Model test of soil deformation response to draining-recharging conditions based on DFOS

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\textbf{Abstract}

Intense groundwater exploitation and seasonal fluctuations of water level in phreatic aquifer cause land subsidence disasters. To better understand the land subsidence mechanism caused by groundwater withdrawal, a small-scale sand-clay interbedded model box was built to carry out consolidation and rebound tests during two drainage-recharge cycles. Distributed fiber optical sensing (DFOS) technologies were introduced for coupled monitoring of soil strain and water content to analyze response characteristics of each layer to water level changes. The results indicate that soil layers were compressed during drainage and rebound during recharge, with clay layer deformation more obvious than sand layer deformation. Deformation of the sand layer was synchronous with water content changes, while that of the clay layer lagged behind water content changes due to its lower permeability coefficient. The segmented compression clay layer curve shows that the water content rapidly decreased when it was lower than the liquid limit, while the clay layer compression rate obviously accelerated during drainage. When the water content was higher than that of the liquid limit during recharge, the rebound rate significantly increased. Under the condition of repeated drainage and recharge, the soil microstructure changed with decreasing porosity and permeability coefficient, leading to a longer consolidation time. With increasing cycling times, the soil gradually tended toward elastic deformation. The observations propose a novel technology, DFOS technology, for monitoring land subsidence caused by seasonal water level fluctuations and dewatering projects. The test results are of great significance to investigate land subsidence mechanisms and evaluate soil layer compression potential.

\section{Introduction}

Excessive groundwater pumping cause the land subsidence due to consolidation of multi-layer clayey and sandy soil, which seriously influences human life and infrastructure construction. Land subsidence has become a worldwide geological hazards problem because it may cause geological, environmental hydrogeological and economic impacts (Mousavi et al., 2000; Sato et al., 2003; Phien-wej et al., 2006; Tosi et al., 2006; Wang et al., 2009b; Tung and Hu, 2012; Pacheco-martínez et al., 2013).

Over the past few decades, scholars have conducted research on land subsidence problems caused by groundwater extraction. Poland and Davis (1969) demonstrated that the centers of subsidence in the Santa Clara valley, California, USA, coincided with the major pumping centers, and due to aquifer pumping, the water head dropped and the effective stress increased, which led to ground subsidence. In 1972, Ramnarong (1989) injected storm water into an underground storage in Bangkok and verified that artificial recharge of groundwater was an effective control measure for land subsidence. Numerous studies have demonstrated that the variation of land subsidence is related to the fluctuation of ground water table (Chang et al., 2004; Chen et al., 2007; Zhang et al., 2015; Chai et al., 2017). To investigate soil compaction and rebound mechanisms, a large number of field and laboratory tests focused on soil deformation characteristics in the process of land subsidence. Based on monitoring data of strata deformation in Shanghai, Zhang et al. (2006) found that the deformation characteristics of soil layers under the different pressure conditions varied. Wang et al. (2009b) noted that strata compression in Changzhou, China, varied significantly and was strongly influenced by groundwater drawdown. Land subsidence was mainly attributed to consolidation of the thick aquitard. Hung et al. (2010) deployed a multi-sensor monitoring system to monitor the aquifer-system compaction within a depth of 300 m and showed that compaction depths were in accordance with pump depths. However, due to complicated geological conditions and interference...
factors in subsidence areas, these field studies have mainly focused on total ground settlement. For laboratory tests on land subsidence, Kitaro (1969) performed the most representative large-scale land subsidence model test, studying the relationship between land subsidence and groundwater extraction in aquifers. Xu et al. (2011) conducted drainage experiments of clayey and sandy soil under different soil combination conditions. Their results showed that the clay layer was the primary cause of soil compaction. However, bubbles in the soil body easily affected the water tube observation data in model tests. Moreover, because of the production process, installation restrictions and limited monitoring points, coupling changes in deformation and water content of the entire soil profile were not obtainable. Both laboratory-made extensometers and borehole extensometers can only reflect deformation and displacement of limited stratum due to point number and distribution limits and this fact leads to urgent need of distributed monitoring technologies.

Fiber Optic Sensing (FOS) technologies have been developed since 1980s and they allow the monitoring of one-dimensional structural physical field along entire optical fiber (Udd, 1995; Grattan and Sun, 2000; Lee, 2003; Barrias et al., 2016). Based on the sensing technique and principle, the optical fiber sensors can be categorized into different types including Fiber Bragg Grating (FBG) Sensors, Extrinsic Fabry-Perot Interferometric (EFP) Sensors, Optical Time-Domain Reflectometry (OTDR) Sensors, etc. (Park et al., 2006; Galindez-Jamío and López-Higuera, 2012; Palmieri and Schenato, 2013). Recently, Distributed Fiber Optic Sensing (DFOS) technologies are widely applied in structure health monitoring due to many advantages over other conventional sensors (Ye et al., 2013; Rodrigues et al., 2015). Sensing cables with both sensing and transmission functions are slender and flexible and hence are suited to bound to the surface or embed inside the geological bodies and related structures without affecting the measured object’s structure to monitor strain, temperature, displacement, moisture, seepage and related parameters using a variety of equivalent transformations (Sun et al., 2014; Zhu et al., 2014a). The stability and safety monitoring based on DFOS technologies of civil engineering structures such as pipelines, dams, tunnels, bridges and historic buildings has been extensively discussed in the last decade (Bastianini et al., 2005; Matta et al., 2008; Rajeev et al., 2013; Gue et al., 2015; Lim et al., 2016). More recently, the application of DFOS for geotechnical structures monitoring were in-depth studied as an urgent problem. The efficiency of monitoring slope stability through BOTDA sensing technology in a media-sized model of soil nailed slop in laboratory was proved by Zhu et al. (2014b). Sun et al. (2014) systematically studied the multi-field information from slopes using DFOS technologies and emphasized its advantages for multi-field information monitoring, including stress, temperature, seepage and deformation in rock-soil mass. Kunitsu and Kokubo (2010) have demonstrated the feasibility of using optical fibers to monitor deformation at different depths in boreholes. Finally, Wu et al. (2015) successfully applied DFOS technologies to field monitoring of land subsidence. According to the monitoring data, significant compressive strain in the aquitard overlying the pumping aquifers was found and the deformation of aquitard was delayed in comparison with the fluctuation of water table in the monitored adjacent aquifer. However, it is hard to obtain the water level and water content changes in each soil layer in the field due to the limit of monitoring technology. Hence, synchronous monitoring of soil deformation and water content of land subsidence has not been reported and the deformation responses of the whole soil profile to water level changes have not been well explained.

