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Application of Back Pressure for Saturation of Soil Samples in Cyclic Triaxial Tests

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Abstract. This paper presents a review on the effect of saturation and back pressure application and discusses the practical aspects of back pressure application. Previous studies suggested use of back pressure during saturation phase; however there is no clear guideline for the level of back pressure that is appropriate to use in triaxial testing. Some researchers even suggested that circulation of carbon dioxide (CO_2) should be used to expedite the saturation process instead of using back pressure to enhance the degree of saturation of tested sands in liquefaction tests. Nonetheless, different levels of back pressure 100, 200, 300 and even higher are typically used in sample preparation to achieve better degree of saturation (higher levels of Skempton's pore pressure coefficient). The level of saturation influences the cyclic behaviour and strength. Therefore, there is a need to assess the proper level of back pressure to simulate real field condition. Examples of testing results on non-cohesive soil are presented to demonstrate how the saturation and back pressure levels affect the behaviour of tested specimen. The importance of realistic representation of field conditions in laboratory testing is demonstrated through examples of soils that are buried under impounded water such as marine sands. In cases where marine sands are buried under high water depths, the level of impounded water can be modelled in triaxial testing by applying different levels of back pressure. Finally, suggestion for the level of back pressure to properly simulate field condition is presented.

Keywords. Triaxial, saturation, back pressure, field behavior.

1. Introduction and background

For laboratory testing of soils, standards and codes specify that a soil sample can be considered to be fully saturated if Skempton's pore pressure coefficient B is over 0.95. Pore pressure parameter B-value is the size of the pore pressure response during an undrained loading increment prior to consolidation. The relationship between pore pressure coefficient B and degree of saturation is associated with the compressibility of soil skeleton and voids (i.e., water-air system) as well as specimen's density [1].

In triaxial testing, the back pressure technique is used to enhance saturation of soil sample. In this technique, the cell pressure and back pressure are increased simultaneously to achieve the desired degree of saturation. Application of back pressures as an effective method to achieve high degree of saturation (i.e., large pore pressure coefficient B) has been suggested and employed in many laboratory studies [2 to 6] For some soil samples, satisfactory level of saturation can only be achieved by applying back

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pressures higher than 500 kPa [2 and 7]In cases that the use of CO_2 might cause alteration of soil mineralogy due to undesired chemical reaction between soil elements, de-aired water and carbon dioxide, large values of back pressure without circulation of CO_2 can also be useful to increase degree of saturation and decrease saturation time [8].

Increase in the specimen pore pressure to achieve desired saturation is based on compressibility and solubility of air in water. Compressibility is controlled by Boyle's law (inversely proportional relationship between gas volume and its pressure); and solubility is controlled by Henry's law (describes dissolution of gas in liquid at constant temperature). Back pressure method is based on Terzaghi's effective stress principle stating that soil exhibits similar behaviour and mechanical properties under same effective stresses (i.e., forces applied on the soil are carried by soil skeleton) [9].

High B-value is an indicator of good saturation for loose sand, but is less critical for stiffer material [10 and 11]. Baldi et al. related acceptable levels of saturation, indicated by B values, to soil compressibility and stated that B value greater than 0.95 may not be a valid criterion of saturation in all cases [12]. Depending on the soil type (i.e., fine grained or course grained), density and condition (i.e. disturbed/reconstituted and undisturbed), different levels of back pressures can be used to achieve a satisfactorily fully saturated specimen. For example, it has been reported that the undisturbed soil samples of till material required larger pressures of about 750 kPa, whereas, reconstituted soil specimens saturated using lower level of back pressure of about 200 kPa. At the same B-value, materials of low compressibility were reported to have higher degree of saturation compared to material of high compressibility [13]. Results of triaxial tests on over-consolidated cohesive soils showed that effect of back pressure on the B-value depends on the soil properties [14]. B-value, could also show the sensitivity of the pore water pressure under undrained loading. Stress condition which affects the compressibility of the sample influences the B-value [15].

