Effect of OCR on sampling disturbance of cohesive soils and evaluation of laboratory reconsolidation procedures

Marika Santagata and John T. Germaine

Abstract: The paper presents the results of an experimental investigation of sampling disturbance in cohesive soils through single-element triaxial tests on resedimented Boston blue clay (RBBC). The first part of the paper discusses the effect of the overconsolidation ratio (OCR) (1−8) of the soil on postdisturbance compression and undrained shear behavior. The results demonstrate that sensitivity to disturbance decreases markedly with OCR. It is also found that for the medium-sensitivity soil tested, the estimate of the preconsolidation pressure is not significantly affected by OCR. The second part of the paper discusses laboratory reconsolidation procedures. For OCR1 RBBC, the recompression method is not effective in recovering the stress–strain behavior of the soil and, for greater disturbance, provides an increasingly unsafe estimate of the strength. For OCR4, provided the reconsolidation path reproduces the path that occurred in the field, this procedure succeeds in recovering the intact stress–strain–strength behavior of the soil. SHANSEP reconsolidation was investigated for normally consolidated RBBC only. For modest levels of disturbance, this is an effective means of evaluating both the stress–strain and the strength behavior of the soil. For greater levels of disturbance, the stress–strain behavior is not fully recovered, but the method continues to provide conservative estimates of the unstrained strength.

Key words: sampling disturbance, clays, overconsolidation ratio, undrained strength, recompression, SHANSEP.

Introduction

Geotechnical engineers have long recognized that sampling techniques and procedures have a significant impact on soil properties measured in the laboratory. As a result, much work has been performed to understand the effects of the sampling process on the engineering behavior of soils and to develop improved techniques for laboratory reconsolidation. Research in this field finds its roots in the early work by Hvorslev (1949) and the fundamental research conducted in the 1960s at Imperial College (Skempton and Sowa 1963), the Massachusetts Institute of Technology (MIT) (e.g., Ladd and Lambe 1963), and the University of California at Berkeley (e.g., Noorany and Seed 1965). Over the past 15 years,
extensive work has continued to be conducted on this topic, a great part of which (Clayton et al. 1992; Georgiannou and Hight 1994; Hird and Hajj 1995; Siddique et al. 1999; Santagata and Germaine 2002) has been founded on the analytical modeling of the tube sampling process and the concept of ideal sampling introduced by Baligh et al. (1987).

At MIT, an extensive research program was undertaken to isolate the fundamental aspects of sampling disturbance of cohesive soils through triaxial element tests (Santagata 1994) and laboratory model tests (Sinfield 1994). The triaxial test experimental program included a large number of tests on both normally consolidated (NC) and overconsolidated (OC) resedimented Boston blue clay (RBBC), in which sampling disturbance was simulated in accordance with the perfect sampling approach (PSA) (Ladd and Lambe 1963) and the ideal sampling approach (ISA) (Baligh et al. 1987) to model block and tube sampling, respectively. The effects of disturbance on compression and undrained shear behavior were then quantified by comparison with the intact behavior of the soil. This work was intended to provide a framework to (i) quantify the effects of tube penetration and stress release; (ii) determine which engineering parameters are most affected by disturbance; (iii) assess the sensitivity of cohesive soils to disturbance as a function of the overconsolidation ratio (OCR); and (iv) evaluate two widely used recompression procedures. The first two objectives were addressed for OCR1 by Santagata and Germaine (2002), who presented and analyzed in detail the data for NC RBBC.

This paper presents results for OC RBBC with OCR ranging between 2 and 8 and compares them with those obtained for the NC soil. Specifically, the paper highlights the sensitivity to disturbance of various engineering properties (preconsolidation pressure, compressibility parameters, undrained strength, strain at failure, etc.) as a function of the OCR.

The second part of the paper evaluates procedures to recover the intact behavior of the soil in the laboratory. In particular, the paper presents additional tests performed to evaluate the validity of the recompression (Bjerrum 1973) and the SHANSEP (Ladd and Fookt 1974) procedures for assessing the undrained strength and undrained stress–strain behavior of cohesive soils in the laboratory. Results of tests performed on RBBC with an OCR of 1 (SHANSEP and recompression tests) and an OCR of 4 (recompression tests only) are presented, and the limitations of the two recompression procedures are discussed.

Supporting technology

Resedimented Boston Blue Clay

RBBC is a soil manufactured in the laboratory from natural Boston blue clay, an illitic low plasticity (CL) clay present throughout the Boston area. The resedimentation process involves mixing under vacuum the soil powder obtained by drying and processing the natural soil with de-aired water to produce a slurry with a water content of 100%. A bacterial growth inhibitor (phenol) and salt (16 g/L required to produce flocculation and achieve a structure similar to that of the natural soil) are also added. The slurry is sprayed under vacuum into a large consolidometer ($D = 30.5$ cm), in which, in a process that lasts about 19 days, it is incrementally consolidated to a maximum stress of 100 kPa and then unloaded to 25 kPa to produce an OCR of 4. In this close-to-hydrostatic condition, the soil cake is removed from the consolidometer and prepared for storage. Triaxial specimens ($H = 8$ cm, $D = 3.6$ cm) are prepared from this block by means of a wire saw and a miter box.

RBBC is uniform and saturated, with a well-known stress history, and is available in a virtually infinite supply, requiring moderate incremental work to produce additional batches. In addition, there exists an extensive database of its index and engineering properties. Finally, and most importantly, although RBBC is not a natural soil, it exhibits behavior similar to that of the natural origin material and of other low-sensitivity marine clays, including stress–strain–strength anisotropy, low to medium sensitivity, and significant strain rate dependency (e.g., see Sheahan 1991; Santagata 1998). The structure of RBBC, evaluated on the basis of the position of the sedimentation compression curve (Skempton 1970; Coteccchia and Chandler 2000), is also consistent with that of natural soils (with sensitivity as great as 10) characterized by a “sedimentation structure”, that is, in which chemical or aging processes (postsedimentation processes, according to Coteccchia and Chandler 2000) have not played a significant role.

