Geotechnical Properties of Cemented Sands in Steep Slopes

Brian D. Collins[1] and Nicholas Sitar[2]

Abstract: An investigation into the geotechnical properties specific to assessing the stability of weakly and moderately cemented sand cliffs is presented. A case study from eroding coastal cliffs located in central California provides both the data and impetus for this study. Herein, weakly cemented sand is defined as having an unconfined compressive strength (UCS) of less than 100 kPa, and moderately cemented sand is defined as having UCS between 100 and 400 kPa. Testing shows that both materials fail in a brittle fashion and can be modeled effectively using linear Mohr-Coulomb strength parameters, although for weakly cemented sands, curvature of the failure envelope is more evident with decreasing friction and increasing cohesion at higher confinement. Triaxial tests performed to simulate the evolving stress state of an eroding cliff, using a reduction in confinement-type stress path, result in an order of magnitude decrease in strain at failure and a more brittle response. Tests aimed at examining the influence of wetting on steep slopes show that a 60% decrease in UCS, a 50% drop in cohesion, and 80% decrease in the tensile strength occurs in moderately cemented sand upon introduction to water. In weakly cemented sands, all compressive, cohesive, and tensile strength is lost upon wetting and saturation. The results indicate that particular attention must be given to the relative level of cementation, the effects of groundwater or surficial seepage, and the small-scale strain response when performing geotechnical slope stability analyses on these materials.

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CE Database subject headings: Sand; Soil cement; Slope stability; Triaxial tests; Cliffs; Soil properties.

Introduction

Steep slopes and cliffs formed in cemented sands often pose either a geologic hazard or an engineering challenge. Although at the stronger end of the cemented sand spectrum (i.e., sandstone), material behavior is dictated by rock mechanics principals, at the weaker end of the spectrum, cemented sands behave neither completely as a soil or a rock. Thus, they provide their own set of challenges with regard to slope stability assessments and engineering design. When exposed in cliffs for example, it is often unclear whether to depend on the cohesion element of shear strength, due both to the often weak nature of the contributing cementation bonds and the possibility of cementation degradation due to environmental factors (e.g., groundwater seepage). Further, both tensile and shear failure modes are often possible, requiring accurate assessment of their strengths. In hazard evaluation or engineering mitigation of cemented sand slopes, the need for a thorough understanding of their geotechnical parameters cannot be overemphasized.

Cemented sands describe a wide range of materials, even at the weaker end of the spectrum. They represent the transitional environment between soils and rocks, often with characteristics of both (Sitar 1983). At one extreme are locked sands that derive their strength from the intimate interlocking geometry of particle contacts and otherwise lack actual cementation, as described by Dusseault and Morgenstern (1979). At the other extreme are carbonate sands where the particles and cementing agent are identical, making a clear distinction between cementing action and intergranular interaction difficult. Many classifications schemes have been proposed, very often dependent on the dry unconfined compressive strength (UCS) [e.g., Shafii-Rad and Clough (1982); Barton (1993)], but with some variability to the lower bound defined as the weakest end member [very weakly cemented with UCS <100 kPa by Shafii-Rad and Clough (1982); slightly cemented with UCS <500 kPa by Barton (1993)].

All cemented sands share many similarities—among them, the pronounced tendency to form steep natural and cut slopes. An additional important common characteristic is that cemented (and locked) granular soils are capable of resisting compression and shear forces similar to uncemented sands but can also withstand at least some minimum tensile stress due to cohesion (cementation and/or interlocking) effects. While most attention has been directed at cementation, the fabric (i.e., particle orientation, particle shape, and packing) also plays an important role in a soil's relative strength and slope stability [e.g., Sitar (1983), Richards and Barton (1999), Martins et al. (2005)]. Regardless of the origin of the cohesive component, the end effects are the same: an ability to form slopes both steeper and taller than their uncemented counterparts, and likewise, a tendency to fail in dramatic, brittle collapse upon excessive loading. Given the obvious utility of understanding and predicting this type of behavior for development and infrastructure located on or near slopes composed of cemented sands, there is still a need for a more generalized understanding of the response of these materials to various changes in stress. These include the reaction of steep slopes to lateral unloading and/or surficial saturation, which are investigated from a material strength perspective in this paper.

