Recent advances in sampling and laboratory characterization of intermediate soils

Don J. DeGroot
University of Massachusetts Amherst, Amherst, MA, USA, degroot@umass.edu

Jason T. DeJong
University of California Davis, Davis, CA, USA, dejong@ucdavis.edu

Øyvind Blaker
Norwegian Geotechnical Institute, Oslo, Norway, oyvind.blaker@ngi.no

William G. Lukas
GEI Consultants, Woburn, MA, USA, wlukas@geiconsultants.com

Christopher P. Krage
GEI Consultants, Rancho Cordova, CA, USA, ckrage@geiconsultants.com

ABSTRACT: Intermediate soils encompass an array of natural deposits, such as silts, silty clays, clayey silts, sandy silts, among others, and they usually exhibit transitional behavior with some properties being clay-like and others being sand-like. Despite intermediate soils being frequently encountered on infrastructure projects worldwide they remain challenging to sample and characterize. This paper presents results from recent research on sampling and laboratory characterization of intermediate soils. An overview of the state of knowledge is presented and the challenges in sampling and laboratory characterization of intermediate soils relative to the more well-established norms for clays and sands are highlighted. Results from sampling and advanced laboratory characterization tests using a suite of reconstituted synthetic soils and intact samples of the low plasticity index Halden silt are presented. Key findings from a synthesis of the research includes: 1) it is possible to collect intact downhole block samples of a low plasticity, low clay fraction silt, 2) poor tube geometry samplers create sample disturbance that results in markedly different undrained shear behavior compared to undisturbed sample behavior, 3) whereas good tube geometry fixed piston sampling can collect samples of similar quality to that of a block sample, 4) significantly disturbed samples show relatively little change in volume and largely recover any loss in shear wave velocity during laboratory reconsolidation to in-situ effective stresses, 5) a recently developed work-based sample quality criteria appears to track well the effects of sample disturbance independent of plasticity and stress history, 6) 1-D consolidation behavior of even good quality samples do not show any visual evidence of stress history, and 7) selection of undrained shear strength for design for intermediate soils that exhibit dilative behavior is complicated and requires careful assessment of the field loading regime and drainage conditions.

Keywords: intermediate soils; silt; sampling; laboratory testing

1. Introduction

Intermediate soils encompass a suite of natural deposits such as silts, silty clays, clayey silts, sandy silts, among others that are frequently encountered on project sites worldwide and violate assumptions inherent in the geotechnical engineering profession's standard of practice for characterization of clays (e.g., undrained response and use of undrained shear strength) and sands (e.g., drained response and use of effective stress parameters). This is due to the transitional behavior of intermediate soils that often exhibit some clay-like properties and, simultaneously, other sand-like properties. The dilative nature of many intermediate soils is such that no unique undrained shear strength (s_u) is observed during laboratory shear testing. Consequently, there are often significant uncertainties with estimating the in-situ s_u for design using laboratory tests on intact, so-called undisturbed samples. Likewise, densification can occur during sampling of loose intermediate soils that are then unknowingly tested in a denser state in the laboratory yielding a dilative behavior response though the in-situ response would be contractive behavior.

These challenges present practitioners and researchers with a dilemma when intermediate soils are encountered on a project: what are appropriate sampling tools and methods; how should the quality of those samples be assessed; and given varying degrees of sample quality, how accurate and reliable are laboratory measured parameters for geotechnical design? These issues arise across a broad spectrum of geotechnical problems, including dams, levees, deep foundations, slopes, mine tailings, embankments, and offshore structures among others. Specific examples where the complex behavior of intermediate soils had a direct role in either the failure of, or use of significant design conservatism, or significant uncertainty in the selection of design parameters, include Harrison Bay Alaska offshore development [1], Brage and Gullfaks C Offshore Fields [2], Wachusetts Dam [3], Kingston Dredge Pond [4], Perris Dam [5], Big Creek Dam [6], Potrero Canyon [7], and Johan Sverdrup Offshore Field [8].

This paper addresses recent advances in sampling and laboratory characterization of intermediate soils. The background section presents an overview of the state of knowledge and highlights the challenges in sampling and laboratory characterization of intermediate soils relative to the more well-established norms for clays and sands. Recent experience by the authors in sampling and laboratory testing of reconstituted synthetic and intact natural intermediate soils is presented. Key findings are summarized and recommendations for practice are offered.

The paper is a synthesis of collaborative research work conducted at the University of Massachusetts (UMass) Amherst, the University of California, Davis (UC Davis), and the Norwegian Geotechnical Institute (NGI) as presented in several UMass Amherst and UC Davis theses [9-13] and corresponding publications [14-21].

2. Background

2.1. Geotechnical Site Characterization

Geotechnical site characterization is best conducted using an integrated approach that combines various geo-disciplines to describe, evaluate, and determine expected site characteristics [22, 23]. Ideally it involves a team of geo-specialists that work towards creating a site ground model which comprises a synthesized database of all collected and interpreted qualitative and quantitative information about a site. For clays and sands best practice methods for conducting a site characterization program are well-established, with an emphasis on selectively combining the advantages of in-situ testing and laboratory testing [24-25]. For many projects, the site-characterization strategy can involve greater emphasis on geophysical methods and in-situ testing, while requiring fewer, high-quality boreholes in which best practice methods to collect good-quality undisturbed samples (particularly for clays) are followed. Use of side-by-side boreholes and in-situ test soundings can be effectively used to develop site-specific empirical correlations from the advanced laboratory test results.

Site characterization for intermediate soils presents a unique challenge given they often exhibit transitional behavior. Sampling methods employed in practice depend on both soil characteristics and project scope. For low risk/low budget projects common practice is to rely on empirical correlations with in-situ tests such as the standard penetration test and the piezocone penetration test (CPTU). Although interpretation of in-situ tests conducted in intermediate soils presents its own set of challenges, especially in understanding likely drainage conditions during testing (e.g., [26]). For high-risk projects, criteria that incorporate soil plasticity and fines content are often employed, such as those proposed by [27, 28], to screen between soils that are more clay-like versus sand-like. Notably, both methods recognized that there is a range of intermediate soils for which high-quality sampling and subsequent laboratory testing are recommended since their behavior is likely more complex and simply employing clay-based or sand-based analysis and design methods without laboratory testing may be inappropriate and unreliable.

2.2. Sampling

Due to their very small interparticle pore sizes, clays have a low permeability (k) relative to other soils and an undrained response during rapid shearing as occurs during sampling. The small pores can develop significant capillary stresses (i.e., suction) which enable undisturbed tube samples to be obtained. Downhole block sampling (e.g., [29-31]) is a state-of-the-art method for collecting high quality clay samples and use of the subsequently laboratory measured behavior provides a valuable frame of reference for evaluating other samplers and sampling procedures. The consensus on the state of best practice methods for sampling soft clays is to perform mud rotary drilling and use a fixed piston sampler with a thin-walled sampling tube that has: 1) an area ratio (AR) of less than 10%, 2) inside diameter (ID) of greater than 72 mm, 3) sharp cutting shoe (≈ 5 to 15°), and 4) zero inside clearance ratio (ICR) [24, 25, 32].

Unlike clays, high-quality sampling in sands is not feasible unless expensive ground freezing is used to fix the in-situ structure. The sampling difficulties in non-frozen ground arise from the relatively high k due to large pore sizes and the inability to maintain suction upon stress relief. As a result, drained conditions prevail during sampling concurrent with effective stress loss, volume change, and possibly failure. The volume changes during the sampling process can cause a loose sand to densify and a dense sand to dilate. Thus, conventional practice for characterization of sands relies mostly on in-situ testing and empirical correlations to estimate various soil properties and construction performance.

Intermediate soils, being a hybrid between clays and sands, could behave undrained, partially drained, or fully drained during sampling depending on particle size distribution (PSD), plasticity, and structure. They have been traditionally sampled using: 1) open-drive U100 or split spoon samplers [33, 34], 2) thin-walled tube samplers [35, 36], and 3) fixed piston samplers using thin-walled sample tubes [8, 27, 37-39]. The drive samplers have a 'poor' geometry with a large area ratio and cutting angle that induces significant disturbance while the thin-walled tube samplers generally have a better geometry. When combined with a fixed piston sampler the thin-walled tube (with a geometry similar to that listed above for clays) has the potential for collecting good quality undisturbed samples depending on drilling methods used and how the sampler is deployed. However, it has been concluded that tube sampling can densify loose silts due to drainage or partially drained conditions [24, 40]. This could change the laboratory measured behavior from what should have been contractive to dilative and can result in a significant increase in s_u (e.g., [34, 41-44]. It is also possible that dense silts could dilate during sampling and reach a looser state prior to laboratory testing. There is limited experience with block sampling for low plasticity intermediate soils; a few examples include [40, 45-46] and more recently [16, 18] as described in Section 4.1.

It remains an open question as to what best practice methods should be for collecting good quality intact samples of intermediate soils over a range of PSD, plasticity and structure. One common approach employed in current practice is to begin with a preferred undisturbed sampling technique for clays but to modify equipment as required until full or nearly full sample recovery is obtained. However, there is no scientific basis for this approach and full recovery does not ensure a good quality sample has been collected. A major challenge is that, unlike for clays, there are no well-established quantitative methods for evaluating the quality of intermediate soil samples.

2.3. Evaluation of sample quality

Hight and Leroueil [24] and Ladd and DeGroot [25] describe various qualitative and quantitative methods for assessing the quality of clay samples. The most widely used quantitative method is measurement of the change in volume during laboratory consolidation to the estimated in-situ effective stress state. The specimen quality designation (SQD) method of [47] uses the volumetric strain ε_{vol} and is mainly applicable for clays with an overconsolidation (OCR) of less than 3-5. Lunne et al. [48, 49] uses the normalized change in void ratio $\Delta e/e_0$ and provides separate quality ratings for OCR 1-2 and 2-4. These indices have proved valuable for clays in assessing the reliability of laboratory measured consolidation and undrained shear properties for design.

Tube sampling in clays is considered an undrained process whereas for intermediate soils it may be undrained, partially drained, or fully drained, and any potential changes in volume occurring during and after sampling are unknown. Yet the lack of a well-established quantitative sample quality rating for intermediate soils has led to use of the clay-based SQD and $\Delta e/e_0$ indices to assess the quality of these soils, often without knowledge of their applicability. Such an approach has been shown to be misleading [14, 15, 18, 38, 40] and is especially problematic for loose intermediate soils that have (unknowingly) densified during sampling and undergo small changes in specimen volume during laboratory reconsolidation to in-situ effective stresses. DeJong et al. [14] recently proposed a work-based method for assessing sample quality based on one dimensional consolidation test data that is applicable to both clays and intermediate soils and is discussed further in Section 3.4.1

Hight [50] proposed a framework for assessing the quality of clay samples using the combined measurement of shear wave velocity (V_s) and pore water suction (u_s or sampling effective stress σ'_s). The guiding principle for application of this method is a comparison of laboratory measured V_s (using bender elements) and u_s (using a suction probe) values on unconfined samples versus field V_s readings obtained in-situ at the effective stress state prior to sampling. Greater similarity between the laboratory and in-situ values indicates higher sample quality. An advantage of this framework over the traditional volumetric measures of sample quality is that the measures of V_s and u_s are nondestructive and thus sample quality can be evaluated before setting up and conducting a laboratory consolidation test (e.g., oedometer, triaxial). Reference in-situ data (i.e., the V_s-$\sigma'_v\sigma'_h$ backbone curve where σ'_v = vertical effective stress, and σ'_h = horizontal effective stress) requires measurement of shear wave velocity using an appropriate geophysical method (e.g., seismic piezocone or dilatometer) and an estimate of K_0. Examples of the application of shear wave velocity and soil suction measurements to evaluate the sample quality of clays include [24, 51-53].

The presumption for soft clays in the V_s-$\sigma'_v\sigma'_h$ framework is that a disturbed state will plot in a region below the backbone V_{vh}-$\sigma'_v\sigma'_h$ curve as shown schematically in Fig. 1 where V_{vh} is the shear wave velocity for a vertically propagating, horizontally polarized shear wave. This is based on published results for soft clays that demonstrate sample disturbance results in a reduction in effective stress and shear wave velocity due to the combined effects of sampling stress relief and destructuring (e.g., [24, 25, 49, 52, 53].

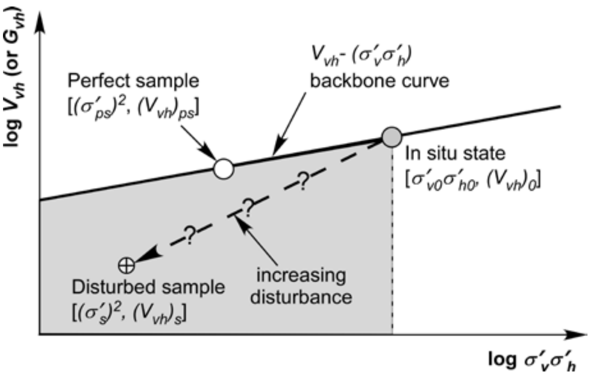

Figure 1. Schematic of shear wave velocity V_{vh} (or small strain shear modulus G_{vh}) – stress state $\sigma'_v\sigma'_h$ framework and reduction in sample state for the perfect sample and a disturbed sample (after [54]).

What is not known is the potential utility of the framework for studying the effects of sample disturbance in non-plastic (NP) and low plasticity index (PI) intermediate soils. While such soils will certainly experience stress relief and possibly destructuring (especially if cemented) during sampling, a complicating factor is the possibility of dilation or contraction taking place during sampling if partial or full drainage occurs. It is thus hypothesized that sample disturbance from destructuring, which is the critical issue for clays [25], and sample disturbance due to volume change, which is the critical issue for sands [5], will both be present for intermediate soils. It is further hypothesized that these combined effects will shift the V_{vh}-$\sigma'_v\sigma'_h$ backbone curve as shown schematically in Fig. 2 (which is a modification of Fig. 1). For initially loose, non-cemented NP to low PI intermediate soils where densification can occur during sampling, V_{vh} will increase since it is directly dependent on void ratio; similarly, an initially dense soil may dilate (loosen) during sampling, resulting in a decrease in V_{vh}. Hence, disturbed sample conditions for intermediate soils can exist above or below Fig. 2 Line O-A and arrive at a condition between points O-B or O-C after reconsolidation to σ'_{v0}.

The authors are not aware of any existing data to test the Fig. 2 hypothesis. One challenge is that NP and low PI soils are likely to develop low suction values after sampling (as shown via laboratory simulated tube sampling in Section 3.3.2) that may be difficult to reliably measure. In the absence of data from intact field samples, Section 3.4.2 presents results from laboratory simulation of tube sampling for a reconstituted PI = 4 soil using a triaxial stress path cell with measurement of shear wave velocity and analyzes the data in a V_{vh}-$\sigma'_v\sigma'_h$ framework.

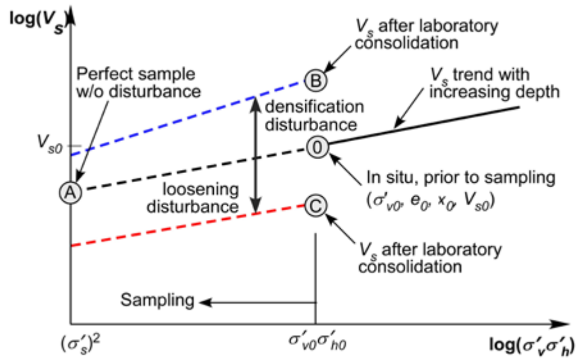

Figure 2. Proposed framework to assess sample quality using shear wave velocity V_{vh} and stress state $\sigma'_v\sigma'_h$ for intermediate soils.

