial consolidation tests were carried out at different effective stress changes σ'_0 , and typical results are presented in Figure 6.

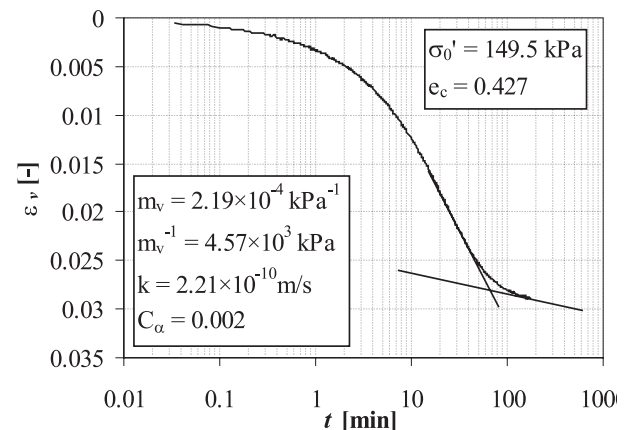


Figure 6. Typical result of a consolidation test; volumetric strain ϵ_v vs. time t .

It has been found that consolidation parameters depend upon void ratio e and effective stress σ'_o . Their average values are given in Table 4. Consolidation coefficient c_v and coefficient of soil permeability k are shown as a function of the void ratio e (Figures 7 and 8) and the effective stress σ'_o . The impact of different stress states is evident from the material loose state (e_c above 0.4).

Table 4. Consolidation parameters according to British standards [9] (the average value for a set of tests).

Parameter	Symbol	Unit	Value
Consolidation coefficient	c_v	[m/s]	$3.2 \cdot 10^{-8}$
Coefficient of volume compressibility	m_v	[kPa ⁻¹]	$2.6 \cdot 10^{-4}$
Coefficient of soil permeability	k	[m/s]	$1.2 \cdot 10^{-10}$
Secondary compression ratio	C_α	[-]	0.001

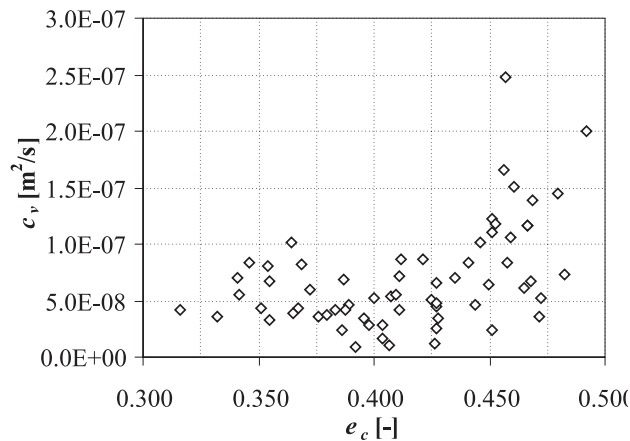


Figure 7. Consolidation coefficient c_v vs. void ratio e_c

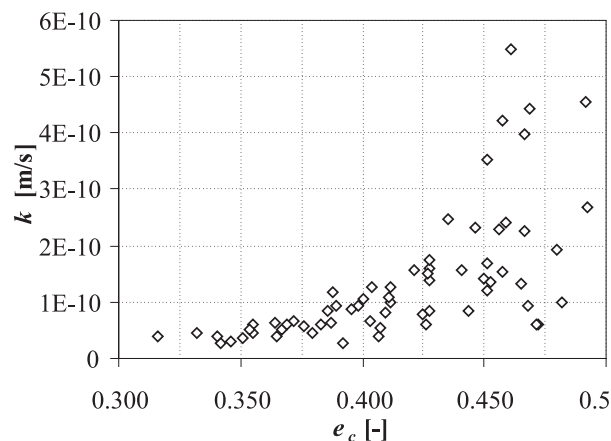


Figure 8. Coefficient of soil permeability k vs. void ratio e_c

6 CYCLIC TRIAXIAL TESTS RESULTS

Figures 9, 10 and 11 show typical varying of pore pressure and its corrected value with increasing number of cycles and shear strain.

