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# Unloading behavior of clays measured by CRS test

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**Abstract**

The unloading behavior of clays was studied by the Constant Rate of Strain (CRS) test, for three clays: two of them are reconstituted and the other was intact. In the conventional CRS test where the stress monotonically increases, the distribution of the pore water pressure in a specimen is assumed to be parabolic, the effective stress is calculated and then the compression behavior is evaluated. However, this assumption cannot be directly applied the unloading condition. In this study, the pore pressure distribution under unloading was simulated by a cubic polynomial under the assumption that hydraulic conductivity does not change in the unloading process. A unique relation in the $e$–log $\sigma'_v$ relation was found, irrespective of both the magnitude of stress or strain and the compression index, $C_c$, at the unloading test, when the consolidation pressure is normalized by $\sigma'_{vmax}$, which is the consolidation pressure before the unloading test. In addition, the creep strain, which is gained by constant loading before the unloading test, was shown to have a great effect on the unloading behavior: that is, the soil behaves stiffly when subjected to a constant load for a prolonged period of time. A strain rate dependency in the unloading process was also observed particularly for heavily unloaded specimens. The unloading behavior was also investigated by the conventional constant load test. The test results show reasonable agreement with those obtained from the CRS test.


*Keywords:* Unloading; One-dimensional consolidation; Creep; Strain rate effect; IGC: D5

## 1. Introduction

The swelling index ($C_s$) is a key parameter in geotechnical engineering, as well as the compression index ($C_c$). $C_s$ may be defined as the slope of the relation between the void ratio ($e$) and the consolidation stress ($\sigma_v$) in the common logarithm scale (in the Cam Clay model, $\kappa$ is used in the natural logarithm scale, ln $\sigma_v$). Characteristics of $C_s$ or $\kappa$ have been investigated by many researchers and used for the prediction of shear strength (see for example, Mitachi and Kitago, 1976; Ohta et al., 1985; Wood, 1990) or shear modulus (for example, Kawaguchi and Tanaka, 2008). $C_s$ is usually measured by a constant loading oedometer test or triaxial tests, where applied stresses decrease step by step, and the negative excess pore water pressure caused by unloading is dissipated through drainage boundaries.

Unlike conventional tests for obtaining $C_c$, i.e., increasing loading stress, the testing method for $C_s$ is not strictly defined. For example, various load removal decrement ratios have been adopted and in an extreme case as the previous Japanese

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Table 1
Geotechnical properties for tested soils in this study.

| Sample | $\rho_s$ (g/cm$^3$) | $w_n$ (%) | $w_L$ (%) | $w_P$ (%) | $I_p$ | $c_v$ (cm$^2$/d) | $k$ (m/s) | Remarks |
|---|---|---|---|---|---|---|---|---|
| Kasaoka | 2.610 | 45.8 | 62 | 36 | 26 | 39 | $7\times10^{-11}$ | Recons. |
| Louiseville | 2.769 | 69.4 | 74 | 23 | 51 | 110 | $3\times10^{-10}$ | Intact |
| Ma13 | 2.693 | 73.6 | 91 | 38 | 53 | 83 | $3\times10^{-10}$ | Recons. |

Industrial Standards (JIS) for incremental loading test (JIS A 1217:1990), $C_s$ was obtained from the change in the void ratio from the final to the first loading stage. In this standard, with eight loading stages and a load increment ratio of one, the decrement ratio for $C_s$ can be as small as 0.0078. This specification was, however, eliminated in the present JIS (JIS A 1217:2009) on the basis that $C_s$ is strongly affected by the decrement ratio. This point will be discussed in this paper. A duration of 24 h is usually adopted for the constant loading test although there is no theoretical reason for doing so. This is probably because it is convenient for those who conduct the experiments. A reason for employing such loosely defined testing methods may be that it is commonly believed that soil under unloading behaves elastically and only the dissipation of the negative pore water pressure should be considered. However, Mesri et al. (1978) studied unloading behavior using the constant loading oedometer and triaxial equipments, and revealed that $C_s$ is not a constant parameter, but is rather strongly influenced by the stress levels and also that the effect of creep (secondary compression) is significant at large stress removals.

