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Saturation of Porous Rock and Measurement of the *B* coefficient

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ABSTRACT: Laboratory testing of fluid-saturated rock is often needed to properly duplicate field conditions, where pores in rock are filled with a liquid. It is desirable that laboratory specimens be fully saturated, because it reduces the number of unknown parameters and allows calculation of some of the rock's poroelastic moduli. However, full saturation is difficult to achieve in rock specimens because of the air bubbles trapped in pores. In order to dissolve air bubbles, back pressure can be applied to the specimens, decreasing the size of air bubbles and reducing the time needed to dissolve them in water. A process to saturate Berea sandstone is presented. Skempton's *B* coefficient was measured at each increment of back pressure while keeping the effective mean stress approximately the same. A *B*-value that was constant and independent of the magnitude of the back pressure indicated full saturation. The maximum *B* achieved for the sandstone at 5 MPa effective mean stress was 0.58. This value was confirmed by calculation of Skempton's coefficient from other poroelastic parameters. Moreover, full saturation was assured by the measurement of a constant P-wave velocity with increasing back pressure.

1. INTRODUCTION

In-situ rock is often fully saturated or at least has a high fluid content. The presence of pore fluid can affect both the elastic parameters and the inelastic deformation process. However, testing of fluid-saturated rock is not typically performed, even though rock-fluid interaction is critical in many applications, such as oil and natural gas search and recovery procedures [1,2].

The limiting conditions of rock-fluid interaction are (a) drained, where the pore fluid is allowed to leave or enter the rock and pressure is maintained at a constant level; (b) undrained, where the mass of the fluid inside the rock stays the same so the pressure is changing; and (c) unjacketed, where increments of pore pressure and mean stress are equal. The influence of these conditions on behavior of porous rock can be investigated to provide the full set of poroelastic parameters [1].

In order that laboratory tests properly duplicate field conditions, it is desirable that lab specimens be fully saturated. When not fully saturated specimens are tested, the degree of saturation can be changing during the test and it brings another undesirable parameter into the problem. Conversely, if the condition of full saturation is achieved, bulk modulus of the pore fluid can be taken as that of the liquid used in the test.

A method of gradual back pressure increase is suggested to achieve full saturation of laboratory specimens [3-6]. Even a highly-permeable rock is a difficult material to completely saturate, as air bubbles get trapped and do not allow a liquid at low pressure to fully take their place. Application of gradual back pressure reduces the size of air bubbles and they are "dissolved" in the pore liquid more readily [3,4,7]. When the pore liquid solutes all air bubbles, its bulk modulus is the one that it has in the pure state [8].

A series of drained and undrained tests were conducted on Berea sandstone to investigate the parameters that govern its poroelastic and inelastic response. This paper describes the back pressure saturation process and measurements of Skempton's *B* coefficient [9] for the rock specimens infiltrated by water. An example of complete saturation of a Berea sandstone specimen is given along with the discussion of the obtained *B*-values.

2. BACKGROUND

2.1. *Skempton's coefficient*

For undrained triaxial tests on soils, Skempton [9] introduced the pore-pressure coefficient *B* to express the pore pressure change Δp that occurs due to a change in the confinement or the all-round pressure $\Delta\sigma_3$. For soils with porosity ϕ

$$B = \frac{\Delta p}{\Delta \sigma_3} = \frac{1}{1 + \dfrac{\phi K}{K_f}} \tag{1}$$

and is approximately equal to 1.0, since the bulk modulus of the soil structure K (order of MPa) is negligible with respect to that of the fluid K_f (order of GPa). While saturating soft and medium soils, B is increasing until it becomes equal to 1.0, meaning that all load is taken by the pore fluid and the specimen is fully saturated. However, for rock, K is on the order of GPa and it is comparable to the bulk modulus of the saturating fluid, such as water (K_w = 2.24 GPa) or hydraulic oil (K_{oil} = 1.3 GPa). It means that rock's stiff framework feels the applied mean stress at least as much as the pore fluid, hence the Skempton's coefficient for rock can be significantly smaller than one. A number of experimental studies, *e.g.* [2, 10-13], suggests that B is decreasing with the effective stress and could be as low as 0.3 for sandstones, even at full saturation.