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Overview of Prior Research on Cemented Sands

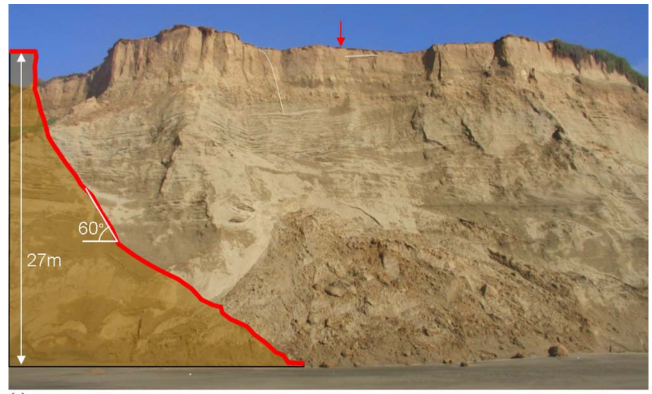
The properties of naturally and artificially cemented granular materials have received periodic interest by the geotechnical community over the past several decades in a variety of contexts and locations. These include studies by Saxena and Lastrico (1978) on naturally cemented sands, Horikawa and Sunamura (1968) on artificially cemented sands, and Sitar (1979) and Clough et al. (1981) on both naturally and artificially cemented sands. Studies aimed at the role of the cementing agent include those by Airey (1993) on carbonate cemented sands in Australia, O'Rourke and Crespo (1988) on volcanoclastic deposits in Ecuador and Columbia, and Barton and Cresswell (1998) on clay and ferruginous cemented sands in England.

Previous research on the geotechnical behavior of cemented sands includes that on their compression, tensile and shear strength (Saxena and Lastrico 1978; Clough et al. 1981; O'Rourke and Crespo 1988; Lade and Overton 1989; Airey 1993; Das et al. 1995; Huang and Airey 1998; Richards and Barton 1999; Fernandez and Santamarina 2001; Schnaid et al. 2001; Cresswell and Barton 2003), fracture behavior (Sture et al. 1999), and constitutive behavior (Reddy and Saxena 1992; Rumpelt and Sitar 1993; Vatsala et al. 2001). Despite the range of lithologic provenance, sampling, and testing conditions in all these studies, several consistencies are found throughout this body of research. First, the behavior of most cemented soils appears to be quite similar regardless of the particular cementing agent (calcareous, siliceous, argillaceous, ferruginous, etc.). In general, linear Mohr-Coulomb strength envelopes apply over typical stress ranges. Friction angles are similar to those obtained for uncemented sand materials whereas cohesion is generally a function of both the cementing agent and the angularity of the particles that provide bonding surfaces. Brittle failure occurs at low confining stresses and at low strain levels (on the order of 0.5–2%,) with increasing ductile response at higher confinement. Finally, when tensile strength has been tested, it is on the order of 10% of the UCS.

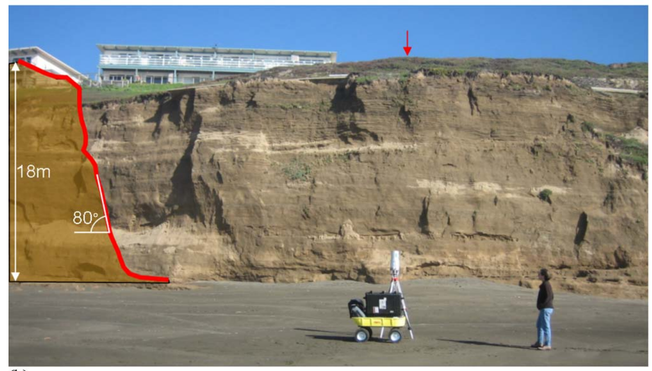
The Clough et al. (1981) study was among the first performed on the cemented sands in the central California coast region, where the present research was undertaken. These studies included those performed by Sitar et al. (1980), Bachus et al. (1981), and Shafii-Rad and Clough (1982). Later studies in this area were also performed by Wang (1986) and Hampton (2002). Clough et al. (1981) identified several key properties of cemented sands, among them, their brittle behavior at low confining stress, the rapid volumetric increase that occurs upon shearing, and the combined roles of cementation and particle interlocking that contribute to shear strength. As this was one of the first detailed studies on the subject, several characteristics now recognized as important to understanding the soil behavior in a steep slope setting were not investigated. These include the role that the stress state of an evolving slope geometry has on shear strength and the response of the compressive, shear, and tensile strengths due to saturation. Likewise, very few of the previously referenced papers present results in the framework specifically applicable to understanding the role of material strength in steep slope and cliff settings. These subjects form the basis for the present investigation.