2.4. Laboratory characterization

So-called advanced laboratory characterization tests that can be routinely conducted in well-established commercial geotechnical laboratories consist of 1-D consolidation (e.g., incremental load IL, constant rate of strain CRS) and consolidated shear tests (e.g., triaxial compression, triaxial extension, direct shear, direct simple shear). If undrained shear strength anisotropy is important for design (e.g., stability problems) then a combination of the consolidated shear tests can be performed. In the absence of ground freezing, sands are characterized by such laboratory tests using reconstituted specimens. Common practice is to use the estimated in-situ density as the target for the laboratory tests and to conduct drained shear tests at different effective stress levels to define a failure envelope.

For clay deposits, establishing the stress history (i.e., σ'_{v0}, preconsolidation or vertical yield stress σ'_p, and OCR = σ'_p/σ'_{v0}) should be a central goal of the test program as it influences all important aspects of clay behavior [25]. Furthermore, the strong link between s_u and stress history within a normalized soil parameter framework is a valuable tool for interpreting and assessing the reliability of the laboratory data (e.g., Fig. 7.1 in [25]). Advanced laboratory testing should couple CRS tests to measure stress history and consolidation behavior with anisotropic or K_0 consolidated strength tests to measure undrained shear behavior. This behavior can be measured using Recompression or SHANSEP strength testing techniques that were independently developed to address the important issues of anisotropy, rate effects and sample disturbance [25, 55].

For intermediate soils, practitioners are often faced with the conundrum of whether the soils of interest are going to exhibit clay-like or sand-like behavior as discussed for example by [5, 27, 28]. It remains an open question as to what the most effective methodology is to use for planning of the advanced laboratory test program. Considerations include:

- will the measured soil behavior fit within a stress history-normalized undrained shear strength framework that is effectively used for clays;
- can stress history be accurately determined from 1-D consolidation tests which often have no visual evidence of a σ'_p;
- how to best select an appropriate s_u for a given design scenario if the observed behavior is dilative;
- can sample disturbance be partially remediated using Recompression or SHANSEP techniques; or
- is the behavior more sand-like such that excessive sample disturbance is inevitable (i.e., densification for looser soils even though there is no definitive laboratory method of proving such has occurred) and the test program should focus on reconstituted samples with the caveat of how representative of the in-situ state and structure they may be?

While the results presented in Section 3 for synthetic intermediate soils and Section 4 for a natural low PI silt do not resolve this intermediate soils testing conundrum they do present some noteworthy new findings.

2.5. Selection of design shear strength

For stability problems such as embankments and levees involving low PI intermediate soils with a high k, consolidation rates are often rapid relative to the rate of construction such that the drained shear strength is the relevant strength for design [39]. For some projects s_u is often a necessary input for analysis and design depending on factors such as loading rate, drainage path, etc. Examples include silt layers confined within lower hydraulic conductivity clay units, pile drivability assessment, relatively rapid installation of offshore structures such as jack-up vessel legs, seabed manifolds, mut mats and gravity-based structures, and certain types of cyclic loading for which the reference s_u is required [21].

However, there are significant uncertainties in selection of appropriate design values of s_u for intermediate soils from laboratory tests due to sample disturbance effects and limitations in reconstitution methods, especially if the soil exhibits dilative behavior (e.g., [37, 40, 56, 57]. Brandon et al. [35] list six criteria for interpreting s_u for silts that exhibit dilative behavior (Fig. 3): 1) maximum deviator stress, $(\sigma_1 - \sigma_3)_{max}$; 2) an assigned limiting vertical strain, $\varepsilon_{v,f}$; 3) state of zero excess shear induced pore pressure at failure $\Delta u_f = 0$, which is equivalent to Skempton's A parameter at failure equal to zero, $A_f = 0$ for B = 1; 4) point at which the effective stress path first reaches the failure envelope, defined by the K_f line; 5) maximum stress obliquity, $(\sigma'_1/\sigma'_3)_{max}$; and 6) maximum shear induced pore pressure, u_{max}. For an interpreted or assumed zero cohesion intercept (c' = 0) criteria 4 and 5 provide the same value of s_u.

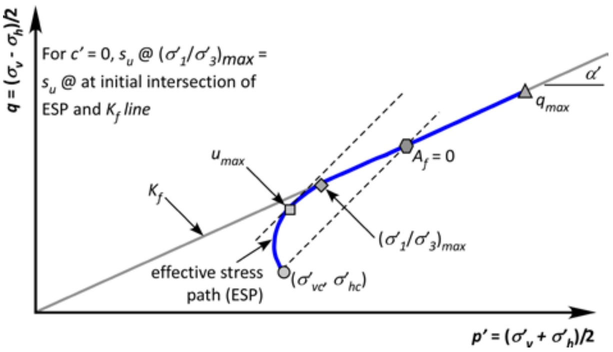

Figure 3. Idealized undrained shear effective stress path showing stress at failure for different failure criteria (modified after [35]).

The differences in s_u among these criteria is often significant and there is no clear consensus on what practitioners should do [34, 35, 38, 56-59]. The major reason for the lack of consensus is the body of research available on how laboratory s_u values for silts defined by the above criteria relate to the in-situ s_u for specific design applications is limited. Furthermore, research on how sample disturbance influences these various s_u selection criteria is also lacking. A pragmatic approach for projects that require upper and lower bound values of s_u, e.g., driven piles, could be to select a criterion that produces a higher end s_u value for drivability analyses and select a criterion that produces a lower end s_u value for capacity analyses. But again, with no clear guidance this could result in excessive conservatism and unknown safety margins.

For designs that can be assumed to involve fully drained shear, data presented in Sections 3.2.2 and 4.4 shows that sample disturbance has little to no effect on the effective stress friction angle for the intermediate soils tested.

2.6. Preparation of reconstituted soils

Preparation of reconstituted specimens for advanced laboratory testing is an approach used in practice because of the challenge in collecting good quality samples of intermediate soils. Clays are typically reconstituted by sedimentation of a slurry while sands are most often reconstituted using pluviation, vibration or moist tamping methods. The choice of method to use is more of a challenge for intermediate soils. Høeg et al. [37] found that intact thin-walled piston samples of a Swedish silt exhibited dilative behavior during triaxial compression testing whereas specimens reconstituted to the same void ratio and initial stress state using both moist tamping and water pluviation methods generally exhibited contractive behavior. Wang et al. [60] also found that moist tamping and layered slurry deposition could not fully replicate the shear behavior measured for intact block samples.

The major advantage of using reconstituted samples is that under a controlled laboratory environment the effects of different variables on consolidation and shear behavior can be studied while recognizing that this may not necessarily be fully representative of the in-situ soil state and structure. The selection of an appropriate preparation method depends on the stated purpose of the test program. Certain soil properties tend to be independent of the preparation method (e.g., effective stress friction angle) while others are strongly dependent on the method used (e.g., small strain stiffness, contractive/dilative tendency, shear strength, cyclic resistance).

Prior studies have not examined the applicability of the slurry preparation approach across a range of predominantly fine-grained intermediate soil mixtures that span nonplastic silts, low-plasticity silty clays, low plasticity clays, and plastic clays. Furthermore, they have not evaluated the behavior of the slurry preparation approach used over a wide range of behaviors and advanced laboratory testing. Section 3.1 presents a recently developed slurry preparation method for intermediate soils and validation data showing the reliability and repeatability of the method using a suite of advanced laboratory tests.

2.7. Laboratory simulation of tube sample disturbance

Baligh et al. [61], using the strain path method [62], developed the ideal sampling approach (ISA) to numerically simulate the process of tube sampling. The research showed that a centerline element of soil undergoes one compression-extension-compression vertical strain cycle during sampler penetration. This strain cycle can be simulated in the laboratory in a triaxial stress path system by applying a loading cycle that consists of a compressive strain ($\varepsilon_{zz,max}$), an extension strain (-$\varepsilon_{zz,max}$), and unloading back to zero vertical strain. Once the strain cycle is complete, the deviator stress is removed, mimicking sample stress relief, to complete the ISA simulation. The magnitude of $\pm\ \varepsilon_{zz,max}$ can be varied to represent different degrees of tube sample disturbance. For a Sherbrooke block sampler $\varepsilon_{zz,max}$ is essentially zero while for a standard US Shelby tube (ASTM D1587 [63]) $\varepsilon_{zz,max}$ is approximately 1.0%.

For clays the ISA simulation is performed undrained, as documented by [64-66]. In general, ISA testing of clays results in a reduction in mean effective stress p', an increase in ε_{vol} or $\Delta e/e_0$ during post-ISA reconsolidation, and a decrease in s_u. Limited ISA testing of intermediate soils have been conducted. Carroll and Long [40] performed undrained ISA testing of an intermediate plasticity silt and found an increase in stiffness and s_u with an increase in the level of ISA strain; opposite the effect found for clays. Sections 3.3.2 and 4.3 present recently collected data from ISA testing of synthetic intermediate soils and a natural low PI silt.

3. Behavior of synthetic intermediate soils

This section presents results from 1-D consolidation and consolidated undrained shear tests recently conducted on synthetic intermediate soils. While laboratory preparation of such soils typically cannot replicate the exact depositional environment and geologic stress history that natural soils undergo, it does provide valuable insight into fundamental stress-strain-strength-flow behavior. These insights can in turn be used as a framework for studying the behavior of intact samples and, in particular, investigating the effects of sampling and sample disturbance. The section presents a new method for slurry deposition of low-plasticity intermediate soils and presents results from 1-D consolidation and anisotropically consolidated undrained triaxial shear tests. Results are also presented for two techniques used to simulate the effect of sample disturbance on this behavior and a recently developed sample quality criteria for intermediate soils.

3.1. Specimen Preparation

Krage et al. [17] describe a new method for slurry deposition of intermediate soils that can produce homogeneous specimens across a range of PIs and PSDs. The primary equipment consists of an 89 mm inner diameter (ID) mixing chamber, mixing blade and a mixing frame within which the mixing chamber can be rotated in a circular motion along its longitudinal axis and also tilted during mixing and subsequent extrusion of the mixed slurry. Sample preparation involves: 1) mixing the soil-water mix in the chamber by spinning a three-blade paddle under vacuum up to 1,400 r/min using a hand drill, 2) removing the mixing paddle and continue mixing under vacuum by rotating the chamber about its longitudinal axis while oriented at an angle of 15 to 30° from the horizontal, and 3) extruding the mixed soil using a piston within the chamber and a tremie tube either directly into an element test cell (e.g., oedometer, triaxial, DSS) or into a larger consolidation chamber.

While the main objective of the work was focused on developing a mixing procedure for low PI intermediate soils, some higher PI clay soils were also tested to evaluate the efficacy of the procedure across a broader spectrum of soil types. The mixing procedure was tested using different mixtures of silt sized ground silica (Sil-Co-Sil 250, US Silica, Ottawa, IL) and kaolin clay (Old Hickory #1 Glaze, Old Hickory Clay Company, Hickory, KY). The dry mass percentages of silica (S) and kaolin (K) clay were varied from 100% silica to 100% clay with a specific mix designated as %S%K, e.g., 85S15K equaled 85% silica silt and 15% clay by mass. Table 1 presents index properties and classification according to the Unified Soil Classification System (USCS, ASTM D2487 [63]) of some example mixes. Liquid limits were determined using the Casagrande cup (ASTM D D4318 [63]) and fines fraction (FC) defined as percent less than 0.075 mm and clay fraction (CF) as percent less than 0.002 mm (ASTM D2487 [63]).

Table 1. Index properties and classification of synthetic soil samples

Soil	LL (%)	PL (%)	PI (%)	FC (%)	CF (%)	USCS	ϕ'_{mo} (°)
0S100K	56	25	31	100	68	CH	19
50S50K	31	15	16	88	48	CL	26
70S30K	24	15	9	83	35	CL	33
80S20K	23	16	7	80	22	CL-ML	35
85S15K	19	15	4	79	16	CL-ML	36
98S02K	18	NP	NP	75	13	ML	40
100S0K	19	NP	NP	75	3	ML	40

Note: LL = liquid limit, PL = plastic limit, PI = plasticity index = LL - PL, FC = fines content = % < 0.075 mm, CF = clay fraction = % < 0.002 mm, USCS = Unified Soil Classification System, ϕ'_{m0} = maximum obliquity from CAUC tests

The dry silica and kaolin constituents for a target soil mixture were initially combined and then hydrated at water contents between 1.5 and 2 times the liquid limit for a minimum of 24 hours. Using too low a water content would result in particle clumping and poor mixing and using too high a water content would result in large consolidation strains for the deposited slurry. For the high silt content samples, the mixture moisture content was limited between 1.5 and 1.7 times the liquid limit to avoid particle segregation. For soils with PI > 4 the primary mixing mechanism was using the mixing blade under vacuum. For the nonplastic soils, for which segregation was a greater concern, the primary mixing mechanism was rotation of the mixing chamber. In all cases mixing under vacuum was conducted for a minimum of one hour. Krage et al. [17] present more details and pictures of the mixing equipment and test procedures.

For triaxial test specimens a 102 mm ID acrylic consolidation chamber was used to create a soil cake large enough for four 35.6 mm diameter specimens. Once the slurry mix was placed into the consolidation chamber, the mix was typically incrementally consolidated (1-D) to 200 kPa so that the consolidated cake could be unloaded, extruded, and dissected for individual triaxial specimens which were sealed and stored in a humid room at a controlled temperature of 11° C and > 85% relative humidity until the time for testing. For PI ≤ 4 specimens that were not self-standing under zero total stress, individual triaxial test specimens were prepared by placing the mixed slurry directly into a triaxial vacuum split mold similar to that described by Wang et al. [60]. The slurry was initially allowed to self-weight consolidate followed by incremental 1-D loading up to 45 kPa while still in the split mold. At the end of incremental loading, the specimen was subject to a vacuum of 30 kPa, the split mold removed, and the vacuum maintained until the specimen was set up in the triaxial cell and subject to an isotropic cell pressure of 30 kPa.

The homogeneity, reliability, and repeatability of the mixing procedure was evaluated using a combination of PSD, water content distribution, CRS, monotonic anisotropically consolidated undrained triaxial compression (CAUC), and monotonic and cyclic direct simple shear (DSS) testing. Fig. 4 presents PSD data for a sample prepared to PI = 7 that was cut into six horizontal slices showing near identical PSDs. The water contents (w) averaged 21.2% with a variation from the average of 0.3 to 0.6% (m/m). Similar water content data with low variability were collected for a PI = 15 mix with an average w = 24.2% and range from the average of 0.0 to 0.4%.

Figs. 5 to 7 present results from: 1) two CRS tests conducted on two different PI = 7 mixes (Fig. 5), 2) pairs of DSS tests conducted on two different mixes of PI = 7 soil and PI = 20 soil (Fig. 6), and 3) two CAUC tests conducted on two different PI = 9 mixes and conducted in two different triaxial chambers and stress path systems (Fig. 7). In all cases near identical behavior was measured. Furthermore, the slurry mixes were prepared and tested independently at UC Davis and UMass Amherst by four different researchers. This confirmation of uniformity and repeatability of the mixing procedure coupled with the observed consistency in soil behavior response measured across a range of mixes (as presented in the following section) demonstrated the efficacy of the preparation method in producing reliable and repeatable results for synthetic soils over a range of plasticity from NP silt to high PI clay.

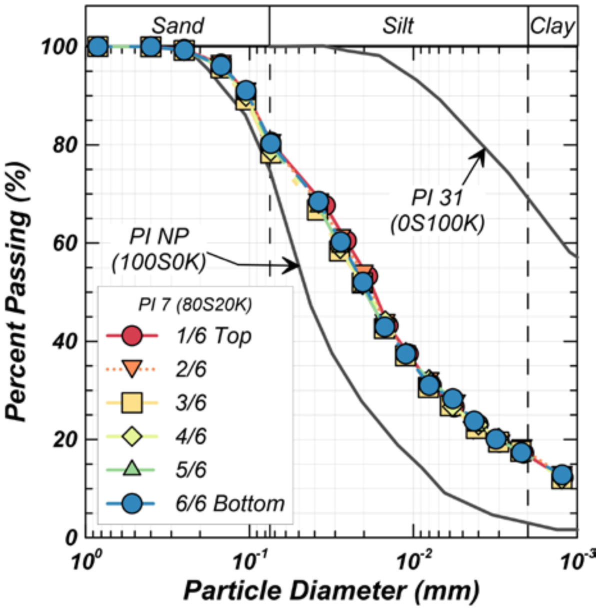

Figure 4. Particle size distribution from six horizontal slices of a slurry deposited PI = 7 reconstituted soil mixture (from [17]).

