# Design Practices in Overconsolidated Clays of New York

VERNE C. MCGUFFEY

The key design practices for construction in overconsolidated clay deposits in New York State are summarized. The differences in prediction technology for heavily overconsolidated soils as compared with normally consolidated soils are highlighted. Rules of thumb for design practices are also included. The design approach used is based on the stress history of the deposit. Overconsolidated clays subject to *overload*-type stress history perform as predicted using classical approaches to settlement and stability. Clay deposits subjected to *desiccated*-type preconsolidation, however, require different approaches in exploration and modeling to properly predict performance. Continuous undisturbed samples are desirable, and many consolidation tests are needed to describe the preconsolidation history accurately. Plots of moisture content versus depth from numerous disturbed samples in the deposit best reflect the type of stress history and therefore are used as a guide to process selection. It is difficult to predict the probability of a slope failure for cuts in natural slopes. The slope stability varies with the rate of shear stress release and rate of water table drawdown. Cutslope failures in overconsolidated clays in New York commonly occur about 7 years after construction. Predictions of time for settlement for desiccated clays overlying normally consolidated clays are difficult to make. Therefore, treatment (such as wick drains) is recommended if the performance objectives cannot be guaranteed should settlement occur in a manner not predicted.

Many design errors have been made and are continuing to be made by engineers inexperienced in prediction of performance of heavily overconsolidated clays. Some of the successful design approaches used in areas of heavily overconsolidated clays in New York State are summarized to help the inexperienced designer. Special features that must be looked into differently than would be done with normally consolidated or lightly overconsolidated clay deposits are also identified. Analysis techniques will not be discussed unless they are unique to overconsolidated clays. These methods and associated rules of thumb can be applied to most heavily overconsolidated clay deposits observed in geotechnical literature.

New York State experience indicates that much of the difficulty in making accurate predictions of soil performance appears to relate to lack of recognition of small variations in soil stratigraphy or parameters that produce a major change in the performance (e.g., major changes in shear strength and consolidation characteristics occur in short distances when the preconsolidation pressure changes from overconsolidated to normally consolidated).

## BACKGROUND

Heavily overconsolidated clays in New York State are defined as those clay deposits with preconsolidation pressure ($P_p$) appreciably higher than the present overburden pressure ($P_o$). This type of deposit is defined further as having an overconsolidation ratio greater than 2 ($OCR > 2$) or a preconsolidation pressure greater than 7000 kPa (1,000 PSF) over the present overburden pressure. The distribution of overconsolidation commonly takes two forms (Figure 1) (*1*): (a) the overload preconsolidation pattern is identified by its relative straight line distribution from clay surface to bottom, usually paralleling the normal overburden pressure diagram; and (b) the desiccated preconsolidation pattern is identified by its very high preconsolidation level near the top of the layer, decreasing in a parabolic shape to the bottom of the layer or to an overload preconsolidation line.

It is common to find a desiccated pattern grading into an overload pattern in the same deposit. It is less common, but not unusual, to find a desiccated pattern underlain by a second or third desiccated pattern. Occasionally there will be an overload pattern over a desiccated pattern (this usually means that there are two separate geologic clay deposits that may have appreciably different characteristics).

Identification of the preconsolidation load pattern and quantification of the preconsolidation loads through the deposit are probably the two most important items in any design activity in overconsolidated clay deposits.

## IDENTIFICATION OF PRECONSOLIDATION LOAD PATTERN

Identification of the preconsolidation load pattern early in the design process is very important so that sufficient sampling and testing can be done to quantify the critical values for the particular design. In New York State (and in technical literature) there is often a wide band of preconsolidation values obtained from tests on the same soil deposit (*1–3*). It is difficult, therefore, to identify clearly the pattern unless a large number of consolidation tests are available (which is seldom economically justified).

