Comparison of Methodologies for Establishing Design Properties of Horizontal Drainage in Soft Cohesive Soils

Clifton B. Farnsworth, A. Tolga Ozer, Steven F. Bartlett, and Evert C. Lawton

Estimating the rate of settlement for foundation soils treated with vertical drains requires an understanding of the horizontal drainage behavior of the soil, because the time of consolidation settlement may be critical to the overall construction schedule and sequencing. This paper provides a case study comparison of the results of methodologies associated with obtaining design parameters for horizontal drainage for use with vertical drain design, including backcalculation of field settlement data, cone penetrometer testing for pore pressure dissipation, and laboratory Rowe cell testing, by means of the soft, cohesive Lake Bonneville soil deposits in Salt Lake City, Utah. Each of these methodologies has an inherent set of strengths and limitations that should be considered when vertical drains are being designed or time of consolidation settlement is being estimated. Backcalculation of field performance data is effective in identifying true in situ settlement behavior but is not always feasible. Rowe cell testing tends to provide values that more closely correspond with those obtained from backcalculation but is not often performed. Testing for pore pressure dissipation is the most used technique, but it can provide drainage values much higher than the other two methodologies.

When large embankments are being constructed over thick, soft clay soils, the time associated with primary consolidation for vertical drainage can be lengthy, in some cases on the order of many years. For large construction projects in an urban environment, waiting periods of that magnitude can become very costly as well as burdensome to the traveling public. Prefabricated vertical (PV) drains are often installed within thick cohesive foundation soils to reduce the time associated with primary consolidation settlement by shortening the drainage path and allowing excess pore water pressures to dissipate primarily in the horizontal direction. This reduction in time of settlement can directly translate into a corresponding acceleration in time of construction.

C. B. Farnsworth, School of Technology, Brigham Young University, 230 SNLB, Provo, Utah 84602. A. T. Ozer, Civil Engineering, Okan University, Tuzla Campus, 34959, Akfirat-Tuzla, Istanbul, Turkey. S. F. Bartlett and E. C. Lawton, Civil and Environmental Engineering, University of Utah, 110 Central Campus Drive, Suite 2000, Salt Lake City, Utah 84112. Corresponding author: C. B. Farnsworth, cfarnsworth@byu.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2511, Transportation Research Board, Washington, D.C., 2015, pp. 1–8.
DOI: 10.3141/2511-01

Using accurate methods for determining the horizontal drainage properties and estimating the rate of settlement of foundation soils treated with PV drains is important because the time of settlement is often critical to the overall construction schedule and sequencing. Underestimating the time of consolidation often results in a delay in scheduled construction activities, and this delay can further adversely affect the project sequencing and potentially cost more time and money. Although being conservative can eliminate the frustration that accompanies an underestimate in the time of consolidation, being overly so is not economical or practical in the time of consolidation estimate.

This paper summarizes the results of a project undertaken to explore the relationships between various methodologies associated with obtaining design parameters for horizontal drainage for use with PV drain design and ultimately making time of settlement estimates. These methodologies include backcalculation of field settlement data, piezo cone penetrometer (CPTU) testing for dissipation of pore pressure, and Rowe cell testing in the laboratory. The study used the highly compressible, fine-grained lacustrine sediments deposited at Pleistocene Lake Bonneville in Salt Lake City, Utah (Figure 1). These soil deposits were selected because they consist of thick cohesive layers that require a long period to achieve primary consolidation, and soil investigation and settlement data for these sediments were readily available. When the embankments—8 to 10 m tall—for I-15 were initially constructed through Salt Lake City during the 1960s, 2 to 3 years were required for the underlying foundation soils to reach near the end of primary consolidation with vertical drainage. In contrast, during placement of the embankment for the more recent reconstruction of I-15 through Salt Lake City, the use of PV drains shortened the time of primary consolidation to between 6 and 12 months with horizontal drainage. Although PV drains allowed the project to be completed in a more compressed time frame, providing accurate end-of-settlement projections proved to be challenging throughout the project.