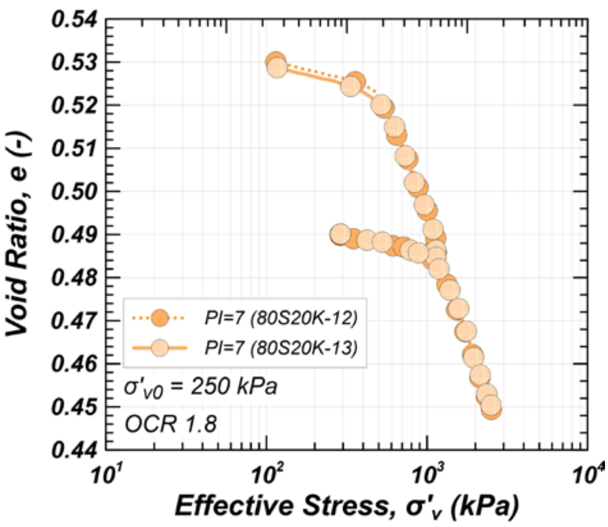

Figure 5. Results from CRS tests on indicating repeatability of consolidation tests of PI = 7 reconstituted soil mixture from two preparation batches (from [17]).

3.2. Stress-strain-strength behavior

3.2.1. 1-D CRS behavior

Fig. 8 presents results from CRS tests conducted on six different mixes with PI ranging from NP to 31. The slurry mixes were first loaded in an oedometer frame to a maximum vertical effective stress $\sigma'_{v,max}$ = 200 kPa and then unloaded to σ'_v = 110 kPa for a laboratory induced OCR = σ'_p/σ'_v = 1.8. The samples were then transferred to a CRS cell and tested in general accordance with ASTM D4186 [63]. The initial void ratio (e_0) decreases markedly with an increase in silt content but then reverses with the NP specimen having a higher e_0 than the PI = 4 specimen. This trend of decreasing and then increasing e_0 with increased silt content is similar to that observed for some sand-silt mixtures and is attributed to the effects of particle packing [67].

The distinguishing features of the measured CRS behavior is the significant increase in the 1-D stiffness of the soils with increasing silt content and loss of any visual evidence of a σ'_p. This highlights a major challenge in characterizing the stress history of low plasticity intermediate soils in that it is often difficult to impossible to determine σ'_p even if a good quality sample was collected. For the tests plotted in Fig. 8, the laboratory mechanical stress history (i.e., σ'_p = 200 and OCR = 1.8) is known and yet no evidence of such is seen in the low PI data. Section 4.2 presents similar results for intact samples of a natural low PI silt.

3.2.2. Undrained CK$_0$U/CAUC shear behavior

Figs. 9 and 10 present results from CAUC and CAU extension (CAUE) tests conducted on five different mixes ranging in PI from NP to 31. The 35.6 mm diameter by 72 mm tall specimens were tested using a stress path triaxial testing system. Internal load cells were used for all tests and membrane resistance and area corrections were applied following [68, 69] and ASTM D4767 [63]. The specimens were back pressure saturated at 300 kPa and K_0 consolidated at an axial strain rate of 0.1 to 0.2 %/hr to σ'_v = 400 kPa and then anisotropically unloaded at a strain rate of 0.05 %/hr to 222 kPa for a final preshear OCR = 1.8. Undrained shear was performed using strain control, typically at 0.5 %/hr, with an increase in σ_v for triaxial compression (TC) and a decrease in σ_v for triaxial extension (TE), both at constant horizontal stress σ_h.

Of note in the results presented in Figs. 9 and 10 is the immediate and continuous dilative behavior of the NP soil compared to all the other mixes, which exhibited varying degrees of contractive behavior, including the PI = 4 specimen. This contrast in behavior highlights another challenge in characterizing the undrained shear behavior of low PI silts, which is the need to anticipate for design whether the soil is going to exhibit contractive (i.e., soft clay-like) or dilative (i.e., dense sand-like) behavior and, in the latter case, how to select a representative s_u for design. This issue is explored in more detail in Section 4.4 for intact samples of a natural low PI silt.

For triaxial extension mode of shear (Fig. 10) the NP, PI = 4, and PI = 7 soils generated positive shear induced pore pressures compared to the more clayey higher PI soils. All five effective stress paths cross the $q = (\sigma'_v-\sigma'_h)/2 = 0$ axis, with the ratio of the mean stress $p' = (\sigma'_v+\sigma'_h)/2$ to the end of consolidation value, i.e., p'/p'_c systematically reducing with a reduction in PI, consistent with [70]. Indeed, the loss in mean effective stress $\Delta p'_c$ for the NP soil is very large and continues up to when the effective stress path reaches the failure envelope, at which point it starts to migrate along it. These results imply a potentially large initial reduction in effective stress during sampling of natural low PI silts as explored more in Section 4.3.

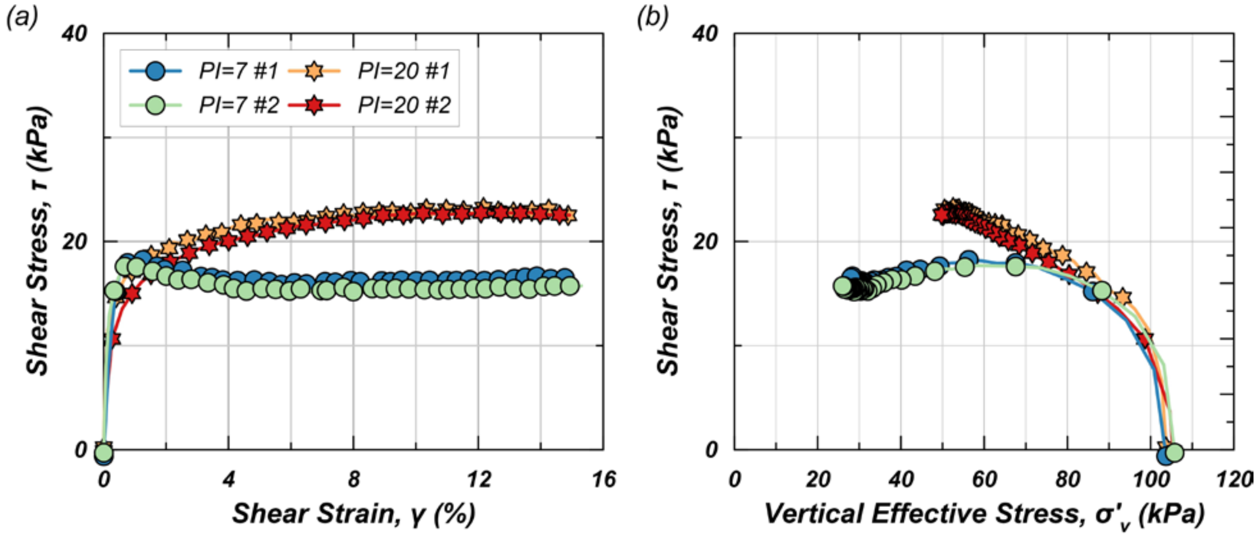

Figure 6. Results from monotonic DSS for the PI = 7 and PI = 20 reconstituted soil mixtures indicating repeatability of (a) shear stress versus shear strain and (b) shear stress plot response (from [17]).

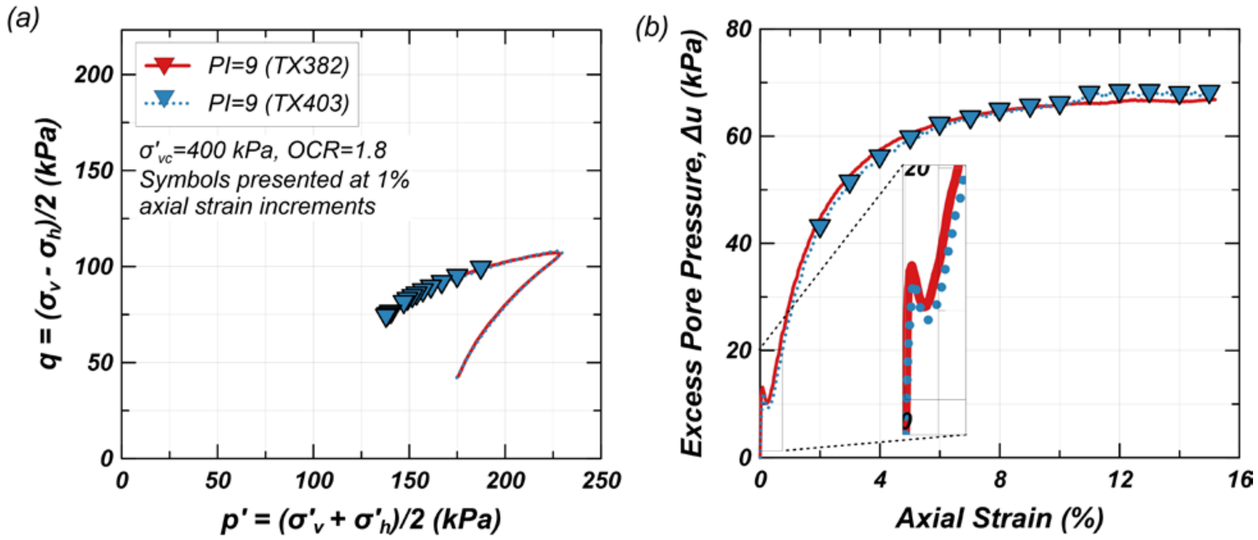

Figure 7. Results from CAUC triaxial tests indicating repeatability of (a) effective stress path and (b) shear induced pore pressure response for the PI = 9 reconstituted soil mixture. Specimens were prepared in separate batches and tested using separate triaxial chambers and stress path systems (from [17]).

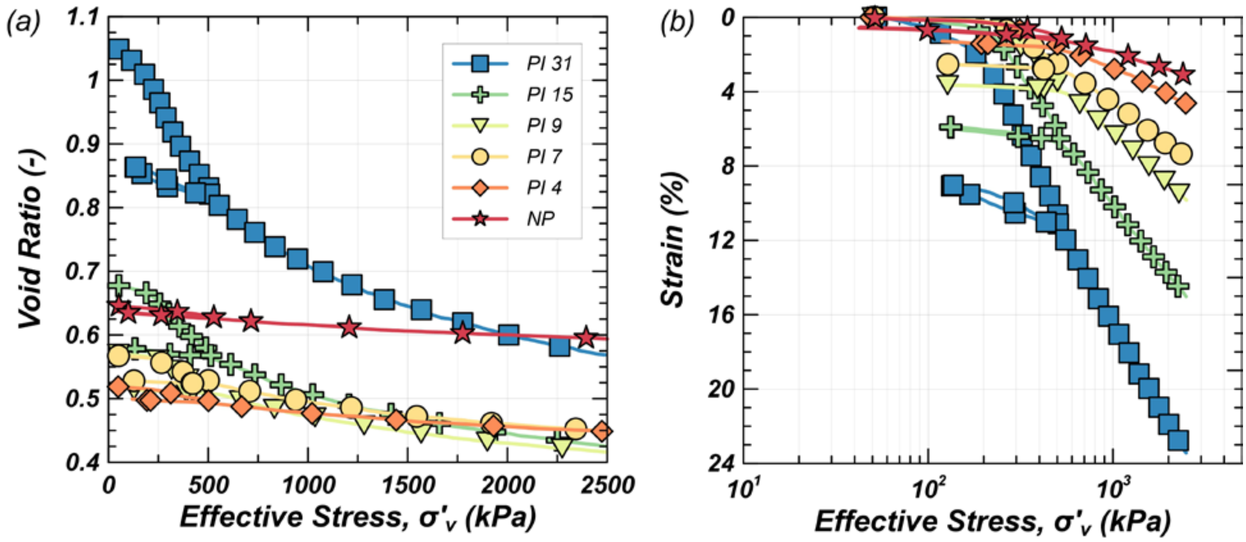

Figure 8. Results from CRS on various synthetic reconstituted intermediate soils (a) void ratio compression curve, and (b) strain compression curve (modified after [17].

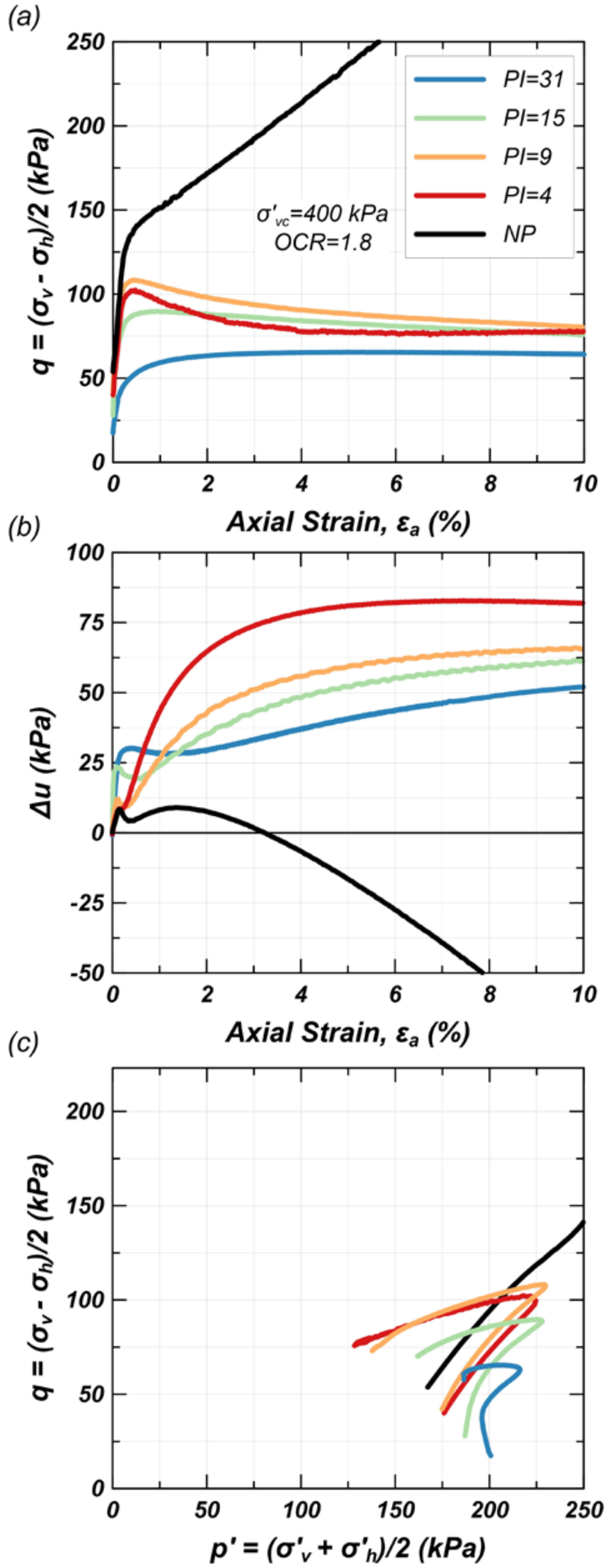

Figure 9. CK_0UC shear behavior of synthetic reconstituted intermediate soils (a) shear stress vs axial strain, (b) shear-induced pore pressure vs axial strain, and (c) effective stress paths (from [15]).