New York State uses natural water content tests as a surrogate method to identify the pattern to support the conclusions needed for the design. New York State and others (*1*,*2*) have observed that natural moisture content ($W_n$) is inversely related to the preconsolidation pressure in a natural soil deposit if the deposit does not change and therefore can be used as a surrogate to identify the shape of the preconsolidation curve for the deposit. Natural moisture content tests can be obtained from most types of disturbed samples with good results. A large number of disturbed samples can be obtained at reasonable cost, and the results can be used to plan a program to get the expensive undisturbed samples for detailed testing. An example of this type of data is shown in Figure 2 (*1*,*2*).

Knowing the expected overconsolidation load pattern from the plots of moisture content ($W_n$) versus depth, an effective undisturbed sampling and testing program can be planned to quantify the

Soil Mechanics Bureau, New York State Department of Transportation, Albany, N.Y. Current address: 22 Lombard Street, Schenectady, N.Y. 12304.

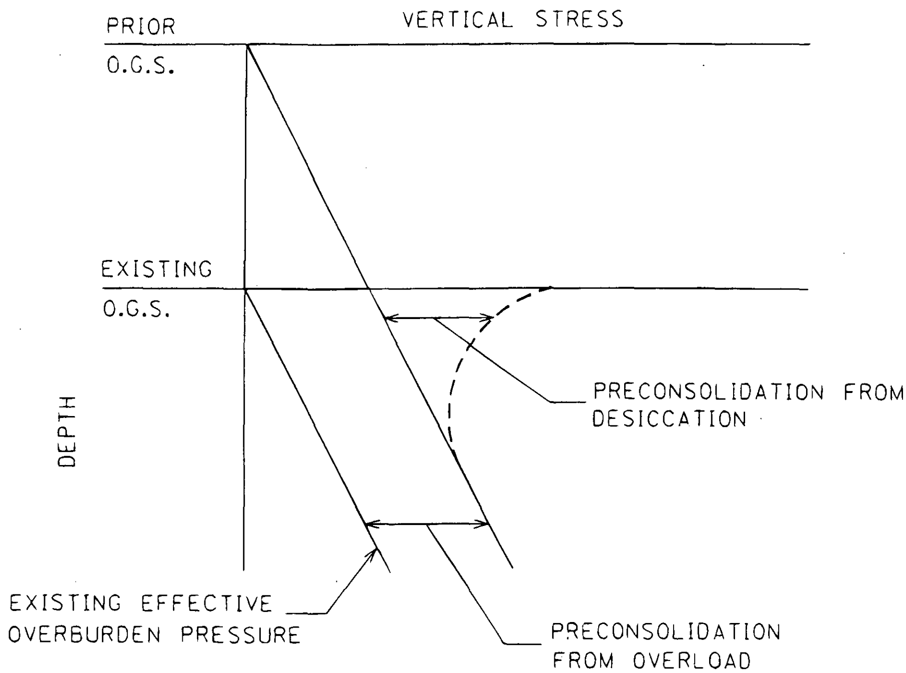


**FIGURE 1 Definitions of overconsolidation patterns [after Ladd and Foott (*1*)].**

preconsolidation pressure values and refine the shape of the curve. Depth versus $W_n$ profiles also can identify where the most severe conditions can be expected so that the proper undisturbed sampling and testing can be done at the most critical depth. Figure 2 shows a typical plot for a desiccated pattern clay, where $W_n$ increases with depth until the preconsolidation load curve changes from desiccated to overload shape; then $W_n$ decreases with depth.

Sampling disturbance seriously affects the consolidation test values for preconsolidation load and therefore special efforts must be taken on critical projects to identify the possibility of sampling disturbance. The natural moisture content seldom is affected by normal sampling disturbance; therefore, the moisture content profile from numerous disturbed sample holes usually will identify clearly the shape of the preconsolidation curve of the deposit. Any consol-