This paper presents a review on the effect of saturation and back pressure application and discusses the practical aspects related to application of back pressure. Previous studies suggested use of back pressure for saturation; however there is no clear guideline for the level of back pressure that is appropriate to use in triaxial testing. Some researchers also suggested that the back pressure technique cannot be used to enhance the degree of saturation of tested sands in liquefaction tests. Alternatively, circulation of CO_2 is suggested as a means to expedite the saturation process and eliminate the need for high level of back pressure. Different levels of back pressure 100, 200, 300 and even higher can be used to obtain higher levels of Skempton's pore pressure coefficient. Examples of testing results on non-cohesive soil are presented to demonstrate how the saturation level and back pressure level affect the tested specimen behaviour. The importance of realistic representation of field conditions in laboratory testing is also discussed. Finally, the level of back pressure for proper simulation of field condition is discussed.

2. Review and discussion of results

This paper reviews the saturation technique by increasing cell pressure and back pressure simultaneously, which involves saturation with drainage open to maintain a set effective stress and then simultaneous increases in the back pressure and the cell pressure. B-value measurement is performed in small undrained stages. Small effective stress changes and consequently volume changes occurs before attainment of saturation which is monitored by changes in specimen's height [12]

This study focuses on samples with degree of saturation of more than 95%, which are considered fully saturated. Nearly saturated non-cohesive soils usually experience pore pressure generation and reduction in effective stress and strength under loading in undrained condition. Shearing of soil can cause volume change. When no volume change is allowed, change in pore fluid pressure occurs due to its incompressibility. Difference between actual pore pressure at each stage of loading and the initial pore pressure is defined as excess pore pressure. It is of interest to know how the level of initial pore pressure would affect the resistance and cyclic behavior. In other words, how the level of initial pore fluid pressure (i.e., water) affects the process of transforming solid state to liquefied state.

Effect of back pressure application on cyclic behavior of sand was investigated by [9]. They reported that under the same effective stress and degree of saturation, liquefaction resistance of the tested sand saturated by using back pressure is higher than resistance of samples saturated without back pressure application. Back pressures of 0, 98.1, 196.2, 298.3 and 382.4 kPa were used in their study and higher resistances were observed in samples saturated under higher levels of back pressures. Changes in the dynamic effective stress path by application of back pressure was observed and stress path showed similar shapes as of denser sand samples compared to the stress path diagrams of samples saturated with lower or zero back pressure. Donaghe and Townsend reported no significant effect of back pressure magnitude on test results of dilative specimens of silt [16].

2.1. Present Experimental Results

As part of the present study, the authors tested silty sand samples that were saturated without circulation of CO_2 and by sole application of high levels of back pressure to achieve B-values higher than 0.95. It was observed that under the same stress state (i.e., effective stress and void ratio) and nearly saturated condition (i.e., B-values of 0.95), samples tested under higher levels of back pressure (i.e., 800 kPa) had higher resistances to cyclic loading (Figure 1) and liquefaction compared to those saturated under lower levels of back pressure (i.e., 700 kPa).

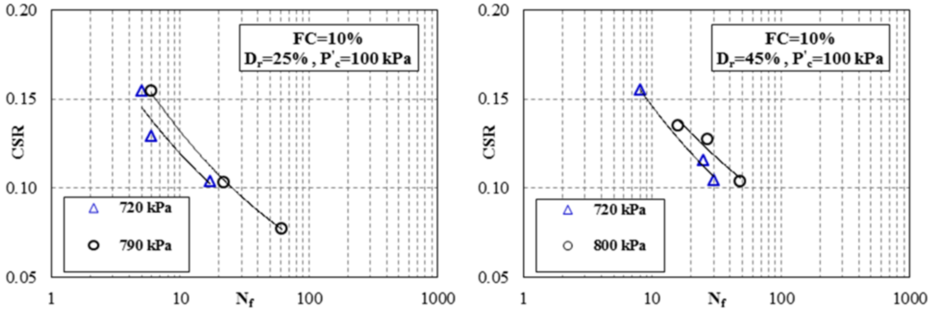

Figure 1. Effect of back pressures on cyclic resistance ratio.