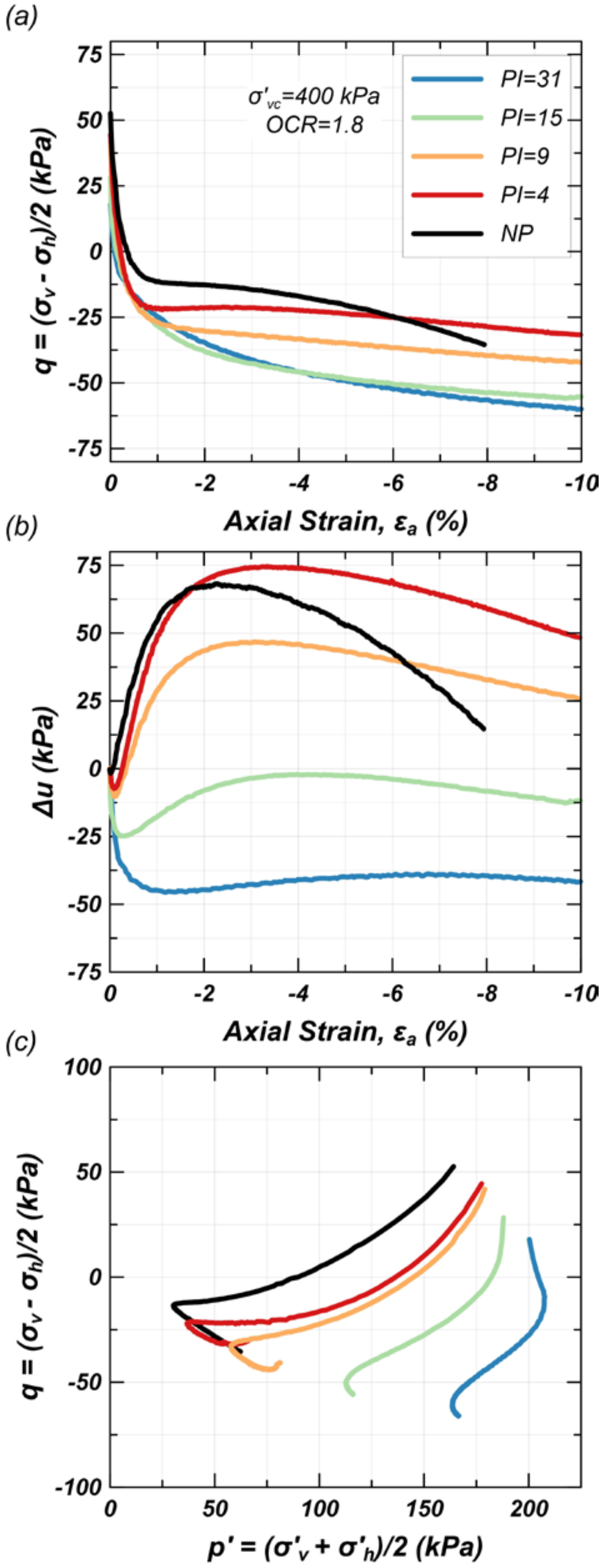

Figure 10. CK_0UE shear behavior of synthetic reconstituted intermediate soils (a) shear stress vs axial strain, (b) shear-induced pore pressure vs axial strain, and (c) effective stress paths (from [15]).

The effective stress friction angle ϕ'_{mo} taken at the maximum obliquity $(\sigma'_1/\sigma'_3)_{max} = (\sigma'_v/\sigma'_h)_{max}$ progressively increases with decreasing PI for the CAUC tests from 19° to 40° for the PI = 31 to the NP soil (with an assumed $c' = 0$). A similar trend was observed for the CAUE tests, although the NP soil had a lower ϕ'_{mo} than the PI = 4 soil. For all the other soils $\phi'_{m0,TE} > \phi'_{mo,TC}$.

Stress normalization of undrained shear behavior of clays has been well studied and is often used to characterize clay deposits, especially for less structured clays [25]. Less is known about the normalized undrained shear behavior of intermediate soils. Fig. 11a plots CAUC data for the PI = 4 soil mix normally consolidated to five values of σ'_{vc} = 108, 198, 401, 600 and 808 kPa. The results exhibit consistent normalized behavior with the normalized undrained shear strength ratios s_u/σ'_{vc} all within the range of 0.28 to 0.30 with s_u taken as equal to q_{max}. Also included in Fig. 11b are CAUC data for reconstituted NP Dedham silt which is a natural silt from Dedham, MA, with specimens prepared in the same manner as the PI = 4 soil and with values of σ'_{vc} = 109, 204, 400 and 601 kPa. The Dedham silt specimens all exhibit dilative behavior with the effective stress paths sharply migrating to the right and up the failure envelope. This type of behavior, as also shown by the NP 98S02K specimen (Fig. 9), makes selection of an appropriate s_u for design more challenging (Section 2.5) and will be discussed in Section 4.4 for a low PI natural silt. As a preview, given the shape of the Dedham silt effective stress paths and that they all converge on the same failure envelope, with ϕ'_{mo} = 35° in this case, any failure criteria that uses a set value for Skempton pore pressure parameter at failure A_f as the definition of failure will produce perfectly normalized results for the Dedham silt independent of σ'_{vc}. However, other failure criteria (e.g., u_{max}) will not necessarily result in selection of the same normalized undrained shear strength.

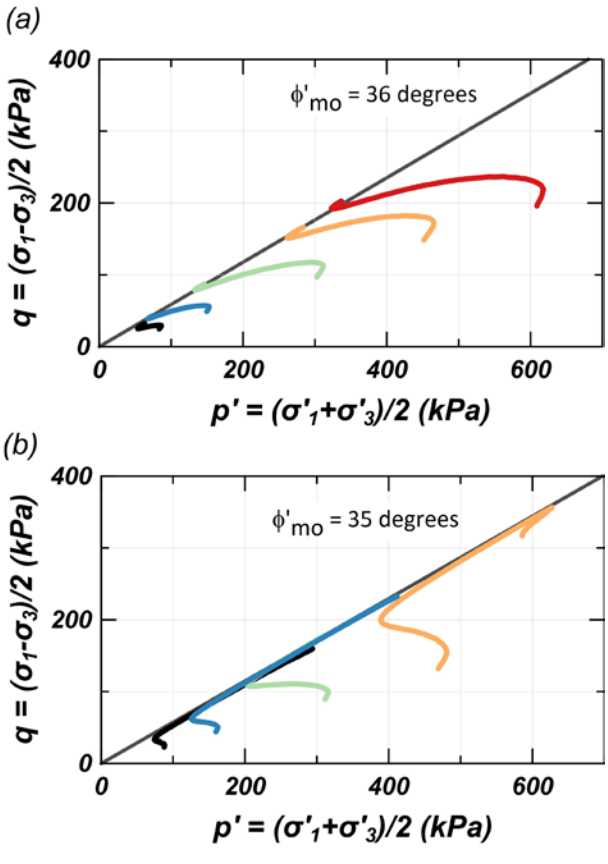

Figure 11. Effective stress paths for specimens consolidated to various preshear effective stresses (a) PI = 4 synthetic reconstituted soil, (b) NP reconstituted Dedham silt.

3.3. Laboratory simulation of sample disturbance

3.3.1. 1-D consolidation behavior

DeJong et al. [14] present results from a test program that was conducted on several of the silt-clay mixtures described in Section 3.1 that were subjected to three levels of simulated sample disturbance. The objective was to evaluate the recompression response in 1-D oedometer loading after inducing varying degrees of disturbance. A reference 1-D perfect sampling (1DPS) procedure was defined as simply the removal of the deviatoric stress within an oedometer specimen as shown in Fig. 12. Specimens were first normally consolidated to a target maximum vertical effective stress $\sigma'_{v,max}$ and then unloaded to an isotropic effective stress state (Fig. 12 Path 1-2-3-4). This is analogous to the method developed by [71] for simulation of perfect sampling in a triaxial cell. But since the horizontal stress cannot be controlled in a conventional oedometer cell the amount of unloading necessary from $\sigma'_{v,max}$ to achieve an estimated $\sigma'_v = \sigma'_h$ was computed using [72] as $K_{0,OC} = (1 - \sin\phi'_{cv})(OCR)^{\sin\phi'cv}$ where ϕ'_{cv} = constant volume friction angle. Values of ϕ'_{cv} were taken as ϕ'_{mo} from CAUC tests conducted on OCR = 1 or 1.8 specimens. Following unloading to K_0 = 1, the 1DPS specimens rested for 30 to 120 minutes, depending on a specimen's PI, and were then reconsolidated without removal of the specimen from the oedometer ring (Fig. 12 Path 4-7). This procedure was selected as the best possible (ideal) condition and provided a baseline for the two other degrees of sample disturbance investigated.

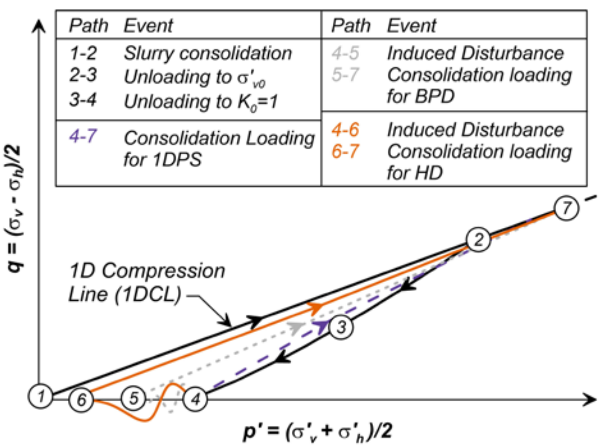

Figure 12. Schematic illustrating 1-D perfect sampling, best practice disturbance (BPD), and heavily disturbed (HD) specimen stress paths in oedometer loading (from [14]).

Best practice disturbance (BPD) was defined as initially following the same loading and unloading path as the 1DPS specimens (Fig. 12 Path 1-2-3-4) but then specimens were removed from the 71 mm diameter oedometer ring and trimmed into a 63.5 mm diameter oedometer ring (Path 4-5). Thereafter the specimens were tested using a conventional CRS loading schedule (Path 5-7). The procedure was intended to simulate disturbance induced by extraction of a sample from a sample tube and subsequent handling and trimming. It does not consider potential disturbance induced by insertion and extraction of a sampling tube in the field, transport, and storage. The third procedure was developed to create highly disturbed (HD) samples. Specimens were initially prepared in the same manner as the BPD procedure but upon removal from the 71 mm oedometer ring, the samples were wrapped in plastic and frozen at –18° C for minimum of 24 hrs followed by thawing at 13° C for 24 hr (Path 4-6). Thereafter the samples were trimmed into a 63.5 mm oedometer ring and tested with the same CRS loading schedule as that used for the BPD specimens (Path 6-7). This method of simulating sample disturbance induces varying degrees of disturbance to the soil mixes because the consequences of the freeze-thaw cycle differ for specimens of varying plasticity. However, the net result was that the samples were all subjected to a consistent procedure that was assumed to destroy the soil's structure and stress history and was considered a proxy for what occurs during poor quality drilling and tube sampling.

The physical effect of the HD freezing procedure varied with PI (Fig. 13); ice crystals and lenses formed in the HD samples after freezing with more ice lenses forming in the higher PI soils while more ice crystals formed in the lower PI soils. After thawing, the PI = 31 clay samples remained intact, albeit with visible laminations from the freeze-thaw cycle, while the NP samples lost all coherence and flowed within the container in which they were stored. Fig. 14 plots CRS compression curves for the three degrees of simulated samples disturbance for soils with PI = 4, 7 and 15. The influence of the BPD and HD disturbance on the PI = 15 clay followed well established trends for the influence of sample disturbance in clays, i.e., increase in recompression ratio $C_r = \Delta e/\Delta log\sigma'_v$ for $\sigma'_v < \sigma'_p$, a decrease in and a poorly defined σ'_p, and a decrease in compression ratio $C_c = \Delta e/\Delta log\sigma'_v$ for $\sigma'_v > \sigma'_p$. Similar effects on C_r and C_c are evident for the PI = 4 and 7 specimens but they are more subtle while any visible evidence of σ'_p (=36 kPa) is lost.

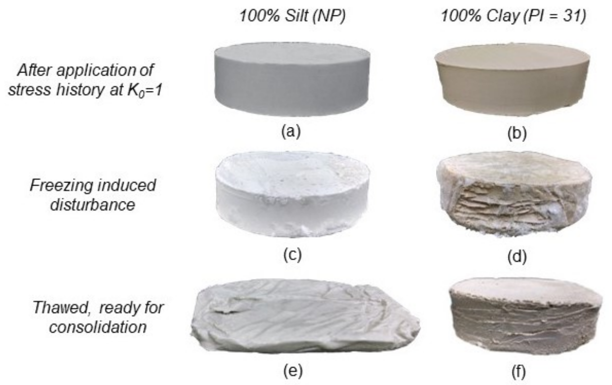

Figure 13. Specimens initially consolidated to σ'_{v0} = 110 kPa, OCR = 1.8 before, during, and after freeze-thaw-induced disturbance (a) NP before, (b) PI 31 before, (c) NP during, (d) PI 31 during, (e) NP after, and (f) PI 31 after (from [14]).

Similar trends as a function of the type of induced disturbance, PI, and stress history are reported by [14] for the full collection of over 100 CRS tests conducted as part of the study. A significant observation resulting from the research was that the NP specimens, and to a similar degree the PI = 4 specimens, flowed as a viscous material upon thawing after the HD freeze-thaw cycle (Fig. 13). Yet the initial void ratios of these specimens after placing them into the CRS ring were nearly identical to that of the 1DPS specimens. Furthermore, ε_{vol} and $\Delta e/e_0$ for loading up to the laboratory simulated σ'_{v0}, while somewhat larger than the 1DPS specimens, were small to very small. This type of 1D consolidation response of low PI intermediate soils highlights one of the challenges of using the well-developed clay-based volumetric measures of sample quality for such soils. That is, in this case the NP specimens were thoroughly disturbed by the HD procedure and yet exhibited very little volume change during subsequent reconsolidation relative the 1DPS specimens. This is explored in more detail in Section 3.4 for the synthetic soil mixtures and in Sections 4.2 and 4.3 for a low PI natural silt.

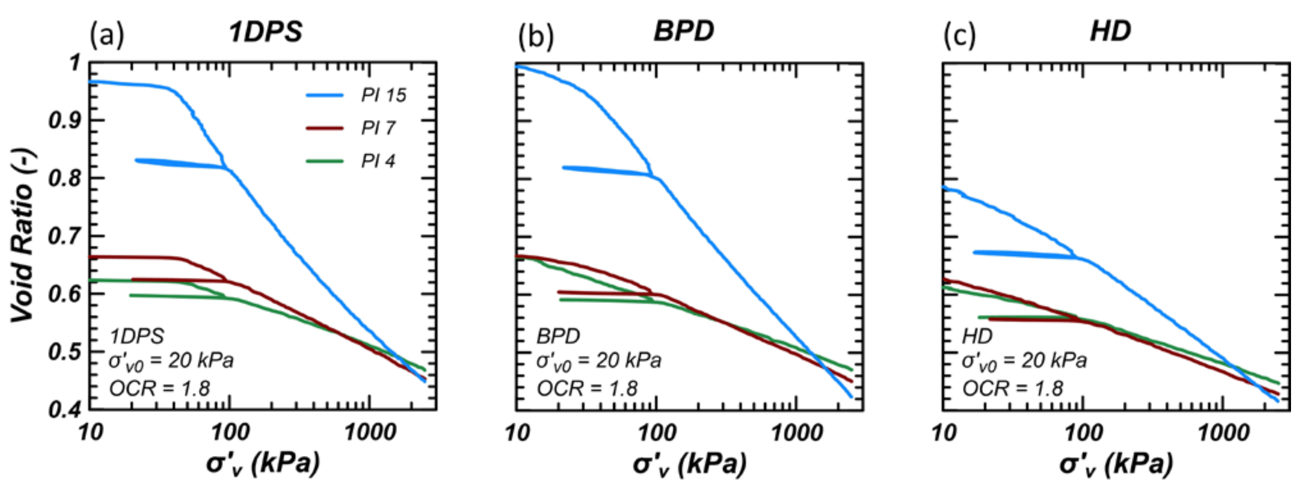

Figure 14. Compression behavior with varying plasticity for (a) 1-DPS specimens, (b) BPD specimens, and (c) HD specimens (from [14]).