On the basis of above description, it can be concluded that deformation of soil layers are closely linked with water level changes. The purpose of this study is to understand the deformation responses of soil under drainage and recharge conditions, respectively. Pulse-PrePump BOTDA and FBG technologies were applied in a model box with sand-clay interbedded layers. The strain and water content distribution of soil profile were synchronously monitored during two drainage-recharge cycles. Finally, soil compression potential and deformation mechanism were discussed for land subsidence prediction and assessment.

2. DFOS technologies

2.1. Water content monitoring principle

Soil water content is monitored using fiber Bragg grating (FBG) technique. According to Bragg’s law, a beam of white light is written in the FBG sensor, and when the light from the broadband source passes through the grating at a particular wavelength, the Bragg wavelength ($\lambda_B$) depend on the effective reflected index ($n_{eff}$) and grating period ($\Lambda$) is reflected (Leng and Asundi, 2003; Zhou et al., 2003). Any local strain or temperature changes alter the index of core refraction and the grating period, followed by changes in wavelength of the reflected light. The variation of the Bragg wavelength caused by temperature can be obtained as follows (Kersey et al., 1997):

$$\frac{\Delta \lambda}{\Delta T} = \gamma (T - T_0)$$

where $\frac{\Delta \lambda}{\Delta T}$ is the wavelength change rate, $\gamma$ is the wavelength-temperature sensitivity factor, and $T - T_0$ is the change in temperature. FBG sensors, kind of typical quasi-distributed sensors, can be multiplexed to measure temperatures at many locations when gratings with different periods are arranged along an optical fiber (Zhu et al., 2014a). Each of the reflected signals will have a unique wavelength and can be easily monitored, thus achieving multiplexing of the outputs of multiple sensors using a single fiber (Li et al., 2004; Zhu et al., 2017) (Fig. 1).

In this experiment, a soil moisture sensor is developed based on a carbon fiber heated sensing tube (CFHST) which is affixed with a series of FBGs sensors. The moisture sensor is embedded in soil and electrified. During the heating process, temperature curves for a specific location are obtained by analyzing the FBG wavelength shift. Because soil thermal conduction primarily changes with water content, the correlation of fiber temperature to soil water content can be established (Côté and Konrad, 2005). The theoretical model between soil temperature rise and moisture has not been established so far, so empirical modes are usually used (Giocca et al., 2012). Sayde et al. (2010) suggested using a piecewise function to describe this relation, which has been adopted by some other scholars (Striegl and Loheide, 2012; Sourbeour and Loheide, 2015). Here, the relationship is expressed as:

![Fig. 1. Principles, multiplexing schemes and wavelength shift of FBG sensors.](image-url)
Table 1  
Relationship between temperature characteristic values and soil moisture.

<table>
<thead>
<tr>
<th>Soil</th>
<th>θ₀ (m³/m³)</th>
<th>z₁</th>
<th>z₂</th>
<th>z₃</th>
<th>k</th>
<th>b</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.03</td>
<td>4.305</td>
<td>0.459</td>
<td>-</td>
<td>0.079</td>
<td>-372.203</td>
<td>20.181</td>
</tr>
<tr>
<td>Clay</td>
<td>0.06</td>
<td>2.460</td>
<td>3.111</td>
<td>0.047</td>
<td>-225.431</td>
<td>24.207</td>
<td>0.963</td>
</tr>
</tbody>
</table>

where \( T_i = \begin{cases} z_1 - z_2 \ln(\theta + z_3), & \theta > \theta_0 \\ k\theta + b, & \theta \leq \theta_0 \end{cases} \) (2)

2.2. Soil strain monitoring principle

Soil strain is monitored by Brillouin Optical Time Domain Analysis (BOTDA) technique which was first demonstrated by Horiguchi and Tatada (1989). Brillouin scattered light is caused by nonlinear interactions between incident light and phonons that are thermally excited within the light propagation medium. This scattered light shifts in frequency by a Brillouin shift and propagates in the opposite direction relative to the incident light (Kwon et al., 2002; Wang et al., 2009a; Iten and Puzrin, 2009). Fig. 2 shows the monitoring principle of BOTDA. There is a linear relationship between strain and temperature and frequency shift within the sensing optic fiber. A significant advantage of Brillouin scattering compared with other scattered light is that the frequency shift caused by temperature is much smaller than that caused by strain (0.002%/°C) (Song et al., 2016). Therefore, while measuring the Brillouin frequency shift (BFS) caused by strain, the temperature influence on the BFS can be neglected if the temperature changes are within 5 °C (Tong et al., 2009). When strain occurs in the optical fiber's longitudinal direction, Brillouin backscattered light undergoes a frequency shift that is proportional to strain (Horiguchi et al., 1989). The BFS as a function of strain is expressed in Eq. (3) (Kwon et al., 2002):

\[
ν(ε) = ν(0) + \frac{dν(ε)}{dε} ε
\]

where \( ν(ε) \) is the center Brillouin frequency under strain \( ε \), \( ν(0) \) denotes its original value, \( \frac{dν(ε)}{dε} \) is a strain coefficient and the proportional coefficient of strain at a wavelength of 1.55 μm is approximately 0.5 GHz/% (strain). In this way, the BFS at each point along the optical fiber can be measured. Using the relationship between BSF and strain, the distributed strain can be obtained.