Bishop [14] extended Skempton's relation (1) to include the influence of the bulk modulus of the solid material of the skeleton (or unjacketed bulk modulus) K_s' for any porous mass with interconnected pores. In terms of bulk moduli, it can be written as:

$$B = \left.\frac{\Delta p}{\Delta P}\right|_{\Delta m_f = 0} = \frac{\alpha}{\alpha + \phi K\left(\dfrac{1}{K_f} - \dfrac{1}{K_s''}\right)} \tag{2}$$

Here Δm_f = 0 means that the mass of pore fluid in the specimen does not change, so B is measured under undrained conditions; K_s'' is the unjacketed pore bulk modulus and α is the Biot coefficient, $\alpha = 1 - K/K_s'$ [1].

In addition to the assumption of interconnectivity of pores, the skeleton and solid material are considered to be elastic and isotropic, and the pore fluid is linearly compressible. Also, for the general case, the change in confining pressure can be replaced with the change in the mean stress ΔP [14]:

$$\Delta P = (\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)/3 \tag{3}$$

Using the relationships between the poroelastic parameters [1], Skempton's coefficient can also be expressed in terms of rock's drained and undrained bulk moduli, K and K_u, respectively:

$$B = \frac{K_u - K}{\alpha K_u} \tag{4}$$

For the drained condition, pore pressure in the specimen is constant and pore fluid is allowed to enter or leave the specimen during the test. In this case, the increase in load is taken by the rock matrix only and hence the specimen deforms more than if the test would be performed under undrained condition, where the pore fluid takes part of the loading and also compresses. Hence,

$$0 < K \le K_u \tag{5}$$

meaning that B should be always non-negative:

$$0 \le B \le 1 \tag{6}$$

Measuring K, K_u, and α experimentally and independently of B, the latter can be calculated from equation (4) and compared to the measured B.

2.2. Saturation

The degree of saturation S, $0 \le S \le 1$, is determined as $S = V_f/V_\phi$, where V_f is the volume of the pore fluid (liquid) in the pores and V_ϕ is the pore volume. If the specimen is saturated with water, then the pore fluid in the nearly saturated (S > 0.95) soil or rock is a mixture of water and gas that has a fluid bulk modulus K_f. Several studies [15-17] describe the relationship between the fluid bulk modulus K_f and the degree of saturation. However, if S > 0.98, then the approximate relationship first suggested by Verruijt [15] is applicable:

$$\frac{1}{K_f} = \frac{1}{K_w} + \frac{1-S}{p} \tag{7}$$

where p is the pore pressure. Equation (7) shows that even a small variation in the degree of saturation strongly influences the bulk modulus of the fluid and hence B (see equation (2)).

Simple injection of water into the specimen at low back pressure (order of 0.1 MPa) provides S ~ 0.98 [4]. However, further increase of S to 1.00 is an involved process. Schuurman [18] suggested that for S > 0.85, the pore air is presented as individual bubbles rather than as continuous phase. Pressures at which air bubbles in the water approach a critical value and collapse are relatively small (order of 0.1 MPa). However, time to fully dissolve all the air trapped in the pores and drainage lines can be on the order of weeks [4,18]. It happens because surface areas of the cramped bubbles and volumes of liquid where they can dissolve are much smaller than in a large water reservoir, hence the diffusion of the pore air in the pore water becomes very time consuming procedure [7]. To facilitate this process, Black and Lee [4] suggested initial flushing of the specimen with deaired water and further gradual application of the back pressure in the pore lines that is significantly greater than the theoretical critical pressures for air bubble collapse.

The purpose of back pressure technique is to achieve 100% saturation by forcing any gas into solution of the pore water. Increase of the pore pressure in a partially saturated specimen affects the volume of the gas in the pores in two ways: (1) by direct compression, the gas is reduced in volume according to Boyle's law; (2) by application of a higher pressure, additional amounts of gas are dissolved in the pore water in accordance with Henry's law of solubility [3]. A large air bubble will take a longer time to diffuse to the critical size than will a smaller bubble; hence the saturation should occur faster for the smaller void size in the geomaterial [18].