Failure Modes and Stress Paths of Evolving Slopes and Cliffs

Cemented sand slopes may evolve under a variety of failure modes. Very often, excavation or toe erosion [Fig. 1(a)] lead to shear failures at inclinations paralleling the slope surface (Sitar 1983; Collins and Sitar 2008). However, erosion by waves is not the only possible mechanism—toe erosion may also occur from human agents such as mining related excavation in quarry pits (Barton and Cresswell 1998). In more indurated, moderately cemented sands, other failure modes are possible, including tensile-strength-loss exfoliation due to fluctuating groundwater seepage conditions [Fig. 1(b)]. The slope geometry in this case must be sufficiently steep to generate tensile stresses in the bluff face, typically upwards of 70° (Sitar and Clough 1983). The difference in geometry, and likewise, failure modes between weakly and moderately cemented sand slopes can be examined using high-resolution survey techniques, such as terrestrial lidar surveying [Fig. 1(b)]. Measurements made through this technique (Collins and Sitar 2002, 2005, 2008) are instrumental in showing, for example, that failures in weakly cemented slopes [Fig. 1(a)] generally occur at shallower slope inclinations compared to moderately cemented slopes [Fig. 1(b)], precluding the development of tensile stresses in those slopes. This type of data also shows that overhangs, which often form at the cliff toe in coastal environments and especially in more strongly cemented sands, are completely absent in weakly cemented sands—their low resistance to both wetting and vertical loading (i.e., existing overburden) prevent cantilevered sections in a coastal environment (Collins and Sitar 2008).



(a)



(b)

Fig. 1. Examples of: (a) weakly cemented sand cliff with toe erosion and shear failure mode; (b) moderately cemented sand cliff with tensile-exfoliation failure mode. Terrestrial lidar data collection [foreground in (b)] provides resultant overlain, high resolution cross sections at location of arrows.

Despite the different possible failure modes, most, if not all, forms of cliff erosion lead to a reduction of confining stress on the face of the cliff. For example, with any form of toe erosion, subsequent failures of the cliff result in the removal of material

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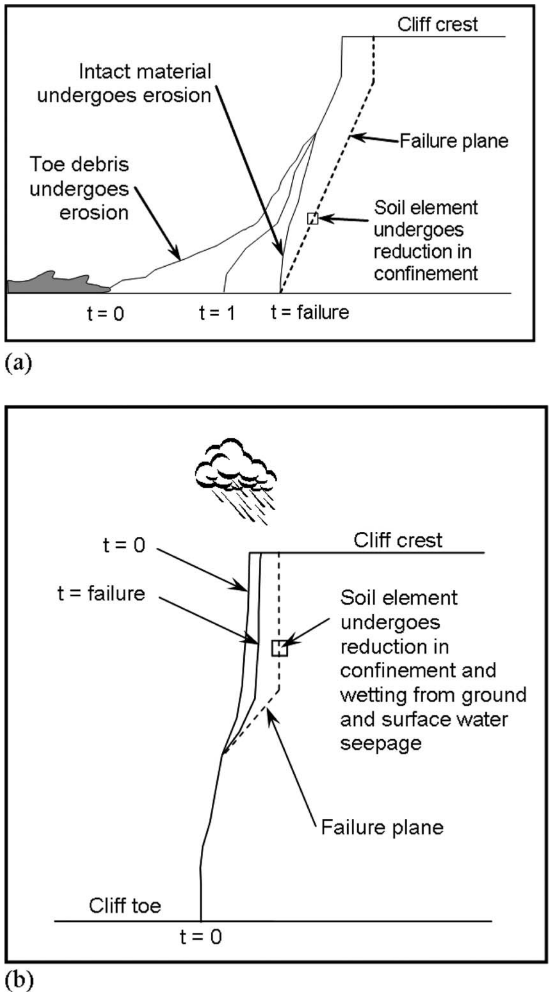

Fig. 2. Idealized models of slope evolution over time (t) for (a) weakly cemented; (b) moderately cemented sand cliffs

