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Selection of Design Strengths for Overconsolidated Clays and Clay Shales

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Abstract

In his 1966 Terzaghi Lecture, Dr. Laurits Bjerrum established the basis for a fundamental understanding of the shear strength of overconsolidated clays and clay shales. These materials have recoverable strain energy resulting from the combination of high levels of past overburden loads and the diagenetic bonds that form after the clay minerals have been distorted into elastically deformed positions. Weathering of these materials causes breaking of the bonds and release of the locked-in strain energy. Rates of weathering depend on, among other factors, whether the formation was formed as a layered deposit or massive deposit. Because the weathering of the materials is variable, the shear strength changes in such earth masses are dynamic processes (periods of weeks to years). The variable time of weathering makes it very difficult to assess relevant design strengths over the life of a slope/wall project from commercial laboratory tests.

This paper looks at the factors, technical and non-technical, affecting selection of design shear strength values for specific projects. For many long-term projects the decision often involves the consideration of residual strength, fully softened strength or some other value. This paper looks in detail at the issue of slope stability of the Cretaceous-aged Potomac Formation in the Washington, DC area.

Introduction

In recent years the term intermediate geotechmical material (IGM) has been introduced to describe earth materials that do not fit well into the description of either soil or rock. The IGMs are typically described as either very hard soils or very soft rocks. Such is the case of the overconsolidated clays and clay shales that are the subject of this paper. In a sense, overconsolidated clays and clay shales only vary in the maximum amount of pressure that was applied to get them to their current state. Both are made up of clay minerals. Both are likely to include some amount of particle-to-particle bonding. The most

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important common feature is that both can, and unless protected, will eventually degrade, i.e., weather, back into a soft clay mass.

The above statement is true, in part, of all sedimentary rocks, except for the qualification that all rocks will eventually weather/degrade into their constituent minerals. Such is the cycle of soil to rock to soil transformation. The key difference between other sedimentary rocks and overconsolidated clays and clay shales is that these materials can make such a transition from rock or IGM to soil within the life of a civil engineering project. It is this transitional characteristic that makes the selection of design shear strengths so challenging.

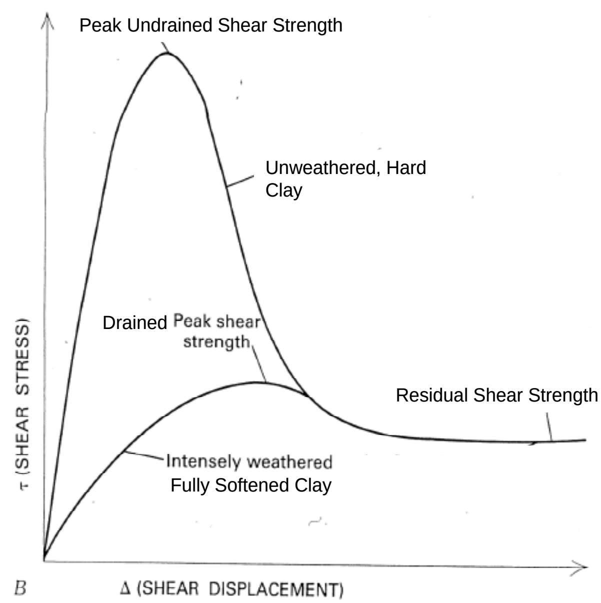

Figure 1: Direct shear Tests on Initially Identical Clay Samples

Given on Figure 1 are shear strength vs. displacement curves for two hypothetical direct shear tests of samples of initially, identical; hard, overconsolidated clay. One sample was tested in undrained shear at its native condition. The second sample was first subjected to intense weathering that caused the sample to reach a fully softened condition and was subsequently tested in drained direct shear. (The fully softened condition is described by Stark et. al. (2005) as the condition achieved when, "... an overconsolidated clay has absorbed as much water as it desires...") The samples are subjected to sufficient shear displacement such that both eventually reached the condition of residual shear strength. The peak undrained shear strength value of the unweathered sample would likely be about ten times the value of the peak drained strength of the fully softened sample. As noted by Stark et al (2005) the residual shear strength, "is controlled by the frictional resistance of face-to-face particles" and "is a function of clay mineralogy." At residual strength, the shear displacement has been enough to destroy all elements of formational structure and bonding such that the clay's strength has been reduced to its lowest level, which is a function only of its mineralogy because the shear displacement has been sufficient to cause parallel alignment of the individual clay particles.

