Mechanism of two-dimensional long-term subsidence in surface peat layer

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ABSTRACT

In urban areas with extensive peat ground, such as Sapporo, long-term subsidence of shallow buried structures such as backfilled pipelines has been reported to pose chronic engineering problems. In addition to natural ground water fluctuations, pavement overlaying and nearby traffic loads, the load imbalance due to backfilling itself is suspected to aggravate the problem, as usually very light peat is replaced by heavier, compacted sandy fills. The objective of this paper is to discuss detailed mechanisms behind the subsidence observed in surface peat layers. In this study, subsidence mechanisms are investigated by performing long-term 2-D model tests and soil-water-coupled finite element (FE) analysis using a Modified Cam Clay-type model tuned to express the stiffness and consolidation characteristics peculiar to peats.

Keywords: peat, consolidation, subsidence, pipeline

1 INTRODUCTION

Peats, formed by remains of plants without enough decomposition, have extremely high natural water content and compressibility. The secondary consolidation observed after the end of primary consolidation is also known to be very large in peat ground. According to Mesri and Ajlouni (2007), the ratio of the coefficient of secondary consolidation $C_u (=\Delta e / \Delta \log t)$ to the compression index $C_c (=\Delta e / \Delta \log \sigma'_v)$ is 0.06±0.01, while the ratio for inorganic clays is generally smaller (0.04±0.01; Mesri and Castro, 1987). The secondary consolidation of the peat can be very large because $C_c$ of many peats is over 2.0. Thus when structures are constructed on peat ground, large long-term settlement is encountered due to combined effects of high compressibility and pronounced tendency for secondary consolidation.

In Hokkaido, Japan, there are large peatlands even in urban areas such as Sapporo. Subsidence of large-scale, heavy structures such as embankments on peat ground has been extensively studied by many local researchers and engineers. On the other hand, settlement problems of smaller-scale structures such as buried pipelines in surface peat layers have been receiving much less attention, despite its widespread occurrence. This problem has been confirmed from 18-year survey records of main gas pipeline subsidence at more than 200 fixed locations in Sapporo. However, the main cause of this problem has not been understood clearly. The following causes are imaginable.

i) The natural process of regional, wide-area subsidence in young peat ground (i.e. pipelines subsiding together with the surrounding ground).
ii) The influence of live traffic loads from roads, along which the gas lines are normally buried (i.e. pipelines subsiding together with roads).
iii) The influence of replacing light peat by heavier sandy backfill (i.e. pipelines subsiding into the stable surrounding ground).

The effective unit weight of peats is usually very small. Therefore, if a part of the original peat ground is replaced with heavier compacted sand, as shown in Fig.1, the effective stress in lower layers increases by modest amounts (approximately 15kPa in the shown example). For peats, this stress increment can be sufficient to cause large deformation.

The objective of this study is to clarify the mechanism behind the long-term subsidence behavior of pipeline structures buried in surface peat layers. 2-D long-term model tests were conducted to observe the 2-D deformation expected to be dominant in the problem, as opposed to the broadly 1-D deformation associated with embankment settlements. In addition, simulation of the model tests was conducted by soil-water-coupled finite element analysis using a Modified Cam Clay-type model with extended sub-models. From the results of these, the mechanism of the long-term settlement of shallow pipelines is explored.
2 TESTING PROCEDURES

The model tests were conducted in a plane-strain container with a transparent front panel for observation. The bottom is double-decked and it is possible to drain the pore water through the interstices in the upper deck during reconstitution of the peat. A typical model cross section is shown in Fig.2. The model tests, reduced to 1/7 of the prototype, assumed two cases. In one case, the whole peat layer thickness was three times as large as the depth of excavation (Case 1), as illustrated in Fig.2. In the other case, the thickness was twice the depth of excavation (Case 2). The peat soil used in the model tests was collected from Bansui district near Sapporo. The physical properties of the sample are shown in Table 1. Compared against existing databases (Yamaguchi et al., 1986, Oikawa and Ishida, 1993), it was confirmed that the relationships between the ignition loss and water contents, wet unit weight or specific gravity of the sample agreed well with those generally observed for a variety of peats. From these results, the tested peat sample can be regarded as representative of peats with water content of \( \approx 600\% \).