For all these reasons, RBBC is an ideal material for the investigation of fundamental aspects of soil behavior without the variability of natural soils having to be taken into account. The average index properties for the five RBBC batches used to perform the tests presented in this paper are summarized in Table 1.

Triaxial apparatus

Figure 1 shows a schematic of the computer-controlled stress-path triaxial testing apparatus used in the MIT geotechnical laboratory for testing cohesive soils (Sheahan and Germaine 1992). The apparatus includes six basic components:

(i) the triaxial chamber, characterized by internal posts, a fixed top cap, a loading piston riding through a low-friction linear bearing, a rolling diaphragm seal, and silicon oil as chamber fluid;

(ii) the system for load application, comprising the pressure–volume controllers (PVCs) used to regulate back and cell pressure, and the 9 kN capacity bench-top Wykeham Farrance (Slough, UK) screw-driven load frame with adjustable gear ratios used for axial loading;

(iii) the motors driving the two PVCs and the load frame, the motor controllers, and drivers.

<table>
<thead>
<tr>
<th>Table 1. Selected index properties of RBBC (series III).</th>
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<tr>
<td>Natural water content, $w_N$ (%)</td>
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<td>Liquid limit, $w_L$ (%)</td>
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<td>Plastic limit, $w_p$ (%)</td>
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<td>Plasticity index, $I_p$ (%)</td>
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<td>% passing No. 200 sieve</td>
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<td>Clay fraction</td>
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(iv) the instrumentation package, including a power supply; a shear beam load cell, which measures the axial load; two high-performance diaphragm-type pressure transducers mounted at the base of the cell to monitor cell and back (pore water) pressures; and two linear variable differential transformers (LVDTs) mounted on the loading piston and the back-pressure PVC piston to measure the external axial strain and the volumetric strain, respectively;

(v) the personal computer (PC) responsible for automated control during all phases of the test (initial pressure up, saturation, consolidation along any stress path or $K_0$ consolidation, and shear in extension or in compression); and

(vi) the central data acquisition system (not shown in the figure), which is based on a PC interfaced with a Hewlett Packard (Palo Alto, Calif.) HP3497A data acquisition unit capable of analog-to-digital conversion and storage of data at a trigger rate of up to 1 Hz.

The triaxial chamber, the load frame, and the two pressure–volume controllers are housed in a temperature-regulated enclosure maintained at 25 °C ± 0.2 °C by an air-circulating unit with a heat source activated by a thermostat.

Research methodology

As mentioned above, the experimental program made use of the frameworks provided by PSA (Ladd and Lambe 1963) and ISA (Baligh et al. 1987) to simulate block and tube sampling, respectively, in the triaxial cell. The following sections provide a brief overview of the two approaches and describe the test program in detail.

Perfect and ideal sampling approaches

The concept of perfect sampling was first introduced in the 1960s by Ladd and Lambe (1963, p. 343) to denote the “[sampling] process where no disturbance has been given to the specimen other than that involved with the release of the in situ shear stress”. Although subsequent work (see below) has identified additional and more significant contributions to disturbance in the case of tube sampling, perfect sampling represents an exhaustive model of minimum sampling disturbance for the case of block sampling.

In more recent years, Baligh et al. (1987) have made use of the strain path method (Baligh 1985) to analyze soil element straining due to deep penetration of a simple sampler ($S$ sampler), which is characterized by a curved cutting edge and a slight reduction of the inner diameter (Fig. 2). Their analysis, which has had considerable impact on the study of sampling disturbance, has shown that soil elements located inside a sampling tube undergo a complex strain history, involving both shear and normal strains. Recognizing that with tube sampling the minimum unavoidable disturbance went beyond the simple release of the in situ shear stresses described by the PSA, they introduced the concept of ideal sampling. In recognition of the fact that the normal axial strain ($\varepsilon_{zz}$) is the dominant strain component in the inner half of the sample, the ISA proposes that the strain undergone by a soil element located at the centerline of the sampler be used to describe the effects of tube sampling. As shown in Fig. 2, such soil elements undergo three distinct phases of undrained triaxial shearing: a compression phase, ahead of the sampler, until $\varepsilon_{zz}(\max)$ is reached at about 0.35$B$ below the tip; an extension phase, during which $\varepsilon_{zz}$ decreases rapidly and becomes zero at the tip of the sampler and $\varepsilon_{zz}(\min)$ (equal to $-\varepsilon_{zz}(\max)$) is reached at about 0.35$B$ inside the sampler; and a last phase, in which the soil element is once again subjected to compression, until $\varepsilon_{zz}$ approaches zero. As shown in Fig. 2, the magnitude of the strains is a function of the geometry of the tube, and the straining increases with the relative thickness of the sampler.

More recently, Clayton et al. (1998) used a finite element approach to investigate the effects of specific features of the sampling tube geometry (inside clearance ratio, cutting edge taper angle, etc.) on the strain history undergone by the soil during sampling. In particular, their results show that (i) the strain path followed by a soil element located at the centerline of the sampler is not generally as symmetric as that shown in Fig. 2 (e.g., for a sampler with high inside clearance ratio, the extension strain is greater than the maximum...
compressive strain experienced by the soil ahead of the sampler); (ii) details of the sampler geometry play a significant role (e.g., the greater the taper angle of the outside cutting edge, the greater the maximum compressive strain); and (iii) the strains generated by a sharp-edge sampler are smaller than those predicted by Baligh et al. (1987) for the S sampler.