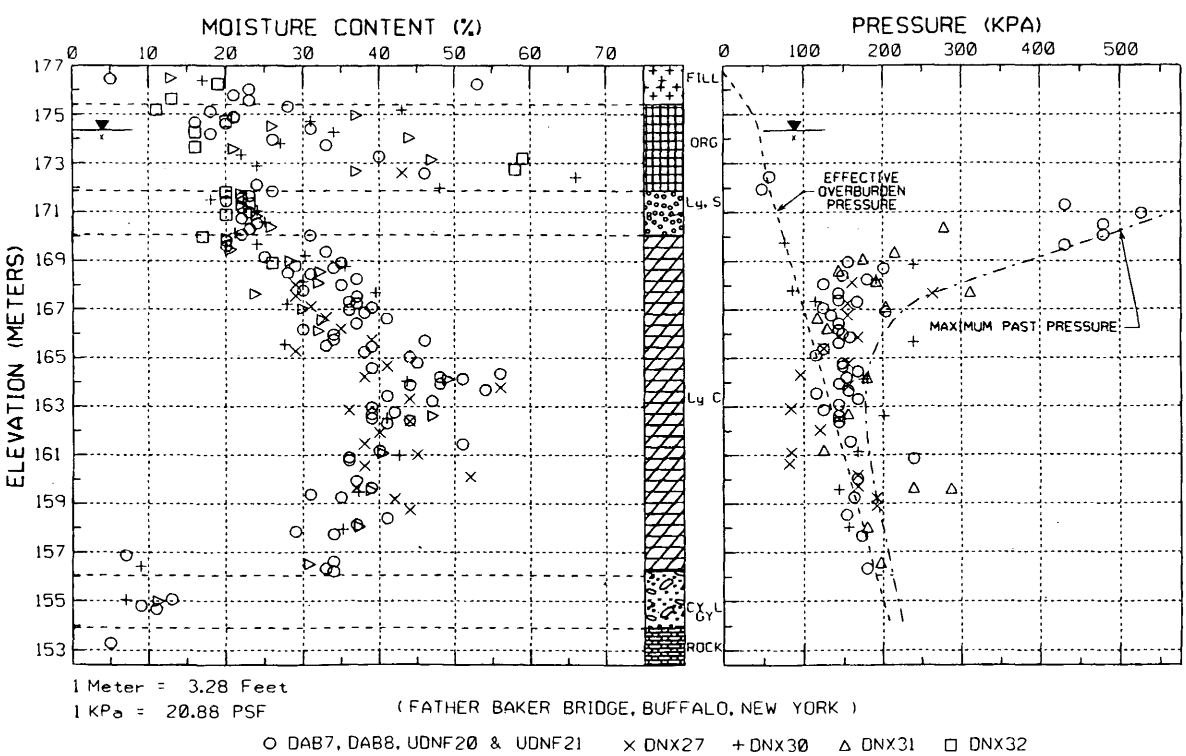


**FIGURE 2 Moisture content and maximum past pressure versus elevation.**

idation test data that do not agree with that shape of curve should be looked at as suspect in quality. Resampling may be needed if it is important to the design. Although a low value of preconsolidation pressure in an otherwise highly preconsolidated deposit is sometimes due to natural landslide disturbance, it is more commonly a direct result of sampling disturbance. The degree of sampling disturbance can usually be identified by X-raying undisturbed sample tubes before running consolidation tests (*4*).

## STABILITY OF NATURAL OR CUT SLOPES

### Short-Term Slope Stability

Most cuts in overload-type preconsolidated clays can be made quite steep for a short term because of the high undrained shear strength ($S_u$), which usually controls short-term cut slope stability. The stability may be estimated using the stability charts from Taylor (*5*) using the shear strength identified from routine testing [consolidated undrained triaxial ($C_u$), unconsolidated undrained triaxial ($U_u$), field or laboratory vane shear, cone penetrometer, or unconfined compression (*UC*)]. Short-term cuts in desiccated preconsolidated clays, however, may become unstable because of fracturing that took place during drying. The material in the fracture may control the stability instead of the average shearing strength from unfractured samples. Literature of successful predictions of cut-slope failures in fractured clay deposits is scant. It is advisable, therefore, to design temporary support or use long-term parameters (e.g., drained friction angle $\Phi_d$) for design of cuts in desiccated deposits.