Selection of Design Strength

Overview- Overconsolidated clays and clay shales are a part of the family of IGMs. Characteristic of sedimentary IGMs is that they have been transformed into hard soil to soft rock configurations by stress history and/or particle to particle bonding. A distinguishing feature of IGMs is that, while they exist in nature in a rock-like form, when disturbed they can regress back to a soil-like material within the life span of a project. In so doing, they typically lose considerable strength.

In a situation involving excavation of a slope into the overconsolidated hard clay, the geotechnical designer has at least three options for selecting shear strength for design of the slope. If the slope is temporary, the peak undrained strength may be appropriate. But if this is to be a permanent slope, the designer is faced with weathering of the clay and the consequential degradation of the clays strength. The question is how much degradation and how much associated strength loss should be expected during the life of the slope.

The degradation vs. time relationship is beyond the current capabilities of geotechnical engineering. This leaves the geoprofessional with two options for long-term design. They can choose to design on the basis of residual strength, the lowest value possible for the given material, or they can choose to design on the basis of the higher fully softened strength value. The decision to chose one or the other has both rationale and consequences.

Degradation of hard clays and clay shales- One of the more lucid discussions of the strength degradation process in overconsolidated clays and clay shales is summarized in Chapter III of the doctoral dissertation of M.E. Botts (1986). Botts summarized the three primary mechanisms of degradation as: 1.equilibration of negative pore pressures inherent in the overconsolidated material, 2. softening of the clay along existing discontinuities in the mass, and 3. progressive failure due to redistribution of stresses in the material.

Equilibration of negative pore pressures- Both the hard clays and clay shales (These are basically a continuum of the same mineral.) began as clay minerals flocculated in a saturated environment. The mass is subsequently stressed and compressed. Such masses typically have very low permeabilities. When these materials are subsequently unloaded, their natural tendency is to expand. The tendency to expand results in high negative pore pressures that contribute significantly to their effective stress state and, as a result, to their very high undrained strengths.

Softening of the clay along existing discontinuities- Many of these formations contain thin, fine sand or silt seams. Others may contain joints or fissures resulting from tectonic deformation. Such discontinuities provide flow pathways for migration of moist air or groundwater. At the clay boundaries of such pathways softening of the clay can occur that reduces the strength of the clay. For the fully softened strength to apply to a mass it does not require softening of the entire mass. It only needs to occur, possibly in very thin zones, along surfaces that are kinematically feasible for mass movement.

Progressive failure- Bjerrum (1967) described a process that can lead to cut slope failures at residual shear strength values in overconsolidated plastic clays and clay shales. These failures result from release of locked-in strain energy caused by the breaking of diagenetic bonding between individual clay particles. The bonds can be broken by weathering or stress concentrations.