In a set of liquefaction tests the pore pressure under cyclic loading rapidly increased, and reached over 50% of effective cell pressure in several tests after only a few cycles. After that the pore pressure increased more slowly and closed before liquefaction rapidly increased again (Figure 9).

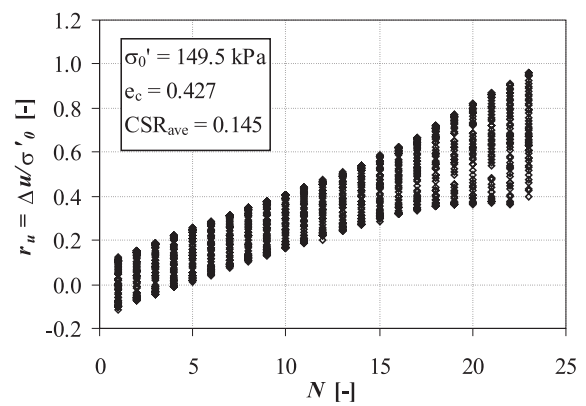


Figure 9. Pore pressure ratio r_u vs. number of cycles N .

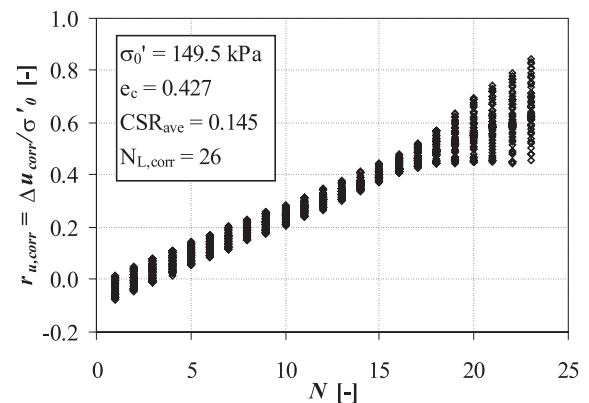


Figure 10. Corrected pore pressure ratio $r_{u,corr}$ vs. number of cycles N .

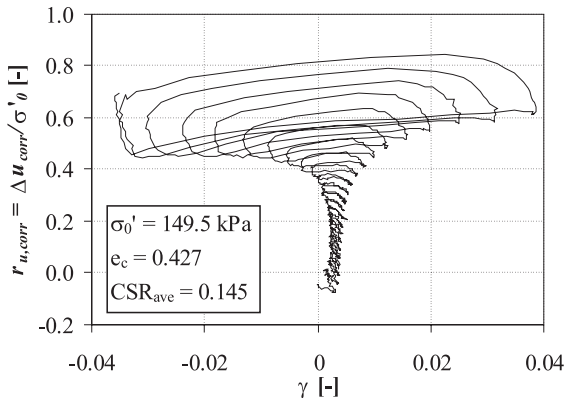


Figure 11. Pore pressure ratio $r_{u,corr}$ vs. shear strain .

Figure 12 shows the double amplitude axial strain ϵ_{da} vs. a number of cycles N .

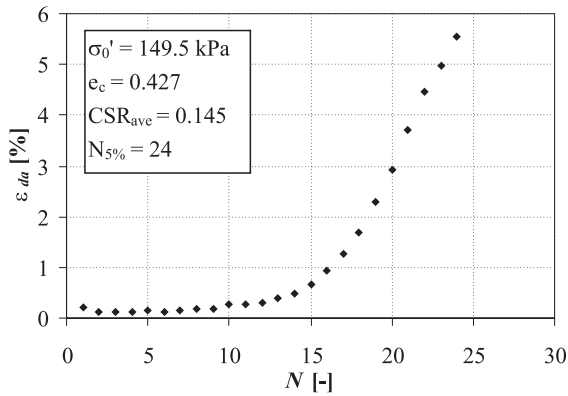


Figure 12. Double amplitude axial strain ϵ_{da} vs. number of cycles N .

A deformation due to cyclic loading depends on the cyclic stress ratio CSR and the porosity e . Generally, deformations are high. A typical stress-strain curve is shown in Figure 13.

