Mechanism of subsidence from pore pressure fluctuation in aquifer layers

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ABSTRACT: Field data revealed that compression in aquifer layers is the major source of ground subsidence, and that compression of saturated granular soils due to fluctuations in pore pressure appears to be a major factor causing the subsidence. To investigate the mechanism of granular soil compression due to repeated pore pressure fluctuations, a Rowe cell was modified to mimic the stress paths in the field. The shakedown theorem was extended to consider saturated, granular soils under Ko conditions. Remolded sand specimens were prepared to study the shakedown behavior. Experimental results showed that the shakedown responses were affected by the pore pressure amplitude, soil composition, and initial state. Comparisons of monotonic and shakedown compressions reveal that the shakedown compression was at least as significant as the monotonic component, and that the shakedown effects should be included in groundwater pumping induced ground subsidence analysis.

KEYWORDS: Granular soil compression, subsidence, shakedown theorem, aquifer layer, pore pressure fluctuation

1 INTRODUCTION

Ground subsidence from excessive groundwater pumping or climate change has been a serious problem in many parts of the world. Hung et al. (2012) reported that the compression of the top three aquifer layers contributed 83% of the ground subsidence during the period of 1997 to 2010.

Based on field monitoring data, Hung et al. (2012) revealed that the major compression was observed in the second aquifer at depth from 52 to 153 m, where the accumulated vertical compression of 63.4 cm has occurred from 1997 to 2010. The ground water level albeit fluctuated on a short term basis, lowered by approximately 2 m initially and then actually raised 10 m after January, 2007. The aquifer within the depth range consisted mostly of fine to coarse sand with occasional layers of gravel. The increase of effective stress due to the initial 2 m lowering of ground water level was not sufficient to cause the soil compression considering the typical monotonic compressibility of granular soils and the high effective stress at 100 m deep. Therefore, the accumulated compression of granular soil layer from repeated ground water variations may be the significant source of ground subsidence in this area.

To ascertain the significance of ground subsidence from the compression of granular soils due to the short-term fluctuation of groundwater levels, a series of soil element tests that mimic the in situ fluctuated stress paths were performed using a newly developed K_o consolidation/compression system. A modified, close-loop servo-controlled Rowe cell (Rowe and Barden 1966) that is capable of controlling the drainage and back pressure was used for the purpose. Specimens made of quartz sand mixed with mica were subjected to sinusoidal variations of pore water pressure while maintaining a constant total vertical stress and K_o conditions. The results were analyzed using the shakedown theorem, which has been widely used in pavement engineering, to describe the permanent deformation of unbounded granular layers subjected to repeated loading. Furthermore, element tests subjected to fluctuated pore pressure from field data and compressions are compared. The importance of compression from fluctuated pore pressure is addressed.

2 LITERATURE REVIEW

2 .1 *Compressibility of granular soils*

Compression of granular soils from increasing effective stress is the net outcome of particle rearrangement and interparticle slip and particle crushing (Mesri and Vardhanabhuti 2009). The plastic strain in confined compression is mainly from particle sliding and rolling at a low stress level, and particle crushing at medium to high stress conditions (Nakata et al. 2001, Pestana and Whittle 1995).

In the field, a soil element is subjected to K_o conditions, in which the lateral strain is prohibited when subjected to variations of vertical stress. Several testing procedures, including the Casagrande oedometer, K_o-triaxial consolidation, constant rate of strain consolidation (CRS), and Rowe cell tests, have been developed to determine the compressibility or consolidation characteristics of soils in K_o conditions. The consolidation cell developed by Rowe and Barden (1966) used a hydraulic loading system on a sealed specimen for drainage control, back pressure application, and pore pressure measurement.

Lambe and Whitman (1969) conducted a theoretical study of the ideal packing of elastic spheres to describe the effects of cyclic loading on stress-strain curves during compression. Their study concluded that a small amount of permanent strain accumulated during the first 10 to 50 cycles of loading for particles without crushing. After this state, a stable hysteretic loop without further accumulation of permanent strain is achieved. The critical stress needed to induce permanent strain is generally small and increases with the previous loading level and rate. Leshchinsky and Rawlings (1988) used a triaxial system to study the effects of stress path on the permanent deformation of dry sand, and found a linear relationship between the permanent axial strain and the logarithm of the number of loading cycles. These studies revealed that characteristics of granular compressibility from repeated loadings are different from those of monotonic loadings.