2.3. Sensing cables and equipment

Two types of sensing cables fabricated by Suzhou NanZee Sensing Technology Co. Ltd., China, were used in this model test. A special tight-buffered single-mode sensing cable—1.2-mm Hytrel sensing cable—was selected as the test strain sensing cable (SSC). To improve the deformation coupling effect between the SSC and soil layers, 4-cm-diameter and 1-mm-thick plexiglass disks were fixed perpendicular to the cable every 10 cm using epoxy resin and protected by 2-mm-long quartz glass tubes at fixed points (Fig. 3a). CFHST, a 3-mm-inner-diameter, 5-mm-outer-diameter and 1-m-long carbon fiber tube with FBGs fixed every 10 cm on the outside surface was used for water content monitoring. The rod resistance of the carbon fiber tube is 19.4 Ω/m. The tube was protected with an outer heat shrinkable tube, and wires were traversed through the CFHST for heating (Fig. 3b).

An NBX-6050A PPP-BOTDA sensing interrogator developed by Neubrex (Osaka, Japan) and an NZS-FBG-A03 FBG demodulation instrument developed by NanZee (Suzhou, China) were used to scan the SSC and CFHST, respectively. Parameters of the equipment and SSC are shown in Table 2.
3. Model test

3.1. Test box and materials

To study the interaction and deformation response characteristics of different soil layers during drainage and recharge in detail, a relatively small model land subsidence test was conducted in a model box with an inner diameter of 42 cm and a height of 100 cm. As shown in Fig. 4, the box was flange-connected by three organic glass cylinders and a foundation. Xiashu soil, which is widely distributed in the Nanjing area, China, was used as a model test clay layer. Physical and mechanical properties of clay and sand are given in Table 3. The grain size distribution curves for sand and clay soils used in the study are plotted in Fig. 5.

3.2. Test procedure

(1) Model construction. As shown in Figs. 4 and 6a, a 10-cm-thick upper sand layer (Sand-1), a 30-cm-thick clay layer (Clay) and a 20-cm-thick lower sand layer (Sand-2) filled the box from top to bottom. To reduce friction between the box walls and soil, a thin polythene sheet with petroleum grease was placed within the test box. The test soil with an initial water content of 12% was positioned in a series of horizontal layers with a thickness of 5 cm each and then tamped equally to obtain a dry density of 1.4 g/cm³. After the box was completely filled, it was kept static for 48 h to ensure soil mass consistency. The SSC embedded in the soil was designed to form a loop, and the ends were connected to BOTDA (Fig. 6b). To better capture compressive strain, the cable was pre-stretched to approximately 7000 με and secured at both ends at fixed points. At the same time, 6 step bench marks (SBM) marked 0#–5# were placed every 10 cm in the soil from top to bottom for comparison with SSC monitoring data (Fig. 6d). Two CFHSTs with the first FBGs at 5 cm and 10 cm from the model bottom were simultaneously connected to A03. Six water tubes were buried from 5 cm to 55 cm deep from the soil surface to observe water level changes in different layers. Water level control was achieved through a water tank and valve in Sand-2. With these devices and sensors, water head and water content change curves with time at varying soil depths, soil strain and settlement curves at different times during drainage and recharge were obtained.

(2) Drainage test. After completion of soil filling, water was slowly pumped into the box by opening the water valve on the bottom right of the model box joined to a water tank (see Fig. 6a). For
complete saturation of the soil sample and to ensure that the soil gases were fully eliminated, the model was kept static for 24 h after saturation. The water tank was then removed, and the water valve was opened below to simulate groundwater extraction. To avoid the influence of heating CFHSTs, soil strain in the depth direction was first measured by BOTDA, and then the water content was measured by continuously heating the CFHSTs for 20 min. Data from the water level tubes and SBMs were recorded simultaneously. Data were collected hourly. The drainage experiment was complete once the water levels in the tubes remained stable.

(3) Recharge test. Water was gradually injected into the model box by connecting the water valve to the water tank. Soil strain and water content were monitored by NBX-6050A and A03, respectively. Data from the water level tubes and SBMs were recorded hourly. Once the water levels in the water level tubes remained stable, the recharge test was completed.

Two drainage and recharge cycles were carried out to study the influence of water level changes on soil deformation.