The Lake Bonneville sediments are approximately 15 m thick and generally identified by upper and lower compressible cohesive layers separated by thin interbedded layers of subaqueous silts, fine sands, and low-plastic clays (Figure 1). The upper and lower Bonneville layers consist of soils that tend to classify as low- or high-plasticity silts and clays (ML, MH and CL, CH) (*1, 2*). Between them lies the interbedded Bonneville layer, which consists of thin alternating soil layers classified as low-plasticity silts and clays (ML and CL) intermixed with clayey or silty sands (SC or SM) (*1, 2*). In general,

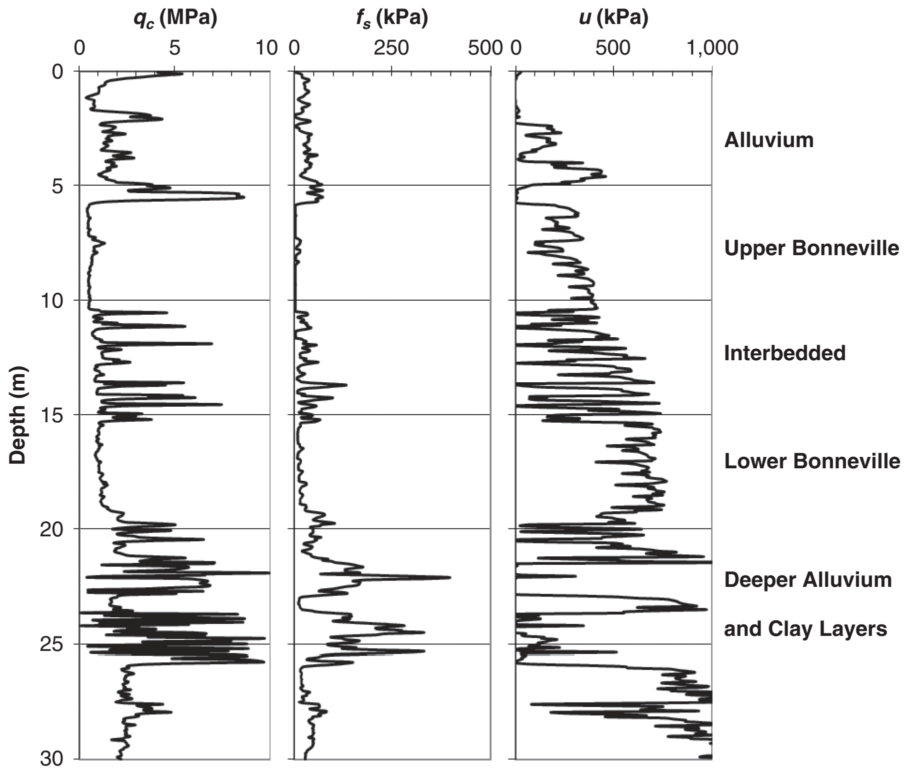

FIGURE 1 Typical CPTU log profiles for soft foundation soils at South Temple site in Salt Lake City.

the Lake Bonneville clay layers are slightly overconsolidated (overconsolidation ratio of approximately 1.5), are readily compressible (compression ratio from 0.1 to 0.35), and have fairly low undrained shear strength (25 to 50 kPa) (*1, 2*).

The location selected for this research was the South Temple site (South Temple Street and 650 West) in Salt Lake City because it exhibited highly compressible subsurface clays and was readily accessible (Figure 1). Furthermore, a large amount of subsurface characterization had previously occurred in conjunction with the Interstate reconstruction and in another research project characterizing the vertical consolidation and drainage properties for the Lake Bonneville clays (*1, 2*).

HORIZONTAL DRAINAGE PARAMETERS

Two primary drainage parameters can be obtained and used for design of consolidation settlement of fine-grained soils: the coefficient of permeability and the coefficient of consolidation, which are directly related. The methodologies evaluated in this research all estimate the coefficient of consolidation, but in situ techniques that measure direct flow can also be used to determine the soil permeability. In either case, knowing one of the drainage parameters and the vertical compressibility of the soil allows for calculation of the other.

Consolidation settlement of a soil takes place after an increase in the loading conditions and a corresponding increase in pore water pressure within the soil. As the induced excess pore water pressures dissipate and water is expelled from the void spaces within the soil mass, simultaneously the soil particles move closer together. Foundations treated with PV drains undergo radial consolidation, the process of vertical compression resulting from horizontal drainage (*3*). The processes of vertical and radial consolidation are similar, with the exception of the direction of dissipation of excess pore pressure. However, radial consolidation is mathematically more complex because the distance between the flow paths decreases as they approach the PV drains, as opposed to remaining constant throughout the vertical drainage system. In either case, the drainage of excess pore water pressure with respect to time is a function of the coefficient of consolidation, either c_v or c_h, for vertical or radial (i.e., horizontal) consolidation, respectively. Thin layers undergoing radial consolidation experience some effect from vertical drainage, but these effects are minor when the compressible layer is more than 5 m in thickness (*4*). To be conservative for thinner layers, researchers often still disregard the vertical drainage. However, in this study, both the Lake Bonneville clay layers are thick enough that any vertical drainage was neglected.