2 .2 *Shakedown theorem*

The shakedown theorem of Melan (1938) and Koiter (1960) has been used to describe the behavior of elasto-plastic materials under cyclic loading. For instance, Johnson (1986) adopted the concept to analyze the behavior of metal surfaces under repeated rolling and sliding loads. Sharp and Booker (1984) applied the concept to pavement design.

In the shakedown theorem, the material response under cyclic loadings is divided into four stages, which are elastic, elastic shakedown, plastic shakedown, and incremental collapse or ratcheting, and the load limits that separate the four stages are elastic limit, elastic shakedown limit, and plastic shakedown limit, respectively. Shakedown stages occur when the applied load magnitude is greater than the elastic limit but smaller than a critical limit that incremental collapse is initiated. In shakedown stages, material deforms plastically in each loading cycle and the accumulation of plastic deformation ceases after a finite number of load cycles. The plastic shakedown occurs when the load magnitude is greater than the elastic shakedown limit but less than the plastic shakedown limit. In this stage, the material achieves a resilient or steady response with hysteretic stress-strain relationship. When material reached the resilient condition, the material is "shaken down". The ratcheting stage indicates that plastic strains accumulate rapidly with failure occurring in a short time.

In the case of ground subsidence of aquifer layers owing to fluctuations of pore water pressure from groundwater pumping, the granular soil layers were within shakedown stage with plastic strain accumulation before they achieved a steady response. In shakedown framework, the aquifer soil is likely to undergo the plastic shakedown condition, in which significant portion of settlement came from accumulation of plastic strain during cyclic loading and the settlement could cease after a finite number of cycles. The significant difference in compressibility between shaken-down and one-cycle-loading conditions provides a potential clue in explaining and predicting ground subsidence owing to pore pressure fluctuations.

3 TESTING SYSTEM AND METHOD

3.1 *State of stress and stress path*

The in-situ state of stress imposed on a level aquifer layer subjected to pore water pressure variations is the outcomes of total vertical stress (σ_v) and pore pressure (u). The total vertical stress is assumed constant if no surcharge is added and the difference in soil unit weight from ground water table variation is ignored. The soil element maintains under K_o conditions throughout the process of pore pressure fluctuations due to pumping and seasonal water table variations. To fit the shakedown theory, a sinusoidal pore pressure variation with a constant amplitude is used in the laboratory element tests. Prior to the application of pore pressure fluctuation, the soil element was fully saturated.

To systematically study the compression of aquifer soils subjected to repeated loadings, sinusoidal pore pressure variations were applied under a constant total vertical stress and K_o condition. This method is referred to as the back-pressure-controlled compression (BPCC), and was adopted in this study. The stress path of BPCC is illustrated in Fig. 1 using the stress parameters defined as:

$$p' = \frac{(\sigma_v' + \sigma_h')}{2} \tag{1a}$$

$$q = \frac{(\sigma_v' - \sigma_h')}{2} \tag{1b}$$

where σ_h' is the effective horizontal stress.

In BPCC, the decrease of q due to stress history induced K_o variations can be more significant than those from the loading paths with a constant u and varying σ_v (i.e., conventional K_o consolidation). The BPCC stress path reduces the stress difference from K_o variations with pre-stress histories to a greater degree than the loading paths with a constant u and varying σ_v. To fit the plastic shakedown condition, a constant amplitude of pore pressure variation is cyclically applied on the saturated soil specimen until the increment of induced permanent vertical displacement approaches zero or a small constant value. To reduce the inconsistencies involved in specimen preparation, a multi-stage testing procedure is adopted. After the completion of consolidation to static vertical effective stress, small pore pressure amplitude is cyclically applied until a steady or resilient state is observed. The pore pressure amplitude is then increased and the process repeated.