**Table 3**

<table>
<thead>
<tr>
<th>Physical and mechanical properties of clay and sand soil.</th>
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</thead>
<tbody>
<tr>
<td><strong>Clay</strong></td>
</tr>
<tr>
<td>2.73</td>
</tr>
<tr>
<td><strong>Sand</strong></td>
</tr>
<tr>
<td>2.65</td>
</tr>
</tbody>
</table>

**Fig. 5.** Test soil particle size distributions.

**Fig. 6.** Land subsidence model soil box test: (a) overall picture of the test; (b) strain sensing cables; (c) layout of sensing cables in the model; (d) assembly diagram of step bench mark.
4. Test results

4.1. Soil strain and deformation response during drainage

Fig. 7a shows the vertical soil strain at different times monitored by SSC during drainage. The initial strain data are subtracted from the later monitoring data, so compressive strain is negative while tensile strain is positive. During the first 4 h of drainage, the strain changes of each soil layer are relatively small, and compressive strain primarily occurs below a depth of 20 cm. Compressive strain increases gradually after 4 h, and the most obvious compressive strain is located at a depth of 20 cm. The compressive strain reaches a maximum of approximately 6000 με in the 10- to 40-cm-deep clay layer and gradually reduces to the sand layer values on both sides. Sandy soil has a larger compression modulus than clay soil (14.9 MPa for sand and 2.6 MPa for clay), so deformation of sand is relatively small under the same stress. In addition, after 24 h of saturation and sandy soil collapse, the sand particles readjust, which lead to a large decrease in sand compressibility.

By integrating the strain curves along the direction of depth, the corresponding deformations are obtained. Fig. 7b shows the accumulative deformation curves at the different times. It is observed that the total amount of soil compression is approximately 1.9 mm. A, B and C represent the compression of Sand-2, Clay and Sand-1, respectively. Similarly, the primary compression is observed in the 10- to 40-cm-deep clay layer, and deformation of Sand-2 is greater than that of Sand-1.

Fig. 8 compares displacements monitored by SSC and six SBMs, representing soil deformation at depths of 0–60 cm, 10–60 cm, 20–60 cm, 30–60 cm, 40–60 cm, 50–60 cm. The SSC results are consistent with SBM, showing that the main deformation occurs in the clay layer, while little deformation occurs in sand layers. At drainage onset, no obvious change is observed in the soil. Compression deformation clearly increases starting at 4 h and stabilizes after 10 h (Fig. 8a). Because of strain loss causes by shear protection, leading to undermeasurement of strain SSC compared to actual strain (Wu et al., 2015), the final stable total compression of SSC is 1.9 mm, which is slightly smaller than 2.1 mm of SBM (Fig. 8b). The relative error was approximately 9.5% relative to the SBM. However, it is found that the SSC measured compression is larger than SBM in Sand-2 (60–50 cm and 60–40 cm) and this may be attributed to the slightly difference of soil density during artificial compaction. Since the compression of sand is relatively small, it is possible for the compression monitored by SSC larger than that of SBM if the density of the SSC position is smaller than the SBM position.

4.2. Soil strain and deformation response during recharge

Fig. 9a presents vertical soil strain at different times measured by SSC during recharge. Tensile strain primarily appears at a depth of 10–40 cm, suggesting a clear rebound in the clay layer. The upper clay layer strain reaches a maximum value of 1300 με, which is greater than that of the lower clay layer. According to water content monitoring results, the average water content of the upper and lower clay are 25.0% and 25.9% at the end of the drainage, respectively, while reach 41.1% and 39.8% when recharge finishes. The change of water content of upper clay is 16.1%, greater than the 13.9% of the lower clay layer. It can be concluded that the upper clay layer close to the surface is affected by evaporation during drainage which results in lower moisture content and increased soil suction (Yong, 1999; Wu et al., 2007; Greco et al., 2010). During recharge, the upper clay layer absorbs more water that increases the spacing of soil particles, therefore more inflation of clay minerals occurs (Tessier, 1999; Cui et al., 2006). The lower clay layer with smaller water content changes has a better water-holding capacity that reduces the rebound (Li et al., 2006). Fig. 9b shows the accumulative deformation curves at different times. The total soil layer rebound is approximately 0.33 mm, which is far less than the 1.9 mm from compression. The clay layer rebound B accounts for most of the deformation and Sand-2, A is approximately 0.05 mm. Sand-1 has the minimum rebound amount C because of its thin thickness.

Tensile test according to the method described in (Wu et al., 2015) has been done to 1.2 mm sensing cables before the model test; the results are plotted in Fig. 10. It shows that monitored displacements at each step are smaller than the actual displacements during tension because of the strain loss results from the sheath. With the increasing tension displacement, strain loss achieves the minimum 0.06% at 3 mm displacement and then turns to small fluctuations. The average strain loss during tensile test is 2.74%.

Fig. 12 compares displacements monitored by SSC and six SBMs in the model test. Both showed the same trend that no obvious changes occur in the soil until 21 h (Fig. 11a). Then, rebound of different layers increases gradually to the stable deformation of 0.33 mm from SSC and 0.34 mm from SBM (Fig. 11b). The relative error is 2.9%, which is slightly larger than the average strain loss of 2.74% in lab. This difference is due the incompletely coupling between optic fiber and the soil under the small confining pressure in model. In fact, there is...
enough confining pressure in the field-drilling hole (Wu et al., 2015), hence, it is possible to quantitatively monitor land subsidence in field using optic fiber once the strain loss is obtained in lab. It is noteworthy that the deformation monitored by SSC was stable after 25 h, which is significantly faster than that measured by SBM, proving the superior sensitivity of SSC to SBM.

Fig. 8. Comparison of vertical displacements from SBM and SSC during drainage: (a) Time history curves of deformation; (b) final compression of different soil depths.

Fig. 9. Soil strain changes during recharge (a) and accumulative deformation (b).