The Constant Rate of Strain (CRS) oedometer provides constant strains to a specimen, instead of the constant loads in the conventional oedometer test, and has gradually come into use as a routine test to measure the compression behavior of soft clays. One of the starting points for the wide use of CRS in Japan may have been the investigation of compressibility for Pleistocene clay layers for the construction of the Kansai International Airport (KIA). Due to the large burden pressure, the yield consolidation stress ($\sigma_{vy}$) for these Pleistocene clay layers cannot be accurately measured by the conventional oedometer test, where the increment of the magnitude of load in each step is the same as at the previous loading step. In CRS test, however, the $e$–log $\sigma_v$ relation can be almost continuously measured so that the $\sigma_{vy}$ value can be clearly defined. Another advantage in the CRS test is that the rate effect on the $e$–log $\sigma_v$ relation can be considered directly. Since the final consolidation stress caused by the weight of the manmade island in the Pleistocene clay layers at KIA is very close to the $\sigma_{vy}$ value, the rate effect greatly influences the prediction of the settlement (for example, see Tanaka, 2005; Watabe et al., 2008). On the other hand, due to the small amount of displacement in the unloading process, it was considered that the CRS test is not suitable for obtaining unloading behavior. Tsutsumi and Tanaka (2011) have developed a new loading apparatus for the CRS test, which provides precise displacements using a step motor controlled by a computer. This paper will present the unloading behavior measured by this CRS test.

## 2. Tested samples and testing method

### 2.1. Soil sample

Three kinds of soil samples were prepared in this study, as indicated in Table 1. Two of the samples (Kasaoka and Ma13) were reconstituted, and the Louiseville clay was an intact sample. Kasaoka clay is commercially available powder clay. Ma13 clay is a natural Holocene clay collected from Osaka bay. These clay samples were thoroughly mixed with water at about 2 times their liquid limit ($w_L$) and consolidated in a cell under 100 kPa consolidation pressure. After consolidation was completed, the samples were extruded from the cell and were wrapped with plastic film and covered by wax. Then the samples were stored in a box under high humidity and constant temperature until testing. The natural water content ($w_n$) in Table 1 is the water content after extrusion from the consolidation cell. Louiseville clay samples were retrieved at the site of Louiseville, Quebec, Canada, following the Japanese standard sampling method using the Japanese standard sampler. Although the main properties of this clay have been reported by Leroueil (1997) and Tanaka et al. (2001), the most prominent consolidation characteristic is non-linearity in the $e$–log $\sigma_v$ relation, as shown in Fig. 1. A sharp reduction in the $e$–log $\sigma_v$ relation after $\sigma_{vy}$ is observed and its gradient gradually becomes moderate as $\sigma_v$ increases. Because of reconstituted samples of Kasaoka and Ma 13 clay, the $e$–log $\sigma_v$ relation is almost linear in the normally consolidated (NC) state, as shown in Fig. 1. The most significant difference between these reconstituted samples is their permeability, as shown in Table 1, where the coefficient of consolidation ($c_v$) and the hydraulic conductivity ($k$) were measured at $\sigma'_v$ = about 300 kPa.

### 2.2. Testing method

The testing apparatus used in this study is the same as that used in previous research (Tsutsumi and Tanaka, 2011), except for a gap sensor to make precise measurements of displacement. The capacity of this gap sensor is 2 mm, and its resolution is 0.0001 mm but considering the noise yielded in the recoding and amplifier systems, accuracy is reliable to about 0.0005 mm. The specimen was 60 mm in diameter and 20 mm in initial height. The upper side of the specimen was kept drained and the pore water pressure ($u$) was measured at the bottom ($u_b$). A back pressure of 100 kPa was applied to allow for precise measurements of the pore water pressure.