Disturbance simulation and evaluation of postdisturbance behavior

Different types of tests were performed to quantify the effects of disturbance on the compression and the undrained shear behavior of RBBC. Every test involved three or four main phases: a consolidation phase, to recover the intact behavior of the soil; a disturbance phase, to simulate either block or tube sampling; and one or two postdisturbance phases, to evaluate the effects of the previous disturbance phase on the behavior of the soil or to evaluate a reconsolidation procedure (see details below).

In the first phase, which was common to all tests, following back-pressure saturation, the specimen was \( K_0 \) consolidated well into the virgin compression line (VCL) (to three to four times the batch preconsolidation pressure of RBBC, which is equal to ~0.1 MPa; see the section on RBBC), maintained at constant stresses for 24 h to allow for some secondary compression and then, if necessary, swelled to the desired OCR. An additional 24 h period of constant stresses followed swelling. Unloading to the OC state occurred through stress path consolidation, targeting a value of \( K_0(OC) \) given by the following equation:

\[
K_0(OC) = K_0(NC)(OCR)^n\]

with average \( K_0(NC) = 0.48 \) and \( n = 0.426 (= 1 - \sin 35^\circ) \). The form of this equation (Schmidt 1966) has been found to be valid for RBBC with the values of \( K_0(NC) \) and \( n \) shown by Sheahan (1991) and Santagata (1994).

RBBC exhibits normalized behavior, and consolidation past 1.2–1.5 \( \sigma'_p \) erases the limited disturbance caused by the removal of the soil cake from the consolidometer and by the setup operations (O’Neill 1985). Therefore, at the end of the consolidation or swelling phase, the soil specimen can be considered to be in a condition similar to that of an element of NC or OC clay in the field prior to sampling, and thus its behavior will be referred to as intact.

The disturbance phase followed the secondary compression phase at the end of consolidation (tests on NC RBBC) or swelling (OC RBBC). Depending on whether the disturbance was simulated according to PSA or ISA, this phase involved either the undrained release of the consolidation shear stress (PSA tests) or the undrained compression–extension–compression strain history shown in Fig. 2, followed by undrained unloading to the hydrostatic state (ISA tests). As indicated in Table 2, the experimental program included ISA tests performed with the nominal amplitude of the strain cycle (\( \varepsilon_c \)) varying between 0.5% and 8%. This was done to correlate the effects of disturbance to the geometry of the sampling tube (e.g., \( \varepsilon_c \approx 1\% \text{ and } 2\% \text{ for } B/t \approx 40 \text{ and } 20, \text{ respectively} \) ) and to gain insight into the degradation of the engineering properties for soil elements not located along the centerline of the tube. These elements, despite not being sheared in a purely triaxial mode during penetration of the sampler, are subjected to significantly higher vertical strains (Baligh et al. 1987). As discussed above, recent work by Clayton et al. (1998) has shown that depending on the specific cutting edge geometry of the sampler, the strain path followed by an element located at the centerline of the tube during sampling may not be symmetric (i.e., \( \varepsilon_{\text{max}} \) in compression not equal to \( \varepsilon_{\text{max}} \) in extension). In this research, however, simulation of the centerline sampling disturbance straining followed the framework provided by Baligh et al. (1987). Although it is recognized that if the presence of a sharp cutting edge is neglected the disturbance undergone by the soil may be somewhat overestimated, this approach limited the number of tests performed. In addition, model testing of the effects of tube sampler geometry performed at MIT by Sinfield (1994) shows that for a given sampler

**Fig. 2.** Strain path results for straining history of soil elements along centerline of S samplers, with \( B/t = 20, 40, \text{ and } 50 \) (modified from Baligh et al. 1987). \( B/t \), aspect ratio; ICR, inside clearance ratio; \( B_e \), minimum diameter.
geometry, the effects of sampling (as measured, for example, by the reduction in effective stresses and the decrease in undrained strength) are substantially greater than those predicted by the triaxial element tests with strain cycle amplitude corresponding to that tube geometry. This is also the reason behind the large range in amplitude of the strain cycle explored in the testing program.

The last portion of the test differed, depending on whether the objective was (i) to evaluate the effects of the previous disturbance phase on the undrained shear; (ii) to evaluate the effects of disturbance on the compression behavior; (iii) to evaluate the SHANSEP reconsolidation procedure; or (iv) to evaluate the recompression reconsolidation procedure. In the first case, the specimen was sheared undrained (postdisturbance unconsolidated undrained (UU) tests with measurement of the excess pore pressure), whereas in the second case, the soil was $K_0$ consolidated to at least 0.9 MPa (postdisturbance $K_0$-consolidated tests). In the case of the postdisturbance SHANSEP tests, the soil specimens were consolidated under $K_0$ conditions to at least 0.9 MPa, allowed to creep for 24 h, and then sheared in undrained compression (SHANSEP tests were performed only for NC RBBC). Note that these tests are also used to study the postdisturbance $K_0$-consolidation behavior. Finally, in the last case (postdisturbance recompression tests), following disturbance the soil specimens were reconsolidated to the predisturbance effective stresses ($\sigma'_v$ and $\sigma'_h$), allowed to creep for 24 h, and then sheared undrained to failure.

Figures 3–5 present examples of the complete stress paths and $e$–log $\sigma'_v$ curves for the tests, and Table 2 summarizes the experimental program. All consolidation and undrained shear phases were conducted with axial strain rates of 0.1%/h and 0.5%–1%/h, respectively.