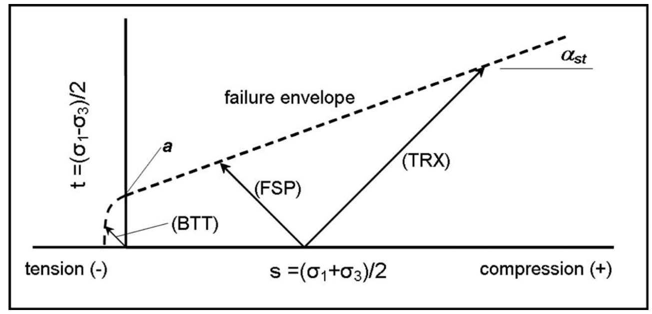

Fig. 3. Stress path plot for FSP loading, conventional triaxial stress path (TRX), and Brazilian tensile test (BTT) stress path in s-t stress space, showing slope (α_{st}) and intercept (a) used to obtain Mohr-Coulomb strength parameters

from the cliff face [Fig. 2(a)]. This results in a reduction of the lateral confining stresses previously provided by the face material. Direct exfoliation of cliff material is also an example of this process, and while often linked to at least some form of additional triggering mechanism [e.g., groundwater seepage—Fig. 2(b)], exfoliation may occur in the absence of any trigger, due simply to the reduction in existing confinement from previous failure.

The stress path taken by an internal soil element in these situations is not often simulated in geotechnical tests; typically the confining stress is maintained with an increase in vertical loading. However, for eroding cliffs, analysis of slope stability is more appropriately performed using strength parameters obtained from a confinement reduction stress path. This field condition results in unloading in the horizontal direction and a constant stress response from overburden in the vertical direction. In terms of stress path testing, the major principal stress, σ_1, remains constant while the minor principal stress, σ_3, is reduced. Using the stress parameters s and t as outlined by Parry (1995)

$$s=(\sigma_1+\sigma_3)/2 \tag{1a}$$

$$t=(\sigma_1-\sigma_3)/2 \tag{1b}$$

results in the stress paths are shown in Fig. 3.

The stress path followed by a soil element subjected to constant σ_1 and decreasing confinement still follows a compression-type stress path, but in a direction opposite to conventional triaxial compression to reach a state of failure described by the linear envelope with intercept ***a*** and inclination α_{st} (Fig. 3). These can be easily converted to typical Mohr-Coulomb parameters c and φ through algebraic manipulation of the stress coordinates. Lambe and Whitman (1969) identified this type of loading as "lateral extension," Parry (1995) as "triaxial compression with constant axial stress," and Dehler and Labuz (2007) as "compression unloading;" it is referred to herein as the field stress path (FSP) to distinguish it from the conventional constant σ_3 compressive stress path. Sitar et al. (1992) and Anderson and Riemer (1995) performed tests with a similar stress path (also called the FSP) by increasing the relative pore-water pressure within the sample. These tests showed that a state of collapse (sudden increase in volumetric strain) could be achieved in sands and clayey silts through rapid dilation at failure. Other stress-path specific testing on bonded soils (Malandraki and Toll 2001) has shown that the resulting Mohr-Coulomb strength parameters are very similar between those obtained from conventional triaxial tests and those from other, more complicated stress paths, including variations of the proposed FSP. However, a decrease in axial strain and increase in volumetric strain at failure and more brittle response is also obtained (Collins 2004). This is similar in many respects to overconsolidated soil behavior, and may partially explain the sudden collapse of cemented sands at failure often observed in the field (Collins and Sitar 2008).

Methods

The steep-slope behavior of weakly and moderately cemented sands was investigated through geotechnical sampling and laboratory testing from a field area that is subject to repetitive cliff failures. The area, located along the northwestern coast of the San Francisco Peninsula in central California, is composed of variably cemented sand marine terrace deposits, and has been extensively studied, including geological mapping, observational field work, and analysis of slope failures (Collins and Sitar 2008). In many respects, it is representative of numerous other areas of the west coast of the United States and elsewhere where steeply sloping cemented sand deposits exist.

Sampling of Cemented Sand

Geotechnical sampling of cemented sands can be difficult, owing to the light cementation that governs internal stability and, like-

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Fig. 4. Sample locations in weakly and moderately cemented cliffs near San Francisco, California. Height of cliff ranges from 27 to 18 m moving north to south (left to right) across image. Dashed line delineates weakly cemented sand (below) from moderately cemented sand (above). Image copyright 2002–2008 Kenneth and Gabrielle Adelman, California Coastal Records Project, www.californiacoastline.org.