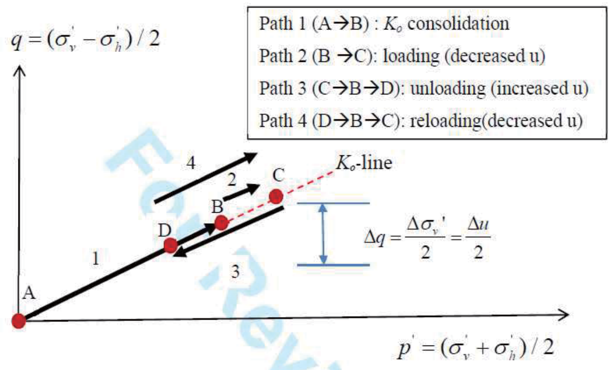

Figure 1. Stress path of back-pressure-controlled compression (BPCC)

3.2 *K_o compression cell*

To ensure the K_o condition during the loading process and extend the testing stress range, the Rowe cell reported by Rowe and Barden (1966) was adopted to perform high pressure K_o compression subjected to pore pressure variations using the BPCC method. Because the new system is for ground subsidence simulation in high stress conditions, it is termed the HPK_o simulator and shown in Fig. 2. The HPK_o simulator consists of three major subsystems: the K_o compression cell, stress control system, and data acquisition (DAQ).

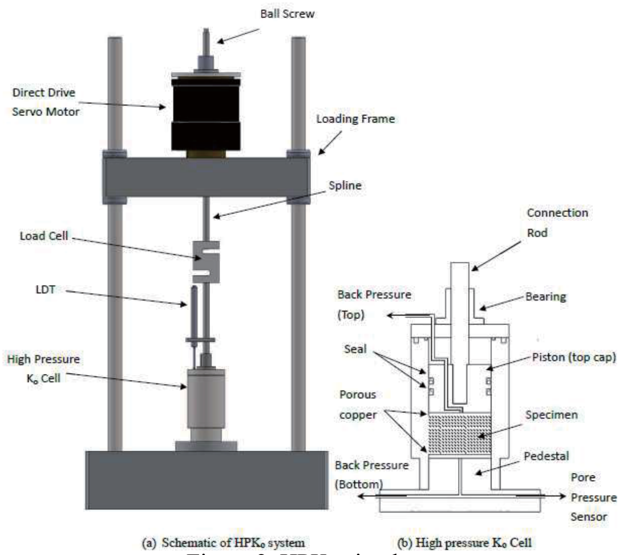

Figure 2. HPKo simulator

Details of the Rowe cell device for high pressure tests are shown in Fig. 2(b). The system is designed to function as a rigid plate applied on the top of the specimen. A cylindrical cell with an inner diameter of 70 mm was machined to be tightly fitted and sealed on a pedestal used in a conventional triaxial cell. The inner wall is coated with a thin layer of Teflon to reduce side friction. The bottom drainage lines are connected to a pore pressure transducer. The top drainage line was connected to a pore pressure control unit and an electronic volume change apparatus. The loading shaft is bolted at the center of top cap and guided by a linear bushing nut. A computer-based 24-bit data acquisition system was integrated to acquire data and provide feedback signals of the axial actuator and pressure regulators for stress control.

To meet the BPCC method in ground subsidence simulation, the total vertical stress and pore pressure must be continuously adjusted to meet the stress conditions. Following the design by Huang et al. (1994), an electromagnetic actuator, consisting of a direct-drive (DD) servo motor, high precision ball screw, and spline shaft, is used to provide servo vertical stress control. The high torque capacity of the motor ensures stability of the close loop control. The combination of the ball screw and spline shaft converts the rotation motion into high precision axial stroke.

An electric-pneumatic (EP) regulator with a resolution of 0.5 kPa is integrated in the HPK$_o$ simulator to automatically control the pore pressure. The back pressure is applied from the top drainage while the pore pressure is measured from the drainage line at the center of pedestal. This setup ensures the full balance of the pore pressure within the specimen, because it measures the longest drainage path for one-way drainage. The volume change is measured by an automatic volume change apparatus with an accuracy of 0.01 cm^3. An external linear displacement transducer (LDT) with a resolution of 0.001 mm is clamped on the loading rod to measure the vertical displacement of the specimen. A comparison between the volume changes of a saturated specimen from the LDT reading and volume change apparatus showed that the error was below the resolution of the volume change apparatus up to a vertical stress of 2.5 MPa, due to the high rigidity of the shaft, rigid top cap, energized seals, and well-aligned linear bushing nut.