Fig. 10. Strain loss of 1.2-mm HY cable during tension test.
4.3. Soil strain and deformation response during drainage-recharge cycles

To study the effects of seasonal water level changes and repeated pumping-recharge on land subsidence, two drainage and recharge cycles were carried out. The strain changes are plotted in Fig. 12a–d, and the corresponding accumulative deformations are plotted in Fig. 12e–h. As seen by comparing Fig. 12c with 12a, clear vertical compressive strains occurs in both drainages. Unlike the first drainage, in the second drainage, Sand-2 strain change is negligible, and deformation is mainly concentrated in the upper clay layer. In addition, the maximum compressive strain is approximately 3000 $\mu$ε, which is significantly less than the maximum 6000 $\mu$ε in the first drainage. Correspondingly, total compression deformation in the second drainage is 0.6 mm (Fig. 12g) less than 1.9 mm in the first drainage (Fig. 12e). As seen by comparing

Fig. 11. Comparison of vertical displacements from SBM and SSC during recharge: (a) Time history curves of deformation; (b) final rebound of different soil depths.

Fig. 12. Soil strain and deformation response during drainage and recharge cycles: (a–d) The soil profile strain changes; (e–h) Integral deformations.
Fig. 12b and d, vertical tensile strains occur in both recharges. However, the main rebound occurs in the whole clay layer the first time but only in the upper clay layer the second time. The stable rebound of the first recharge is 0.33 mm (Fig. 12f), which is slightly larger than 0.30 mm in the second recharge (Fig. 12h). We conclude that the compressibility of the lower clay layer is reduced after a single drainage-recharge cycle. Parker et al. (1982) reported that changes in structure which accompany drying of soils and clays may affect the magnitude of expansion on rewetting. Parallel orientation may be induced by drying of dispersed clays (Rowell, 1963). Before the first cycle, soil is relatively loose; its microstructure must be defined by dominating face to edge particle arrangement with a large contact angle. When drainage, water is expelled from the soil, giving rise to collapse of the face-edge structure and rendering clays particles more and more oriented. With the decreasing contact angle, soil microstructure transforms from face-edge structure to face-face structure (Cui et al., 2006). Tang and Shi (2011) pointed that because the position and arrangement of clay particles changed greatly in this process, the changes in the structure to a certain extent, is irreversible.

5. Discussion

5.1. Deformation characteristics of sand and clay layers

Fig. 13 presents deformation change-time curves for Sand-1, Clay and Sand-2. We find that compression and rebound of the clay layer are significantly greater than those of the sand layers. The clay layer compression curves have obvious segmentation. At drainage onset, compression is not obvious in this slow deformation stage (Stage A); with the release of water, compression increases rapidly with increased pressure and transforms into a rapid deformation stage (Stage B). Soil compression until this stage is mainly complete. At Stage C, deformation is still slowly continuing with time, although the deformation is almost complete. Stage A and B are the primary consolidation stages, while Stage C shows secondary consolidation, which is mainly due to the creep of clay soil. Creep deformation refers to the continuous compression with time while the stress state remains constant (Hyde and Burke, 1988; Lade and Liu, 1998). Although only a small amount of creep compression is monitored in this test, it has been proved by the field observation data that time-dependent creep effects play an important role in land subsidence, which can lead to the lag phenomenon occurrence in both aquifers and aquitards (Shi et al., 2007; Wang et al., 2009b). The slowness of the drop and rise of the groundwater level gives the creep deformation enough time to develop. Thus, the aquifer system keeps compressed on whole even during recharging, only the subsidence rate reduces temporarily (Ye et al., 2005). Wang et al. (2013) stated that creep deformation should not be neglected after groundwater pumping stops and/or during the groundwater recharging, as it is an important factor for land subsidence prediction.

The compression initiation time in the first drainage, \( t_{d1} = 4 \) h, is greater than that in the second drainage, \( t_{d2} = 2 \) h (Fig. 13a and b), and the rebound initiation time in the first recharge, \( t_{r1} = 21 \) h, is greater than that in the second recharge, \( t_{r2} = 6 \) h (Fig. 13c and d). Because clay permeability is comparatively lower, an initial hydraulic gradient develops when pumping the aquifer and dropping the water level. Once the hydraulic head pressure difference between the aquifer and clay layer is enough to overcome the bonding force between particles, water is released out of the voids (Su et al., 2014). Due to the inhomogeneity of Sand-2 in artificial compaction, pore channels between sand particles are obstructed; hence, it is slow for the overlying clay layer to release and absorb water. After a single drainage-recharge cycle, sand pores gain uniformity by grain rearrangement, restructuring and position readjustment, which cause the lag between release and absorption of...
water to become shorter in the overlying clay layer.

However, the time for main deformation fulfillment has the opposite trend showing that \( t'_{21} = 4 \, \text{h} < t'_{22} = 6 \, \text{h} \) and \( t'_{11} = 5 \, \text{h} < t'_{12} = 12 \, \text{h} \). It is approved that the clay layer consolidation time is mainly determined by soil permeability and thickness (Liu, 2005). Clay soils before and after two drainage-recharge cycles were sampled to measure the hydraulic conductivity. Before the test, hydraulic conductivity is \( 2.15 \times 10^{-4} \, \text{cm/s} \) and after 2 cycles it becomes \( 9.70 \times 10^{-5} \, \text{cm/s} \). This suggests that the clay soil permeability decreases greatly after drainage-recharge cycles. Lapierre et al. (1990) mentioned that there is a relationship between the permeability and the pore-size parameters of the clay. Seeing from the micro view, pore size distribution changes with different consolidation stage and consolidation depths (Griffiths and Joshi, 1991; Cui et al., 2016). During 1-D consolidation, deformation is primarily the result of the compression of inter-aggregate pores and the intra-aggregate pores (Wang and Xu, 2007; Kanayama et al., 2009). Clay aggregates adjust positions to minimize porosity between aggregates (Yu et al., 2016). In addition, clay minerals within the aggregates change their orientation alignment and rotation, which decrease the internal pores within the clay aggregates. As a result, void ratio and permeability of soil decreases, which result in consolidation time increases.