Results of 1-D consolidation tests on peat samples are shown in Fig.3. A step loading consolidation test was conducted on a specimen of 100mm in diameter and 70mm in height, while a constant-rate-of-strain (CRS, at a strain rate of 0.02%/min) consolidation test was conducted on a smaller specimen (60mm diameter and 20mm height). From the \( e \)-\( \log \sigma_v' \) curve of natural peat sample, the yield stress of peat sample is unclear. Results of Fig.3 were used reconstituted. In this study, therefore, the model ground was prepared by reconstitution by aiming to reproduce the in-situ water content, not by trying to achieve the same (unclear) apparent yield stress. The applied surcharge load was 5kPa.

In general, sand is a preferred material for backfilling around pipelines, due to its wide availability and ease of handling. However, if sand is used in this model test as it is, the scaling law cannot be satisfied in a reduced scale because the stress of the model ground becomes small proportionally to the model scale. Therefore, lead

![Diagram](image)

**Fig.1.** Typical cross-section of backfill around utility pipeline.

![Diagram](image)

**Fig.2.** Model cross-section (Case 1).

**Table 1.** Physical properties of Bansui peat.

| Water content (%) | Unit weight (kN/m³) | Ignition loss (%) | Degree of separation (%) | Specific gravity |
|-------------------|---------------------|-------------------|--------------------------|-----------------|
| 588               | 10.6                | 60.0              | 69.9                     | 2.15            |

![Diagram](image)

**Fig.3.** Compression curve, coefficient of permeability and consolidation of reconstituted peat samples.
shot of 1mm in diameter, having effective unit weight 7 times larger than that of sand, was used to satisfy the stress scaling law. The similitude between each physical variable is shown in Table 2. The scaling ratio $N$ is set as 7. For the peat, its effective unit weight is very small (0.8kN/m$^3$) and regarded as zero, again broadly satisfying the scaling law. After completing preparing the model ground, sheet-piled and braced-excavation was conducted in a similar way as real construction, and then the lead shot was gently pluviated. From the start of the excavation, the ground deformation was monitored with a digital camera.

3 ANALYSIS PROCEDURES

The adopted FE meshes and boundary conditions are shown in Fig.4. The FE analysis was conducted to simulate the model tests in the prototype scale by considering the right half of the models due to symmetry. A small surface surcharge of 1.26kN/m$^2$ was applied to avoid numerical instability. Drainage was allowed only along the ground surface in a same way as the model tests. The original water level was set same as the ground surface.

A model based on the Modified Cam Clay model was adopted for the peat ground and a linear elastic model for the backfill sand. The input parameters for the peat ground and the backfill sand are shown in Table 3. Here, $\lambda_k$ is the permeability coefficient index representing the slope of the $e$-$\log k$ curve. The coefficient of

### Table 2. Scaling law of model test.

|          | Prototype | Model |
|----------|-----------|-------|
| Length (Displacement) | 1 | $1/N$ |
| Stress | 1 | 1 |
| Effective unit weight | 1 | $N$ |
| Strain | 1 | 1 |
| Consolidation time | 1 | $1/N^2$ |

### Table 3. Input parameters in FE analysis and determination of values (including parameters that are calculated from the others).

| Symbol | Meaning | Peat: Formulation A | Peat: Formulation B | Sand** | Determination |
|--------|---------|---------------------|---------------------|--------|---------------|
| $\gamma_t$ | Total unit weight (kN/m$^3$) | 10.4 | 10.4 | 19.0 | From measurement |
| $E$ | Elastic modulus (kN/m$^2$) | - | - | 5600 | |
| $\lambda$ | Compression index | 1.06 | 1.06 | - | $0.434C_c$: From consolidation test |
| $\kappa$ | Swelling index | 0.11 | 0.11 | - | $0.1\kappa^*$ |
| $e_0$ | Initial void ratio | 11.5 | 11.5 | - | $e$ corresponding to 5kPa |
| $\varphi'$ | Effective angle of shear resistance (°) | 43.4 | 43.4 | - | $\varphi'=0.19L_i+0.32^*$ |
| $M$ | Critical state parameter | 1.79 | 1.79 | - | $M=6\sin\varphi'/(3-\sin\varphi')$ |
| $v_f$ | Eventual Poisson’s ratio at large strain | 0.24 | 0.24 | 0.33 | $v=(1-\sin\varphi')/\sin\varphi'*$ |
| $k_0$ | Initial coefficient of permeability (m/s) | $4.5 \times 10^{-8}$ | $4.5 \times 10^{-8}$ | - | $e$ corresponding to 5kPa |
| $\lambda_k$ | Permeability coefficient index | 0.346 | 0.346 | - | $0.434C_k$: From consol. test |
| $\nu_0$ | Initial Poisson’s ratio | 0.24 ($=v_d$) | Variable according to $p'$ (0 at $p'=5$ kPa) | ($=v_d$) | Assumed *** |
| $G_0$ | Initial shear modulus (MPa) | Variable according to $p'$ | 0.85 | - | $G_0=3(1-2\nu_0)K_{tan}/(2(1+\nu_0))$ |
| $\alpha$ | Constant | - | 1000 | - | Yamazoe et al. (2014) |