### Behavior of intact RBBC

Definition of the intact behavior of RBBC was based on results from this testing program, as well as data from other investigations. Specifically, the one-dimensional compression behavior was characterized from data from the first consolidation phase of tests presented here, as well as from constant rate of strain (CRS) tests conducted to stress levels as great as 0.8 MPa by Cauble (1993). For NC RBBC, the disturbance phase (with the exception of the PSA tests) provides most of the data for the undrained shear behavior, as the soil fails at a strain of 0.1%–0.2%. In the case of OC RBBC, however, the strain imposed during the first compression phase of the ISA disturbance cycle is almost always not large enough to cause failure of the OC soil. Thus, the intact undrained shear reference behavior was derived from the database provided by Sheahan (1991).

### Compression behavior

Figure 6 presents a set of curves derived from the constant rate of consolidation portion of direct simple shear tests performed by Cauble (1993) on soil obtained from one of the batches used in this testing program. The curves, which extend to a vertical stress of about 0.8 MPa, define the typical one dimensional compression behavior of RBBC.
pression curves show a well-defined break corresponding to the batch preconsolidation pressure and the approximately constant slope of the VCL over the entire stress range investigated (for some batches, a modest reduction in compression ratio, CR (= ∆ε/∆log σ’v), of about 7%–9% is observed in the same stress range). Similar behavior is observed in the one-dimensional consolidation phases of the triaxial tests performed for this testing program for all batches of RBBC, despite some limited batch-to-batch variability, and in general the compression behavior is quite repeatable.

As mentioned above, the coefficient of earth pressure at rest, K0, increases with increasing OCR following the empirical equation of the type proposed by Schmidt (1966): K0 = K0(NC) (OCR)n, with K0(NC) = 0.48 and n = 0.426 (Sheahan 1991; Santagata 1998).

Undrained shear behavior

Figures 7a and 7b illustrate the typical undrained triaxial compression behavior of RBBC (OCR1–8) sheared at a rate of 0.5%/h. The figures show that NC RBBC reaches peak strength at a relatively small axial strain (~0.1%), followed by significant softening; and because of the development of large positive pore pressures, p’ decreases markedly. Increasing OCR causes a decrease in the peak value of the strength normalized to the maximum vertical effective stress (σ’vmax), an increase in the axial strain at failure (εp ~ 1%, 3.5%, and 6% for OCR = 2, 4, and 8), and a reduction in strain softening. At larger strains, the effective stress paths approach a common failure envelope. Associated with increasing OCR is also a decrease of the shear-induced (u_s = ∆u - ∆σ_oct) pore pressure beyond 0.5%–1% axial strain, as well as a reduced development of excess pore pressure (u_e), which causes the transition from a fully contractive (OCR1) to an entirely dilatant (OCR8) undrained shear behavior.

For all OCRs the soil is observed to exhibit normalized behavior, and the following SHANSEP parameters apply for

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undrained triaxial compression (de/dt = 0.5%/h): S = 0.33; and m = 0.7 (q_f/σ’_{vc} = S(OCR)^m). The undrained strength ratio of NC RBBC is observed to decrease with increasing value of the lateral stress ratio, K, in a manner described by the following relationship: q_f/σ’_{vc} = 0.54–0.45K (r^2 = 0.79) (Fig. 8).

Effect of OCR on postdisturbance behavior

Preshear stress state

Figures 9a–9f present the stress–strain curves, stress paths, and curves of excess and shear induced pore pressure for the ISA ± 2 disturbance phase of tests conducted on NC and OCR4 RBBC. It is shown that for NC RBBC, basically all phases of sampling generate positive pore pressure, with the first two phases playing the major role. This results in a continuous decrease of the effective stresses (note that for lower disturbance, the second compression phase actually results in a small increase in the stresses; Santagata and Germaine 2002).

The major difference with OC RBBC is that the soil dilates during the initial compression to ε_a = ε_{max} with consequent increase of the effective stresses. The positive pore pressure produced during the subsequent extension phase results in an overall reduction of the effective stresses, which is more marked for decreasing values of the OCR. The last two phases (second compression phase and shear stress release) produce a further, less significant decrease of the stresses.

Figure 10 presents the change in mean effective stress (p’_{m} = (σ’_{vmax} + 2σ’_{vc})/3) normalized by the value at the end of consolidation (i.e., predisturbance value), p’_{m0}, caused by disturbance versus the strain cycle amplitude, ε_c. For all OCRs, the effective stress decrease is more marked as the amplitude of the strain cycle increases. Note that in this figure as throughout the paper, the results for the PSA tests are plotted at ε_c equal to zero. There is consistency in the results for each level of straining and OCR, as well as significantly greater effective stress loss with decreasing OCR, for a given level of disturbance. With the exception of the PSA tests, the changes in p’_{m} are substantial even for the higher OCRs.

Just as the stress change associated with swelling is quantified through the OCR, the undrained decrease in effective stress caused by sampling disturbance can be expressed through a similar parameter, the apparent overconsolidation ratio (AOCR), first introduced by Ladd and Lambe (1963) and here redefined as the ratio between σ’_{vmax}, the maximum vertical consolidation stress, and σ’_{vc}, the effective stress at the end of sampling. For OC RBBC, it is useful to distinguish between the stress decrease associated with the swelling phase (σ’_{vmax} to σ’_{vc}) and that caused by sampling (σ’_{vc} to σ’_{s}). This second contribution can be quantified through the induced overconsolidation ratio (IOCR) (= σ’_{vc}/σ’_{s}; Santagata and Germaine 2002). From the definitions above, it is clear that IOCR = AOCR/OCR and that for NC clay, IOCR = AOCR. In addition, for intact soils, IOCR = 1, and the AOCR is equal to the actual OCR.