3.3.2. Undrained CAUC behavior

Lukas et al. [15] present results from a laboratory investigation of the influence of simulated tube-sampling disturbance on lightly overconsolidated intermediate soils by varying plasticity and sampling disturbance. The specimens were tested in a triaxial stress path cell using the ideal sampling approach (ISA) using axial strain cycles of ±0.5%, ±1.0%, and ±3.0% to simulate three different degrees of tube-sampling disturbance. Test specimens were prepared as described in Section 3.1 and baseline CK_0UC/CAUC tests were performed as a reference state for an undisturbed condition. These tests are described in Section 3.2.2 and Fig. 9 plots the undrained shear results. ISA testing was performed by preparing and consolidating specimens in the same manner to a final σ'_{vc} = 222 kPa and OCR = 1.8 but then at the end of consolidation, ISA undrained shear was performed strain-controlled at 0.5% axial strain per hour with a single strain cycle using peak ISA strains of ±0.5%, ±1.0%, or ±3.0%. At the end of the strain cycle, the deviator stress was removed by undrained-unloading to an isotropic stress state. The post-ISA pore pressure was treated as the new back pressure, drainage valves opened, and the specimens were anisotropically reconsolidated from the post-ISA stress state back to the pre-ISA σ'_{vc} = 222 kPa. This post-ISA reconsolidation procedure was intended to simulate the recompression method [25, 55] for which laboratory shear specimens are consolidated to the estimated in-situ effective stress state (i.e., σ'_{vc} in the case of these laboratory simulations of tube sampling).

Fig. 15 presents the effective stress paths for the reference undisturbed specimen and specimens with ISA strain cycles of ±0.5%, ±1.0%, or ±3.0%. Nearly identical stress-strain behavior was measured during the initial shear phase for each ISA test, further confirming that the sample preparation procedure described in Section 3.1 and the triaxial test procedures were repeatable. The stress paths show that the development rate of shear-induced positive pore pressures, as well as cumulative pore pressure, increased with decreasing PI. The unloading total stress path that followed the initial peak positive ISA strain resulted in a significant excursion of the effective stress path for the lower PI soils toward a small p' with the failure envelope from the undisturbed CK_0UE/CAUE tests providing a lower boundary. By the end of the ISA strain cycle, the NP and PI = 4 specimens were at an effective stress state significantly lower than that for the PI = 31 clay specimens. The loss in mean effective stress relative to the pre-ISA mean stress $\Delta p'/p'_c$ for the largest ISA strain cycle of ±3.0% was 98% and 97% for the NP and PI = 4 specimens compared to only 37% for the PI = 31 clay. These results show a markedly different response of low PI intermediate soils to simulated tube sampling disturbance compared to clays (e.g., [65]).

Fig. 16 presents the post-ISA effective stress paths for the Fig. 15 specimens. All specimens showed a decrease in the initial prepeak stiffness, a decrease in strain softening response, and increases in s_u and the strain to failure ε_f with increasing ISA strain. The magnitude of these changes increased with decreasing PI. The PI = 31 clay specimens all exhibited contractive behavior throughout shear with little difference in $s_u = q_{max}$ for the different degrees of disturbance. In contrast, the NP soil exhibited significant dilative behavior in both the undisturbed state and after the simulated tube sampling disturbance. The most dramatic changes in behavior relative to the reference undisturbed specimen was found for the PI = 4 specimens. The simulated tube sample disturbance resulted in a complete reversal in the undrained shear behavior from contractive for the reference undisturbed specimen to highly dilative for even the lightly disturbed specimens. Such dramatic changes in behavior of this low PI intermediate soil have significant implications for selection of s_u for design.

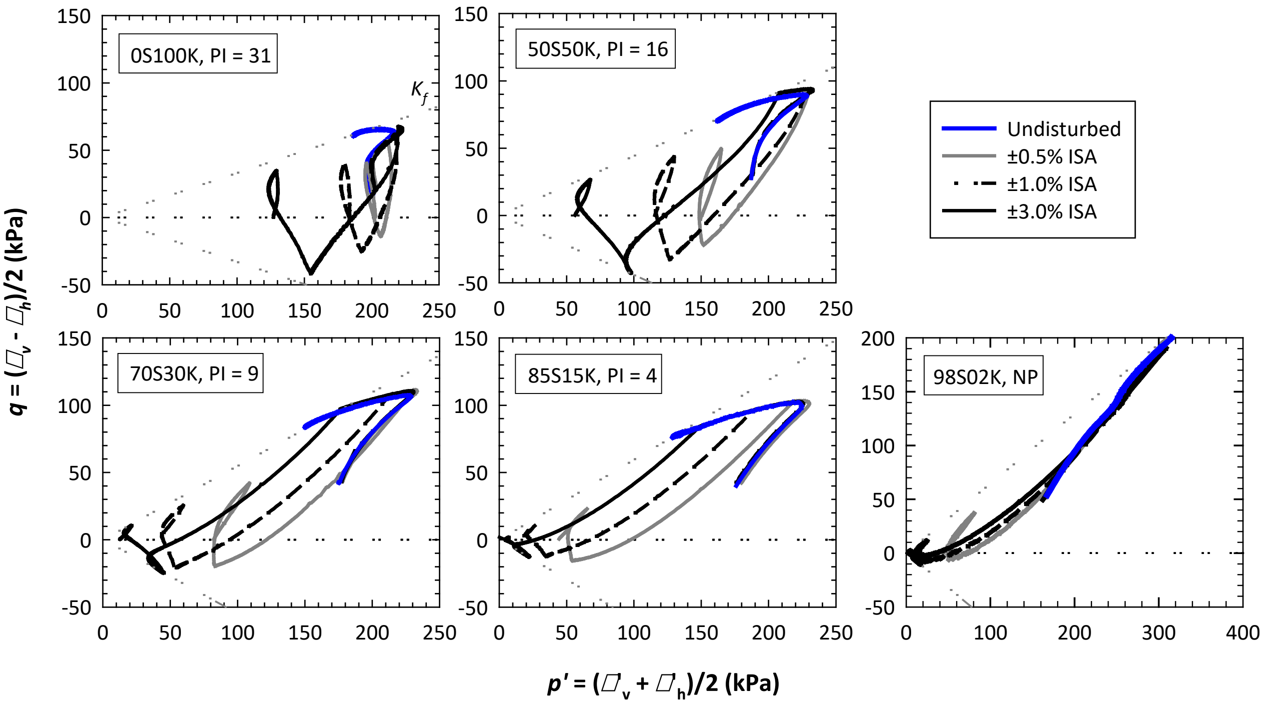

Figure 15. Effective stress paths for undisturbed CAUC tests and during ISA straining for synthetic reconstituted intermediate soils (after [15]).

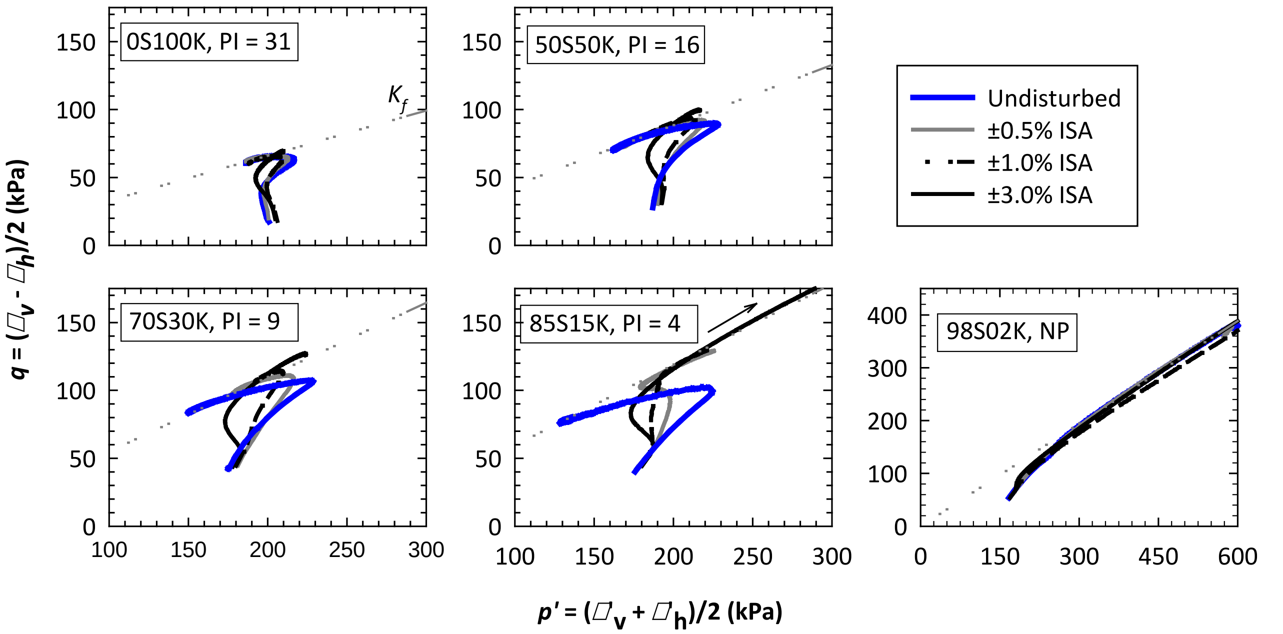

Figure 16. Effective stress paths for undisturbed CAUC tests and post-ISA undrained shear for synthetic reconstituted soils (after [15])

Lukas et al. [19] present results from a series of drained ISA tests conducted on the NP and PI = 4 and 16 soils. The tests were conducted in an identical manner to the undrained ISA tests except that the drainage lines were open during ISA shearing. The tests were performed fully drained to represent simulation of the extreme case of full drainage during tube sampling. The effective stress path during ISA by default for drained shear followed a 45° line in *q-p'* space which in some cases for the NP and PI = 4 soils reached the compression and extension failure envelopes during the strain cycle. This effective stress path during ISA shearing was completely different than that for the undrained ISA tests where significant loss in mean effective stress occurred (Fig. 15). Furthermore, the post-ISA reconsolidation volume change ε_{vol} or $\Delta e/e_0$ was negligible for the drained ISA tests; the majority of the volume change took place during ISA shearing and in the end the net volume change from start of ISA to end of post-ISA reconsolidation was larger for the drained ISA tests. The main difference in shear behavior between the undrained and drained ISA tests is that the latter specimens tended to have greater rate of strain hardening during post-ISA undrained shear, consistent with the greater ISA induced volume change.

3.4. Evaluation of sample quality

3.4.1. 1-D consolidation tests

As noted in Section 2.3 the $\Delta e/e_0$ method of evaluating sample quality is based upon data primarily from mostly moderate to highly sensitive marine clays with PI ranging from 5 to 55 and OCR between 1 and 4. For the dataset presented by [14] described in Section 3.3.1, the post disturbance $\Delta e/e_0$ values were dependent on PI as shown in Fig. 17. HD specimens with PI < 7 exhibited a sample quality rating of good to fair or better despite undergoing significant disturbance. In fact, the NP HD specimen produced a very good to excellent sample quality rating even though the specimen was destroyed during the freeze thaw process. These results confirm the caution of [49] in use of the clay-based $\Delta e/e_0$ criteria for soils outside the range of properties of their database.

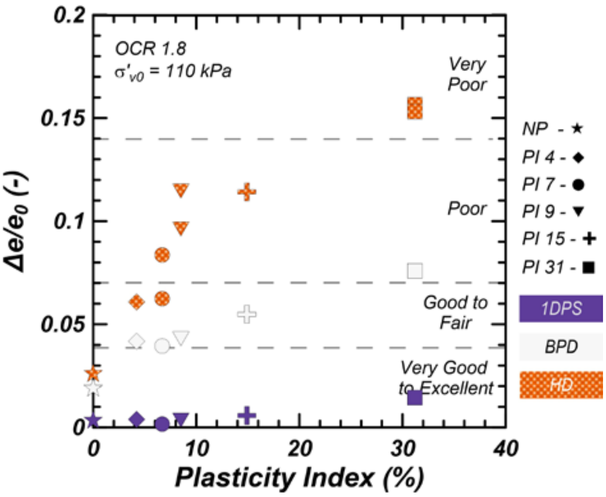

Figure 17. Sample quality trends with PI for synthetic intermediate soils using existing clay-based $\Delta e/e_0$ criteria (from [14]).

The data presented by [14] highlights the need for an alternative sample quality assessment that is applicable to a wider range of soils and in particular low PI intermediate soils. As such, [14] proposed a work-based approach for evaluating sample disturbance as shown in Fig. 18. The method is based on the work per unit volume, or strain energy method developed by [73] for estimating the preconsolidation stress for clays and is similar to that proposed by [74] using 1-D constrained modulus data. The [14] approach is based on the following observations from the full data set described in Section 3.3.1 that included variations in type of disturbance (1DPS, BPD, and HD), PI from NP to 31, and stress history (σ'_p < 1000 kPa and OCR between 1.2 and 3.8): 1) the strain energy-based index for recompression loading from the seating stress to σ'_{v0}, $C_{rw,i}$, tended to increase with an increase in degree of induced disturbance (e.g., HD vs 1DPS), 2) the strain energy-based index for virgin compression loading C_{cw} tended to decrease with an increase in degree of induced disturbance, 3) both $C_{rw,i}$ and C_{cw} increased with an increase in PI, and 4) C_{cw} was approximately constant for a given PI, σ'_{v0} and OCR indicating that it could be used to track the difference in the compressibility of the different soils tested. These observations suggested that the ratio $C_{rw,i}/C_{cw}$ could serve as a useful indicator of sample disturbance for a range of soil types as shown in Fig. 19. Furthermore, $C_{rw,i}/C_{cw}$ was also found to track the different degrees of induced sample disturbance independent of stress history (i.e., σ'_{v0}, σ'_p).

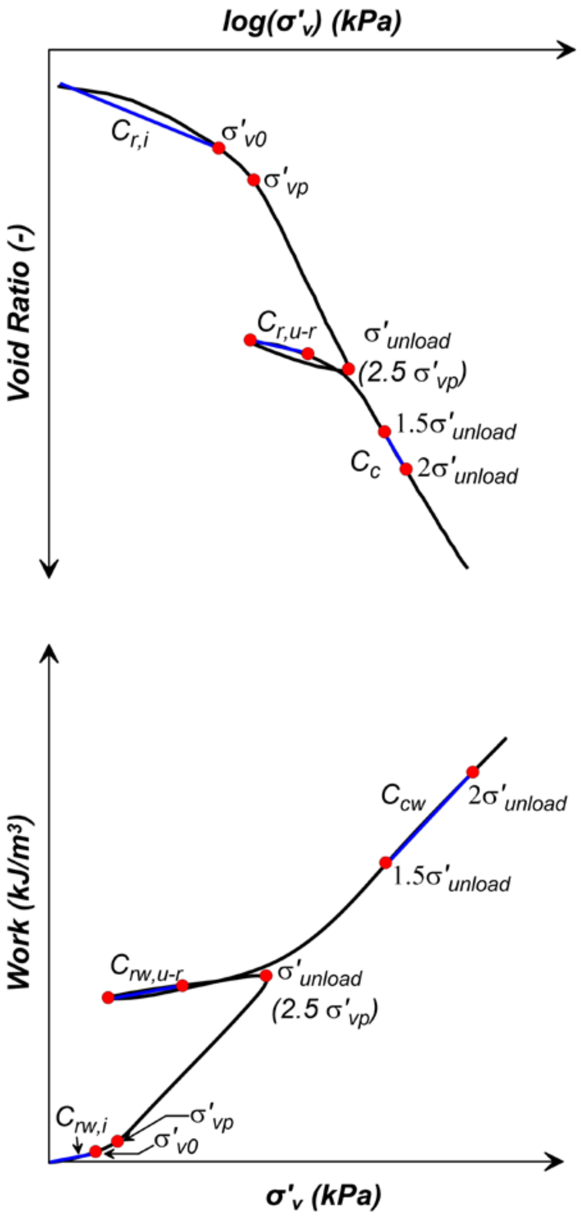

Figure 18. Stress ranges for defining compression indices (a) compression curve, and (b) strain energy plot (from [14]).