$C_c$: Compression index, $C_k$: Permeability coefficient index, $L_i$: Ignition Loss, $K_{tan}$: Tangential bulk modulus

*From Hayashi et al. (2008)  **For parameters of sand, general values were assumed.

*** Typical values for clays at small strains, around 0 (Nishimura, 2014), were tentatively adopted for peat here.
consolidation, $c_v$, of the peat ground was large at initial low confining pressures, as shown in Fig.3(b). However, it significantly decreases when the consolidation pressure increases. Therefore, this characteristic was reflected in the model by the following equation (Yamazoe et al., 2011), where $\gamma_w$ is the unit weight of water and $p'$ the mean effective stress. See Table 3 for the meaning of the other constants.

$$c_v = \frac{k_v \exp \left( \frac{2 - \ln \frac{p'}{\gamma_w}}{\alpha} \right) p'}{1 + e_0}(1 + e_0)$$ (1)

The volumetric strain - time curve from one step (the effective vertical stress of 6 to 12kPa) in the step loading consolidation tests and that of the FE simulation using the mesh illustrated in Fig.4(c) are shown in Fig.5. From Fig.5, the settlement rate is overestimated if $c_v$ is set constant, but approached the observed rate if its decrease at higher stress is considered.

Typically though not necessarily, the shear modulus, G, in the Cam-Clay model is set proportional to the effective confining pressure in a same way as the bulk modulus, K. This formulation, named “A” tentatively in this paper, provides extremely small G for low confining stresses as encountered near the ground surface. It is therefore difficult to evaluate properly the immediate shear deformation near the ground surface. In this study, the formulation allowing initially fixed values of G, as proposed by Yamazoe et al. (2014) and named “B”, was used. This model assumes two regions, as shown in Fig.6. The tangential shear modulus, $G_{\text{tan}}$, becomes variable proportionally to the bulk modulus via a fixed value of Poisson’s ratio, $\nu_f$, in the large strain range, just as assumed in formulation A. However, in the small to middle strain range, the $G_{\text{tan}}$ value are set as a function only of the strain level. Its variation is represented by the following equation.

$$G_{\text{tan}} = \frac{1}{1 + \nu_f^2} \sqrt{\frac{p'}{p_0}}$$ (2)

Here, $G_0$ is the initial shear modulus, $\alpha$ is a constant (Mirjalili et al., 2011) and $\nu$ is the shear strain ($\nu = (2e_{12}e_{12}/3)^{1/2}$). The deviatoric strain tensor, $\varepsilon_{ij}$, is defined from the strain tensor, $e_{ij}$, as $e_{ij} = e_{ij} - e_{kk}\delta_{ij}/3$, where $\delta_{ij}$ is Kronecker’s delta ($i = j : \delta_{ij} = 1$, $i \neq j : \delta_{ij} = 0$).

4 OBSERVED AND SIMULATED PIPELINE SETTLEMENTS

The observed and simulated settlement of the backfill bottom is shown in Fig.7. About 0.2m of settlement was observed by Day 5,000. In this figure, the time scale is magnified to the prototype scale according to the consolidation scaling law shown in Table 2. Therefore, the settlement curve in the model time scale is shifted in parallel to the (log) time axis as illustrated as curve (a) in Fig.8. This is a rational scaling if the effective stress – strain relationship is independent of the strain rate. However, if the settlement curve obeys an isotache-type law, as suggested by Tanaka and Hayashi (2014) for peats, the secondary consolidation behavior should be independent of the scale and the above $N^2$ scaling rule will not hold. In such a case, a real prototype behavior will be represented by curve (b) shown in Fig.8. Matsuo et al. (1992) stated that both results could be experimentally observed in 1-D consolidation tests of peats. If the tested peat possesses an isotache property, larger settlement will occur in real ground. Therefore, the 0.2m settlement observed in the model test should be considered as a lower bound value to the actually anticipated settlement for a peat of $w \approx 600\%$.