Compression behavior

Compression curve and compressibility parameters

Figures 11a–11c compare the intact and post-ISA ± 2 disturbance compression curves for NC and OC RBBC with OCR equal to approximately 2 and 4. Each figure presents the regression line through the steepest portion of the compression phase (with slope C_{c1} = ∆ε/∆logσ’), which defines the intact VCL, as well as the line (with slope indicated as C_{c2}) through the postdisturbance compression curve at σ’_{c} = 3σ’_{c0} (slightly lower value in the case of the OCR2 test). The figure indicates a less significant difference between C_{c1} and
Thus, a reduced sensitivity to disturbance, for increasing values of the OCR. This is clearly shown in Fig. 12, in which the slope of the compression curve ($\Delta e/\Delta \log \sigma_v'$) is normalized by $C_{c1}$ and plotted versus $\sigma_v'/\sigma_{v\max}'$, the current effective stress normalized by the maximum vertical effective stress. The figure demonstrates that in the OCR4 test (and possibly also for OCR2 RBBC) reconsolidation to $3\sigma_p'$ is effective in recovering the intact compression curve (some difference between $C_{c1}$ and $C_{c2}$ may derive from stress level effects). This is not the case for the NC soil. More substantial differences between the two slopes are observed for higher degrees of disturbance (Santagata and Germaine 2002).

Figure 12 also indicates no clear effect of the OCR on the recompression portion of the disturbed compression curve, as for $\sigma_v'/\sigma_p' < 1$ the three curves are basically overlapping; that is, for a given magnitude of disturbance, the recompression behavior of the soil is affected in a similar manner, independently of the OCR. This can be explained by observing the limited variation in the AOCR at the end of disturbance for the three tests (AOCR is equal to 10.3, 7.1, and 11.7 for OCRs of 1, 2, and 4, respectively). In fact, the values of the AOCR for the two OC tests fall within the range of data for NC RBBC subjected to ISA ± 2 disturbance (6.9–12.1). This indicates that for this level of disturbance (ISA ± 2), possibly because of the limited structure of RBBC, disturbance does not affect the mechanisms of deformation in the recompression region.

**Reconsolidation strains**

Figure 13 plots the volumetric strains measured during reconsolidation of the specimen to the in situ (i.e., pre-
The data are consistent and demonstrate that for a given OCR, the volumetric strain associated with reconsolidation to the in situ stresses is closely related to the degree of disturbance undergone during sampling, as measured by the strain cycle amplitude. For any level of disturbance, the reconsolidation strain decreases with increasing OCR. As a result, any evaluation of the quality of a soil sample as a function of the reconsolidation strain measured in the laboratory must be a function of the OCR of the soil. This is recognized, for example, by Lunne et al. (1997), who proposed two separate criteria for evaluating sample disturbance in soft, low-plasticity soils, one for OCR1–2 and one for OCR2–4. For example, in the case of RBBC, the criterion for an excellent sample proposed by Lunne et al. (1997) (which is expressed in terms of \( \Delta e/e_0 \)) would correspond to a measured volumetric reconsolidation strain smaller than 2% for OCR1–2 and less than approximately 1.5% for OCR2–4.

**Preconsolidation pressure**

For NC RBBC, the effects of disturbance on the estimation of the known in situ preconsolidation pressure were evaluated by analyzing the one-dimensional compression beh-
behavior of the soil following ISA ± 1, ISA ± 2, and ISA ± 5 disturbance. The main conclusions from these tests, extensively discussed by Santagata and Germaine (2002), can be summarized as follows:

(i) The uncertainty in the estimate of $\sigma'_p$ increases with disturbance.

(ii) Best estimates of $\sigma'_p$ have an accuracy of 10% ± 5%.

(iii) The reloading behavior of the soil after disturbance appears to be concurrently influenced by the decrease in effective stresses and the destructuring produced by disturbance, which would cause the preconsolidation pressure to be overestimated and underestimated, respectively. Which of these two mechanisms prevails depends on the type of soil and the amount of disturbance.

(iv) For a low- to medium-sensitivity soil such as RBBC and the levels of disturbance considered, the first mechanism appears to control the behavior of the soil; as a result, disturbance may lead to an overestimate the preconsolidation pressure.

As illustrated in Fig. 11, ISA ± 2 disturbance tests were conducted on RBBC with nominal OCR of 1, 2, and 4 to evaluate the effect of the OCR. Table 3 summarizes some relevant data for these tests. Comparison of these data sheds further light on the observations listed above. First, the impact of disturbance on the determination of the preconsolidation pressure does not appear to be affected by the OCR of the soil. This is highlighted by the very consistent values of $(\sigma'_p^{\max} - \sigma'_p^{\min})/\sigma'_p$, which quantifies the uncertainty associated with the estimate of the preconsolidation pressure.

Second, with the exception of the $\sigma'_p^{\min}$ data, the results indicate that disturbance leads to overestimation of the preconsolidation stress. These two observations suggest that the behavior of the soil is indeed controlled by the loss in effective stress associated with disturbance. In fact, as discussed above, for all three tests the AOCR is approximately the same. This is consistent with the results for the compressibility data in the recompression portion of the curve. Although the varying effect of disturbance on the slope of the VCL should be reflected at least in the values of $\sigma'_p^{\min}$ (the determination of which is independent of the initial portion of the curve), comparison of the results for OCR1 and OCR4 (these two tests are characterized by the same value of $\sigma'_p$ and were conducted on soil from the same batch, and reconsolidation was extended to $3\sigma'_p$) suggests that for RBBC, this effect may be secondary. More data are needed before a conclusive statement on this point can be made.