The coefficient of consolidation is a lumped parameter having the material properties that govern the consolidation process by combining both the effects of compressibility and permeability of the soil. The only practical difference between the coefficients of horizontal and vertical consolidation is the difference between the horizontal and vertical permeability. The other two parameters that contribute, the unit weight of water and the vertical compressibility of the soil, remain the same for both conditions. The vertical coefficient of consolidation is typically obtained from a one-dimensional laboratory consolidation test. Different methods of obtaining the horizontal coefficient of consolidation are discussed and compared later.

Most clays tend to have a larger permeability in the horizontal direction than the vertical because of the depositional environment that exists when clay layers are formed under subaqueous conditions. Representative values of the ratio (k_h/k_v) of horizontal permeability (k_h) to vertical permeability (k_v) for the Lake Bonneville clay layers have often been thought to best fit within a description of slight layering (e.g., sedimentary clays with occasional silt dustings to random silty lenses) and therefore assumed to have a horizontal to vertical permeability ratio somewhere between two and five (*5*). This ratio for the Lake Bonneville clay layers was also explored as part of the research and is addressed later.

METHODS FOR OBTAINING c_h

Backcalculation of Field Data

Field performance data of primary consolidation settlement can be used to back-calculate the horizontal coefficient of consolidation for soils treated with PV drains by using the technique described by Bergado (*6*) in conjunction with the Asaoka projection method (*7*), an observational technique that uses actual settlement data for determining the end of primary consolidation. Successive equal time steps for the actual settlement data are required, and the resulting settlement for each time step is plotted against the settlement for each respective previous time step. The corresponding first-order difference line intercepts a 1:1 slope line and provides both an estimate of the magnitude of settlement and the time to end of primary settlement. The slope of this Asaoka projection line is then used to back-calculate the effective coefficient of consolidation, $c_{h(e)}$, an average or composite value for the entire layer (*8*).

Bergado demonstrated the technique for backcalculating $c_{h(e)}$ and showed that the calculated value is a function of three unknowns: the ratio (k_h/k_s) of the horizontal permeability of the soil to the horizontal permeability of the soil in the smear zone (k_s), the ratio (d_s/d_w) of the diameter of the smear zone (d_s) to the diameter of the PV drain (d_w), and the discharge capacity of the PV drain (q_w) (*6*). Underestimating the smear effects of PV drain installation or overestimating the discharge capability of the drain will result in underestimation of $c_{h(e)}$ values. However, for PV drains that provide sufficient flow for the drainage capacity of the soil fabric, the discharge capacity term becomes insignificant and does not affect the calculated $c_{h(e)}$ value. For this research, the discharge capacity of the PV drains, q_w, was assumed to be sufficiently large not to limit the drainage of the excess pore pressures because the drains were effective in significantly reducing time of settlement. On the basis of the geometry of the installation mandrel for the PV drain, the ratio of d_s/d_w was determined to be about 3.65. For soils with fairly well to well-developed macrofabric (e.g., sedimentary clays with discontinuous lenses and layers of more permeable material), the ratio of k_h/k_s is about two to four, while, for varved clays and other deposits containing embedded and more-or-less continuous permeable layers, the ratio of k_h/k_s may vary from about three to 15 (*6*). Initially, a value of four was selected for this research because the Lake Bonneville clay layers seemed to fit within the two descriptions provided above (*8*). However, a more accurate lower value was subsequently determined and is provided later in this paper.

During the more recent I-15 reconstruction project, a spider magnet extensometer was placed within the foundation soils at the South Temple site to capture the compression of the individual layers during embankment construction (Figure 2) (*8, 9*). The magnet elevations were selected to correspond with the interface between major subsurface soil layers. The readings from this instrument were used in conjunction with the method discussed above to back-calculate the effective horizontal coefficient of consolidation for each of the different magnet intervals, as well as a composite (or average) value for the entire subsurface profile (*6*). The resulting effective horizontal coefficient of consolidation, $c_{h(e)}$, values are shown in Figure 2. A single settlement point from the surface was used to calculate a composite $c_{h(e)}$ value of 14 mm^2/min for the entire subsurface profile. Likewise, individual $c_{h(e)}$ values of 9, 17, and 12 mm^2/min were calculated for the magnet intervals corresponding with the lower Bonneville, interbedded Bonneville, and upper Bonneville clay layers, respectively. These values essentially represent the baseline values for this study because they indicate the true behavior of the soil–PV drain system.