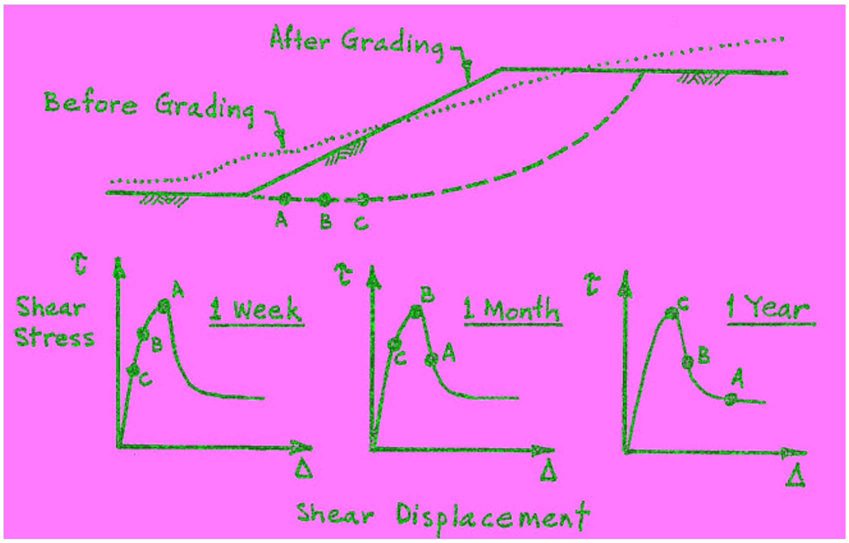

Figure 2. Mechanism of Progressive Failure (From Duncan 1977)

As depicted on Figure 2, the failure starts at the point of maximum stress concentration, the juncture point of change in excavated slopes. At this point the clay is strained to failure in undrained shear strength, and following adequate shear strain, the shear strength drops to the level of drained residual shear strength. This causes the excess load to be transferred deeper into the clay resulting in a progression of the failure until it finally defines a failure surface through the entire clay mass. The source of the initial stress concentration is removal of the clay mass that was providing a high level of lateral restraint to the unexcavated mass. High lateral loads, i.e., restraint are a common characteristic of heavily overconsolidated plastic clay masses.

<u>Selecting Shear Strength Values for Design</u>- The dilemma of selecting shear strength values for design of specific projects is that these strength values are subject to change, i.e., decrease, during the life of the project. Unfortunately the technical capability of predicting the rate of strength loss with time does not exist today, and because of the complexity of the variables impacting the time-loss relationship, it is unlikely that reliable relationship will be developed in the foreseeable future. As a result we must accept a pragmatic approach to this problem.

In 1948 H. Q. Golder (Hansen 1953) defined the type of pragmatism needed to address this dilemma.

"For the engineer, … , there are many possible answers, all of which are <u>compromises of truth and time</u>, for the engineer <u>must have an answer now</u>; his answer must be sufficient for a given purpose, even if not true. For this reason an engineer must make assumptions he knows to be not strictly correct – but which will enable him to arrive at <u>an answer which is sufficiently true for the immediate purpose</u>." (Underlining added for emphasis.)

Strength values for overconsolidated clays and clay shales range from peak undrained shear strength to fully softened shear strength down to as low as residual shear strength. Decision regarding the design strength of these materials includes both technical and non-technical issues such as:

- Structural and groundwater conditions of the material
    - Inclination of bedding planes
    - Presence of ancient landslides
    - Other discontinuities in the mass
- Design life of the project
- Long-term project owners and their risk tolerance

- Cost impacts of initially selecting conservative design values
- Estimated extent of future ground failures
- Cost impacts of remediating future ground failures

Two examples of the factors involved in such decisions are:

- Design of residential subdivisions – In this case the developer of such properties are very short-term owners, who will quickly sell the individual housing units to individuals who were not a party to risk decisions made by the developer. As a result many local governments have mandated that such projects must be designed using conservative (costly) residual strength values for slopes in the subdivision in order to protect the homeowners.
- Design of new highway alignments – In this case it often makes sense to design slopes using shear strength values well above residual strength values. The long-term owners of these projects are typically state or local government units who want to minimize their initial costs and often have permanent staffs that can participate in future repairs of failed slopes. However, the development of slopes/right-of-ways for such projects must take into account the location of nearby properties owners such as residential homeowners who could be affected by landslides that occurred as a result of shear strength values at residual levels.

A Case History - Washington, DC Area

Background - Following the War of 1812, the U.S. Congress authorized the building of a new fort on the Potomac River to guard the U. S. Capital from the type of naval invasion that allowed the British to invade and burn the city of Washington. The Fort is located on a ridge on the north side of the Potomac about a mile west of the Mount Vernon Plantation on the south side of the river. Construction of the Fort began in 1818 and it remained in active service until 1872.