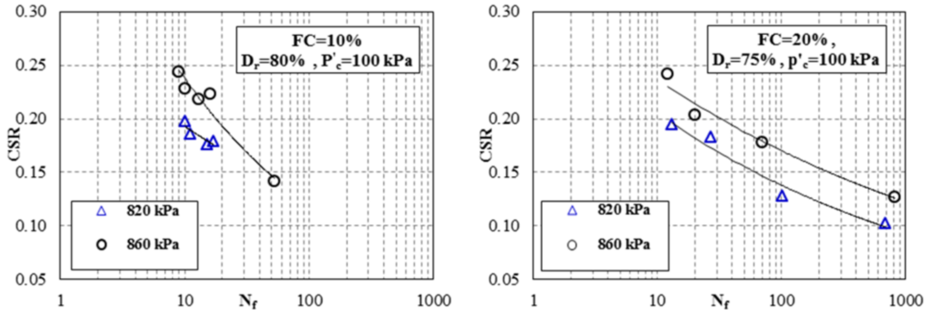

Figure 1. (*continued*) Effect of back pressures on cyclic resistance ratio.

Unit energy required to liquefy samples was also calculated from the accumulated area enclosed by the hysteresis loops of stress-strain cycles up to the onset of liquefaction. Unit energy required for liquefaction was higher in samples saturated under higher levels of back pressure (Figure 2).

It was observed that dense samples that showed dilative behavior generally required higher levels of back pressure (i.e., 850 kPa compared to 700 kPa) to enhance saturation and to achieve B-value of higher than 0.95.

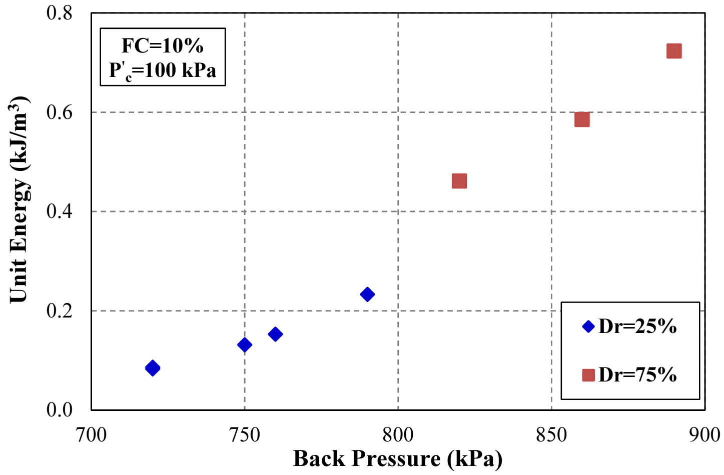

Figure 2. Effect of back pressure on required unit energy for liquefaction.

2.2. Discussion

Xia and Hu suggested that the back pressure can be viewed as the initial hydrostatic pore pressure that initially existed in sand before application of loading and that different dynamic effective stress paths and higher resistance to liquefaction are related to additional confinement to the movement of the sand particles caused by the extra hydrostatic pressure (i.e., higher back pressures) on the samples [9]. They related this behaviour to the microstates change in the transfer (e.g., point, line, or surface contacts) and interaction (e.g., frictional actions or push actions) of the inter-particle forces while the macro effective stress states (i.e., resultant forces on the soil skeleton) are constant.

They showed that the use of back-pressure technique in liquefaction test of the sand distorts the test results and related this to the change in the shrinkage property of the soil skeleton because of the change in the inter-particle forces transfer and act due to the viscous property of water and molecular forces in the absorbed water layer. They stated that the observed differences in liquefaction resistance under different levels of back pressure could add a condition to Terzaghi's effective stress principle that soil properties not only depends on the effective stress state but also on the interparticle forces.