DeJong et al. [14] thus suggested the following criteria for assessing sample quality using $C_{rw,i}/C_{cw}$: 1) < 0.15 = High quality, 2) between 0.15 and 0.40 = Moderate quality, and 3) > 0.40 = Low quality. It is recommended that C_{cw} be evaluated over the stress range of $2.5\sigma'_p$ to $5.0\sigma'_p$ if an unload-reload (U-R) cycle was not performed or $1.5\sigma'_u$ to $2.0\sigma'_u$ if an U-R cycle was performed where σ'_u = the stress at which the U-R cycle was conducted (Fig. 18). Fig. 20 compares $C_{rw,i}/C_w$ and $\Delta e/e_0$ which shows that for the soils and test conditions used both methods accurately characterized the 1DPS specimens as very good to excellent $\Delta e/e_0$ quality or High $C_{rw,i}/C_w$ quality. However, the $\Delta e/e_0$ method ranked some the low PI HD specimens as very good to excellent whereas the $C_{rw,i}/C_{cw}$ method more accurately classified disturbance in these specimens. The $C_{rw,i}/C_{cw}$ method was further evaluated using a database created from prior studies of sample quality and general agreement was found between disturbance classifications using $\Delta e/e_0$ and $C_{rw,i}/C_{cw}$ for natural samples (Fig. 20).

DeJong et al [14] note that since strain energy is the integration of compression strains the trends observed using $C_{rw,i}/C_{cw}$ can be extended to e-$\log\sigma'_v$ space with use of $C_{r,i}$ and C_c calculated over the same stress ranges as in work space (Fig. 18) and using the same High, Moderate and Low criteria values listed above for $C_{rw,i}/C_{cw}$ and again with no apparent trend with PI and stress history.

3.4.2. CAUC tests

For the ISA tests presented in Section 3.3.2 values of $\Delta e/e_0$ were computed for post-ISA reconsolidation phase back to the pre-ISA consolidation state for each test. In all cases, $\Delta e/e_0$ increased as the ISA-imposed strain damage increased from ±0.5% to ±3.0%. Although all specimens had one of the two highest $\Delta e/e_0$ sample quality ratings of either very good to excellent or fair to good. This, as noted in Section 3.4.1, would be a misleading application of this clay-based sample quality rating especially for the ±3% ISA tests because the imposed ISA strains were well beyond the undisturbed peak shear strength for the plastic soils and the subsequently measured undrained shear behavior was markedly different for the PI = 4 and 7 specimens.

All the triaxial tests described in Sections 3.2.2 and 3.3.2 were conducted with vertical bender elements which were constructed following the method presented by [75]. The transmitting signal was generated with a Wavetek model 29 10 MHz Direct Digital Synthesis function generator with a single ±10 V amplitude sine wave trigged at a 10 Hz delay. The transmitted and received signals were both read using a Pico PC-based oscilloscope (Model 4226) with variable 12- or 16-bit resolution and up to 125Ms/s sampling rate and PicoScope 6 software. The received signals were averaged over time and a 30 kHz low-pass filter was applied. Arrival times were determined using the first zero crossover method [76] and corrected for the calibrated system delay.

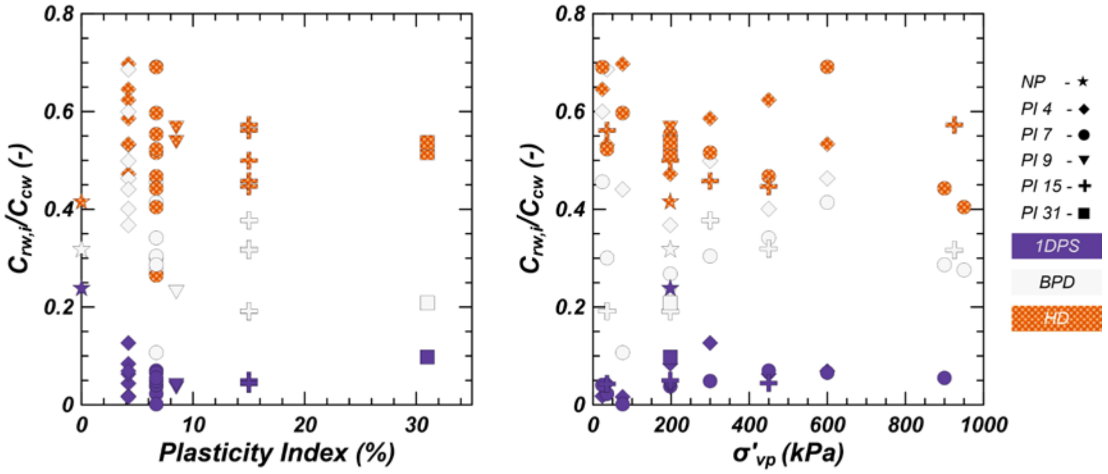

Figure 19. For all synthetic intermediate soils with 1DPS, BPD, and HD disturbance, $C_{rw,i}/C_{cw}$ versus (a) PI, and (b) σ'_{vp} (from [14]).

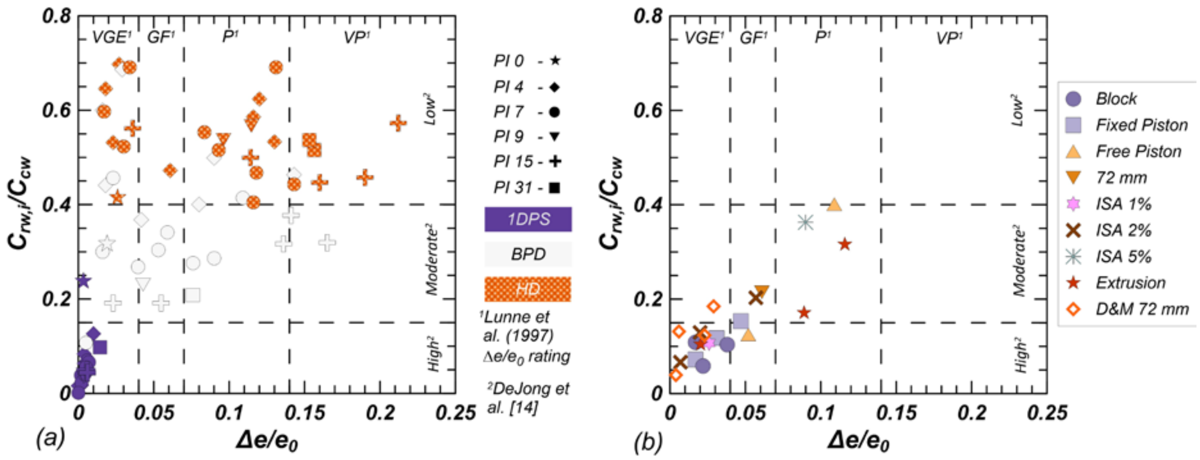

Figure 20. $C_{rw,i}/C_{cw}$ vs. $\Delta e/e_0$ using (a) reconstituted synthetic intermediate soils, and (b) historical data (from [14]).

To investigate the efficacy of the [50] V_{vh}-σ'^{2}_{s} framework as a means for evaluating the influence of tube sampling disturbance on a low plasticity intermediate soil the backbone V_{vh} versus $\sigma'_v\sigma'_h$ curve (Fig. 2) was determined. This was achieved by performing a CK_0UC test with multiple load-unload cycles to collect data on the interrelationship among e, σ'_v, σ'_h, OCR and V_{vh}. Fig. 21 plots data for a test on the PI = 4 soil and analyzed following the procedure outlined in [24]. V_{vh} values were converted to small strain stiffness $G_{vh} = V_{vh}^2\rho_t$ where ρ_t = total soil density. Fig. 21d represents the laboratory determined backbone G_{vh}-e-σ'^2 relationship for the PI = 4 soil and provides a frame of reference for mapping the effects of laboratory induced sample disturbance.

ISA tests were performed at pre-ISA OCRs = 1.8 and 3.6 and with ISA strain cycles of ±1% and ±3%. Fig. 22 plots the ISA ±3% effective stress paths which shows the significant reduction in mean stress during ISA straining for both the OCR = 1.8 and OCR = 3.6 specimens. Fig. 23 tracks V_{vh} at key stages of the tests and Fig. 24 plots V_{vh} and G_{vh}/e^{-m} versus $\sigma'_v\sigma'_h$ as measured 1) pre-ISA, 2) end of ISA, and 3) at end of post-ISA reconsolidation to the pre-ISA consolidated state. The results show a significant loss in V_{vh} due to the ISA straining which corresponds to the large reduction in $\sigma'_v\sigma'_h$. Upon post-ISA reconsolidation to the pre-ISA effective stress state both specimens essentially fully recovered G_{vh} or V_{vh} values. Fig. 25 plots the post ISA undrained shear behavior for the OCR = 3.6 specimens. As shown in Section 3.3.2 and Fig. 16 for OCR = 1.8 tests, the ±1.0% and ±3.0% ISA tests show remarkably different behavior than their reference undisturbed counterpart. In both cases, the specimens exhibit a limited initial contractive response, followed by significant dilative behavior, especially for the ±3.0% specimen. The differences between undisturbed and ISA disturbed for OCR = 3.6 shown in Fig. 25 are less but still significant.

The shear wave velocity-stress state data suggest that little to no destructuring occurred during ISA disturbance given that the end of ISA V_{vh} and G_{vh}/e^{-m} (Fig. 24) values plot on or close to the backbone curves despite the significant loss of $\sigma'_v\sigma'_h$. This is consistent with the samples all being young, reconstituted soils that are presumed to have little to no structure. Furthermore, during post-ISA reconsolidation G_{vh}/e^{-m} or V_{vh} essentially returned to the pre-ISA values, implying that the changes in the specimen state due to ISA disturbance were fully recovered. However, the subsequently measured undrained shear behavior was markedly different than that of the undisturbed reference behavior. These results, while based on only one soil, indicate that tracking V_{vh} may not be a robust indicator of sample quality for non-structured low PI intermediate soils. This topic is explored further for intact samples of a low PI silt in Section 4.3.

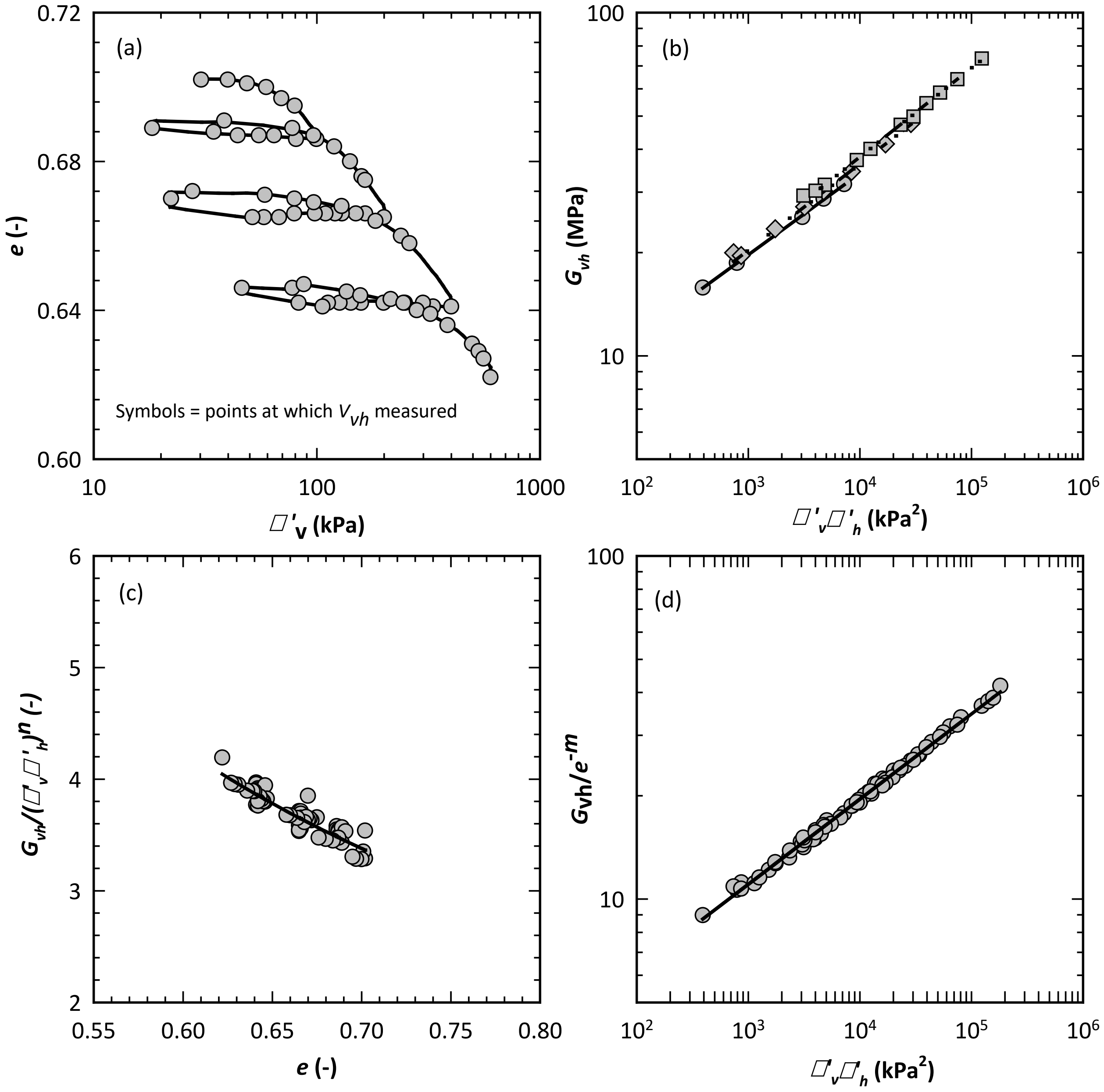

Figure 21. Development of shear wave framework for the synthetic reconstituted PI = 4 soil (a) shear wave velocity V_{vh} during K_0 consolidation loading and unloading, and data for determination of parameters (b) n, (c) m, and (d) S_{vh} and confirmation of n.

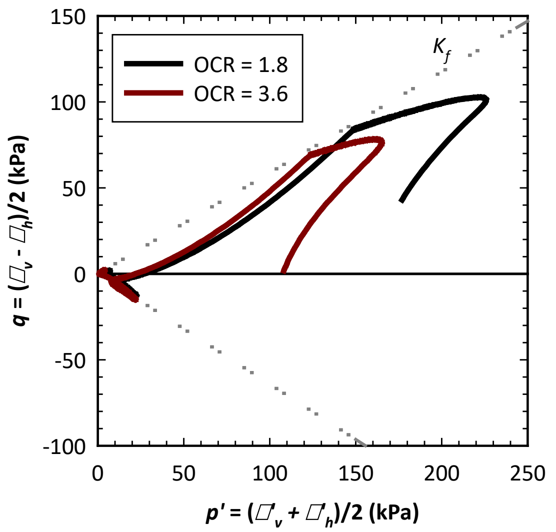

Figure 22. Effective stress paths during ±3% ISA straining for OCR = 1.8 and 3.6 synthetic reconstituted PI = 4 soil.

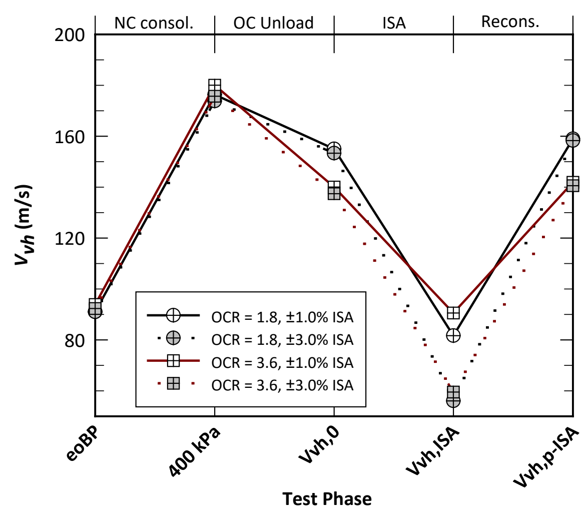

Figure 23. Evolution of V_{vh} at ISA test stages for the synthetic reconstituted PI = 4 soil (eoBP = end of back pressure, $V_{vh,0}$ = pre-ISA, $V_{vh,ISA}$ = end of ISA, and $V_{vh,p\text{-}ISA}$ = after post ISA reconsolidation).