### Long-Term Slope Stability

The long-term performance of cuts in natural slopes of overconsolidated clays (both overload and desiccated) is difficult to predict. The undrained shear strength at the time the cut is made may be very high, but this value reduces with time (apparently because of the stress relief) until the shear strength governing failure approaches the drained friction angle at the new overburden stress. This has been discussed more thoroughly previously (*6–8*).

Appropriate circular arc- or wedge-type stability analyses (e.g., Bishop's circular or NAVFAC wedge) (*9*) usually will be adequate to describe the condition of stability of an overconsolidated clay slope, provided that the drained strength ($\Phi_d$) is used in the analysis. A usable value of the drained friction angle can be obtained from drained triaxial, drained direct shear, or consolidated undrained triaxial tests with pore pressure measurements. These values should be compared with existing charts of drained friction angle versus plasticity index (*7,8*). Backfigured data from failed slopes in overconsolidated clays demonstrate that using the drained friction angle ($\Phi_d$) will usually give an appropriate expression of the stability at the time of failure ($FS = 1 \pm 0.05$). (Computerized slope stability analyses with automatic search patterns may produce artificially low factors of safety when using a zero cohesion input because the search moves to the skin of the slope. It may therefore be necessary to include a low value of cohesion in the computer stability analyses so that the computer will search at realistic locations where the observed failure occurs. Inserting 344 to 1033 kPa (50 to 150 psf) of cohesion allows the automated stability analyses to identify correctly the location of the failure surface without changing the factor of safety appreciably.)

The static groundwater table in the system is disrupted when a cut is made and the groundwater surface is lowered below the surface of the new cut slope. It is believed that the excavation produces a negative pore pressure below the cut slope surface, which contributes to the overall short-term stability of the slope. The negative pore pressures dissipate with time, resulting in a reduction of shear strength to a value approaching the drained value ($\Phi_d$). Few clay slopes would remain stable in the long term in normal highway cuts of 1 on 2 (26.5 degrees) to 1 on 3 (18.5 degrees) if the groundwater table remained at the surface of the slope when the preconsolidated clays reach the drained shear strength condition (usually 20 to 27 degrees). Often preconsolidated clay cut slopes fail from 1 to 10 years after construction (an average of 7 years in New York State). One possible explanation, and a methodology to predict the time to failure for long-term stability of cuts and overconsolidated clays, has been given (*7*). This explanation assumes that the negative pore pressure dissipation is the reverse of the normal loading pore pressure dissipation and therefore that the same time factor curves apply. The time to failure then can be estimated from the coefficient of consolidation ($c_v$) of the soil and the depth to the probable failure plane [usually about 4.6 to 6.1 (15 to 20 ft) in New York State clay slopes]. For example $t = T \times H^2/c_v$, substituting typical numbers, $t_{90} = 0.848 \times$ [6.1 m (20 ft)]$^2$/0.0093 m$^2$ (0.1 ft$^2$)/day $\times$ 365 days/year = 9.3 years) (*7*).]

Care is needed to identify slopes that could have been subjected to old landslide activity. Large excess pore pressure may still exist along the original failure plane that may reduce shear strengths to values below that described by the drained friction angle and the present overburden. A conservative approach is to backfigure an equivalent shear strength along the failure plane assuming a factor of safety of 1 on the old failure surface, and then to complete the design assuming no increase in shear strength. A detailed investigation must be conducted to define clearly all necessary parameters if the conservative approach is not acceptable or cost-effective. This investigation might include extensive explorations and testing along with long-term pore pressure and movement measurements.

One way of reducing the risk of slope instability from a permanent high water table in overconsolidated layered silt and clay systems (which seldom exhibit much drawdown of the water table) is to excavate an additional 2 to 3 ft of the clay along the slope and replace it with an open stone fill slope protection (*10*), which has the effect of lowering the water table.

## STABILITY OF EMBANKMENTS ON OVERCONSOLIDATED CLAYS

Expectation of very high in-situ undrained shear strength in heavily overconsolidated clay can lull the investigator into complacency because a stability situation seldom arises from embankments constructed on overconsolidated clays when undrained shear strength controls. There are situations in which an exception in the soil system controls the performance of the construction, and therefore some exceptions will be discussed.