Wissa [6] suggested a method to check if a very stiff soil specimen is fully saturated by determining Skempton's coefficient B at gradually increasing back pressures while keeping the effective stress approximately constant. If the rock is not fully saturated, then the B-value will be increasing with increasing pressure, as more air is forced into solution. A measured B-value that is constant and independent of the magnitude of the back pressure indicates full saturation.

Once the air is driven into solution, the air-water mixture behaves as a fluid with a bulk modulus equal to that of pure water [8,18], so dissolved air in the water has no influence on K_f. Hence, pore fluid bulk modulus becomes a fixed parameter and can be used in calculation of some other poroelastic moduli presented in equations (2) and (4).

Application of high back pressures (> 10 MPa) in the lab can be limited by the pore pressure system. Also, sometimes it could be necessary to duplicate the field conditions, where the pore fluid pressure in the in-situ rock is on the order of a few MPa. In the next section, the experimental method of achieving and assuring full saturation with minimum back pressure is presented.

3. EXPERIMENTAL METHODS

The University of Minnesota Plane-Strain Apparatus [19] allows application of three principal stresses and measurements of the strains in corresponding directions on prismatic (100 x 87 x 44 mm) rock specimens. Additionally, the apparatus allows liquid saturation of the rock, sealed with polyurethane to prevent the penetration of confining fluid into it. A standard pore pressure system is presented in Fig. 1, and consists of upstream and downstream channels with pressure transducers, and platens with porous stones, which distribute the pore fluid over the whole specimen area.

After the rock specimen was prepared, connected to the pore pressure system, and placed inside the plane-strain apparatus, a small axial seating load was applied. Then, the specimen was stressed axially and laterally to 5-7 MPa, depending on desired pore pressured to be applied later in the test.

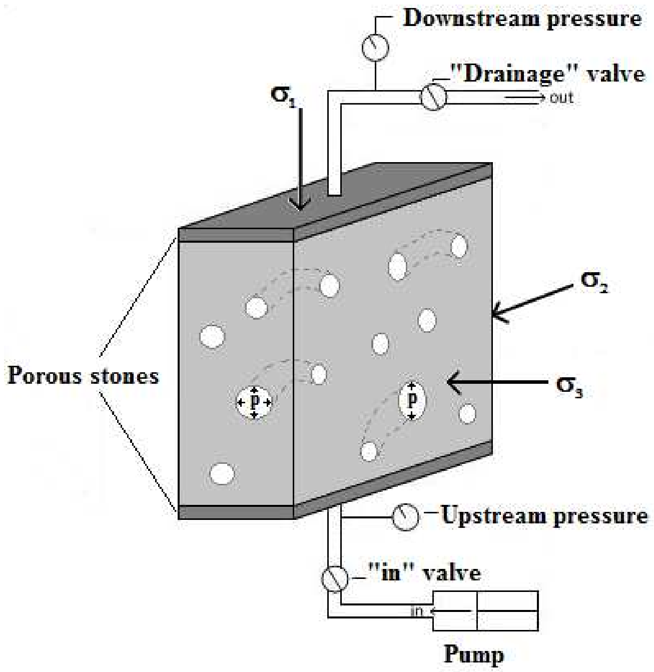

Figure 1. Setup for plane strain testing with pore pressure.

Once the confining pressure was applied, the process of water saturation of the specimen started. Deaired water was pumped from downstream into the specimen with the upstream valve open to the atmosphere, releasing excess air, as the pore water permeated through the specimen (Fig. 1). For each step of water injection (5-10 ml/step), the upstream pressure increased to approximately 0.3 MPa and decreased in a minute or so to about 0.1 MPa before the next step of injection. The water started flowing from the upstream valve after injecting about 80 ml, the estimated volume of pore space in the specimen. Further pumping of 100 ml of water through the specimen showed that a mass balance was approximately achieved.