3 .3 *Testing procedure*

The BPCC testing procedure is divided into three phases, including specimen saturation, K$_o$ consolidation, and application of pore pressure variations. Air was expelled from the specimen by water flushing from bottom to top. After this, vertical pressure and back pressure were applied simultaneously with a stress increment of 5 kPa for 3 minutes to reach stress equilibrium. When the back pressure reached 110 kPa, the stress components were hold for saturation. A B-value check in K$_o$ condition (Chang et al., 2014) was performed, and a B-value greater than 0.95 was needed for saturation.

After the completion of saturation, K$_o$ consolidation was performed using a vertical stress increment of 5 kPa to the effective vertical consolidation stress while maintaining the back pressure of 110 kPa. The completion of K$_o$ consolidation was determined based zero excess pore pressure at bottom and no volume change for 5 minutes. The dissipation time for different stress levels was recorded.

After the completion of K$_o$ consolidation, sinusoidal or varied pore pressure variations were slowly applied while maintaining a constant total vertical stress. For each pore pressure amplitude, the pore pressure was slowly ramped to the predetermined values over a set period based on K$_o$ consolidation results. The suitability of this ramping period was checked from the pore pressure measurements at the bottom drainage. The pore pressure variation procedure was programed using a computer-based controller and DAQ to automatically perform the BPCC. The repeated loading stage stopped when the induced increment of permanent vertical displacement of five consecutive cycles was constant. The next loading amplitude was performed after the data were saved.

4 TESTING RESULTS AND DISCUSSIONS

4 .1 *Results of sinusoidal loading*

A mixture of Ottawa sand with 0, 10 and 20% of mica contents (MC) by weight was used in the testing program. The Ottawa sand is a uniform sand (SP) with a mean grain size (D_{50}) of 0.65 mm, and primarily comprises quartz with a specific gravity of 2.65. The maximum and minimum void ratios of the sand are 0.75 and 0.48, respectively. The mica content is a low plasticity powder from a commercial supplier of painting additives. The USCS classification of the mica powder is CL-ML, with a mean grain size of 0.013 mm. The mica powder is added with Ottawa sand because mica content is the key factor to induce high compressibility of granular soils in central-western Taiwan (Huang et al. 1999). The sand specimens with mica contents of 10 and 20% are classified as SC-SM in USCS classification system, and were tested to investigate the effects of mica content on the compressibility of granular soils.

The multi-staged results for the specimen with MC of 20% and a void ratio after Ko consolidation (ec) of 0.651 are shown in Fig. 3. Fig. 3(a) and 3(b) show the time histories of pore pressure and axial strain. Fig. 3(c) shows the vertical effective stress-axial strain relationship. The axial strain is calculated from the vertical displacement divided by the height of the specimen before the application of sinusoidal loading. Fig. 3(b) shows that the permanent axial strain accumulates as the load cycle increases under a constant pore pressure fluctuation amplitude. The hysteretic loops between the vertical effective stress and axial strain are shown in Fig. 3(c). The increase in the plastic strain reduces as the loading cycle increases.