From the comparison of the test and FE analysis result, a difference appears even at the backfilling stage. It is inferred that this is caused by difficulty in executing perfectly constant-rate filling in the experiment, as assumed in the analysis. However, the quantity of observed settlement and settlement rate of primary consolidation is generally reproduced in Case 1. The analysis overpredicted the primary consolidation in Case 2. It turned out later that the model ground produced for Case 2 had lower water content (around 530%). The identical sets of model parameters used for both cases did not take account of the larger degree of overconsolidation in Case 2. There was no significant difference in the simulated subsidence rate after the end of backfilling between formulation A and B. However,
the amount of the initial immediate settlement due to backfilling was affected by the difference in the initial values of \( G \) in these two different approaches. The analysis results suggest that the primary consolidation was completed by around Day 100-200 in both cases 1 and 2, although this was not clear in the experiments. The reason for apparently accelerating subsidence in the experiments after this point in the experiments is not clear at the moment. The time-independent model adopted in this study was not capable of simulating the subsequent secondary consolidation observed in the model test. Incorporating the time-dependent formulation, possibly based on the isotache theory (e.g. Leroueil, 1996 and 2006, Watabe et al., 2012), is the next step in future, as Tanaka and Hayashi (2014) suggested the theory’s applicability to peat ground.

Fig. 9 shows the displacement vectors and the shear strain distributions on Day 100, computed by PIV analysis for the experiment. The experimental results show that the deformation is concentrated around the

\[
\text{Volume strain} = \frac{\Delta V}{V_0} \times 100\%
\]

\[
\sigma - \epsilon = \left( \epsilon_x - \epsilon_y \right)^2 - \gamma_{xy}^2 \right)^{0.5}
\]

(a) Case 1 (Model test) (b) Case 1 (FEM)* (c) Case 2 (Model test) (d) Case 2 (FEM)*

Fig. 9. Vector diagrams of deformation and distributions of shear strain (Prototype scale, around Day 100). *The results shown are based on formulation B
backfill without clear occurrence of shear localization and the region of influence is fairly limited. On the other hand, the analysis predicted relatively large displacement in the far field, with particularly pronounced lateral spreading. The authors’ ongoing triaxial tests suggest that the peat is likely to possess peculiar plastic potential which leads to limited lateral bulging against vertical loading. Predicting the ground deformation mode will require further elaboration of the model.

5 CONCLUSIONS

The mechanisms behind the subsidence of a backfilled pipeline in peat ground were studied by performing long-term model tests, 1-D consolidation tests and soil-water-coupled finite element analysis focusing on the consolidation of a peat layer due to the backfill’s weight. The conclusions drawn from this study are as follows.

1) About 0.2m settlement was observed by a sand backfill of typical cross-section in peat ground of some 600% water content. Allowing for potentially physical scale-independent nature of secondary consolidation, even larger settlement is likely. Of the actually observed settlement of up to 0.8m in Sapporo city, therefore, 30-40% is easily explainable by the consolidation due to the backfill weight.

2) The very low stress level involved in the problem requires a careful consideration into determination of elastic parameters in the adopted Cam Clay-type model. Comparison of the simulation based on two different formulations confirmed robustness of the primary consolidation prediction.

3) The backfill subsidence can be simulated with accuracy satisfactory for practical purposes until the end of primary consolidation by considering realistic decrease of the coefficient of consolidation at increased stress levels. The future work must address describing the secondary consolidation behavior by a time-dependent constitutive model.

4) The ground deformation associated with the backfill subsidence was seen only in the backfill’s vicinity in the model tests. On the other hand, in the FE analysis using a Modified Cam Clay-type model, the ground deformation propagated more extensively. Completely describing the peculiar characteristics of peats’ deformation requires further studies.

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