The variance of the $c_{h(e)}$ between layers was smaller than expected. The authors anticipated that the granular layers would exhibit a much larger drainage potential than the adjacent clay layers. However, this smaller variance could potentially be attributable to a smear zone

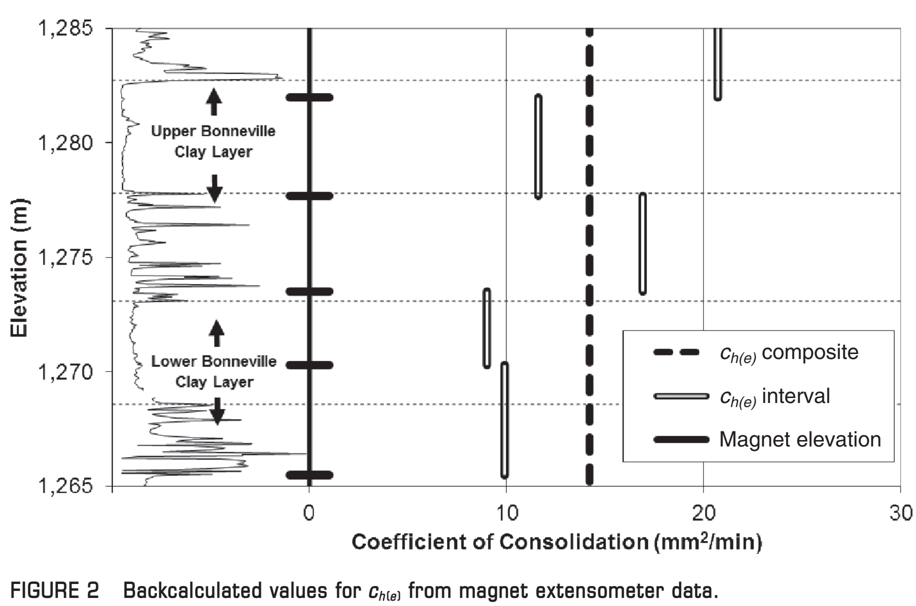

FIGURE 2 Backcalculated values for $c_{h(e)}$ from magnet extensometer data.

created for the entire length of the PV drain during its installation. In other words, the granular layers may not be reaching their natural drainage potential because of clay smeared along the interface between the drain and the adjacent granular soil from overlying clay layers. However, these results represent the reality of practical construction application and true field behavior.

CPTU Test for Dissipation of Pore Pressure

The test for dissipation of pore pressure is the most common in situ technique for determining the coefficient of consolidation, c_h, and is readily performed with a CPTU test. Although a number of methods for interpreting the data have been developed, the Teh and Houlsby method (*10*) is the most accurate (*11*). This method follows the premise that theoretical dissipation curves dependent on soil stiffness can be normalized by the square root of the rigidity index (I_R) to provide a single theoretical dissipation curve for soils of variable stiffness (*10*). As the cone penetrometer is advanced through the soil, the shearing action of the soil displacement generates excess pore water pressures. When the depth of the dissipation test is reached, the advancement of the probe is halted, and pore water pressure is measured until the excess pressure has reached at least 50% dissipation.

Use of the pore pressure dissipation test has several limitations. First, this test requires knowing the rigidity index of the soil. Because measuring this value at each test depth would be impractical, a general value is typically used for the soil type. However, this generalizing assumption potentially limits the accuracy of the c_h calculations. The Lake Bonneville clay deposits are thought to have an $I_R = 50$, the value used in this research (*8*). Another limitation of this test is the tendency of the pore pressure element within the cone to become desaturated when the cone is pushing through dry or dilating granular layers. Constant complete saturation is crucial to producing high-quality reliable pore water pressure readings. When desaturation occurs, the pore pressure element generally resaturates during the dissipation test. However, the data gathered between test initiation and resaturation are erroneous because the true peak excess pore pressure value is not measured correctly and the corresponding time to 50% dissipation may also be inaccurate. Finally, dissipation of excess pore pressure occurs over a small area and can be heavily influenced by thin drainage lenses, meaning that the results may not accurately represent the true macrostructure of the soil layer.