The technique of back-pressure saturation was used next. Pore pressure p was increased in the specimen by pumping water in and having the drainage valve closed; a maximum back pressure of p = 5-6 MPa was applied. During this procedure, the axial and lateral stresses were always kept approximately 5 MPa higher than the pore pressure. After each new increase in back pressure, the pore pressure was allowed to equilibrate, meaning that upstream pressure would be equal to the downstream pressure and the change in pressure would not exceed 1 kPa/min. During this process, the remaining gas was dissolved into the pore fluid. When the equilibrium of pressure was achieved, Skempton's coefficient was measured through a procedure called a B-check.

The B-check is not straightforward under a plane strain condition, which in our case was achieved by wedging the specimen in the thick-walled steel frame that constrained the deformation of the specimen in intermediate principal stress direction. Hydrostatic loading cannot be applied [20], and the increment of the intermediate principal stress can be written as:

$$\Delta\sigma_2 = \Delta\varepsilon_2 E + \nu(\Delta\sigma_1 + \Delta\sigma_3) \tag{8}$$

For tests with pore pressure, the specimen size was chosen so that when equal increments in axial $\Delta\sigma_1$ and confining $\Delta\sigma_3$ pressures ($\Delta\sigma_1 = \Delta\sigma_3 = \Delta\sigma$) were applied, the frame deformation due to the stress increment applied by the specimen was compensated by the negative strain on the inner side of the frame due to the increase in cell pressure (according to Lame's solution, the inner part compresses due to equal pressures applied on the inside and outside of the cylinder). So, $\Delta\varepsilon_2 = 0$ in equation (8) and the expression for the measured B takes the form:

$$B^{meas} = \frac{\Delta p}{\Delta P} = \frac{3\Delta p}{2(1+\tilde{\nu})\Delta\sigma} \tag{9}$$

where $\tilde{\nu}$ is current Poisson's ratio of the rock and it changes from ν in dry (or drained) condition to ν_u upon saturation, while $\nu_u > \nu$. When the undrained elastic loading performed on the saturated specimen, ν_u independent of B-values can be calculated from the plane strain compression data.

While performing an undrained test in the laboratory, the specimen is connected to the drainage system of the cell and also to the pore pressure transducers. As the drainage system has a non-zero volume filled with water, it experiences volume changes due to its compressibility. Wissa [6] and Bishop [21] modified the expression for B to include terms representing the compressibility of the pore-pressure measuring system. The following equation was obtained for the corrected value of Skempton's coefficient B^{cor}:

$$B^{cor} = \frac{1}{\dfrac{1}{\left(\dfrac{\Delta p}{\Delta P}\right)_{observed}} - \dfrac{V_L}{V}\dfrac{K}{\alpha K_f} - \dfrac{K(C_L + C_M)}{\alpha V}} \tag{10}$$

where V is the volume of the specimen, V_L is the volume of the fluid in the pore-water lines, C_L is the compressibility of the pore-water lines, and C_M is the compressibility of the pore-pressure measuring element. However, several researchers [10,21,22] noticed that for modern high-pressure tubes and pressure transducers, the main contribution (more than 95%) to the correction term comes from the extra fluid volume in the system V_L. So, only the first two terms in the denominator of the right hand side of (10) are considered. The final equation for calculating B^{cor} in plane strain experiments is

$$B^{cor} = \frac{1}{\left(\dfrac{2(1+\tilde{\nu})\Delta\sigma}{3\Delta p}\right) - \dfrac{V_L}{V}\dfrac{K}{\alpha K_f}} \tag{11}$$

During the back-pressure saturation procedure, several immediate increments of $\Delta\sigma_1 = \Delta\sigma_3 = \Delta\sigma = 0.34$ MPa were applied and the corresponding increase in pore-water pressure Δp was measured in a few minutes when p equilibrated. If the system is completely saturated, the pore pressure response should be constant and independent of back pressure. On the other hand, if the system is not saturated, the pore pressure response will increase with growing back pressure, because the degree of saturation is augmented and consequently, the bulk modulus of the pore fluid increases according to equation (7) as the back pressure is growing [6]. If this is the case, the new increment in back pressure is applied and equilibrium is established before performing a new B-check. When Δp becomes the same for each increment $\Delta\sigma$ = const, the rock is considered to be saturated.