wise, collapse. Reconstituted samples are generally not acceptable for strength testing because they cannot replicate the original fabric of the deposit and hence cannot replicate the apparent cohesion of the in situ materials. Samples obtained by conventional geotechnical drilling methods undergo extensive disturbance compared with results from hand-carved block sampling (Bachus et al. 1981) and generally result in higher densities and water contents, a more ductile stress strain response, and a lower shear-strength envelope. Because of these limitations, artificially cemented samples are commonly used (e.g., Clough et al. 1981; Lade and Overton 1989; Reddy and Saxena 1992; Das et al. 1995; Huang and Airey 1998; Fernandez and Santamarina 2001). Whereas these do mimic the stress-strain behavior of the natural materials, they do not adequately reproduce the internal fabric and interlocking of the grains. Hence, the only sampling method that stands a chance to maintain the internal fabric is hand-carved block sampling, which was selected as the preferred sampling method for this study. Whereas the process is time consuming, the results are more consistent with true field conditions. In the past this approach has been selected by Frydman et al. (1980), Clough et al. (1981), O'Rourke and Crespo (1988), Richards and Barton (1999), and Dittes and Labuz (2002), among others.

Samples were extracted from the face of two, unclassified, late Pleistocene, sea-cliff outcrops composed of paleo–dune, beach, and alluvial deposits (Smith 1960; Brabb and Pampeyan 1983; Collins 2004) located south of San Francisco, California, approximately 3 m above beach level (Fig. 4). The two outcrops consist of a weakly cemented uniform sand in a unit approximately 24 m tall capped by up to 3 m of moderately cemented sand, and a moderately cemented, uniform sand in a unit approximately 18 m tall. Herein, weakly cemented sand is defined as that with an UCS below 100 kPa, in contrast to moderately cemented sand with UCS above 100 kPa but below 400 kPa. These categories are referenced to a modified form of those defined by Shafii-Rad and Clough (1982) and are consistent with differing cliff failure modes observed between the materials (Collins and Sitar 2008).

Degraded surface materials were first scraped away to expose a fresh surface a few tens of centimeters into the cliff. Hand tools, including shovels and spatulas were then used to carve 0.3-m^3 block samples from the face. Samples were wrapped in plastic to preserve the in situ moisture content and were transported to the laboratory while maintaining the vertical orientation of the sample to minimize failure along horizontal bedding planes. Each sample was hand carved using a succession of finer steel blades to trim the blocks to 7.1-cm-diameter, 15.2-cm-tall cylinders (i.e., conventional 2.8-in by 6-in specimens).

Laboratory Testing

The laboratory testing program included standard index testing (ASTM 2004a,b,c,e), triaxial testing, and Brazilian tensile testing at various states of saturation. Triaxial testing included unconfined tests (ASTM 2004g), conventional compression tests, and the FSP tests described previously. In total, 18 triaxial tests were performed on the weakly cemented sand, whereas 12 triaxial and 4 Brazilian tensile tests were performed on the moderately cemented sand. In some cases, triaxial tests were performed under suction (vacuum) to attain low relative confinement levels (referred to as "TRXv" tests). No tensile tests could be performed on the weakly cemented sand due to the extremely weak structure of this material. Further, tests on weakly cemented sand were only performed at the in situ water content—they collapsed when introduced to water. Tests on moderately cemented sand were performed at both in situ and wetted (soaked for several minutes within the triaxial cell) water content to mimic typical storm field conditions (heavy precipitation, surface, and groundwater runoff), although FSP tests were not performed at the in situ water content due to the focus of the investigation on the wetted-dominated response of this material.

All tests were performed under drained conditions using an electronic stress-controlled triaxial testing device; volume change was measured for those samples performed under wetted conditions. In the FSP tests, the incremental reduction in confinement (σ_3) was counterbalanced in the axial direction by steadily increasing the axial stress by an identical amount. Brazilian tensile tests were conducted with modifications to the ASTM (2004d) test for rock specimens and ASTM (2004f) test for concrete specimens as described by Collins (2004). In these tests, a bottom platen constructed of foam with a thin steel rod insert and a top platen consisting of a curved, rigid, plastic hemicylinder was used to account for the highly delicate nature of the samples.

Table 1. Geotechnical Index Properties of Weakly and Moderately Cemented Sand from Pacifica, California

Material	USCS desig.	Unit weight, γ (kN/m^3)	In situ gravimetric water content (%)	Wetted gravimetric water content (%)	Void ratio, e	Specific gravity,[a] G_s	D_{50} (mm)	%< #200 sieve
Weakly cemented	SP	17.0	8.9	N/A	0.64	2.6	0.21	1.4
Moderately cemented	SM	18.7	12.6	22.1	0.60	2.6	0.15	12.1

Note: N/A=not applicable.