For this research, two sets of pore pressure dissipation tests were performed at 0.5-m intervals within the Lake Bonneville clay deposits at the South Temple site (*8*), and interpretation of these tests followed the Teh and Houlsby method (*10*). The resulting values for the horizontal coefficient of consolidation, c_h, for the upper Bonneville clay layer varied between 22 and 780 mm^2/min, with an average value of 175 mm^2/min, and for the lower Bonneville clay layer, c_h varied between 13 and 122 mm^2/min, with an average value of 44 mm^2/min (Figure 3). Under the assumption that the maximum outliers (two in the upper layer) could be removed from the data set because they appeared to have been heavily influenced by porous seams, the average value of c_h decreased to 114 mm^2/min for the upper Bonneville clay layer (Figure 3). Dissipation testing was not performed in the interbedded Bonneville layer. For this data set, the median values were similar to the average values but slightly lower. Although average values were used in this research, the use of median values might be considered given the tendency for the thin porous seams to affect the test results.

When compared with the backcalculated c_h values, the dissipation results were significantly larger, by a factor of approximately five and 10 for the lower and upper Bonneville clay layers, respectively. Reduction of the assumed value of I_R and a decrease in the disparity in the results are possible. However, the value used in this research was already on the lower end of the expected range for clays, and therefore the results are more likely being influenced by adjacent thin porous drainage seams. In addition, removal of additional large values may be possible from the data set for those tests with results that appear to be influenced by adjacent drainage layers, with subsequent reduction in the overall average. However, with so much scatter in these data, determining which test results can appropriately be removed and which should remain is difficult. One approach might be simply to average the bottom third of each

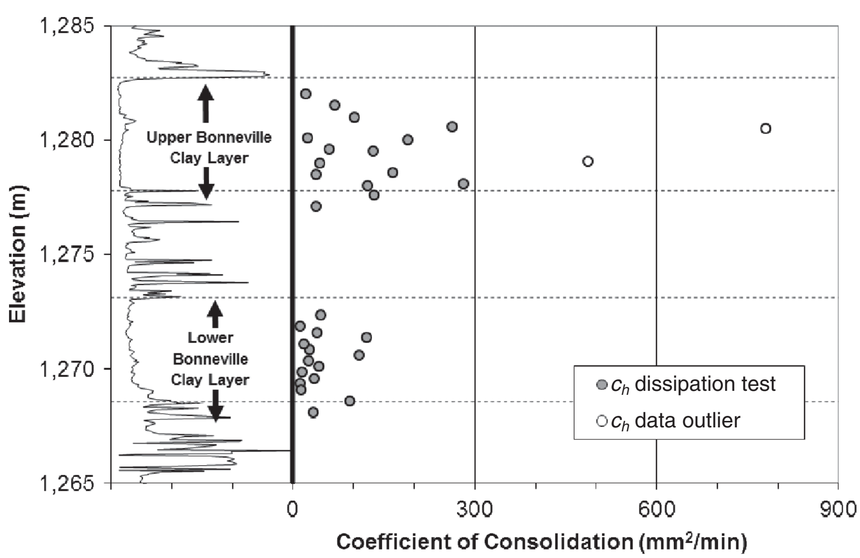

FIGURE 3 Values for c_h from pore pressure dissipation test.

data set; doing so in this case would decrease the ratio of dissipation to backcalculated values to factors of two and three for the lower and upper Bonneville clay layers, respectively. However, the minimum dissipation value measured for each of the clay layers in this study is still 1.5 to two times the average backcalculated values representing actual drainage behavior. The values measured with the dissipation test simply overestimated the actual horizontal drainage properties of the soil.

Laboratory Testing

One of the primary challenges of laboratory testing of soils is the difficulty in obtaining minimally disturbed samples in the field. Samples for this research were obtained by pushing a Shelby tube with a piston sampler, a method shown to provide much less sample disturbance than conventional methods (*12*). Despite the best efforts to minimize sample disturbance during sample collection, handling, transportation, storage, and laboratory testing, some disturbance will inherently occur and affect the test results. The permeability of a soil sample is especially sensitive to changes in the void ratio and soil fabric. Another inherent issue with laboratory testing is the size of the small test specimens, which might not accurately represent the macrostructure of the layer. Furthermore, specimen preparation tends to use the homogeneous parts of the clay sample, potentially introducing a bias toward the least permeable parts of the clay layer.