- Even small embankments placed on slopes that have failed (old landslides) can set off new failures even though the average shear strength is very high.
- Some overconsolidated clays have been subjected to major previous failures resulting in micro shear planes (low-strength clays between blocks of very-high-strength overconsolidated clays) (New York State Department of Transportation, Morrows Corners, West Granville, 1958, unpublished data). A special sampling and testing program may be needed to identify the situation and obtain suitable

parameters for shear strength. Use of a drained strength parameter ($\Phi_d$) is usually suitable if there is no residual pore pressure.

- Overconsolidated clays of the desiccated pattern usually have very high strength clays over much softer clays. It is usually not appropriate to use the very high shear strength of the surface soils because they are often fractured and fissured. Because there is no easy way to obtain quality test results for these fissures, a conservative approach is usually taken. One such approach is to assume that fissures are filled with sand and use a 35 degree drained friction angle in the heavily overconsolidated clay instead of the measured shear strength of the clay. The measured shear strength can be used if there is confidence that the data were obtained from soil below the zone of fissures.
- In urban areas, discontinuities should be expected in the stiff clay where old foundations or utility lines were excavated through the very strongest part of the desiccated clay. This often leaves areas of very low strength (sand, debris, etc.) in an otherwise high-shear-strength clay deposit. It is suggested that a conservative approach be used as in the previous paragraph.
- Strength gain from loading of soft clay beneath heavily overconsolidated clay in a desiccated clay system should not be depended on without extensive study. The heavily overconsolidated surface clay has a low permeability that may slow the vertically upward drainage from the soft clay, thereby making it very difficult to predict time for strength gain. The concepts of predicting time for drainage will be discussed further under the section "Settlement of Overconsolidated Clays."
- Some overconsolidated clays are very sensitive and may dramatically change characteristics under loading when overstressed (*11*). The St. Lawrence clays in New York State have natural water content more than 10 percent over the liquid limit and are very sensitive. Failures in sensitive clays may occur at the post peak or residual strength (which may be 20 to 70 percent of the peak natural strength) because of progressive failure. The lower value of shear strength may control fill stability if the embankment being constructed creates a condition of overstress at any place beneath the embankment (*11,12*). As a general rule any overconsolidated clay that has a natural moisture content 5 percent or more above the liquid limit is highly susceptible to this large, and sometimes rapid, loss of strength (*13*). A method of analyzing the stability of sensitive clay systems subjected to overstress is described by Gemme (*11*).

## SETTLEMENTS OF OVERCONSOLIDATED CLAYS

The magnitude of settlement of overconsolidated clays usually can be predicted quite accurately using the standard consolidation equations developed by Terzaghi (*14*) using the recompression ratio (*RR*) up to preconsolidation pressure ($P_p$) and using the compression ratio (*CR*) above the preconsolidation pressure (*13*).

The time for consolidation to occur follows the standard consolidation equations using standard test parameters for the overload pattern preconsolidated system.

The time for consolidation to occur for the desiccated system, however, seldom can be predicted accurately. Often the surface soil is so heavily preconsolidated that it may effectively block the drainage of the underlying near-normally-consolidated clays (the coefficient of permeability may become very small—less than $10^{-7}$ cm/sec). If this occurs a vertically upward component of drainage may no longer occur in the underlying soft clay. If the bottom of the layer has an impermeable boundary such as rock, there may be nearly a zero rate of vertical drainage and most of the pore pressures must dissipate laterally. Rough estimates of the rate of lateral pore pressure dissipation can be made using the procedures described by Ladd and Foott (*15*) or an approximation from McGuffey (*16*). Some desiccated deposits have shrinkage cracks (fissures) filled with silt or sand, which can allow upward drainage. Unfortunately, it is nearly impossible to estimate the overall contribution of this effect to the rate of settlement expected. One project in Buffalo (Young Street Arterial) exhibited changing boundary conditions with time. The pore pressure measurements indicated primarily one-way downward drainage to a thin gravel layer over rock at the beginning of loading. As loading continued, the downward component stopped and all drainage appeared to be lateral with a small component vertically upward through the desiccated clay.