Third, the data confirm that for the level of disturbance investigated, it is possible (particularly in the case of a continuous compression curve, such as that provided by CRS consolidation) to estimate the preconsolidation pressure with considerable confidence (the $\sigma'_p^{\text{be}}$ data are within 10% ± 5% of the true value).

Undrained shear behavior

The UU postdisturbance tests described above (see Fig. 3) were used to investigate the effects of PSA and ISA disturbance on the undrained triaxial compression behavior of RBBC.

Figures 14a and 14b show postdisturbance stress–strain curves for NC and OCR4 RBBC. For NC RBBC (Fig. 14a), as already discussed by Santagata and Germaine (2002), as disturbance increases, the stress–strain curve loses all the characteristics typical of NC RBBC, and the behavior is more like that observed for intact OC RBBC (see Fig. 7). Associated with increasing levels of disturbance (PSA to ISA ± 5) are an increased reduction of the undrained strength; an increase in the strain at peak shear stress; a loss of strain softening; and an increased reduction of the soil stiffness. The stress paths presented by Santagata and Germaine (2002) for these same tests indicate a change from contractive to increasingly dilatant behavior and an increase in maximum obliquity.

Although qualitatively similar effects are observed in the case of OCR4 RBBC (Fig. 14b), the results demonstrate a significantly reduced sensitivity to disturbance. In particular, because the intact $K_0$ value for OCR4 is close to 1, PSA disturbance has no effect on the stress–strain–strength behavior of the clay, and the results for this test fall within the range for the intact material. The results of the ISA tests suggest more significant effects of disturbance on the stress–strain behavior than on the undrained strength.

Figure 15 compares the undrained strength data for NC and OC RBBC following disturbances of different magnitude (PSA tests plotted at $\varepsilon_c = 0$). The data for the intact soil are also included. The range shown reflects some inherent variability, as well as variations due to the value of the preshear $K_0$, and the results are quite consistent for each level of straining and OCR. NC RBBC shows a well-defined trend of decreasing strength with strain cycle amplitude (strength is reduced by as much as 45% in the ISA ± 5 tests). As the OCR increases, the sensitivity to disturbance decreases substantially; and for OCRs larger than 2, the undrained strength appears essentially unaffected by the disturbance cycle, at least for strain cycle amplitudes of ≤2%. This is most likely associated with the increase in the failure strain with OCR: whereas NC and OCR2 RBBC fail at approximately 0.1% and 1%, respectively, at 2% strain neither OCR4 nor OCR8 RBBC has reached the peak strength.

Figure 16 highlights the effect of disturbance on the prefailure stiffness (data below 0.1% axial strain not shown, as measurements were performed with external LVDTs). Santagata and Germaine (2002) discussed how for NC
The effects of disturbance on the prefailure behavior derived from three distinct influences: the effective stress decrease and the destructuring process associated with sampling, which both cause the stiffness to decrease; and the load reversal occurring as a result of shear, which produces an increase of the stiffness in the prefailure nonlinear region.

For NC RBBC, it was observed that for all ISA tests the first two effects dominated and, further, that destructuring played a significant role, as the decrease in the effective stress alone could not account for the reduction in stiffness. In an analysis of the OC data, the direction of the preshear stress path need not be distinguished, as both the intact and disturbed shear are associated with load reversal. It is of interest, however, in this case to understand to what degree the reduction in stiffness derives from the effective stress decrease. For the ISA ± 1 and ISA ± 2 tests the difference in stiffness can be explained simply by the reduction in effective stress caused by sampling (i.e., the stiffness is approximately the same as would be measured on a soil specimen consolidated in stiffness derives from the effective stress decrease. For the ISA ± 1 and ISA ± 2 tests the difference in stiffness can be explained simply by the reduction in effective stress caused by sampling (i.e., the stiffness is approximately the same as would be measured on a soil specimen consolidated if not disturbed).

### Table 3. Evaluation of the preconsolidation pressure.

<table>
<thead>
<tr>
<th>OCR</th>
<th>( \sigma_{p}^{\text{true}} ) (kPa)</th>
<th>( \sigma_{p}^{\text{min}} ) (kPa)</th>
<th>( \sigma_{p}^{\text{be}} ) (kPa)</th>
<th>( \sigma_{p}^{\text{max}} ) (kPa)</th>
<th>( \frac{\sigma_{p}^{\text{max}} - \sigma_{p}^{\text{min}}}{\sigma_{p}^{\text{true}}} ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OCR1</td>
<td>306</td>
<td>289 (−5.4%)</td>
<td>314 (+2.6%)</td>
<td>343 (+12.2%)</td>
<td>18</td>
</tr>
<tr>
<td>OCR2</td>
<td>430</td>
<td>432 (+0.5%)</td>
<td>471 (+9.5%)</td>
<td>515 (+19.8%)</td>
<td>19</td>
</tr>
<tr>
<td>OCR4</td>
<td>306</td>
<td>295 (−3.6%)</td>
<td>316 (+3.3%)</td>
<td>348 (+13.7%)</td>
<td>17</td>
</tr>
</tbody>
</table>

**Note:** An example of the determination of the various \( \sigma_{p} \) values is provided in Santagata and Germaine (2002).

- \( ^{a} \) Known true \( \sigma_{p} \) adjusted to account for the effects of secondary compression (Mesri and Godlewski 1977).
- \( ^{b} \) From strain energy procedure (Becker et al. 1987), assuming that the initial slope of curve is zero.
- \( ^{c} \) From strain energy procedure, enlarging the low stress portion of the curve for greater accuracy.
- \( ^{d} \) From strain energy procedure, choosing a higher initial slope, as would typically occur in the presence of only a limited number of data points (e.g., incremental oedometer results).
to $\sigma'_s$ with $OCR = AO\sigma CR$ corresponding to ISA $\pm 1$ or ISA $\pm 2$ disturbance). For the ISA $\pm 8$ test, this is not the case, and the rearrangement of the soil structure causes the additional reduction in stiffness (~50%). Note that these assessments were made using the data and the interpretative framework for the prefailure behavior of RBBC provided by Santagata (1998). These results substantiate the existence of a critical threshold strain (see, for example, Georgiannou and Hight 1994), beyond which the soil undergoes major fabric distortion (destructuring).