Radial Consolidation Test

The Rowe cell is a laboratory device used to perform one-dimensional consolidation testing of soil samples. These devices differ from standard laboratory consolidometers in that they include measurement of pore water pressure, drainage control, and application of back pressure to the specimen. A Rowe cell may be configured to perform consolidation testing with either vertical or horizontal drainage, thus providing either the vertical or horizontal drainage characteristics of the soil. This research used horizontal drainage following the general one-dimensional consolidation test standards in ASTM D2435-96, for sampling, specimen preparation, and general consolidation test procedures and British Standards Institution procedure BS 1377-6 associated with radial consolidation. The horizontal coefficient of consolidation, c_h, values were obtained by using the average of two empirical curve-fitting methods: the plot of settlement against the log of time and the plot of settlement against the square root of time (*8, 13*). The method using the log of time is probably a more consistent method of interpretation because the method using the square root of time is extremely sensitive to the location of the line drawn through the linear portion of the early part of the curve. However, for more highly permeable materials, the primary consolidation portion of the test occurs so rapidly that the resulting data do not readily fit the distinct consolidation shape for the log of time (backward S). When this rapid consolidation occurs, identifying the break between primary and secondary settlement is not easy. Therefore, the method using the square root of time is still considered a worthwhile tool for estimating the coefficient of consolidation (*8, 13*).

The Rowe cell test was run on 22 specimens from the South Temple site for this research, and the horizontal coefficient of consolidation, c_h, results are shown in Figure 4 (*8, 13*). As with the values from the other methodologies examined in this research, the c_h values were estimated for the full-height embankment load and in essence represented the behavior expected during consolidation. These results were therefore obtained by interpolating between the load steps in the consolidation test to a c_h value corresponding with the equivalent vertical effective stress of the soil column and embankment load. Both the upper and lower Bonneville clay layers had values apparently influenced by higher permeability layers that did not seem to represent the overall behavior of the full layer (Figure 4). Although shown, these outliers were not used in calculating the average drainage behavior for the clay layers. The resulting values of c_h varied between 9 and 96 mm^2/min, with an average value of 37 mm^2/min, and 4 and 15 mm^2/min with an average value of 9 mm^2/min, for the upper and lower Bonneville clay layers, respectively.

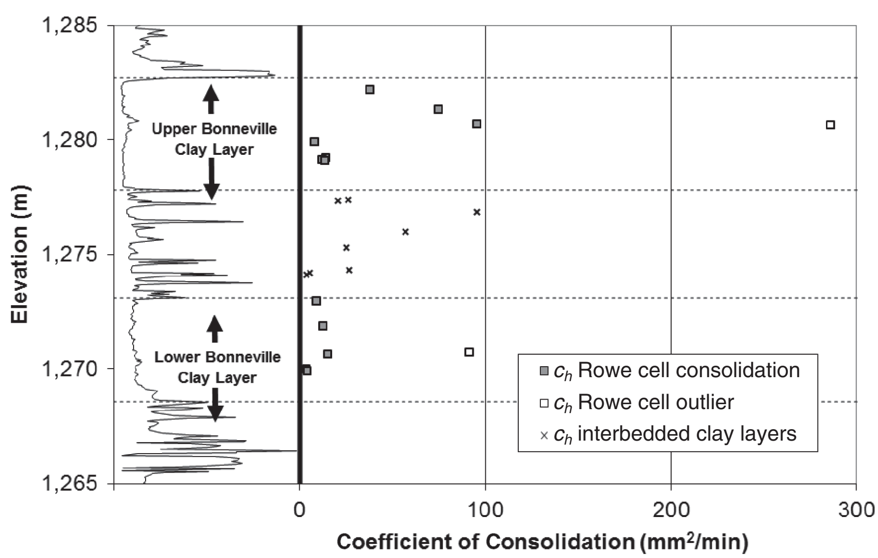

FIGURE 4 Values for c_h from Rowe cell consolidation testing.

When compared with the backcalculated c_h values, the Rowe cell results greatly resembled the actual drainage behavior of the Lake Bonneville clay layers. With the removal of the single outlier for the lower Bonneville clay layer, the average value of 9 mm^2/min was the same for both methods (Figure 4). With the removal of the single outlier from the upper Bonneville clay layer data set, the average value for the Rowe cell was three times that of the backcalculated value (Figure 4). However, several of the other large data values in the Rowe cell results seemed to be heavily influenced by adjacent drainage layers. Removing the three additional three large values from the data set made the average Rowe cell value identical to the backcalculated value, 12 mm^2/min (Figure 4). This result implies that removal of data appearing to be influenced by adjacent drainage layers may be warranted. Furthermore, the lower-end data measured with the Rowe cell in this study (excluding the apparent outliers) were nearly identical to the actual drainage behavior of the foundation soils during consolidation settlement.