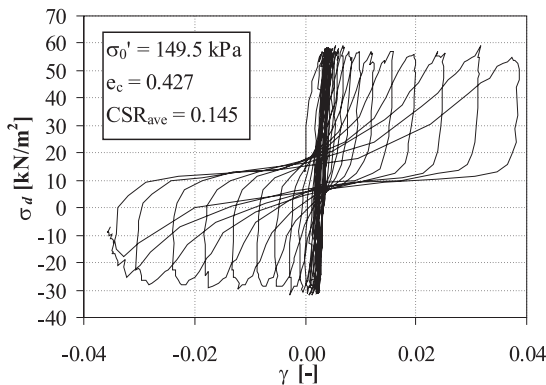


Figure 13. Deviator stress σ_d vs. shear strain

The damping ratio was calculated based on hysteresis loops in the stress-strain curve for each test. Equation (11) was used. It has been found that the damping ratio increases in the first few cycles. After reaching its maximum value, the damping ratio starts to decrease. It has been observed that during every cyclic test the damping ratio limits to a certain value when the material has liquefied. The results, presented in Figure 14 show the dependence of damping ratio limit values on the void ratio of a specimen.

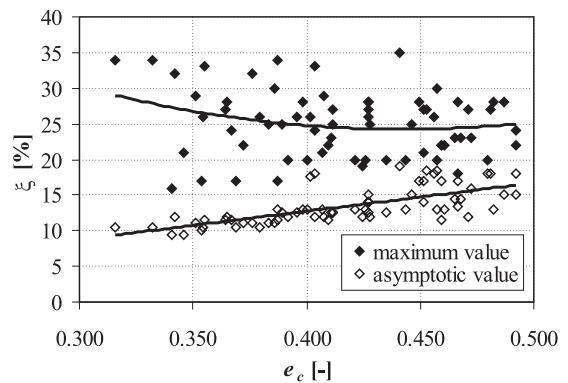


Figure 14. Range of limit values of the damping ratio upon the void ratio e_c

The varying of material stiffness was observed during cyclic loading. Young's modulus E and the shear modulus G rapidly decreased after some cycles and after that they strongly decreased to lower values (Figure 15). The impact of compression or extension is negligible.

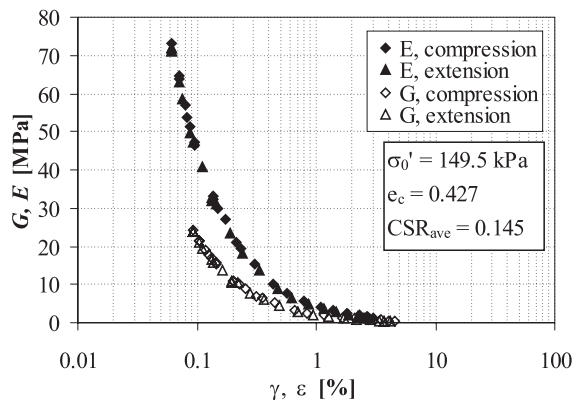


Figure 15. Young's modulus E and shear modulus G vs. strain γ, ϵ .

The following figures 16 and 17 summarize the cyclic triaxial test results. Two aspects of soil liquefaction are presented, taking into account the above liquefaction criteria: the pore pressure increase and the strain increase. It is obvious that the material is much more sensitive in a state with a higher void ratio. The number of cycles needed to reach liquefaction (pore pressure ratio or strain) is low in that case. The number of cycles needed for liquefaction substantially increases in a dense state of material.

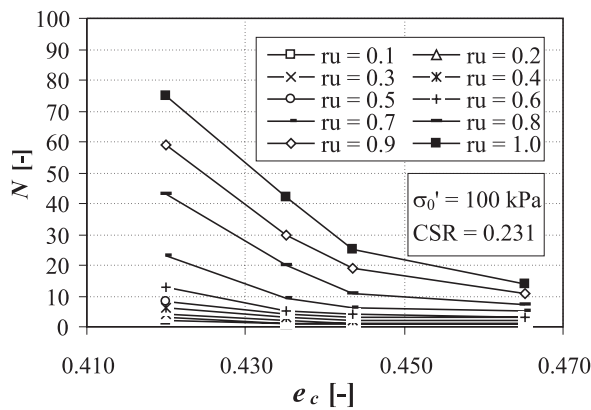


Figure 16. Number of cycles N needed to reach given r_u vs. void ratio e_c .