In 1987 Woodward-Clyde Consultants was commissioned to make a study of landslides that were threatening the Fort, then a National Park. This study provided access to the historical records of the Fort, which notably included, detailed records of landslides in the very stiff clays near the river level. The records indicated that these materials, the Potomac Group Formation, were responsible for slides that damaged or destroyed the numerous segments of the river wall where loading and unloading of transport barges took place. This is one of the earliest historical records of what came to be known as the "marine" clay problem.

As residential development grew rapidly in Fairfax County, VA (directly south of DC) during the 1950s and 1960s the incidences of landslides in the "marine" clays grew with dire consequences. In 1973 a property known as Rose Hill Park was submitted to the County for residential development. County officials knew that this property had been initially slated for development in the 1960s, but the process had been halted by a landslide that occurred during what, reportedly was, a relatively minor excavation for the main street into the property. Concerned that the County did not have the technical capabilities to address the new submittal, the County Director of Design Review retained a geotechnical engineering consultant, Woodward-Gardner & Associates (WGA), to aid in the review.

WGA's review (1973) agreed with the developer's geotechnical consultant that the site clays were "stiff and highly overconsolidated" and possessed "appreciable unconfined compressive strength." However WGA did not agree with the consultant's position that there was "… <u>no</u> evidence to indicate deep seated earth movements of measurable or damaging magnitude." WGA went on to note that the "physical properties and history of slides in the area tend to fit that category of soils described in Dr. Laurits Bjerrum's Third Terzaghi Lecture." WGA noted that progressive failure in these overconsolidated clays leading to development of residual shear strengths in the materials, as noted by Bjerrum (1967), was a very likely explanation for many of the slides in these very stiff "marine" clays.

To resolve the difference of opinions between WGA and the developer's consultant, a widely recognized geotechnical consultant, John P. Gnaedinger of Soil Testing Services, Inc., was retained to advise Fairfax County on the dispute. Gneadinger (STS, 1974) sided with the WGA position and subsequently worked with the County to develop policy for development in the area directly underlain by the so-called marine clays (Dallaire, 1976). The new policies identified the zones of "marine" clay outcrops, established a three person Geotechnical Review Board to review plans for projects in the affected area, and guidelines that required the use of residual shear strength unless the submitting engineer could convince the Geotechnical Review Board that this approach was not appropriate.

<u>Potomac Group Soils</u> – Figure 3 shows the outcrop of the Lower Cretaceous Potomac Group Soils of the Atlantic Coastal Plain Province in the vicinity of Washington, DC. These soil formations, consisting of beds of almost pure clay, sandy clays and sands, were deposited in fluvial and deltaic environments made up of meandering rivers on

broad plains near the coast. Although the clay units of this formation are known locally as “marine” clay, the origin of these materials has led to a far more complex array of soils than would have developed in a true marine depositional environment. Later, during post-Cretaceous time (Obermeier and Langer, 1986), hundreds of feet of the sediments were eroded off the surface of these soils, leaving them significantly overconsolidated. Additionally Obermeier (1984) notes the presence of post-Cretaceous faulting in these materials resulting in swarms of fractures in the stiff to hard clays.