Vertical Consolidation Test

A laboratory method commonly used for estimating the horizontal coefficient of consolidation, c_h, is to run a one-dimensional consolidation test, obtain the vertical coefficient of consolidation, c_v, and then make an assumption about the relationship between c_h and c_v. As indicated earlier, the permeability in the horizontal direction is often greater than in the vertical, and for the Lake Bonneville clay layers, the ratio of horizontal to vertical is often thought to be somewhere between two and five. Research performed by Ozer (*2*) measured the vertical coefficient of consolidation for the Lake Bonneville clay layers by using constant-rate-of-strain consolidation testing in the laboratory (Figure 5) (*14*). Values of c_v were obtained at in situ stress, between the in situ stress and preconsolidation stress, and beyond the preconsolidation stress through the end of the test. Much more scatter occurred in the results before the preconsolidation stress was reached (Figure 5). The values of c_v measured after the preconsolidation stress was exceeded best represent the stress levels achieved beneath large embankment placement and likewise the controlling rate of consolidation settlement. Therefore, only the c_v values beyond the preconsolidation stress were selected for further comparison in this research. The values of c_v varied between about 1 and 32 mm^2/min, with an average value of 12 mm^2/min, and 2 and 10 mm^2/min, with an average value of 5 mm^2/min, for the upper and lower Bonneville clay layers, respectively. When compared with the results from backcalculation and the Rowe cell test, the values of c_h are approximately 1.0 and 1.8 times the values of c_v for the upper and lower Bonneville clay layers, respectively. This difference indicates that the anisotropy within the Lake Bonneville clay layers may not be as great as previously thought in relation to practical construction application. These results also support recent research regarding the c_h/c_v ratio attributable to the smear effect during PV drain installation (*15*). Although these results are specific to the Lake Bonneville clay layers, this type of comparison could be useful in establishing the anisotropy relationship for other cohesive soils, thus making the vertical consolidation test results more readily available for estimating the horizontal drainage properties of the soil.

COMPARISON OF c_h RESULTS

Thus far, three methodologies used to obtain the horizontal coefficient of consolidation, c_h, have been identified and briefly described: backcalculation of field data using the Asaoka method, the CPTU pore pressure dissipation test, and the laboratory Rowe cell. Each of these methods was used to calculate c_h values for the Lake Bonneville clay layers. The principal purpose of this paper is to compare and contrast the results obtained by using these methods. The average c_h values for the Lake Bonneville clay layers obtained from the different methodologies are shown in Figure 6. The average vertical coefficient of consolidation, c_v, values obtained from one-dimensional constant-rate-of-strain consolidation testing are also shown in Figure 6. The backcalculated values represent actual field behavior at the South Temple site and thus serve as the practical control values against which to compare the other methodologies.

This research demonstrated that the c_h values obtained in the laboratory with a Rowe cell are quite similar to the $c_{h(e)}$ values obtained

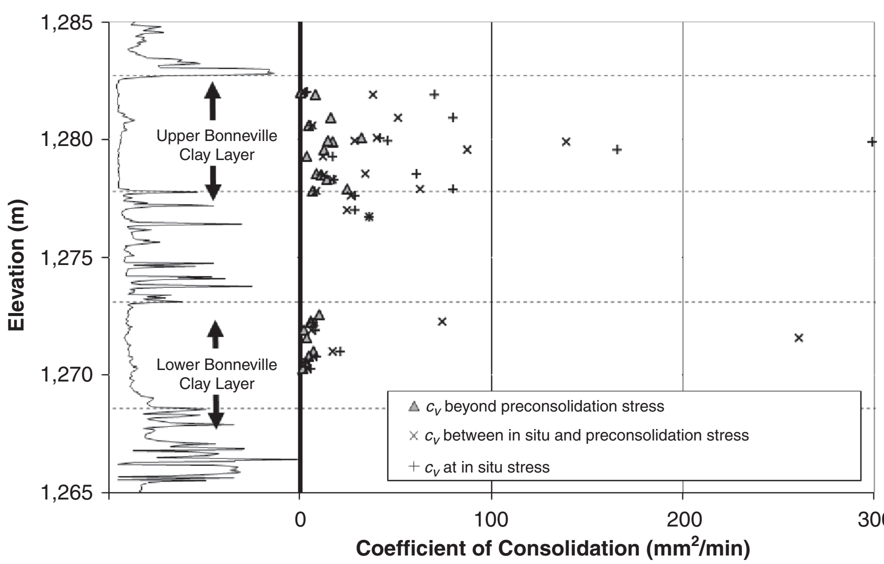

FIGURE 5 Values for c_v from constant-rate-of-strain consolidation testing.