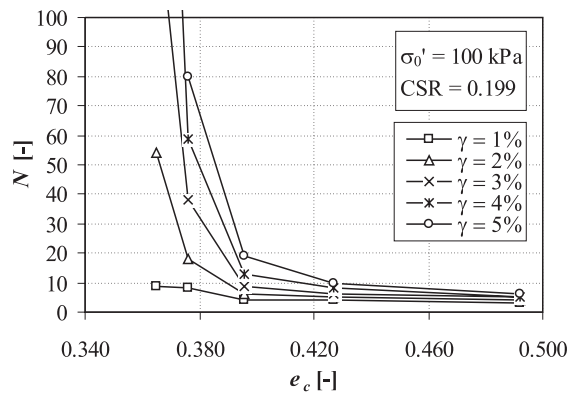


Figure 17. Number of cycles N needed to reach given γ vs. void ratio e_c .

7 CONCLUSIONS

The introduction of cyclic triaxial tests presents important completion of the research of geomechanical characteristics of lacustrine carbonate silt from the landslide Stože.

The non-linear behavior of the tested material is presented as a decrease of soil stiffness with an increase in the strain amplitude. Regardless of compression or extension, the Young modulus and the shear modulus of material change similarly (a decrease with an increase of strain), while the damping ratio increases at the beginning of cyclic loading and starts decreasing later on. Both, the maximum damping ratio value and the final value of damping ratio after soil liquefaction, depend on the void ratio of a specimen.

Perceivable changes in material behavior start when the pore pressure ratio exceeds $r_u = 0.4-0.5$. The number of load cycles needed to cause an increase of the pore pressure ratio to this value does not greatly depend on the material state, while differences in the behavior of loose and dense materials are noticeable above this value. The impact of the void ratio on the liquefaction depends on the liquefaction criteria. When talking about the pore pressure increase, the boundary value is around $e_c \approx 0.44$. Liquefaction resistance of the material with the void ratio below this value significantly increases. When liquefaction is estimated based on strain dependent criteria, the void ratio boundary value is around $e_c \approx 0.40$.

Based on cyclic triaxial test results, lacustrine carbonate silt was recognized as a highly sensitive material. With the aim of avoiding later liquefaction triggering, a proper maximum void ratio of the deposited landslide material should be assured.

Performed cyclic triaxial tests show dynamic characteristics of lacustrine carbonate silt and the applicability of such cyclic loading triaxial tests.

Presented research is still in progress and will be finished in 2005.

ACKNOWLEDGMENTS

This research is financed by the Slovenian Ministry of Higher Education, Science and Technology. The financial support of the Motorway Company in the Republic of Slovenia - DARS and the company Prevent is also gratefully acknowledged.

REFERENCES

- [1] Das, B.M. (1993). Principles of soil dynamics. *PWS-KENT Publishing Company*, 397-453.
- [2] Petkovšek, A. (2001). Geological - geotechnical investigations of the Stože landslide. *Ujma*, No.14-15, 109-117.
- [3] Lenart, S. (2004). Stože landslide - a case history. *Proc. 5th International Conference on Case Histories in Geotechnical Engineering*, New York.
- [4] Lenart, S. (2002). Shear modulus and damping ratio of reconstituted moraine samples in the small strain range. *Proc. 15th European Young Geotechnical Engineers' Conference*, Dublin, Ireland, pp. 67-76.
- [5] Lenart, S. Deformation characteristics of lacustrine carbonate silt in the Julian Alps. *Soil Dynamics and Earthquake Engineering*, (to be published).
- [6] Brandl, H. (1992). Retaining structures for rock masses. *Engineering in Rock Masses*, Butterworth Heinemann, Oxford, England.
- [7] Ishihara, K. (1993). Liquefaction and flow failure during earthquakes. *Geotechnique* 43, No. 3, 351-415
- [8] Lenart, S. (2003). Dynamic characteristics of soils in the case of reconstituted gravely sandy siltstone from Stože landslide (in Slovene). *MSc. Thesis*, University of Ljubljana, Ljubljana.
- [9] British Standards Institution (1999). Consolidation. *BS 1377*, London