The practical example of the case can be soil layer under impounded water in a reservoir, lake or at seabed. Following discussion presents that pore fluid compressibility plays an important role in the amount of energy that is required to generate pore water pressure by changing the compressibility of the whole medium.

Two fully saturated soil elements under similar effective stress that are buried under two different levels of water height are assumed for comparison (Figure 3). These two soil elements have been saturated under different levels of impounded water or hydrostatic pressures. Laboratory studies showed that these two elements have different resistances to cyclic loading under undrained condition.

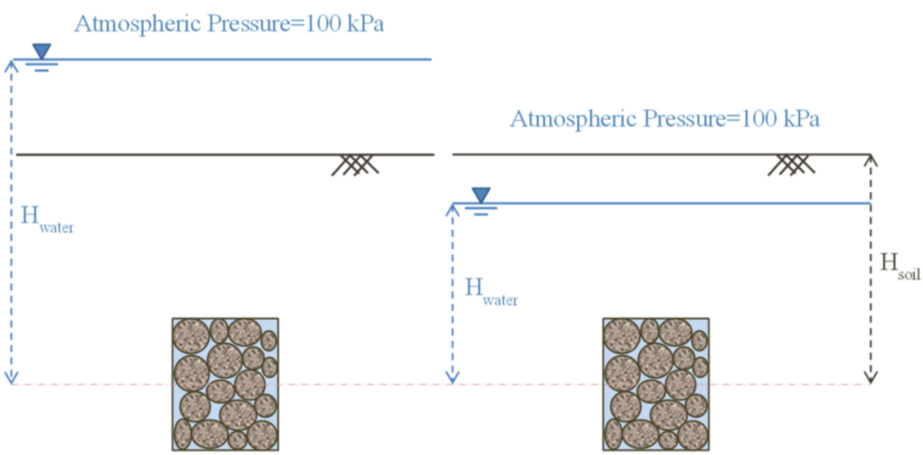

Figure 3. Sample soil element different hydrostatic pressures and similar effective stresses.

Each soil element can be considered as two-phase medium composed of solid material/soil skeleton and pores filled with water and gas. Tendency of compressive volume change of this two-phase medium depends on soil skeleton compressibility and pore fluid compressibility. This means that compressive stiffness of soil medium is a combination of soil skeleton stiffness and pore fluid stiffness (Figure 4). In other words, soil specimen exhibits compressibility of soil skeleton and pore fluid. Soil skeleton behaves macroscopically as a solid material and shear forces can be transferred through solid intergranular contacts if normal forces are transferred at the same time. Soil skeleton and pore fluid transfer normal forces whereas shear forces can only be transferred through soil skeleton by friction action [17].

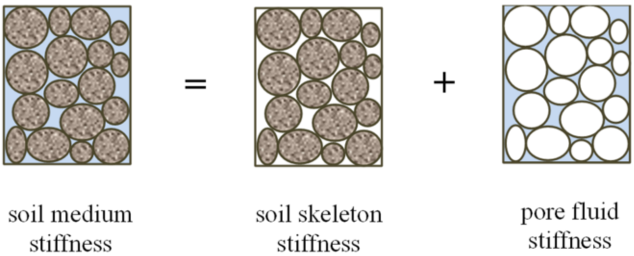

Figure 4. Compressibility of soil as two-phase medium.

Compressibility of pore fluid (i.e. water) is a function of pressure and temperature. Compressibility decreases with increase in pressure and decrease in temperature as a result of dissolution of gas in liquid. While pore fluid stiffness increases under pressure, compressibility of the soil skeleton is not influenced by this change.