[a]G_s determination based on Bachus et al. (1981).

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Table 2. Engineering Strength Properties of Weakly and Moderately Cemented Sand from Pacifica, California

Material	UCS (kPa)	Friction angle,[a] ϕ (°)	Cohesion,[a] c (kPa)	Tensile strength, σ_t (kPa)	Elastic modulus at 20 kPa confinement, E_{20kPa} (kPa)
Weakly cemented—in situ wc	13	39	6	0	23,000
Moderately cemented—in situ wc	340	46	69	32	115,000
Moderately cemented—wetted wc	124	47	34	6	50,000

Note: wc=water content; UCS=unconfined compressive strength; TRX=triaxial compression test; and FSP=field stress path compression test.
[a]Mohr-Coulomb shear strength parameters are from a linear best-fit to the data.

Results

Index test results (Table 1) show that the cliff-forming deposits consist of uniform fine dense sands and include a minor amount of silt in the moderately cemented sands cliffs. The silt content and resultant decrease in void ratio in the moderately cemented sand are thought to be the primary factors for the increased strength in comparison to the weakly cemented sand; the larger surface area of the particles allows more contacts for cementation. Other indices (Table 1), including unit weight and in situ moisture content (measured during the California dry season, May through October) are typical compared to other uniform sands.

Strength test results generally fall within the range of values obtained in previous research of nearby materials (e.g., Sitar et al. 1980; Clough et al. 1981). UCS for the weakly and moderately cemented sand at their in situ water content are 13 kPa and 340 kPa, respectively (Table 2). Conventional triaxial results (Figs. 5 and 6) indicate a steep, near-linear increase of stress leading up to slight ductility, followed by brittle failure between 0.2 and 1.2% axial strain. This brittle behavior is accentuated in the FSP test results with failure at only 0.05–0.3% axial strain; a fourfold decrease compared to the conventional triaxial stress path. However, volumetric strain at failure is 50% higher in the FSP due to lateral expansion. An average Poisson's ratio of 0.295 was obtained (measured at the initial tangent modulus to the maximum point of curvature of the stress-strain curve using generalized Hooke's Law) from moderately cemented sand stress and volume measurements, and is consistent with that for typical sands (Gercek 2007). The elastic modulus (measured at the identical point as Poisson's ratio) is an order of magnitude larger between weakly and moderately cemented sand. Overall, the soils become stiffer with increasing cementation and less stiff with increasing water content. In all cases, samples were more ductile at higher confinement, consistent with expectations.

Linear Mohr-Coulomb effective shear-strength parameters can be well-fit to both materials (Figs. 7 and 8, Table 2), although the weakly cemented sand shear-strength envelope is more obviously steeper and curved at low stresses. For sands at low confinement, dilatancy occurs upon shearing and is responsible for higher shear strength in terms of the friction angle (Ponce and Bell 1971). This may also explain the higher than expected friction angles measured for both soils, some 5–10° above expected values for typical uniform sands (30–35°), but also expected for testing under such low confinement. Sitar (1983) also noted this behavior in coarse sediments at low confinement due to fabric strength and dilatancy influences.

Wetting has a pronounced effect on both soils. In the weakly cemented sand, complete disaggregation occurred upon wetting, precluding further testing. Further, due to the lack of sample stability at the in situ moisture condition, a tensile strength value of zero is presumed (Table 2). In the moderately cemented sand, saturation increase from 55% to 96% (gravimetric water content increase from 13 to 22%, Table 1) resulted in a 60% decrease in the UCS, 50% decrease in cohesion, 55% decrease in elastic modulus, and more than 80% decrease in tensile strength (Table 2). The friction angle remained nearly unchanged as expected. The result of tensile strength loss in the moderately cemented sand was observed most dramatically in the sample condition upon failure (Fig. 9). However, at the in situ water content, the tensile strength is on the order of 10% of the UCS, consistent with previous research (Clough et al. 1981; Das et al. 1995).