Table 4 summarizes the final stable deformation of Sand-1, Clay and Sand-2 over two drainage-recharge cycles. All three layers show the same state of compressed → rebound → recompressed → rebound. The rebound is less than compression over 2 cycles revealing obvious irreversible deformation. Tang et al. (2010) explained that, as the soil is compressed at a low stress, the volumetric deformation is mainly attributed to sliding of the particles and this process is irreversible. With the increasing of compaction load, the aggregates may develop elastic deformation or pore fluid may be expelled, this part of deformation will rebound when pressure releases. Meanwhile, the soil mass water absorbing capacity in the recharge stage is smaller than that in the drainage stage under the corresponding pressure. According to Wang and Wang (2007) the quantity of water absorption during recharge only accounts for 12%–19% of water release during drainage under the corresponding pressure. Compression in the first drainage is greater than that in the second drainage because the soil void ratio, compressibility, permeability coefficient decrease after the first cycle. Laboratory test of compression-rebound-recompression shows the compression coefficient of clay and sand decreases from 0.50 MPa\(^{-1}\) to 0.23 MPa\(^{-1}\), and 0.10 MPa\(^{-1}\) to 0.05 MPa\(^{-1}\) respectively. However, clay layer rebound in the first recharge is only slightly larger than that in the second recharge. The Sand-1 and Sand-2 rebounds are basically the same. This stable rebound of the sand layer represents elastic rebound of the sand skeleton.

The unit water head deformation (UWHD) is a characteristic parameter reflecting the stress-strain relationship of soil under the actual stress state and can be used to determine residual soil deformation characteristics for simple subsidence prediction (Riley, 1969; Helm, 1976). The UWHD is defined as:

\[
I_s = \frac{\Delta S_s}{\Delta h_s}, \quad I_c = \frac{\Delta S_c}{\Delta h_c}
\]

where \( I_s \) and \( I_c \) are UWHDs of water level rising or falling, respectively (mm/m); \( \Delta h_s \) and \( \Delta h_c \) are the ranges of water level rise or fall, respectively (m); \( \Delta S_s \) and \( \Delta S_c \) are the corresponding rebounds or compressions when water level rises or falls (mm). Define \( C_p = |I_s|/|I_c| \), which characterizes the soil elastic characteristics. In the periodic rise and fall of the water level, \( C_p \) is generally less than 1. When \( C_p \approx 1 \), residual deformation disappears, and soil enters the elastic deformation stage (Su, 1979). In this model box test, the ranges of water level rise and fall are the same, so \( C_p \) is simplified as \( |S_s|/|S_c| \).

Table 2, the \( C_p \) value of the entire soil layer increases from 0.17 in the first cycle to 0.50 in the second cycle, showing that subsidence continues under repeated water level fluctuations due to residual deformation. The \( C_p \) of each soil layer is significantly larger than \( C_p \), indicating that residual deformation of soils gradually decreases with the increasing repetitive water level fluctuations. This is approved by Ili and George (1966) on the micro level that although changes in microstructure, which accompany compression and rebound, are not likely to be entirely reversible, after repeated cycles of drainage and recharge, compression and rebound become very nearly equal. Similar observations are also found both in Shanghai and Taiwan land subsid- ence area by Zhang et al. (2003) and Liu et al. (2004). When water level fluctuations in a relatively stable range, soil deformation shows a certain degree of elastic characteristic. That is, the soil consolidation will gradually come to an end, and further seasonal fluctuation of groundwater level will only cause the soil deformation response in an elastic or recoverable manner. Only if the groundwater level drop exceeds a certain yielding limit again, the compression behavior of the soil layer will exhibit inelastic or irreversible deformation.

5.2. The relationship between soil deformation and water content in land subsidence

Land subsidence is closely related to soil seepage, and soil-water coupling effects have an important influence on soil deformation. Fig. 14 describes the water change content in the soil’s vertical direction during drainage and recharge (take the second cycle as example). Before drainage, the clay and sand layers are in a saturated state with saturated water contents of 55% and 32%, respectively. During 0–2 h, water in Sand-2 first drains away, and its water content subsequently decreases. The main drainage is from 2 h to 8 h when the water content of Sand-2 decreases continuously and the pore water seepage of Clay starts from the side near Sand-2 under the hydraulic gradient effect. The water content of Clay gradually decreases, and leakage recharge from the upper Sand-1 occurs at the same time, which quickly reduces its water content, and drops the water level. From 8 h to 24 h, the water contents of the Sand-2 and Clay layer decreases slowly. Until the cessation of drainage, the stable water content of Clay is larger than that of the sand layers on both sides.

For the first 6 h of recharge, the water content of Sand-2 increases as water fills the model box. From 6 h to 18 h, under the hydraulic gradient effect, water enters the clay layer, whose water content gradually increases due to continuously water absorption. Meanwhile, penetration occurs in the clay layer with increasing pore water pressure, and a concomitant rise in the water level leads to an increase in the water content of Sand-1.