Completely saturated water has negligible compressibility compared to the compressibility of the soil skeleton only. Compression stiffness of the pore fluid has inverse relationship with pore fluid compressibility and depends on the absolute pressure as well as content of free gas that is not solved in liquid. Following expression show these relationships [17]:

$$\text{Compression Stiffness of Pore Fluid} = \frac{1}{\text{Pore Fluid Compressibilty}} \tag{1}$$

$$\text{Compression Stiffness of Pore Fluid} \approx \frac{\text{Absolute Pressure}}{\text{Percentage of Free Gas in Fluid}} \tag{2}$$

At room temperature and under atmospheric pressure of approximately 100 kPa, pore water with 1% of gas will have compression stiffness of about 10 MPa. Where the absolute pressure equals approximately 0.2 MPa (e.g. at 10 m below mean sea level), pore water with 1% of gas will have compression stiffness of 20 MPa. This level of pore fluid compressibility may be in the same order of the compressibility of the soil skeleton and for many types of liquefaction cannot be neglected if the gas content is large [17]. For example, sandy seabeds have often gas content of larger than 0.3% [18].

Under large levels of hydrostatic pressures, that is higher absolute pressures, compressibility of the pore water decreases and pore fluid stiffness increases. The higher pore fluid stiffness adds up to the soil skeleton stiffness and results in generally lower compressibility of the whole medium under shearing deformation as it prevents normal loads be transferred through soil particle contacts. This results in reducing soil contraction tendency and rate of pore pressure generation. Work done through shear forces and displacements is transformed to compressive forces carrying by pore fluid in the form of pressure change. Under higher back pressures more energy is required to reach threshold strain and produce excess pore pressure. Therefore, larger effort is required to decrease shear strength and higher cyclic resistances are obtained as such. Larger effort means more work/energy is required for generation of excess pore water pressure. Absolute pressure in field condition and laboratory condition can be defined as follows:

- **In field condition:** Absolute Pressure=Hydrostatic Pressure + Atmospheric Pressure
- **In laboratory testing (controlled condition):** Absolute Pressure= Back Pressure

Since pore water transfer normal/compressive forces, therefore, higher resistance to liquefaction is expected from samples under higher in-situ hydrostatic pressure or laboratory back pressure. As larger energy is required to increase pore pressure and trigger liquefaction. Therefore, back pressure can be estimated based on hydrostatic pressure (i.e., head of impounded water) plus atmospheric pressure. This means that lower levels of water head in field suggests using lower levels of back pressures which requires longer time for saturation whereas large head of water suggests using large back pressures which requires less time and effort for saturation. Following table summarizes examples of pore fluid stiffness in different conditions assuming that pore fluid has 1% of gas content.

Table 1. Pore fluid stiffness under different pressure conditions.

Back Pressure (kPa)	Absolute Pressure (kPa)	Hydrostatic Pressure (kPa)	Equivalent Water Height (m)	Gas Content	Pore Fluid Stiffness (MPa)
200	200	100	10	1%	20
300	300	200	20	1%	30
500	500	400	40	1%	50
700	700	600	60	1%	70

For soil elements under high level of water table or impounded water that contains oxygen (O_2) gas, application of large back pressure would be more realistic and no circulation of CO_2 is required to facilitate the saturation process as the high level of initial pore pressure/back pressure enhances saturation. While for samples located at shallower depths and under shallow depths of water that have higher possibility of getting exposed to rainfall which contains amounts of CO_2, circulation of CO_2 along with application of lower initial pore pressure/back pressure will model in-situ condition more appropriately. CO_2 get dissolved in water under lower pressures as its solubility is different than O_2. Therefore, decision on using CO_2 and the level of back pressure totally depends on the in-situ condition. Samples under CO_2 circulation and lower back pressure versus samples under higher back pressures are both considered saturated if B is higher than 0.95, however, pore pressure stiffness is different and results in different levels of resistances to development of excess pore pressure.

As stated by De Groot et al. [17], complete liquefaction due to full development of pore pressure and strength reduction is not a frequent event around marine structures [17]. However, partial pore pressure generation and strength reduction resulting in a large deformation is observed more often. Large level of impounded water in sea which results in negligible pore fluid compressibility could be one of the reasons of less frequent occurrence of complete liquefaction.