**Recovering the intact behavior in the laboratory**

Recompression tests  
Figures 17a–17c compare the intact undrained stress–strain behavior of NC RBBC with that observed in the postdisturbance recompression tests for three levels of disturbance (PSA, ISA $\pm 1$, and ISA $\pm 2$). Note that for the ISA tests the first portion of the disturbance cycle is used to depict the intact behavior, whereas for the PSA test the data from another test with the same $K_0$ were selected. For reference, the curves derived from the UU tests performed after the same degree of disturbance are shown. Comparison of the curves for the UU and recompression tests indicates that this reconsolidation procedure does partially recover the intact behavior of the soil. The differences between the recompressed and the intact curves are, however, still quite marked: the soil fails at a much larger strain (Fig. 18), and strain softening is essentially eliminated in the ISA tests. Most important, the results show that recompression to the in situ stresses is consistently associated with an overestimate of the undrained strength of the NC soil (Fig. 19). Although the overestimation is negligible (<1%) in the case of the PSA test, it becomes very significant in the case of the ISA tests, increasing from +8% for the ISA $\pm 1$ test to almost 20% in the ISA $\pm 2$ test. This effect is expected to be even more significant for higher degrees of disturbance.

Figure 20 presents a similar set of plots, as well as the corresponding stress paths relative to ISA $\pm 2$ disturbance of OCR4 RBBC. In this case, the first compression phase of the ISA disturbance cycle shear does not reach failure. Therefore, the results from another test that had the same OCR and that displayed similar stress–strain behavior up to 2% axial strain (see both tests plotted with continuous lines in Fig. 20a) are used to represent the intact behavior of the soil. The figure shows that in the case of OC RBBC, recompression to the presampling stresses yields a satisfactory estimate of the undrained strength of the OC soil (Fig. 19). Although the overestimation is negligible (<1%) in the case of the PSA test, it becomes very significant in the case of the ISA tests, increasing from +8% for the ISA $\pm 1$ test to almost 20% in the ISA $\pm 2$ test. This effect is expected to be even more significant for higher degrees of disturbance.

The figure also demonstrates that there remain significant differences in stress–strain behavior compared with that of the intact soil. To explain this observation, it should be noted that by reconsolidating the soil specimen to the presampling effective stress through a continuous loading path, the reconsolidation procedure did not reproduce the loading history that generated the OC conditions in the first place. During the initial consolidation phase the predisturbance field stresses were reached as a result of unloading from higher stresses, and as a result, the following shear involved a load reversal. Researchers have pointed out that the direction of the preshear stress path affects the undrained stress–strain behavior (e.g., Jardine et al. 1991), and Santagata (1998) has shown that for OC RBBC this effect is very marked. This suggests that for a realistic representation of the intact stress–strain behavior of OC soils (that have reached the current state of stress as a result of unloading), the reconsolidation process should involve reaching the target stress conditions through an unloading path. This may be
achieved by first loading the soil slightly beyond the target vertical stress and then unloading. Although such a test was not performed in this experimental program, results are available for the stress–strain behavior (0%–1%) of intact OCR4 RBBC sheared following reloading from a higher OCR (Santagata 1998), that is, with a preshear consolidation path similar to that followed in the recompression test. These data, which are shown as hollow symbols in Fig. 20a, fall exactly on the postrecompression curve. This indicates that for OCR4, at least for this level of disturbance, when differences in the preshear path are accounted for the recompression procedure is indeed effective in reconstructing the intact stress–strain–strength behavior of the soil.

**SHANSEP tests**

Figures 21a–21c compare the stress–strain curves for the undrained shear phase following SHANSEP reconsolidation after ISA ± 1, ISA ± 2, and ISA ± 5 disturbance with the curves for intact NC RBBC determined during the first portion of the disturbance cycle (compression shear to $\varepsilon_{\text{c, max}}$). Again for reference, the figures report the data from the postdisturbance UU tests. For comparing the intact behavior of RBBC (derived from tests with $\sigma'_c \sim 0.3 \text{ MPa}$) with that following SHANSEP reconsolidation (derived from tests with $\sigma'_c \sim 0.9–1 \text{ MPa}$), the effect of stress level must be taken into account. On the basis of the results of direct simple shear tests performed at stress levels of 0.1–1.2 MPa, Ahmed (1990) observed that for RBBC the effective stress level has a significant effect on the strain at peak ($\varepsilon_{\text{peak}}$ increased by about 100%–120% over the stress range examined) but negligible influence on the undrained strength. Another aspect requiring consideration is the effect of the preshear lateral stress ratio. It was discussed earlier in the paper that the undrained strength and to a lesser degree the strain at peak are influenced by the preshear value of $K_0$ (Fig. 8). Therefore, some difference between the strength and the strain at peak after the SHANSEP reconsolidation and the corresponding intact strain at peak may arise if the $K_0$ value after reconsolidation is different from the in situ value. The limited number of SHANSEP tests performed has not resolved whether, and to what degree, disturbance affects $K_0$: whereas in the ISA ± 1 and ISA ± 5 tests, the $K_0$ at the end of reconsolidation was found to be greater than the in situ value, the two values were virtually identical (0.444 vs. 0.445) in the ISA ± 2 test. As shown in Fig. 8, the results for the SHANSEP tests fall within the $K_0$–($q_f / \sigma'_c$) band determined for the intact soil and provide a reasonable and conservative estimate of the undrained strength of NC RBBC for the levels of disturbance investigated.
In terms of stress–strain behavior, Figs. 21a and 21b indicate that for both ISA ± 1 and ISA ± 2 SHANSEP tests, the reconsolidation procedure proposed by Ladd and Foott (1974) yields a realistic description of the undrained behavior of NC RBBC, including the small value of the strain at peak and some strain softening.