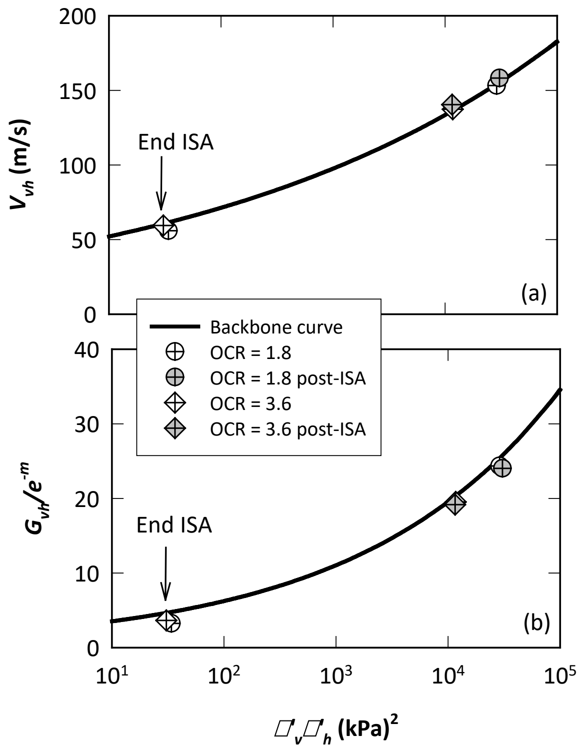

Figure 24. Backbone curves of (a) V_{vh} and (b) G_{vh}/e^{-m} versus $\sigma'_v\sigma'_h$ and pre-ISA, end of ISA, and post-ISA data from ±3% ISA tests with OCR = 1.8 & 3.6 for the synthetic reconstituted PI = 4 soil.

3.5. Findings from tests on low PI synthetic intermediate soils

The results presented in this Section from a variety of tests conducted on reconstituted intermediate soils confirmed some previous research and established new findings, some of which indicate completely different behavior from that of well establish norms for clays and sands. The newly developed reconstitution procedure enabled preparation of test specimens of varying PI in a reliable and repeatable manner which resulted in the measurement of repeatable and consistent stress-strain-strength-flow data across a range of PIs. Some key findings, with a focus on the low PI intermediate soils tested, include:

- it is often difficult or impossible to determine σ'_p using conventional clay-based methods even for good quality samples;
- small changes in silt content/PI can result in dramatic differences in undrained shear behavior that can change from contractive to dilative behavior;
- the clay-based ε_{vol} and $\Delta e/e_0$ sample quality criteria do not track sample disturbance even for highly disturbed samples as cautioned by [49];
- a new work-based sample quality criteria based on 1-D consolidation tests appears to track well the effects of sample disturbance independent of PI and stress history;
- simulated tube sample disturbance results in a significant loss of mean effective stress and can completely change the undrained shear behavior from a contractive response in the undisturbed state to a dilative response after disturbance; and
- shear wave velocity did not track well sample disturbance; once the sampling induced large reduction in p' was recovered upon reconsolidation, the corresponding large reduction in V_{vh} was also near fully recovered.

These findings highlight the challenges in sampling and laboratory characterization of low PI intermediate soils. In this work the synthetic soils tested were young, reconstituted soils that did not exhibit the sensitivity or structure that is common for some natural occurring soils resulting from various depositional and geologic stress history mechanisms such as cementation, aging, ground motion, etc. Furthermore, the sample disturbance induced was a laboratory simulation and may not necessarily fully represent the various sources and extent of disturbance that can occur in sampling of natural soils. Results from sampling and testing of intact samples of a natural low PI silt are presented in the following section.

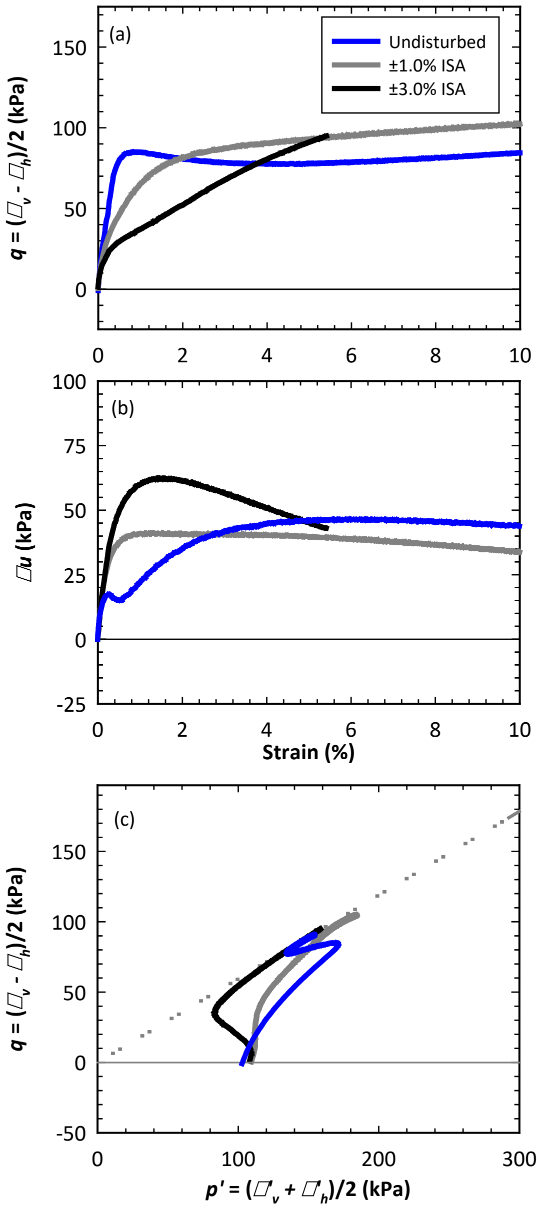

Figure 25. (a) Shear stress (q), (b) shear induced pore pressure (Δu), and (c) effective stress path during undrained shear for OCR = 3.6 reference undisturbed, ±1% ISA, and ±3% ISA synthetic reconstituted PI = 4 soil specimens.

4. Sampling and laboratory behavior of a natural low PI silt

4.1. Block and tube sampling

Sampling of the low PI Halden silt was conducted at the Halden, Norway research site which is one of five Norwegian National Geo Test Sites (NGTS). The primary deposit of interest at Halden consists of post-glacial, marine and fjord-marine sediments that are believed to be geologically normally consolidated. The soil profile consists of a silty sand down to about 4.5 meters depth followed by two primary silt units down to about 15 meters depth which are underlaid by a clay unit. Mean ground water table is located at approximately 2 m depth and in-situ pore water pressures are close to hydrostatic in the silt units. Blaker et al. [16] provides detailed site characterization data including geology, geophysical, in-situ testing and laboratory test data.

The focus of the sampling and laboratory results presented in the following sections is on the samples collected within the depth interval of approximately 11 to 12 meters [18]. Typical index and classification properties were water content w = 27%, fall cone liquid limit [77] w_L = 29%, plastic limit w_P = 21%, liquidity index I_L = 0.7, silt fraction (between 2 and 63 μm) = 89%, and clay fraction (% < 2 μm) = 9%. The plasticity index PI_{FC} = 8 is based on the fall cone liquid limit whereas the liquid limit determined using the Casagrande cup (ASTM D4318, [63]) $w_{L,CC}$ = 23% results in a PI_{CC} = 2. Such a difference between the fall cone and Casagrande cup liquid limits are common at this low end of liquid limits (e.g., [78]). The Casagrande plasticity data classify the Halden silt as a low plasticity silt ML in the Unified Soil Classification System (ASTM D2487 [63]).

Sampling consisted of Sherbrooke block sampling and four types of piston samplers with each sampler deployed in a dedicated borehole (Table 2): 1) NGI 54-mm ID composite piston sampler [79], 2) a 71 mm ID gel-push static (GP-S) sampler [80], 3) a 76 mm GUS hydraulic piston sampler (Acker Drill Company, Scranton, PA, USA), and 4) a 61 mm Dames & Moore (D&M) fixed piston hydraulic sampler (GeoMatic, San Bernardino, CA, USA). The NGI 54 mm sampler has a relatively poor geometry but has been used in research projects as a valuable frame of reference of poor-quality sampling (e.g., [49]). The GP-S was included in the research as this relatively new sampler has been used for projects investigating liquefaction potential of silty soils (e.g., [81-83]). It uses a core catcher and injects a water-soluble polymeric gel from the sampler shoe to lubricate and reduce friction between the sample and sampler wall.

The GUS and D&M samplers were included as they are used in USA drilling practice. The GUS sampler uses a standard Shelby tube (ASTM D1587 [63]) which for the sampling performed at Halden were constructed of galvanized steel. The standard Shelby tube has an ICR = 3% and a hunched cutting edge and were modified by removing the bottom 5-10 mm of the stock tube to create an ICR = 0 and machined in a lathe to create a straight cutting angle of about 20°. The Authors have used a shaper cutting angle of 5 to 10° for sampling in clays (e.g., [84]) but find this angle can be delicate. Furthermore, the Authors believe that the cutting angle is of secondary importance to sample quality compared to AR, ICR and especially use of a fixed piston. The GUS piston head uses two leather packers which were conditioned before drilling by setting the piston head inside a Shelby tube and placing the assembly into a bucket of water to allow the packers to undergo constrained swelling overnight. Issues with zero recovery with the GUS sampler is often due to use of old degraded packers or poorly conditioned packers resulting in the inability for suction to be maintained during sampler extraction. The stock D&M sample tubes were constructed of brass and have a good geometry with an ICR = 0 and AR <10% although the tube ID is somewhat smaller than the generally recommended minimum of approximately 72 mm [24, 25]. The D&M sampler is compact and relatively easy to handle during set-up and removal from the drill rig compared to the much longer, and heavier, GUS sampler system.

Table 2. Sampler dimensions and properties

Sampler	D_i (mm)	ID (mm)	OD (mm)	t (mm)	AR (%)	ICR (%)	Angle (°)
Block	-	-	~ 250	-	-	-	-
NGI 54 mm	54	54.3	65	5.5	45	0.6	5
GP-S	71.5	72.1	90/93	10.8	69	0.8	†
GUS	72.9	72.9	76.2	1.6	9	0	~20
D&M	61.4	61.4	63.5	1.1	7	0	~30

Notes: D_i = ID at cutting shoe, ID = inside diameter of sampler tube; OD = outside diameter of sampler tube, for GP-S cutting shoe is 3 mm larger than OD of sampler tube; t = sampler tube wall thickness; AR = area ratio = $(OD^2-D_i^2)/D_i^2$, ICR = inside clearance ratio = $(ID - D_i)/D_i$; angle = angle of the tube or cutting shoe if used. †cutting shoe does not have a straight cutting angle.

Mud rotary drilling is not commonly performed in Norwegian drilling practice and thus the block sampling was conducted using water as the cutting fluid. The NGI 54 mm sampler was directly pushed to the target sampling depth, i.e., full displacement sampler. GP-S sampling used a steel casing that was installed by a SONIC drill rig. Whereas for the GUS and D&M samplers, which are normally deployed in a mud rotary borehole, the drill string with a pilot bit was advanced to within approximately 100 mm of the target sample depth, retracted from the borehole, pilot bit removed, sampler attached, reentered into the borehole and pushed the final distance to the target sampling depth. The soil cuttings created during advance of the drill string were left in the borehole to form a drilling mud but without circulation. The NGI 54 mm sampler was rod activated while the GP-S, GUS and D&M were hydraulically activated using water pressure from the drill rig. During sampling, the GP-S core catcher is fully open and is triggered into the closed position at the end of the tube push. The NGI 54 mm and GP-S samples were left to rest for several minutes before retrieval. The GUS and D&M samplers were left to rest for 15 min and thereafter rotated trough two full rotations before retraction to the ground surface.

The Sherbrooke block samples were successfully collected down to a depth of 15.2 meters despite the low clay fraction and PI of the Halden silt. Although there were occasional problems with fine silt causing the lower cutting blades from being fully activated. It is likely that these soil properties are close to the lower limit of what is possible with this open type of sampler (i.e., the ability of the soil to maintain a minimal suction to keep the sample intact). The block sample from 11-12 m was considered in the laboratory test program as being the best representation of the undisturbed intact soil and the NGI 54 mm sample as being the most disturbed. The GUS and D&M samples were transported from the USA to Norway and the authors worked closely with the drillers in their deployment. Both samplers have several O-ring seals and the authors' standard practice is to dismantle the samplers between each sampling event to clean and repack the O-rings as fine flowable silt has the potential to compromise their effectiveness. This did not add significant time to the sampling program for which 6 GUS and 6 D&M samples, all with full recovery (610 mm for GUS and 457 mm for D&M), were collected in 1.5 days using separate boreholes for each sampler and down to a depth of 19 meters. The bottom ends of the tubes were sealed with layers of plastic foil, a flexible plastic cap and electric tape. The top ends of the tubes, with a gap between the top of the sample and the tube top, were sealed with a 50:50 mix of petroleum jelly and paraffin wax [85]. The samples were left in the sample tubes, wrapped in bubble wrap, packed in a shipping crate, and transported by commercial airline and surface courier to UMass Amherst.

4.2. 1-D Consolidation behavior

Fig. 26 presents results from two incremental load (IL) oedometer tests performed at NGI following the procedures outlined in [86] on the block sample [18] and CRS tests performed at UMass Amherst (ASTM D4186 [63]) on the GUS and D&M samples. The rapid consolidation during individual load increments did not lend themselves to log time or square root of time interpretation and the data plotted in Fig. 26 is for a constant time t_c = 4 minutes for each increment which was considered beyond end of primary consolidation. Volumetric strains of 1.3% and 1.4% at σ'_{v0} were measured for the two specimens which corresponds to $\Delta e/e_0$ of 0.031 and 0.032. Values of $C_{rw,i}/C_{cw}$ equal 0.16 and 0.20. The compression curves show no visual evidence of a σ'_p and while graphical procedures such as Casagrande's construction will produce a value of σ'_p such an interpretation is not considered reliable. The geologic history of the site [16] indicates that the deposit is geologically normally consolidated, although it is likely slightly overconsolidated due to aging processes. The lack of a distinct σ'_p highlights the challenge of characterizing such low PI silts within a stress history-undrained shear strength framework which is well-established and useful in practice for many clays as discussed in Section 2.4.

For reference Fig. 26 also plots the compression curve from a reconstituted specimen prepared as a slurry (SD) in a manner similar to the methods described by [17] and [60] in Section 3.1. The slurry was prepared at a water content between 1.5-2.0 times the fall cone liquid limit using soil cuttings from the block that had not been previously tested. The SD specimen ended up with an initial void ratio almost identical to that of the block sample specimens but thereafter was significantly more compressible and did not end up merging with the block sample compression curve within the maximum stress used in the tests.

The initial void ratio and compression curve for the GUS sample is essentially identical to that for the block samples with ε_{vol} = 1.6% and $\Delta e/e_0$ = 0.037 at σ'_{v0} and $C_{rw,i}/C_{cw}$ = 0.29 ($C_{r,i}/C_c$ = 0.22). Research has shown that clay samples can be successfully transported without loss of sample quality (e.g., [84]) but the good comparison between the block and GUS sample is noteworthy for such a low PI soil given the handling and transport required to ship the sample from Norway to the USA. The D&M sample has a lower initial void ratio but is more compressible than the block and GUS samples with ε_{vol} = 2.3% and $\Delta e/e_0$ = 0.055 at σ'_{v0} and $C_{rw,i}/C_{cw}$ = 0.28 ($C_{r,i}/C_c$ = 0.21). Koutsoftas [39] presents CRS compression curves for D&M samples of one NP and two PI < 7 soils and reports $C_r/C_{c,max}$ values all less than 0.10 for the samples.