The average water contents of each layer at different times are calculated to determine the relationship between soil deformation and water content. As plotted in Fig. 15, the saturated water contents of three layers at the onset of drainage are larger than those at the end of recharge, indicating that the soil internal void ratio decreases; that is, the soil layers compressed after a single cycle. During drainage, soil layers show compression deformation with decreasing water content. During recharge, soil layers show rebound deformation with increasing water content. This trend is quite similar to the results found by Chang et al. (2004) that when the land subsidence is significant, in the dry season, the water table is lowered; while when the land subsidence is minor, in the wet season, the water table is uplifted. Deformations of
Sand-1 and Sand-2 layers show slight fluctuations that are basically synchronous with water content changes. Nevertheless, the Clay deformation curve is relatively smooth and lags slightly behind the water content changes. It is proved that the deformation lagging behind water level changes is the result of infiltration characteristics of clayey soil and slow dissipation of transient excess pore water pressure (Riley and Stewartson, 1969; Ding et al., 2012).

For the Clay layer, it is worth mentioning that the water content changes during the second drainage and recharge cycle.

![Fig. 14. Evolution of water content distribution in during the second drainage and recharge cycle.](image)

![Fig. 15. Water content and deformation changes versus time curves during the second drainage and recharge: (a) Sand-1; (b) Clay; (c) Sand-2.](image)
decreases rapidly when it is lower than its liquid limit, and concomitantly, the compression rate obviously increases during drainage. The rebound rate significantly increases when the water content is higher than its liquid limit during recharge (Fig. 15b). This occurred because clay particles have an electric double layer structure (Mitchell and Soga, 2005) and the combination of water and clay particles has a relationship with water content (Fukue et al., 1999). When the water content is higher ($\omega \geq \omega_p$), clay particles are dispersed, and free water appears around the periphery of the soil particle diffusion layer when the soil porosity is larger. The distance between clay particles is beyond the scope of gravitational attraction; hence, intergranular binding strength almost disappears. The pore water is mostly free water, which is easily discharged under the effect of small hydraulic gradients, making clayey soil compressed. With a gradually reduction in water content ($\omega_L > \omega > \omega_p$), pore water mainly consists of adsorption bound water within the diffusion layer and a small amount of free water. Under the effect of hydraulic gradient, a small amount of free water is released first. When water extraction increases, the hydraulic gradient increases enough to overcome the shearing strength of penetration adsorption bound water within the diffusion layer, and then the penetration adsorption bound water becomes free water and is released (Dixon et al., 1992; Chen et al., 2001). The distance between soil particles decreases, and large pores between clay aggregates collapse, resulting in further compression of the clay layer. Drainage consolidation of clay soil is basically completed. If the moisture content is lower ($\omega < \omega_p$), pore water is given priority to strongly bound water, which interacts by strong electrostatic attraction with clay particles and hydrogen bond connections. It is not easily compressed, as its properties are similar to those of a solid (Fine, 1973).

5.3. The mechanism of soil deformation

When extracting groundwater, the pore water pressure of confined aquifer subsequently decreases and the effective stress increases concomitantly. The relationship between changes in pore fluid pressure and compression of the aquifer system is based on the principle of effective stress first proposed by Terzaghi (1925):

$$\sigma_e = \sigma_T - \mu$$  \hspace{1cm} (5)

where effective or inter-granular stress ($\sigma_e$) is the difference between the total stress ($\sigma_T$) and the pore water pressure ($\mu$). The sands in aquifier loosely packed with large pore spaces are unstable due to the granular structure of overhead contact. Sand particles contact each other and its connection strength depends on the $\sigma_e$. As shown in Fig. 16a, contact force between sand particles can be decomposed into the tangential force ($T$) and normal force ($P$). As the water level changes, $P$ simultaneously changes making the soil structure instability. After two drainage-recharge cycles, grain crushing can been found in Sand-2 from the electron microscope image (Fig. 16b). Chuhan et al. (2003) noted that sand compression is initially due to grain-scale frictional slip, rotation, and sliding, and, if the effective stress is sufficiently high, crushing of the particles becomes prominent. Crushing generates large relative motions between sand particles, enhances deformation, results in tighter grain packing, and reduces the primary porosity. Bowmani and Soga (2004) also observed that soil particles, which go through the process of arrangement, restructuring (sliding and rolling, etc.) and broken are squeezed, making water sand layer compressed. However, Chuhan et al. (2002) suggest that although grain crushing enhances compression, the local crushing at the grain contact increases the contact area between the grains, which further reduces the compressibility of sands. As a result, the deformation curves of sand show that the rate of compression decreases with increasing time.

When the aquifer water level continues to decline, the pore water pressure of adjacent clay layers relatively rises, forming the hydraulic gradient that points to the aquifer. The clay layer is finally compressed by discharging pore water to the aquifer. According to the monitored compression curves of clay soil, compaction process can be roughly divided into primary consolidation and secondary consolidation. Similar compression behavior are also observed in land subsidence field studies (Shi et al., 2007; Wang et al., 2009b; Mahmoudpour et al., 2016). The macrolevel behavior of soil, to a great extent, is controlled by its microstructure and mechanical properties of its particles (Cui and Jia, 2013). Under a smaller hydraulic gradient, the initial drainage from aquiclude is the free water. Clay layers begin to be compressed as the pore water pressure reduces and the effective stress increases. With the continuously reducing of water content, infiltration adsorption combined water within the diffusion layer turns into free water and is concomitant released. Meanwhile, leakage yield from the leaky layer occurs. The distance between soil particles decreases, large pores between clay aggregates are extruded and damaged which further reduces the pores and increase the compression of clay layers, as illustrated by Yu et al. (2016) in Fig. 16c. Seepage force caused displacement, rotation and orientation of clay particles and sharply increasing effective stress has thinner the hydration film as well as caused the soil microstructures failure and destruction, that is, plastic compression of clay soil. In addition to the decrease of pores and elastic compression of soil skeleton, the plastic compression is the major part of the land subsidence. Secondary compression is considered as the deformation that occurs at a constant effective stress, i.e., after the dissipation of excess pore pressure (Griffiths and Joshi, 1991). Taylor and Merchant (1940) described secondary consolidation as owing to viscous and structural reorientation caused by shear stresses developed during primary consolidation. Yong and Mckyes (1969) considered secondary compression to be the result of adjacent particles sliding relative to each other owing to the action and change of micro stresses. Wang (1992) and Rajapakse (2016) both suggested that the primary consolidation and secondary consolidation happen at the same time. The secondary consolidation deformation accounts for the proportion to be few at the beginning. With the dissipation of pore water pressure and increase of effective stress, the proportion of secondary consolidation deformation gradually increases.