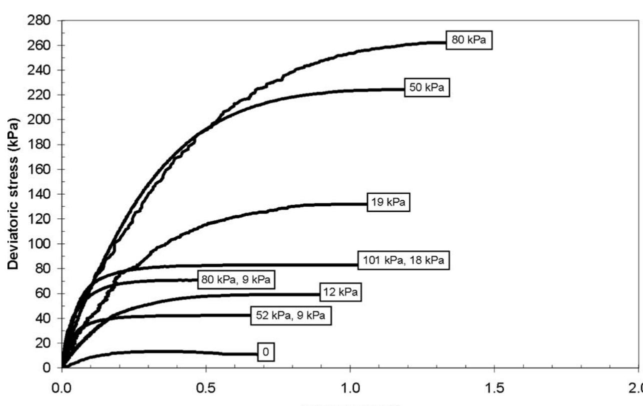

Fig. 5. Typical weakly cemented sand tests at in situ moisture content for unconfined (UNC) and triaxial (TRX) compression and FSP loading. XX kPa=σ_3 (UNC and TRX), XX kPa, YY kPa =constant σ_1, σ_3 at failure (FSP)

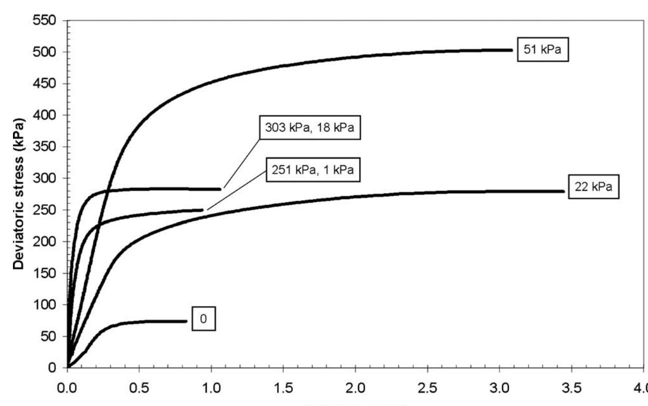

Fig. 6. Typical moderately cemented sand tests at wetted moisture content for unconfined (UNC) and triaxial (TRX) compression and FSP loading. XX kPa=σ_3 (UNC and TRX), XX kPa, YY kPa =constant σ_1, σ_3 at failure (FSP)

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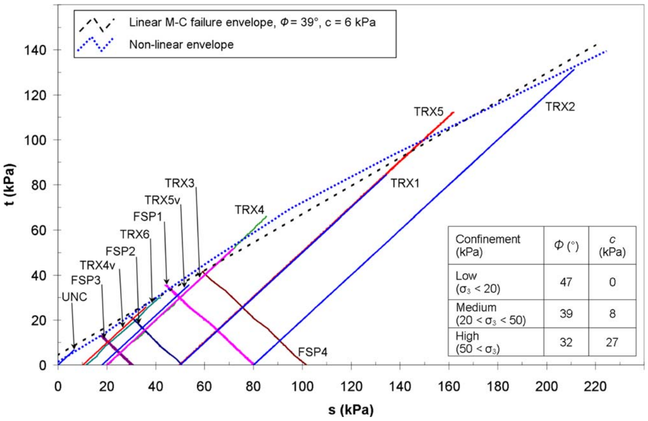
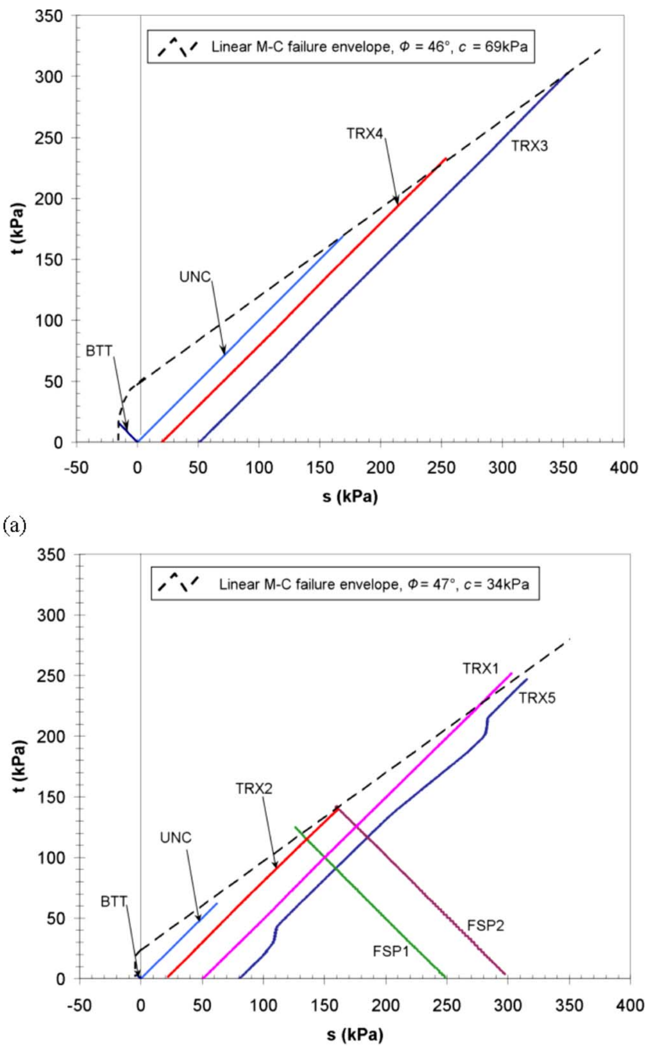