4. EXPERIMENTAL RESULTS AND DISCUSSION

The experiments were conducted on Berea sandstone with porosity $\phi = 0.23$ and dry bulk density $\rho = 2.1$ g/cm^3. The rock is flat-bedded, light gray, medium- to fine- grained protoquartzite cemented with silica and clay. Permeability of the sandstone is about 40 mD (at 5 MPa effective confining pressure), Young's modulus E = 14-16 GPa, Poisson's ratio $\nu = 0.31$, uniaxial compression strength UCS = 41-43 MPa.

The results of the back pressure saturation process are presented in Fig. 2. The increments of pore pressure Δp for constant increments of $\Delta\sigma = 0.34$ MPa are plotted versus back pressure.

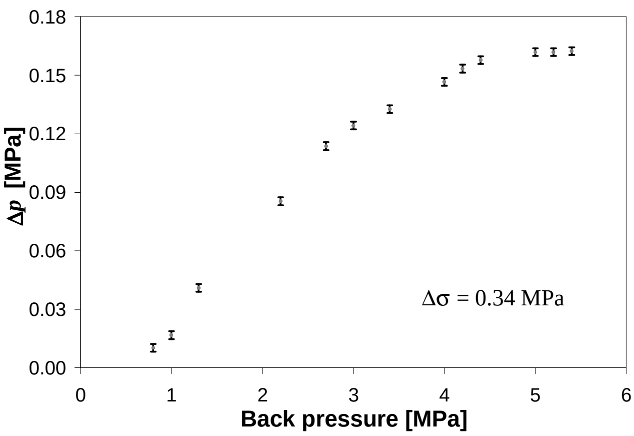

Figure 2. Pore pressure change with increasing back pressure.

It can be observed that at low back pressures, increments of pore pressure due to increments in mean stress are very small, which can be explained by the load mostly taken by the rock matrix. The pore fluid at p < 4 MPa consists of water and air, and hence is more compressible than just pure water. However, with increasing back pressure, air bubbles dissolve in water, Δp is increasing and becomes constant when p > 4 MPa. At back pressures above this value, the specimen is fully saturated and K_f can be taken as that of pure water K_f = K_w = 2.24 GPa.

Drained bulk modulus and Biot coefficient were measured to be $K = 9.6\ GPa$ and $\alpha = 0.67$ [23]. B^{cor} can be calculated from equation (11), assuming $\tilde{\nu}$ is changing from $\nu = 0.31$ in a drained condition to ν_u = 0.35 in an undrained case [24], and that pore water lines were filled with deaired water, K_f = 2.24 GPa, from the beginning of the back pressure saturation process. The calculated B-values are presented in Figure 3.

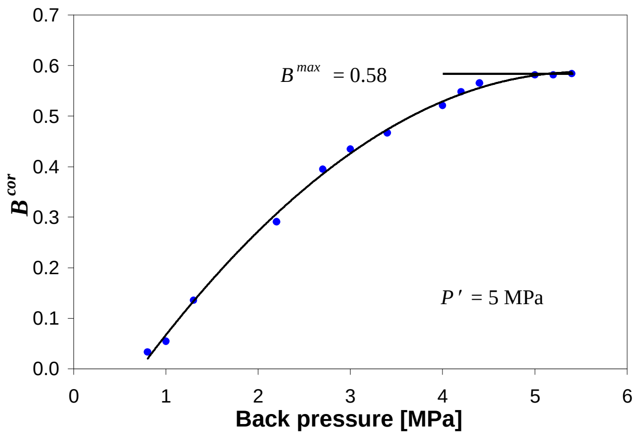

Figure 3. Change in B^{cor} with back pressure.

The maximum value of Skempton's coefficient achieved in our experiments and corrected for the compliance of the measuring system is B^{cor} = 0.58.