The above type of performance is nearly impossible to predict, and therefore it is often prudent to treat the area with sand drains or wick drains as an economical guarantee of performance if vertical drainage is uncertain in desiccated clay systems.

## HEAVE EXPANSION OF OVERCONSOLIDATED CLAYS

Heave in excavations of overload preconsolidated clays in New York State is generally so small as to be neglected (*17,18*). To summarize New York State experience (*17*), "The swell potential is not considered to be large unless the soil is *desiccated*, the groundwater table at a considerable depth, and the soil contains clay mineral particles with expansive characteristics."

Heave in overconsolidated clay due to desiccation often occurs when excavations expose the clay to free moisture. This is most noticeable with high bentonite clay that has been dried back below the shrinkage limit during much of its previous history. Damage similar to frost heaves can occur unless special treatment is used such as allowing preexpansion before installation of the final roadway surface or designing so that the expansive clay is unable to obtain additional moisture.

## STRUCTURE FOUNDATIONS

### Shallow Foundations

Shallow foundations usually are no problem on either type of overconsolidated clay because the footing loads are very small compared with the previous loading of the clay system and therefore the clay has adequate strength and exhibits little or no compression from the structure loading system. A few situations that should be investigated in detail for shallow foundations on overconsolidated clays are given below:

- Preconsolidated clays with a natural moisture content below the plastic limit may heave if subjected to free water as a result of construction (e.g., inadequate footing drainage). Clays with natural moisture content below the shrinkage limit are highly susceptible to damaging heave when exposed to free moisture, and special precautions are required.
- Footing loads seldom will cause settlement of either overload or desiccated clay systems, but if fill is being placed around the structure foundation in a desiccated clay, the settlement of the softer underlying clay will have to be evaluated to determine the effect of grading settlements on the structure performance.

## Deep Foundations

Deep foundations in overload pattern preconsolidated clays usually do not present problems and perform as predicted using standard design practices. Deep foundations through a desiccated pattern preconsolidated clay may be damaged by settlement of the underlying softer clay if settlement is not predicted accurately or accounted for. This requires careful investigations to determine whether any settlement will occur. The design may also have to consider large pile drag if there is grading around the structure that would cause consolidation of the underlying softer clay.

It may be more economical sometimes to redesign the structure for zero net load than to increase the number of piles to account for pile drag. The situation becomes more complex when settlement is still occurring from a previous load on the site. The net load must then be reduced to a value below the preconsolidation pressure curve or back to the previous original ground surface load. On one building project in New York City, the cost of the deep foundation was more than doubled to account for the settlement remaining from old fill.

Deep foundations should go completely through the softer material under the very stiff surface layers and not get founded in the stiff upper layer by those using designs based on pile-driving blow counts alone.

## ADDITIONAL RULES OF THUMB

- Overload pattern preconsolidated clay deposits exhibit similar stress history over large areas and often can be related to a definable geologic deposit [e.g., glacial Lake Albany clay is not found above USGS elevation 70.15 m (230 ft), and therefore the preconsolidation load anywhere in the deposit can be closely estimated by subtracting the present ground elevation from elevation 70.15 m (230 ft) and multiplying this by the soil effective unit weight of 1.042 g/cm$^3$ (65 $\pm$ lb/ft$^3$)].
- Desiccated pattern overconsolidated clay deposits also cover wide areas, but local variations can have a major effect on performance. For example, the depth to rock on the Lockport Expressway Project in western New York State varied across the project. There were areas where rock was immediately under the stiff clay with no underlying soft clay adjacent to areas with appreciable amounts of soft clay underlying the stiff clay. This required berms next to moderate fills and no berms next to higher fills, and there were large differences in the settlement for the same height of fill. To prevent being surprised by these types of variations, a large number of subsurface explorations are desirable to define the controlling conditions.
- Samples taken at 5-ft or wider intervals in desiccated pattern soils can miss the most critical (weakest and most compressible) soil in the deposit. Therefore, continuous samples are essential in desiccated clay deposits.
- A small error in identifying and testing the weakest portion of a desiccated pattern deposit can result in shear failures in situations where the stiff surface layer has been removed (e.g., streams or canals—Figure 3). The lowest undrained shear strength strongly controls stability in these situations.