When the degree of disturbance is much greater (ISA ± 5), it appears that the effects of disturbance are not erased, as the stress–strain curve shows a much larger strain at peak and the absence of any strain softening. The higher stress level and the variation in the lateral stress ratio cannot explain the large difference in the undrained behavior of the soil. In this case, it is likely that because of the high level of disturbance, either the structure of the soil is altered irreversibly or consolidation to $3.2\sigma_p$ is not sufficient to bring the soil back to the VCL (Santagata and Germaine 2002), and thus the final shear is not representative of a truly NC soil.

### Summary and conclusions

Triaxial element tests were conducted to investigate the effect of sample disturbance on measured soil properties. The goal of the study was to investigate (i) the variation of the effects of disturbance with OCR; and (ii) the effectiveness of recompression and SHANSEP reconsolidation for recovering the intact behavior of the soil.

Comparison of the behavior of NC and OC (OCR2–8) RBBC shows that for the same degree of disturbance (as measured by the magnitude of the strain cycle amplitude imposed when disturbance is being simulated), as the OCR increases the effects of disturbance (effective stress loss, reduction in strength, increase in strain at peak, magnitude of reconsolidation strains, etc.) are less significant.

The results for OC RBBC confirm previous observations made for NC RBBC (Santagata and Germaine 2002) that the decrease in the effective stresses, and the destructuring of the soil are the fundamental mechanisms occurring as a result of sampling. Unless the magnitude of the disturbance straining is sufficient for the soil to fail, it appears that the effects of disturbance on the undrained strength are modest. The stress–strain behavior is more significantly affected. For lower degrees of disturbance, the observed behavior can be explained by the effective stress decrease. The effects of destructuring of the soil appear to come into play when disturbance is associated with strain exceeding a critical threshold that depends on the stress history of the soil.

With regard to the postdisturbance $K_0$-consolidation behavior, the results substantiate the observation previously made for NC RBBC (Santagata and Germaine 2002) that the preconsolidation stress is affected to a lesser degree by disturbance. Moreover, the data for OC RBBC confirm that at least for the degrees of disturbance investigated, the $K_0$-reloading behavior is primarily controlled by the effects of effective stress reduction. This, for the medium-sensitivity soil tested, leads to an overestimate of the preconsolidation pressure as a result of disturbance.

The dependence of the magnitude of the reconsolidation strain on the OCR of the soil limits the use of this parameter as a practical indicator of sample quality.

The investigation of the recompression reconsolidation procedures substantiates statements by other researchers (e.g., Ladd and De Groot 2003) that for NC soils, this method provides a highly unsafe estimate of the undrained strength. For these soils, the SHANSEP method appears a more appropriate solution, as it always generates a safe estimate of the strength and for modest disturbance can provide a reasonable stress–strain curve.

For OC soils, the recompression procedure is appropriate for recovering the strength. If characterization of the stress–strain behavior is needed, the reconsolidation path must be representative of that which took place in the field.

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References


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List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AOCR</td>
<td>apparent overconsolidation ratio</td>
</tr>
<tr>
<td>Cc</td>
<td>compression index</td>
</tr>
<tr>
<td>CR</td>
<td>compression ratio</td>
</tr>
<tr>
<td>e</td>
<td>void ratio</td>
</tr>
<tr>
<td>Ea</td>
<td>undrained secant stiffness</td>
</tr>
</tbody>
</table>

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IOCR  induced overconsolidation ratio

\(K\)  lateral stress ratio

\(K_0\)  coefficient of earth pressure at rest

\(m\)  SHANSEP equation exponent

OCR  overconsolidation ratio

\(p'\)  average effective stress = \((\sigma'_v + \sigma'_h)/2\)

\(p'_m\)  mean effective stress = \((\sigma'_v + 2\sigma'_h)/3\)

\(p'_e\)  end of consolidation mean effective stress = \((\sigma'_v + 2\sigma'_h)/3\)

\(q\)  shear stress = \((\sigma_v - \sigma_h)/2\)

\(q_f\)  shear stress at failure

RR  recompression ratio

\(S\)  undrained strength ratio at OCR = 1

\(u_e\)  excess pore pressure = \(\Delta u\)

\(u_s\)  shear-induced pore pressure = \(\Delta u - \Delta \sigma_{oct}\)

\(\varepsilon_a\)  axial strain

\(\varepsilon_c\)  strain cycle amplitude

\(\varepsilon_p\)  axial strain at failure (peak)

\(\varepsilon_v\)  volumetric strain

\(\varepsilon_{zz}\)  normal axial strain undergone by soil elements during tube sampling

\(\sigma_{hc}\)  horizontal consolidation stress

\(\sigma'_p\)  preconsolidation stress

\(\sigma_{p[be]}\)  best estimate of preconsolidation stress

\(\sigma'_s\)  sampling effective stress

\(\sigma'_v\)  vertical effective stress

\(\sigma'_{vc}\)  vertical consolidation stress

\(\sigma_{max}\)  maximum vertical effective stress reached during consolidation

\(\sigma_{oct}\)  octahedral stress = \((\sigma_1 + \sigma_2 + \sigma_3)/3\)