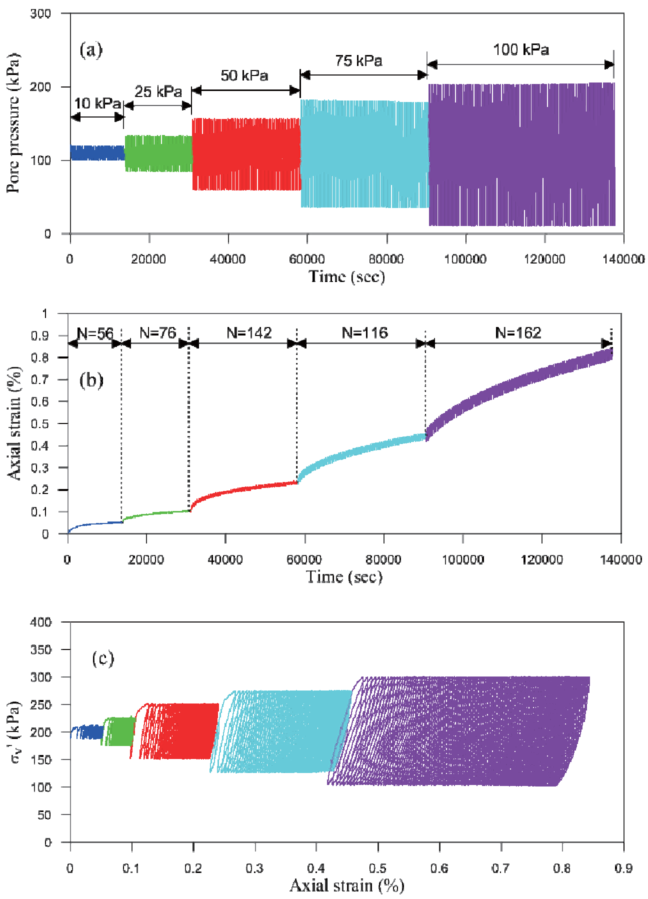

Figure 3. Results of sinusoidal loading for MC=0% and e_c= 0.651: (a)pore pressure; (b)axial strain; (c)stress-strain curves

To show the significance of compression considering the shakedown effect, the ratio of the accumulative plastic strain at the initiation of plastic shakedown ($\varepsilon_{p,ps}$) to the plastic strain induced in the first cycle ($\varepsilon_{p,1}$) is defined as shakedown compression ratio (SCR), which is expressed as:

$$SCR = \frac{\varepsilon_{p,ps}}{\varepsilon_{p,1}} \tag{2}$$

The SCR represents the increase in accumulative plastic strain from repeated loadings.

Fig. 4 shows the variation of SCR with pore pressure amplitude for all specimens. Although nonlinear relationships exist between the SCR and Δu, a relatively constant was observed for Δu less than 50 kPa, which is also the boundary value dividing plastic shakedown and plastic creep. This feature is important for predicting the compression of granular soils subjected to pore pressure fluctuations. Additionally, the results also reveal that the SCR values between the shakedown state and first cycle could be up to 29.4 times for sand with mica, and 18.5 times for clean sand. The SCR values between the plastic shakedown state (i.e., Δu not greater than 50 kPa) and first cycle values are reduced to 8.2 times for sand with mica and 6.6 times for clean sand. These values provide the basis for explaining the field monitoring data, and indicate the significance of shakedown compression for ground subsidence.

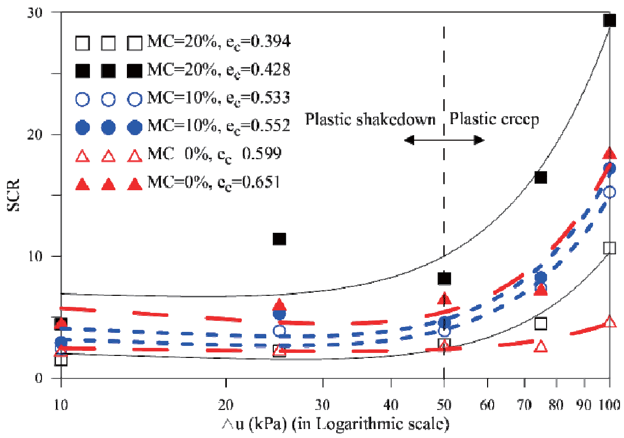

Figure 4. Effects of repeated loading on compression of granular soils

4.2 *Results of field simulation*

Remolded silty sand specimens by mist pluviation method (Huang et al. 2016) using the in-situ aquifer soil were subjected to field pore pressure history. The pore pressure histories were recorded for over a year in central-western Taiwan (Fig. 5). Mist pluviation method is used to prepare silty sand specimens without inducing particle segregation. Due to lack of field void ratio data, the specimens were subjected to the same pore pressure history repeatedly by assuming the process has occurred before and will repeat in the future.

The vertical compression result from the second cycle of the pore pressure history and the in-situ measurements from multi-point measurement in the second aquifer layer (52~153 m from surface) is shown in Fig. 5. Two borehole data (STA-3 and STA-9) in the vicinity are presented to show the consistency of the data. The results evidently show that the subsidence simulator is capable of accurately modeling the field behaviors. Additionally, the inconsistency in fluctuations of the volumetric strain and pore pressure reveals the significance of compression from repeated loading.