## COMMON PROBLEMS

Three common errors that can be disastrous to the designer when working with overconsolidated clays are as follows:

1. Failure to recognize a highly sensitive overconsolidated clay that may be subjected to high shearing stresses where a part of the deposit is overstressed. This error can result in rapid loss of shearing strength and resultant shear failures under loading (and flow slides of cuts in natural slopes).
2. Failure to recognize a cutslope where the groundwater table will not draw down sufficiently (before shear stress relief occurs) to allow for long-term stability of the cutslope.
3. Failure to recognize the potential for heave in desiccated clay deposits.

The first situation usually is easily identified in New York State, and there has not been a failure of this type for over 20 years. The second situation is common in New York State, with failures occurring about once every 2 to 3 years. These failure can be expected to continue because the prediction technology is not dependable, and the cost of a detailed design often exceeds the cost of correction if there

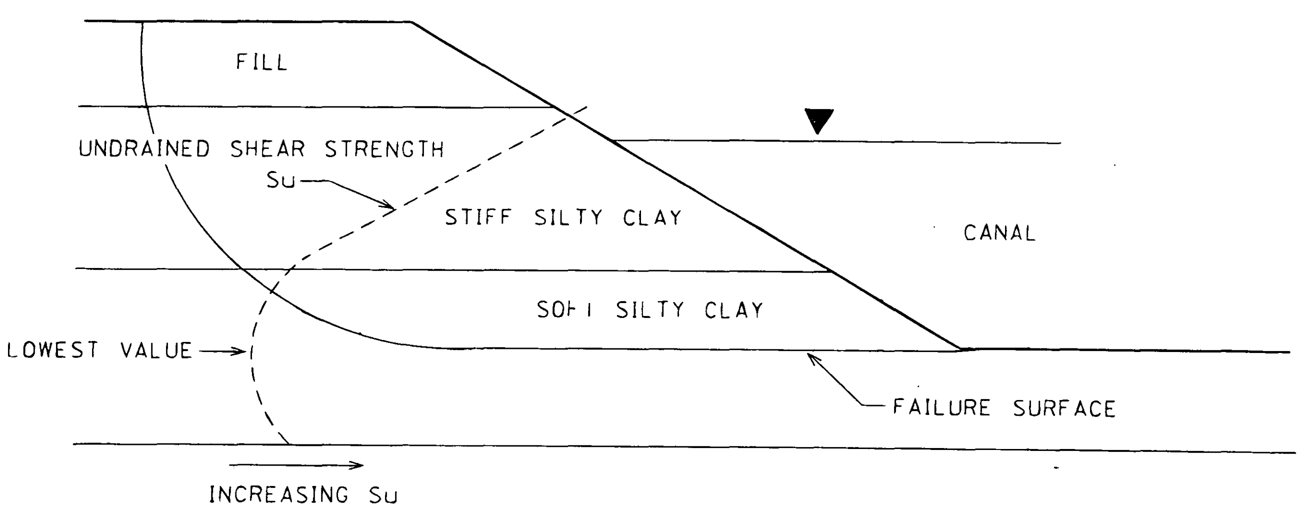


**FIGURE 3 Example site where lowest value of undrained shear strength $S_u$ controls performance.**

is a failure. The third situation seldom is important for transportation facilities in New York State because the design requirements for frost protection also prevent heave problems. It is important for buildings, and the availability of moisture must be controlled to prevent costly damage.