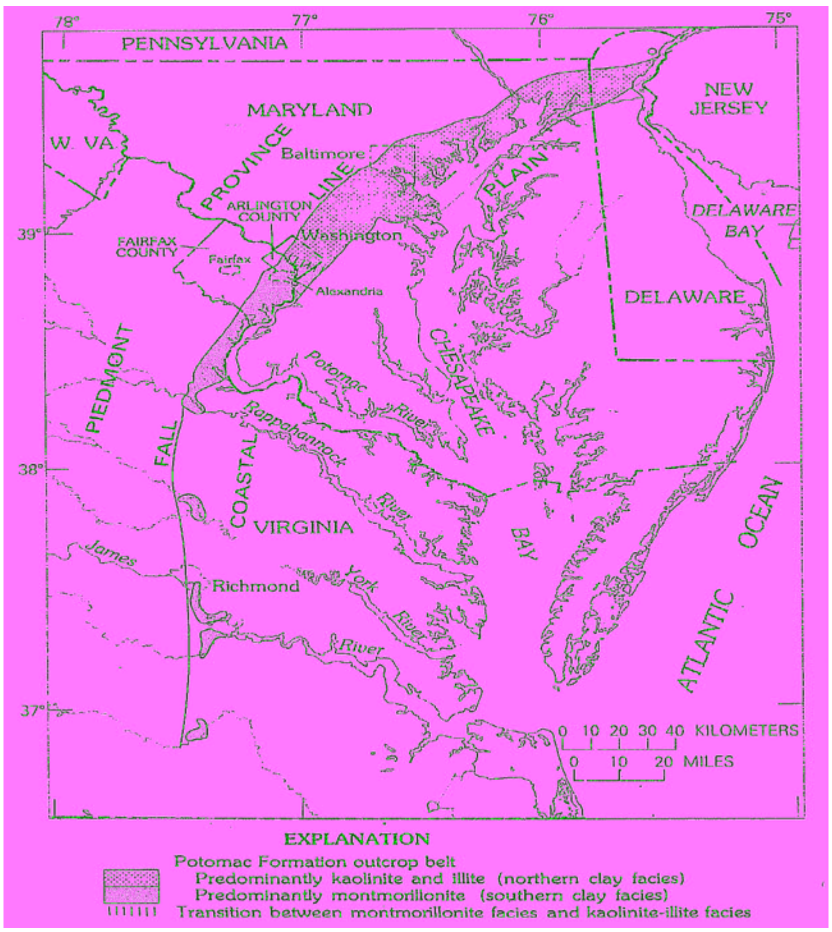

Figure 3. Potomac Formation Outcrop Belt from Virginia to New Jersey (after Force and Moncure, 1978)

The clays in Virginia, south of the current Potomac River, contain high percentages of montmorillonite minerals and are typically classified as CH materials. Liquid limits are typically on the order of 70+%, and as high as 100+%, with Plastic Limits on the order of 30%. Natural moisture contents are typically at or near the Plastic Limit (Obermeier, 1984). Standard Penetration blow counts typically range from 20 to 40 blows per foot. Unconfined compression shear strength values for the clay rich materials commonly range between 4 and 6 ksf (Obermeier, 1984). Important to the possible development of progressive failure in these materials, a recent site investigation (Kleinfelder, 2006) used a dilatometer to measure insitu lateral pressures in these clays. At-rest lateral earth pressure coefficient values of 1.5 to 2.3 were assessed based on correlations developed for similar hard clays. Therefore vertical cuts in these materials would result in the removal of significant lateral restraining forces. This action could trigger the kind of events represented in Figure 2.

Owing to the Fairfax County ordinance requirements relative to development in the Potomac Group Formations, many residual shear strength tests have been performed on these clays. These test are almost exclusively pre-split, direct shear tests where the strength is measured only after numerous back and forth movements, known as "yoyo" direct shear cycles. Predominantly the measured residual shear friction angles are between 10 and 15 degrees. With sufficient cyclic movements, the failure surface is sufficiently "polished" such that the cohesion is reduced to zero.

Conclusions

In the words of Obermeier (1984), "Both naturally occurring and construction-related landslides are commonplace and widespread in the belt of Lower Cretaceous Potomac Formation deposits extending from Virginia to New Jersey." This is a band that has relatively limited topographic relief. Often the landslides come to rest at slopes of 1:4 (vertical to horizontal). The complexity of the geology of these heavily overconsolidated, plastic clays (depositional, mineralogical and tectonically) produce slope failures in the area that fit all three of the failure mechanisms described earlier in this paper, and additionally these landslides are often precipitated by groundwater conditions. No single mode of failure is sufficient to explain all such failures.

This situation brings us to the primary point of this discussion, namely that the selection of shear strength values for design of slopes in these type of materials requires more than simply technical considerations. Design strength values must also consider non-technical factors such as risk tolerance and economics. The Washington, DC area is a land area that is highly valued as one of the most important political locations in the world. It cannot be abandoned because of its inherent risks, nor can it be built such that it causes undue harm to people, nor can it be universally built to the most conservative standard, namely residual strength. Selection of design clay strength values must achieve a measured balance of technical and non-technical factors.

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