5 CONCLUSIONS

To investigate the mechanism of granular soil compression due to repeated pore pressure variations, a modified Rowe cell is developed in this work to apply a stress path that simulates the pore pressure fluctuation while maintaining a constant vertical stress. The system is called the HPK0 simulator, and the testing procedure is termed back-pressure-control consolidation (BPCC). Remolded sand specimens with different mica contents are prepared to different void ratios to study the shakedown behaviors.

The testing results reveal that the shakedown theorem is valid for saturated, granular soils under Ko condition. Analytical and experimental results show that saturated granular soils under Ko condition will always be below the plastic shakedown limit and reach a plastic shakedown state that shows a resilient response with a zero or small but constant plastic strain increment. Comparisons of monotonic and shakedown compressions reveal that the shakedown compression is at least as significant as the monotonic component for plastic shakedown cases, and thus that the shakedown effect should be included in ground subsidence analysis.

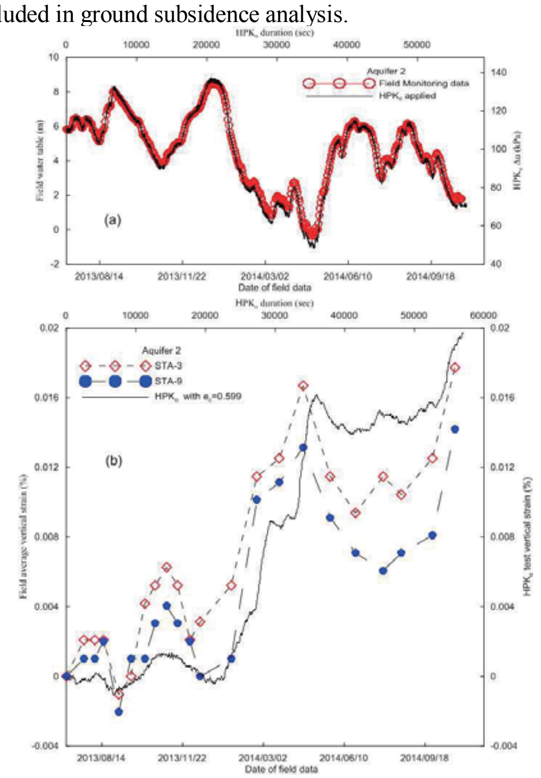

Figure 5. Comparison of field monitoring and laboratory modeling

6 REFERENCES

Chang, W.J., Chang, C.W., and Zeng, J.K., 2014. Liquefaction characteristics of gap-graded gravelly soils in K_o condition. Soil Dynamics and Earthquake Engineering, 56, pp. 74-85.

Huang, A.B., Hsu, S.P., Kuhn, H.R., 1994. A multiple purpose soil testing apparatus. Geotech. Test. J. 17(2), 227-232.

Huang, A.B., Hsu, H.H., Chang, J.W., 1999. The behavior of a compressible silty fine sand. Can. Geotech. J. 36(1), 88-101.

Hung, W.C., Hwang, C., Liou, J.C., Lin, Y.S., Yang, H.L., 2012. Modeling aquifer-system compaction and predicting land subsidence in central Taiwan. Eng. Geol. 147-148, 78-90.

Koiter, W.T., 1960. General theorems for elastic–plastic solids. In: Sneddon IN, Hill R (ed) Progress in solid mechanics, pp. 165–221.

Lambe, T.W., Whitman, R.V., 1969. Soil mechanics. Wiley, New York.

Mesri, G., Vardhanabhuti, B., 2009. Compression of granular materials. Can. Geotech. J. 46, 369–392.

Pestana, J.M., Whittle, A.J., 1995. Compression model for cohesionless soils. Geotechnique 45(4), 611-631.

Rowe, P.W., Barden, L., 1966. A new consolidation cell. Geotechnique 16(2), 519-539.

Sharp, R.W., Booker, J.R., 1984. Shakedown of pavements under moving surface loads. J. Transp. Eng. ASCE 110, 1–14.

Yu, H.S., 2005. Three-dimensional analytical solutions for shakedown of cohesive-frictional materials under moving surface loads. Proc. R. Soc. A. 461, 1951-1964.