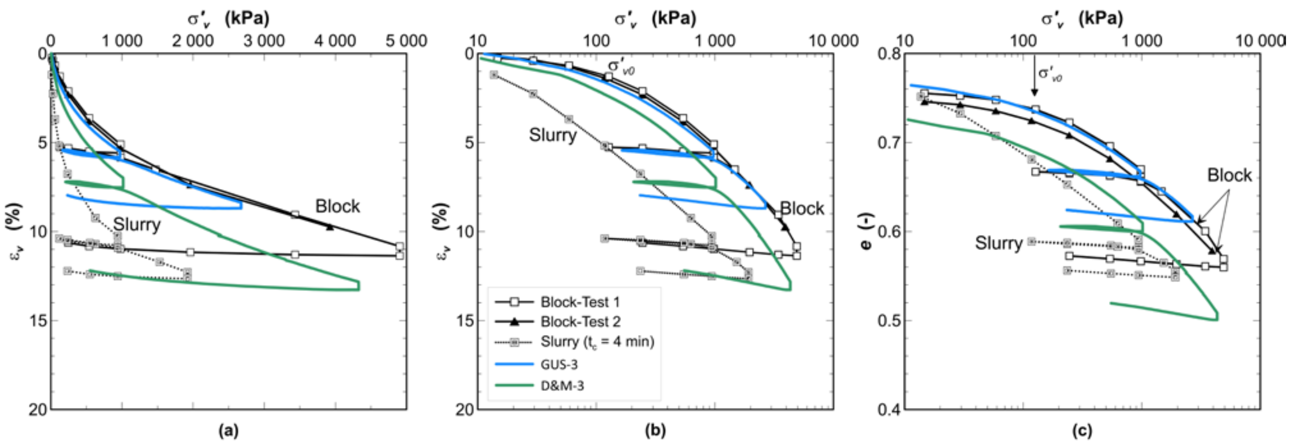

Figure 26. 1D consolidation of Sherbrooke block, GUS, D&M and reconstituted (slurry) Halden silt. Vertical effective stress versus vertical strain on (a) linear and (b) semi - log axis, and (c) void ratio versus log stress. (modified after [18]).

4.3. Undrained shear behavior

CAUC/E tests were performed on the block, NGI 54 and GP-S samples at NGI in general accordance with the procedures described in [87][1]. All specimens were anisotropically consolidated to σ'_{v0} and an estimate of the in-situ horizontal effective stress σ'_{h0} using K_0 = 0.5 [16]. Undrained shear was performed at an axial rate of 0.5%/h. Shear wave velocity V_{vh} was measured with bender elements using equipment similar to that described in Section 3.4.2. ISA tests were also performed on specimens trimmed from the block sample using the procedure described in Section 3.3.2 with ISA axial strain cycles of ±0.5%, ±1.0%, and ±3.0%. More details on the test program and results are presented in [18].

End of consolidation ε_{vol} and $\Delta e/e_0$ values for the seven CAUC/E tests performed on the block samples were in the relatively narrow range of 0.8% to 1.3% and 0.014 to 0.031, respectively. Likewise normalized shear wave velocity values measured at the end of consolidation $V_{vh}/V_{vh,SDMT}$ were in the range of 0.83 to 0.87, where $V_{vh,SDMT}$ was from in-situ downhole measurements using a seismic flat dilatometer [16]. Sample quality cannot be rated using the new [14] procedure as it requires 1-D normally consolidated compression data.

Fig. 27 plots results from the undrained shear phase of one the CAUC tests which exhibited an initial contractive type of behavior up to 1%-2% axial strain followed by significant tendency for dilative behavior. Both the CAUC and CAUE (not plotted in Fig. 27) tests migrated along a failure envelope with the same value of ϕ'_{mo} = 36°. Fig. 27 also includes the results from CAUC tests performed on NGI 54 mm and GP-S samples. Values of ε_{vol} and $\Delta e/e_0$ at the end of consolidation were 1.1% and 0.024 for the NGI 54 mm and 1.1% and 0.026 for the GP-S. These values are similar to that measured for the block samples and yet the undrained shear behavior is markedly different. Both specimens have a much greater rate of shear stress and negative pore pressure development with axial strain, although both arrive at the same failure envelope as the block sample specimens.

Fig. 27 also plots the post-ISA undrained shear behavior of the ISA tests performed on the block sample specimens with axial strain cycles of ±1.0% and ±3.0%. Similar to that measured for the low PI synthetic soil presented in Section 3.3.2, the specimens underwent a significant loss of mean stress p' and V_{vh} during the ISA straining with $\Delta p'/p'_c$ equal to 95% and 98% and $V_{vh,ISA}/V_{vh,0}$ = 0.56 and 0.41 for the ±1.0% and ±3.0% tests. Upon post-ISA reconsolidation to the pre-ISA effective stress state the ε_{vol} and $\Delta e/e_0$ values were low and V_{vh} fully recovered to the pre-ISA values. Yet the subsequently measured undrained shear behavior was very different for the ISA test specimens compared to the reference block sample specimen.

The data plotted in Fig. 27 for the poor area ratio NGI 54 mm and GP-S tube samplers and the ISA strain damaged block samples contain trends expected for significant tube sampling disturbance. The effect appears to be greatest for the NGI 54 mm sampler (as expected) although the GP-S has a poorer geometry and perhaps some compensation occurred due to the reduction in friction from the polymer gel. It is, however, evident that no matter the degree of disturbance, there is no effect on the location of the failure envelope.

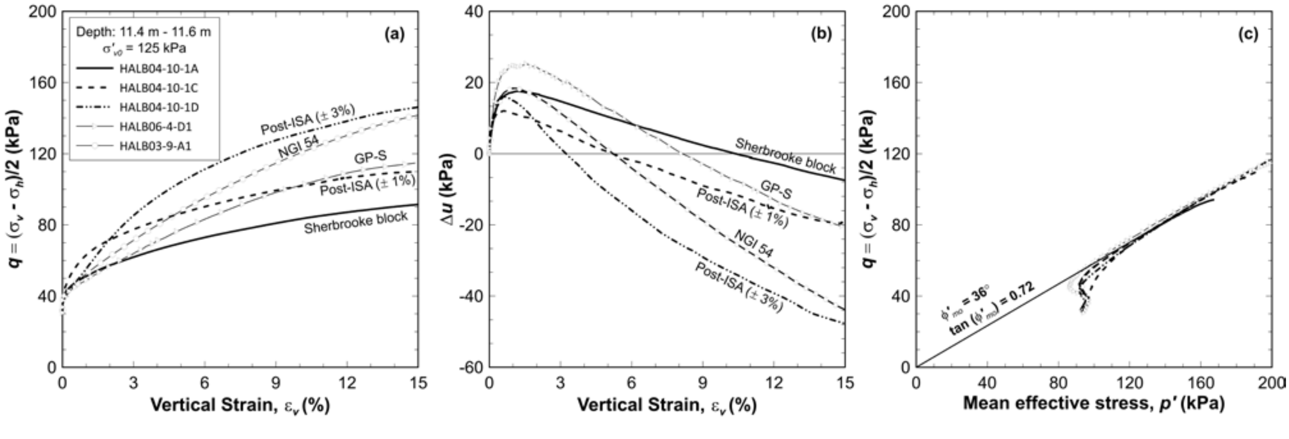

Figure 27. Effect of simulated ISA disturbance and true sample disturbance on undrained shear behavior. (a) Stress – strain, (b) pore pressure - strain, and (c) stress – path (from [18]).

4.4. Selection of Design s_u

Fig. 28 plots the CAUC results for tests conducted on the block, NGI 54 and GP-S samples together with an indication of the six undrained shear strength criteria presented by [35] as listed in Section 2.5. The differences in s_u among the various criteria for a given sampler type is very large and for some of the criteria the difference in s_u between the block sample and the more disturbed tube samples is also very large. The lower bound for all samples is Criterion 6 (u_{max}) as the specimens have not yet started exhibiting dilative behavior. Criterion 3 with the selection of a representative design value for A_f (e.g., 0.0 or 0.25 or 0.5 etc.) will result in essentially the same s_u value for the three samples since they all converge onto

[1] CAUC/E tests on the GUS and D&M samples planned for fall 2019 were initially delayed and then further delayed due to Covid-19 lockdown. Regrettably, the results were not available in time for preparation of this paper.

the same failure envelope. For the other criteria, s_u for the tube samples are as much as 150% greater than that of the block sample and in the extreme case for the combination of the three different samplers and the six s_u criteria the selected s_u can range from about 50 kPa to 120 kPa (tabulated data provided in [18]). The results from the different samplers confirm the findings from the ISA tests; the s_u estimates increase with increasing magnitude of ISA induced strain for all but the u_{max} and $A_f = 0$ criteria (Fig. 29). This implies that undrained triaxial testing of tube sampled silt specimens can lead to selection of an artificially high undrained shear strength for design.

It is evident that selecting the relevant s_u for design requires assessing the field loading regime and the likely drainage conditions (an often-challenging task). Furthermore, even if good quality samples are tested this challenge is exacerbated by the fact that there is limited research on what is an appropriate s_u for a given design scenario, as noted in Section 2.5. At a minimum, the $A_f = 0$ criterion provides a valuable reference s_u that would be equal to the drained shear strength. For strongly dilative soils like the Halden silt any strength criterion that corresponds to $A_f < 0$ requires careful consideration unless higher values of undrained shear strength are conservative, e.g., for extraction assessments, skirt penetration, pile driving, etc. For stability problems, lower values of s_u are more conservative and consideration should be given to estimated field strain levels and pore pressure dissipation conditions.

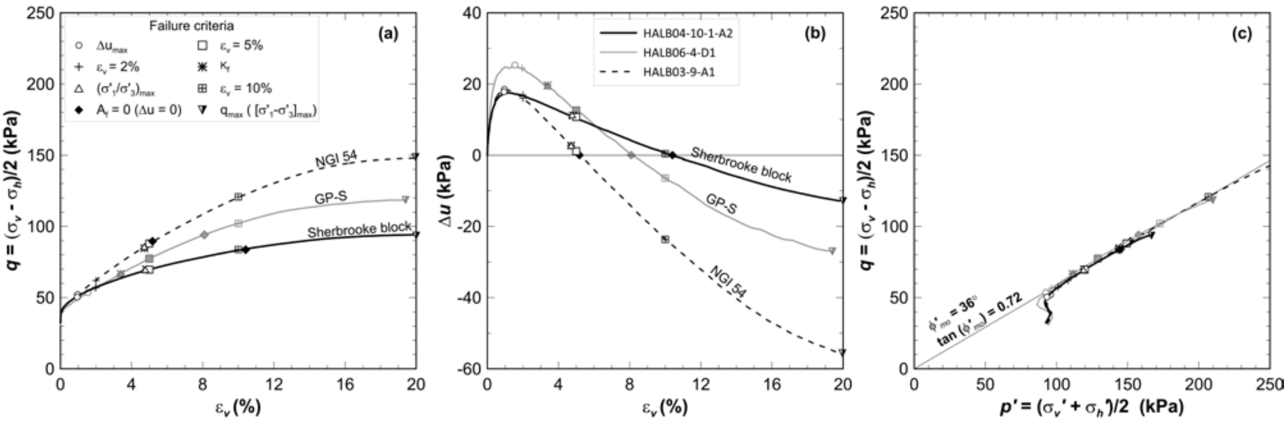

Figure 28. Undrained shear strength criteria [35] illustrated for CAUC tests on three types of Halden silt samples (NGI 54, GP-S and Sherbrooke block). (a) Stress – strain, (b) pore pressure - strain, and (c) stress – path (from [18]).

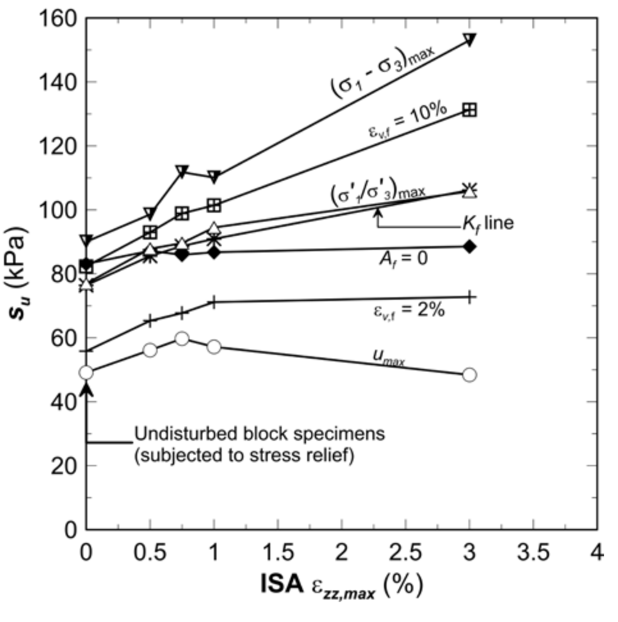

Figure 29. Effects of ISA simulated sampling disturbance on selection of undrained shear strength from CAUC tests on Sherbrooke block samples of Halden silt for various criteria (from [18]).

5. Conclusions

The results presented in this paper from tests conducted on synthetic intermediate soils provided a sound framework for studying the fundamental stress-strain-strength behavior of these soils. It provided insight into the effects of simulated sample disturbance on their behavior and assessment of the degree of disturbance. This foundational knowledge guided the conduct of a sampling and advanced laboratory test program conducted on the PI = 2 Halden silt and in interpretation of results. Some key findings from testing this natural silt include:

- it is possible to collect intact Sherbrooke block samples of a PI = 2 soil with 89% silt and 9% clay;
- 1-D compression curves on the good quality block samples show no visual evidence of σ'_p;
- ε_{vol} and $\Delta e/e_0$ at σ'_{v0} were similar for the block and tube samples and yet samples from the poor geometry NGI 54 mm and GP-S samplers had markedly different undrained shear behavior with a higher shear stress at all strain levels;
- shear wave velocity did not track sample disturbance;
- ISA tests performed on the block sample provided supporting evidence of the likely disturbance induced during sampling for the poor geometry NGI 54 mm and GP-S samplers;

- the GUS sampler with a good tube geometry did result in a 1-D compression curve that was essentially identical to that measured for the block sample;
- the D&M sampler, which also has a good tube geometry but smaller ID, produced what appears to be a somewhat more disturbed sample compared to the block and GUS samples;
- sample disturbance had no effect on measurement of the effective stress failure envelope (ϕ'_{m0}) and thus has no effect on the A_f criterion for selection of s_u;
- otherwise, large differences in possible s_u for design result from the dilative nature of the Halden silt and is magnified by tube sample disturbance, and;
- if s_u is required for design, selection of a representative value is highly dependent on the state of the laboratory test specimens, strength criterion, and the design application, i.e., whether lower-bound or higher-bound values are required and the likely field drainage conditions.

Research is currently ongoing in evaluating the behavior of samples collected using the GUS and D&M samplers and use of screw plate load tests [20] to evaluate the in-situ behavior of the Halden silt relative to that measured in the laboratory consolidated shear tests.

Acknowledgements

The Authors are grateful to several funding agencies that supported various portions of the work presented herein. They include the US National Science Foundation under grants CMMI-1436617, CMMI-1436793, CMMI-1138203, and CMMI-1300518; the California Department of Water Resources (contract 4600009751); the Norwegian Geotechnical Institute; the Research Council of Norway (RCN) through project Norwegian GeoTest Sites (NGTS) Grant No. 245650; and the Norway-America Association's (NORAM) Graduate Study and Research Scholarship Program. Any opinions, findings, and conclusions or recommendations expressed in this article are those of the authors and do not necessarily reflect the views of these agencies.

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