## CONCLUSIONS

- Identifying the pattern of preconsolidation load of the clay systems is one of the most important features of any investigation in overconsolidated clays.
- Identifying the pattern can best be made by a large number of plots of moisture content versus depth from numerous disturbed sample borings.
- The quantification of the preconsolidation stress level can be obtained by good-quality undisturbed samples and normal consolidation testing programs.
- The undrained shear strength of the deposit is satisfactory for determining stability under loading conditions on flat terrain.
- For unloading conditions (cuts) or loading on sloping terrain, it is essential to determine the drained friction angle of the deposit.
- It is nearly impossible to predict accurately the time for settlement to occur for the desiccated pattern systems because of the difficulty in defining the boundary conditions and the drainage parameters. Therefore, treatments such as sand drains are often preferable to attempting to get enough explorations to make accurate predictions.
- The potential for heave should be of concern on most desiccated pattern overconsolidated clay systems.

## REFERENCES

1. Ladd, C. C., and R. Foott. *Foundation Design of Embankments Constructed on Varved Clays.* FHWA, U.S. Department of Transportation, 1977.
2. *Proceedings of the Foundation Deformation Prediction Symposium.* Report FHWA-RD-75-515. FHWA, U.S. Department of Transportation, 1975.
3. Bergado, D. T., A. S. Enriquez, C. L. Sampaco, M. C. Alfaro, and A. S. Balasubramaniam. Inverse Analysis of Geotechnical Parameters on Improved Soft Bangkok Clay. *Journal of Geotechnical Engineering,* ASCE, Vol. 118, No. 7, July, 1992.
4. Hall, E. C., and L. D. Suits. The Use of X-Rays in Soil Testing. *Highway Focus,* Vol. 8, No. 1 Jan., 1976.
5. Taylor, D. W. *Fundamentals of Soil Mechanics.* John Wiley & Sons Inc., New York, 1948, 455 pp.
6. McGuffey, V. C. Earth Cut Slope Design in New York State. In *Highway Research Record 457,* HRB, National Research Council, Washington, D.C., 1973.
7. McGuffey, V. C. Design of Cut Slopes in Overconsolidated Clays. In *Transportation Research Record 873,* In TRB, National Research Council, Washington, D.C., 1982.
8. Bjerrum, L., and N. E. Simons. Comparison of Shear Strength Characteristics of Normally Consolidated Clays. In *Proceedings for Shear Strength of Cohesive Soils, ASCE,* Boulder, Colo. 1960.
9. U.S. Department of the Navy. *Soil Mechanics Design Manual 7.1, NAVFAC DM 7.1, May, 1982.*
10. New York State Department of Transportation, In *Design Manual,* Soil Mechanics Bureau. *Installation Details for Granular Fill—Slope Protection.* Standard Sheet #203-3R1. 1982.
11. Gemme, R. L. Overstress Analysis on Sensitive Clays. In *Design Manual,* Soil Mechanics Bureau, New York State Department of Transportation, March, 1978.
12. Jurgenson, L. The Application of Theories of Elasticity and Plasticity to Foundation Problems. *Contributions to Soil Mechanics 1925–1940,* Boston Society of Civil Engineers, 1940.
13. Raymond, G. P., and H. E. Wahls. Estimating 1-Dimensional Consolidation, Including Secondary Compression, of Clay Loaded From Overconsolidated to Normally Consolidated State. In *Special Report 163,* TRB, National Research Council, Washington D.C., 1976.
14. Terzaghi, K. *Theoretical Soil Mechanics.* John Wiley & sons, Inc., New York, Feb., 1954.
15. Ladd, C. C., and R. Foott. New Design procedure for Stability of Soft Clays. *Journal of Geotechnical Engineering,* ASCE, Vol. 100, No. GT7, Society of Civil Engineers 1974.
16. McGuffey, V. C. Approximate Method for Lateral Drainage. In *Soil Mechanics Bureau Design Reference Book,* New York State Department of Public Works, 1964.
17. Department of Highways—State of Colorado & University of Colorado in Cooperation with U.S. Department of Commerce. *A Review of Literature on Swelling Soils.* Bureau of Public Roads, Washington, D.C., 1964.
18. Muesser Rutledge Consulting Engineers. *South Mall Instrumentation Report.* New York State Department of Transportation, 1964.