Reported Skempton's coefficient can be compared with B calculated from equation (4): B = 0.51 – 0.65. The latter one has a wide range, because of the variation in measured drained and undrained bulk moduli and Biot coefficient [23]. However, we can state that reaching B = 0.58 means that full saturation was achieved. These results are also in agreement with the experimental observation of Hart and Wang [2], who measured B-values for their Berea sandstone to be in the range 0.61-0.68 from conventional triaxial tests conducted at 4.0-5.7 MPa effective confining pressure.

The alternative way to check saturation of a lab specimen is based on the observation that compressional waves (P-waves) propagate in saturated soils with a velocity that is strongly affected by the water filling the interstices of soil grains [25]. Strachan [26] suggested the following criterion for ensuring full saturation: P-wave velocity in the saturated material becomes constant with increasing back pressure keeping effective stress constant. Two sensors were attached to the opposite sides of prismatic specimen and P-wave velocity was measured during the saturation process at constant effective mean stress P' = 5 MPa (Fig. 4).

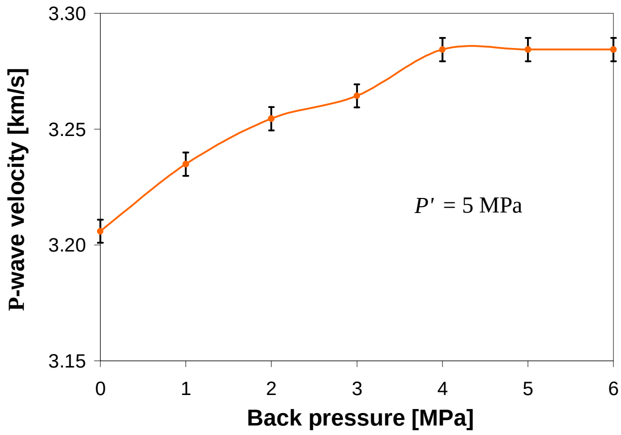

Figure 4. Change in P-wave velocity with back pressure.

The change in P-wave velocity with back pressure in Berea sandstone is not strongly pronounced as in soil [25,26]. However, it can be seen that the velocity is increasing with the back pressure and becomes constant when p exceeds 4 MPa, which supports the proposed B-check method in achieving full saturation.

5. CONCLUSIONS

It is frequently necessary to determine poroelastic parameters or strength characteristics of fluid-saturated rock specimens. This condition is difficult to achieve because of the air bubbles trapped in the rock pores. The application of gradually increasing back pressure decreases the size of the bubbles and facilitates their diffusion in the pore fluid.

This method was implemented for saturation of Berea sandstone. It was observed that with increasing back pressure Skempton's coefficient B was increasing, if the effective mean stress P' acting on the specimen was preserved to be the same: P' = 5 MPa. The value of B increased until p ~ 4 MPa was reached and then it became constant and equal to 0.58, meaning that 100% saturation was achieved. This value is in the range of B predicted by the independent calculations from the relationships of poroelasticity. Full saturation was also confirmed by no change in P-wave velocity in the sandstone at p > 4 MPa and P' = 5 MPa.