Tani and Hasegawa reported that earthquake damage due to slide failure and lateral deformation of small earth dam embankments occurs more often on the upstream side of a dam and when sandy soils are presents in foundation or embankment [19]. Fukushima Prefecture, related the failure of upstream side of the dam to strength decay of saturated sandy soils and high level of impounded water [20]. Sendir et al. reported greater deformation on the upstream side of the dam than on the downstream side even for gentler upstream slopes [21]. Figure 5 shows how the level of initial pore pressure/back pressure on soil cyclic resistance can affect stability of the upstream slope of an embankment. Soil elements located in the body of the earth structure are under similar soil height (i.e., similar effective stress) and different water height (i.e., different hydrostatic pressures). Soil element A has lower cyclic resistance compared to soil element D as they are buried under different levels of impounded water resulting in varying pore fluid stiffness. Soil element A is also prone to higher cyclic stresses due to amplification of motion. This variation of resistances in different soil elements could dictate the failure surface pattern. Stress method for liquefaction assessment considers use of effective stress. If the effect of pore fluid stiffness be ignored then analysis using soil resistances derived from lab tests without proper simulation of field condition might results in misleading interpretations.

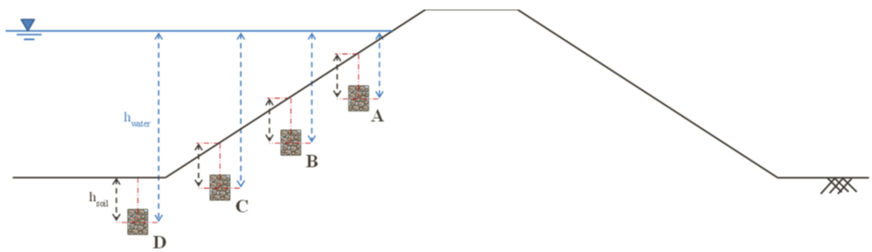

Figure 5. Hydrostatic pressure and overburden pressure on soil element in body of an embankment.

3. Conclusions

Discussion presented in this study showed that factors that contribute to liquefaction resistance include both B-value and pore fluid stiffness. Tendency for volume change of soil medium depends on soil skeleton stiffness and pore fluid stiffness. Therefore, hydrostatic pressure acting on sample in real field condition needs to be considered for choosing the level of back pressure. Due to time limitation in performing laboratory testing, saturation is often achieved using high levels of back pressures that are not necessarily representative of field conditions. For testing samples of loose to medium density that are expected to have contractive behaviour and large development of pore water pressures, it is recommended to test at least three samples under different levels of back pressures to estimate the effect of initial pore pressure in behaviour. At least one of the levels of back pressure needs to be representative of field condition even if application of lower back pressures requires longer period of time to achieve high levels of B-values.

The effect of applied back pressure on tests of cohesive samples that have the capability for water absorption needs to be investigated in future studies.

References

[1] Skempton, A. W., The pore pressure coefficient A and B, *Geotechnique* **4** (1954), 143-147.
[2] Lowe, J., and Johnson, T. C., Use of back pressure to increase degree of saturation of triaxial test specimens, *Proceedings of Research Conference on Shear Strength of Cohesive Soils*, American Society of Civil Engineers (1960), 819-836.