Fig. 7. Stress path failure data and envelope plotted in *s-t* stress space for weakly cemented sand. UNC=unconfined, TRX =conventional triaxial, TRXv=triaxial with vacuum, FSP=field stress path triaxial. Equivalent nonlinear Mohr-Coulomb strength parameters are shown in table.



(a)

(b)

Fig. 8. Stress path failure data and envelope plotted in *s-t* stress space for moderately cemented sand at (a) in situ moisture content; (b) wetted moisture content. UNC=unconfined, TRX=conventional triaxial, FSP=field stress path triaxial, BTT=Brazilian tensile test

(a) (b)

Fig. 9. Moderately cemented, Brazilian tensile test samples after failure for (a) in situ gravimetric water content (12.6%); (b) wetted/soaked gravimetric water content (22.1%)

Discussion and Conclusions

The geotechnical properties of weakly and moderately cemented sands were investigated in the context of their role in the stability of steep slopes and cliffs. In general, many properties are similar regardless of their setting. Most index parameters, for example, are not specific to cliffs, with the exception of the degree of saturation (water content), which should vary with depth into a cliff face. Whereas this characteristic was not investigated in this study, Hampton (2002) showed that the gravimetric water content increases by 1–3% between the cliff face and a point some 20–50 cm into the cliff. This difference likely plays a role in the development and decrease of tensile stresses at the bluff face.

The results of the laboratory test program highlight the contribution of strength parameters to cliff stability. First and foremost, the UCS provides an indication of the relative contribution of cohesion through the cementation and soil fabric network. Cliffs composed of soils with low UCS (i.e., weakly cemented) are more likely to be governed by their frictional component, whereas those composed of high UCS materials (i.e., moderately cemented) are likely to be more dependent on their cohesive strength, and potentially, their related tensile strength. Thus, when evaluating the stability of weakly cemented sand cliffs, cohesion and tensile strength should not be overly relied upon to prevent failure, especially if the sediments are exposed to water. Here, physical processes such as excavation (e.g., slope or toe erosion) are more likely to govern stability. However, in moderately cemented sands, cohesion-related tensile strength plays an important role in maintaining the integrity of steep slopes and cliffs and stability is more likely to be governed by environmental conditions, such as surface or groundwater seepage. In either case, brittle failure, consistent with the testing results, should be expected.

Second, for both materials (weakly and moderately cemented sand), the Mohr-Coulomb strength parameters are similar between those obtained using either conventional triaxial or the proposed FSP triaxial tests. However, the more brittle response under FSP conditions leads to failure at strains an order of magnitude lower than that obtained in conventional triaxial compression. Since typical cemented sand cliffs fail under FSP conditions, and most often, very close to the bluff face, these small strain conditions may be potentially undetectable by observation (i.e., tension cracks) and should be both closely monitored (e.g., using high resolution surveys) and analyzed.

Finally, the influence of wetting on material strength was well established in both materials with important results for cliff stability. Regardless of the failure mode, whether compressive, tensile, or shear, all strength parameters in cemented sands, with the

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exception of friction angle, decrease upon wetting by up to 100% for weakly cemented sand, and by between 50 and 80% for moderately cemented sand. Martins et al. (2005) and Lin et al. (2005) also measured similar decreases in these properties in residual sandstone soils and weak sandstones respectively. It should therefore be expected that cliff stability will also decrease as a result and that this condition be modeled appropriately in either hazard mitigation or engineering design studies. In summary, the variable degree of cementation found within sands may manifests itself through differing failure modes—which failure mode should be expected and designed for will depend directly on an understanding of this degree of cementation and the environmental conditions present in the field.

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