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REFERENCES

1. Detournay, E. and A. Cheng. 1993. Fundamentals of poroelasticity. In *Comprehensive Rock Engineering, Vol. II*, ed. C. Fairhurst, 113-171.
2. Hart, D.J. and H.F. Wang. 2010. Variation of unjacketed pore compressibility using Gassmann's equation and an overdetermined set of volumetric poroelastic measurements. *Geophysics*, 75:9-18.
3. Lowe, J. and T.C. Johnson. 1960. Use of back pressure to increase degree of saturation of triaxial test specimen. In *Conference on Shear Strength of Cohesive Soils*. 819-836.
4. Black, D.K. and K.L. Lee. 1973. Saturating laboratoty samples by back pressure. *J. Soil Mech. & Found. Division, ASCE*. 99:75-93.
5. Chaney, R.C., E. Stevens, and N. Sheth. 1979. Suggested test method for determination of degree of saturation of soil samples by *B* value measurement. *Geotech. Testing J.* 2(3):158-162.
6. Wissa, A.E. 1969. Pore pressure measurements in saturated stiff soils. *J. Soil Mech. & Found. Division, ASCE*. 95:1063-1073.
7. Lee, K.L. and D.K. Black. 1972. Time to dissolve air bubble in drain line. *J. Soil Mech. & Found. Division, ASCE*. 98:181-194.
8. Fredlund, D.G. 1976. Density and compressibility characteristics of air-water mixtures. *Can. Geotech. J.* 13:386-396.
9. Skempton, A.W. 1954. The pore-pressure coefficients A and B. *Geotechnique*, 4:143-147.
10. Mesri, G. K., K. Adachi, and C.R. Ullrich. 1976. Pore-pressure response in rock to undrained change in all-round stress. *Geotechnique*, 26(2):317–330.
11. Green, D.H. and H.F. Wang. 1986. Fluid pressure response to undrained compression in saturated sedimentary rock. *Geophysics*, 51:948–956.
12. Berge, P.A., H.F. Wang, and B.P. Bonner. 1993. Pore pressure buildup coefficient in synthetic and natural sandstones. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* 30(7): 1135-1141.
13. Blöcher, G., D. Bruhn, G. Zimmerman, C. McDermott, and E. Huenges, E. 2007. Investigation of the undrained poroelastic response of sandstones to confining pressure via laboratory experiment, numerical simulation and analytical calculation. In *Rock Physics and Geomechanics in the Study of Reservoirs and Repositories*, eds. C. David and M. Le Ravalec-Dupin, The Geological Society, 284:71-87.
14. Bishop, A.W. 1973. The influence of an undrained change in stress on the pore-pressure in porous media of low compressibility. *Geotechnique*. 23(3): 435-442.
15. Verruijt, A. 1969. Elastic storage of aquifers. In *Flow through Porous Media*, ed. R.J.M. De Wiest, 331-376.
16. Allen, N.F., F.E. Richart, Jr., and R.D. Woods. 1980. Fluid wave propagation in saturated and nearly saturated sands. *J. Geotech. Eng. Division, ASCE*. 106:235-253.
17. Vardoulakis, I. and D.E. Beskos. 1986. Dynamic behavior of nearly saturated porous media. *Mech. Materials*. 5:87-108.
18. Schuurman, I.E. 1966. The compressibility of an air/water mixture and a theoretical relation between the air and water pressures. *Geotechnique*. 16(4): 269-281.
19. Labuz, J.F., S.T. Dai, and E. Papamichos. 1996. Plane-strain compression of rock-like materials. *Int. J. Rock Mech. Min. Sci.* 33(6): 573-584.
20. Makhnenko, R. and J. Labuz. 2013. True triaxial testing with passive restraint. Submitted for *Rock Mech. Rock Eng.*
21. Bishop, A.W. 1976. Influence of system compressibility on observed pore pressure response to an undrained change in stress in saturated rock. *Géotechnique*, 26(2):371-375.
22. Ghabezloo, S. and J. Sulem. 2009. Effect of the volume of the drainage system on the measurement of undrained thermo-poro-elastic parameters. *Int. J. Rock Mech. & Mining Sci*. 47:60-68.
23. Makhnenko, R.M. and J.F. Labuz. 2013. Unjacketed bulk compressibility of sandstone in laboratory experiments. In *Proceedings of The 5th BIOT Conference on Poromechanics, Vienna, Austria, 10-12 July 2013*, paper No. 304.
24. Makhnenko, R. and J. Labuz. 2012. Drained and Undrained Plane Strain Compression of Porous Rock. In *Proceedings of XXIII ICTAM, Beijing, China, 19-24 August 2012*, paper No. FS10-020.
25. Bardet, J.P. and H. Sayed. 1993. Velocity and attenuation of compressional waves in nearly saturated soils. *Soil Dyn. Earthquake Eng.* 12: 391-401.
26. Strachan, P. 1985. Alternative test method for ensuring full saturation in triaxial samples. *Geotech. Testing J.* 8(1):43-46.