[3] Bishop, A. W., and Henkel, D. J., *The Measurement of Soil Properties in the Triaxial Test*, 2nd ed., Edward Arnold, London (1962).
[4] Black, K. D. and Lee, K. L., Saturating laboratory samples by back pressure, *Journal of the Soil Mechanics and Foundation Division (ASCE)* **99** (1973), 75-93.
[5] Saada, A. S. and Townsend, F. C., State of the art: Laboratory strength testing of soils, In R. N. Yong and F. C. Townsend, (eds.), *Laboratory Shear Strength of Soil*, American Society for Testing and Materials (ASTM), Philadelphia, STP **740** (1981), 7-77.
[6] Berre, T., *Suggested International Code of Soil Engineering Practice for Consolidated Triaxial Tests*. Norwegian Geotechnical Institute, Internal Report 56103-30, 1986.
[7] Soliman, N., Effect of geologic history on the design parameters of heavily overconsolidated till, In R. N. Yong (ed.), *Geological Environment and Soil Properties*, Special Publication of the American Society of Civil Engineers, Geotechnical Engineering Division, 1983.
[8] Naghavi, N., *Liquefaction Assessment of Carbonate-Silica Silty Sands Using Energy, State Parameter and Shear Wave Velocity*, Electronic Thesis and Dissertation Repository 4820, 2017.
[9] Xia, H. and Hu, T., Effects of saturation and back pressure on sand liquefaction, *Journal of Geotechnical Engineering* **117** (9) (1991), 1347-1362.
[10] Mulilis, J.P., Townsend, F.C., Horz, R.C., Triaxial testing techniques and sand liquefaction, *Dynamic Geotechnical Testing ASTM* STP **654** (1978) 265-279.
[11] Tatsuoka, F., Sonoda, S., Hara, K., Fukushima, S., Pradhan, T.B.S., Failure and deformation of sand in torsional shear, *Soils and Foundations* **26** (4) (1986), 79-97.
[12] Baldi, G., Hight, D. W., and Thomas, G. E., A reevaluation of conventional triaxial test methods, In Robert T. Donaghe, Ronald C. Chaney, and Marshall L. Silver (eds.), *Advanced Triaxial Testing of Soil and Rock*, American Society for Testing and Materials (ASTM), Philadelphia, STP 977 (1988), 219-263.
[13] Baldi, G., Nova, R., Membrane penetration effects in triaxial testing. *Journal of Geotechnical Engineering* **110** (3) (1984) 403–420.
[14] Lipiński, M. J., Wdowska, M. Saturation criteria for heavy over-consolidated cohesive soils. *Annals of Warsaw University of Life Sciences – SGGW. Land Reclam.* **42** (2) (2010), 295-279.
[15] Sugiyama,Y., Kawai K. and Iizuka. A., Effects of stress conditions on B-value measurement, *Soils and Foundations* **56**(5) (2016), 848-860.
[16] Donaghe R. T. and Townsend F. C., *Effects of Back Pressure Saturation Techniques on Results of R Triaxial Compression Tests.* Miscellaneous Paper GL-79-12, Geotechnical Laboratory U. S. Army Engineer Waterways Experiment Station, Final Report (1979).
[17] De Groot, M. B., Bolton, M. D., Foray, P., Meijers, P., Palmer, A. C., Sandven, R., Sawicki, A., and The, T. C., Physics of liquefaction phenomena around marine structures, *Journal of Waterway, Port, Coastal, and Ocean Engineering* **132** (4) (2006), 227-243.
[18] Sandven, R., Husby, E., Husby, J.E. and Jonland, J., Development of a sampler for measurement of gas content in soils, *ASCE Journal of Waterway, Port, Coastal, and Ocean Engineering*, (133)(1) (2005).
[19] Tani, S., Hasegawa, T., Earthquake damage on earth dam by The Nipponkai-Chubu Earthquake, *Journal of Japanese Society of Agri- culture Engineering* **55** (10) (1987) 17–25 (in Japanese).
[20] Fukushima Prefecture, *Report of the Cause of Fujinuma-ike Failure*, <http://www.pref.fukushima.lg.jp/download/1/nosonkeikaku_kensyo_houkoku1-1.pdf> (in Japanese), 2011.
[21] Sendir, S., Sato, J., Towhata, I., Honda, T. 1-G model tests and hollow cylindrical torsional shear experiments on seismic residual displacements of fill dams from the view point of seismic performance-based design, *Soil Dynamics and Earthquake Engineering* **30** (2010), 423–437.