When recharging groundwater, a slight rebound of sand occurs with the rising water level of confined water sand. Under the effect of hydraulic gradient, water begins to enter into the clay layer. With the constantly water absorption, pore water pressure in the clay layer increases and the effective stress reduces which brings about the rebound,

![Fig. 16. Illustrations of sand and clay microstructure deformation: (a) Contact force between sand particles; (b) electron microscope image of sand-2 after 2 cycles; (c) the compression of pores and collapse of card-house structure subjected to isotropic consolidation modified from (Yu et al., 2016).](image-url)
reflecting the elastic characteristics of clay soil. However, the rebound that occurs when penetrating leakage appears (from the aquifer to clay layer) may be related to the microstructures and mineral composition of soil (Gong, 2002; Luo et al., 2014). For this reason, groundwater exploitation ban have been carried out (Shi et al., 2012), artificial recharge of groundwater have been implemented during the last two decades to mitigate the negative impacts of land subsidence in China (Han, 2003; Tu et al., 2011; Shi et al., 2016). However, field data have revealed that land subsidence did not cease but continued when the groundwater level in aquifers began to increase, and that land uplift often occurred after the groundwater level had risen for many years (Schmidt and Bürgmann, 2003; Chen et al., 2007). Zhang et al. (2015) believed that the lagging is attributed not only to the visco-plastic deformation and the consolidation deformation of aquitard units, but also to the visco-plastic deformation of the aquifer units. However, these phenomena are not clearly observed in the model test. This is because compared to the model test, groundwater level usually drops and rises slowly in land subsidence areas, which gives creep deformation plenty of time to develop. Moreover, the soil layers with much larger thickness and lower permeability leads to the great lagging of rebound behind the recovery of groundwater level.

Under the condition of repeated pumping-recharge, the soil structure will reach a relatively stable state after large plastic deformation and creep deformation. Compression deformation caused by drainage will become smaller and smaller, while rebound deformation will relatively increase. When the soil layers mainly occur elastic deformation, the recharge effect could be best.

6. Conclusions

Detailed and distributed monitoring of aquifer systems is important for understanding deformation laws in pumping and artificial groundwater recharge processes, which can serve as a guide to judge land subsidence trends. In this paper, distributed fiber optical sensing (DFOS) technologies were applied to a model test to simultaneously monitor soil strain and water content changes during drainage and recharge cycles. The response mechanisms of sand and clay layers in the process of drainage and recharge were subsequently analyzed. A few conclusions are drawn as follows:

(1) The DFOS technologies have significant advantages that can achieve distributed, sensitive and precise acquisition of multi-fields information (strain, moisture, etc.) during the land subsidence processes compared to traditional step branch marks. It offers a novel monitoring method for land subsidence mechanism analysis and early warning.

(2) During drainage or recharge, soil layers shows compression or rebound with decreases or increases in water content. The sand layer deformation shows slight fluctuations that are basically synchronous with water content changes. Nevertheless, the clay layer deformation curve is relatively smooth and lags slightly behind water content changes due to its lower permeability. In addition, deformation of the sand layers is much smaller than that of the clay layer.

(3) In draining-recharging cycles, decreases in the soil porosity and permeability coefficient will increase the consolidation time. Residual deformation gradually decreases with an increase in repetitive water level fluctuations. Compression caused by drainage becomes smaller and smaller, while rebound increases. After a number of cycles, the soil structure reaches a relatively stable state after large plastic deformation and creep deformation and tends toward an elastic deformation stage.

(4) According to the compression curve, the clay layer compression process can be divided into primary consolidation and secondary consolidation. There is a good corresponding relationship between deformation and water content. The water content decreases rapidly when it is lower than the liquid limit, and the compression rate clearly increases during drainage. The rebound rate speeds up significantly when the water content is higher than the liquid limit during recharge.

It must be pointed out that this small scale and short duration model test can qualitatively reflect the deformation response characteristics of sand and clay layer to water level changes. These results are suitable to explain the deformation mechanism of soil layers near the water table, such as, land subsidence caused by groundwater level seasonal volatility in shallow phreatic aquifer and uneven consolidation settlement resulted from dewatering in excavation engineering. Regional land subsidence is usually large in scale and longer duration. Soil deformations are closely associated with soil permeability coefficient, drainage distance and drainage time, etc. When extracting deep groundwater in aquifers, groundwater flows are relatively complicated in pumping aquifers and adjacent weakly permeable layers since anisotropic and interlayer seepage both exit. Hence, further researches on large-scale model test and long-term field monitoring based on DFOS technologies need to be carried out as the influence of the factors mentioned above on soil deformation mechanisms are not yet quite clear.

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References