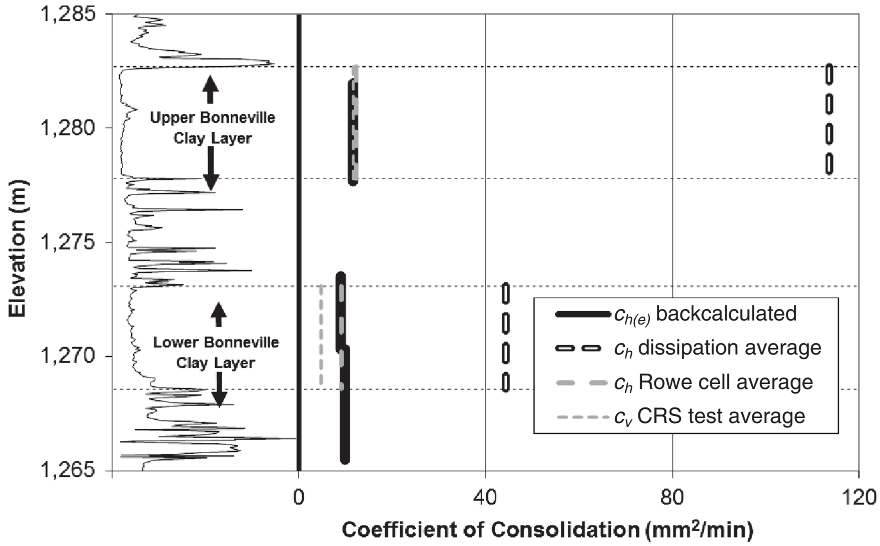

FIGURE 6 Comparison of values for average coefficient of consolidation (CRS = constant rate of strain).

through backcalculation with the Asaoka method. However, the values of c_h obtained with the CPTU test for dissipation of pore pressure were significantly higher. While adjacent drainage layers may have some influence on Rowe cell testing, the process of selecting and preparing clay samples within the laboratory seems to use the most cohesive portions of the tube samples and to avoid the use of any specimen with silt or sand lenses. In contrast, the test for dissipation of pore pressure appears to be heavily influenced by adjacent drainage layers. The testing for dissipation of pore pressure for this research was performed strictly on a depth basis as opposed to attempting to target specific cohesive layers. Perhaps if more selective depth targeting had been performed, attempting to avoid edge effects from adjacent granular layers, the values might have been slightly smaller.

In relation to the practical difference in c_h values obtained, an engineer will more than likely select a single value for the coefficient of consolidation when performing an estimate of settlement time for a clay layer not much unlike the average values presented (Figure 6). An engineer may opt to be more conservative and select the lower-bound data, but for this research the average values are presented. Calculating the time to 98% consolidation on the basis of average c_h values for the Rowe cell (12 mm^2/min) and pore pressure dissipation (114 mm^2/min) tests, with triangular spacing for the PV drains of 1.5 m, provides quite different estimates of time for consolidation in the upper Bonneville clay layer: 201 and 21 days, respectively. While an experienced engineer might recognize that 21 days to 98% consolidation for the Lake Bonneville clays is too aggressive, the investigation data still suggest that this is the case. This small example is provided simply to demonstrate the importance of obtaining appropriate estimates of the drainage characteristics of soft soils when estimates of time of settlement are made. The unfortunate results of a poor time estimate are construction delays that can negatively affect the time and ultimately the cost of construction. Further problems occur when construction progresses too quickly and the final structure must absorb the remaining settlement, possibly leading to poor performance, high maintenance costs, or even premature failure.

The results also indicate that c_v values obtained with constant-rate-of-strain consolidation testing are at or slightly below c_h values obtained from laboratory Rowe cell consolidation testing and backcalculation of field settlement data (Figure 6). This research has also demonstrated that the anisotropy of the Lake Bonneville clay layers may not be as high as previously thought. Having a better understanding of the anisotropy of the Lake Bonneville soils makes it possible to use vertical consolidation data more readily in conjunction with PV drain design.

CONCLUSIONS

Having a good estimate of the horizontal drainage characteristics of the soil is critical for making an accurate estimate of the time of settlement for soil treated with the PV drain. This paper compared the results from three methods for obtaining the horizontal coefficient of consolidation: backcalculation using the Asaoka method, CPTU testing for dissipation of pore pressure, and laboratory Rowe cell testing. This research has demonstrated that the Rowe cell testing provided results nearly identical to those from actual backcalculated field behavior. However, the test for dissipation of pore pressure provided results that were five to 10 times as large.

Backcalculation of field data is the most accurate approach for obtaining the horizontal drainage characteristics of the subsurface soils because the results identify true subsurface settlement behavior. This methodology can be used for calculating an average estimate for the performance of global subsurface settlement or, when used in conjunction with magnet extensometer data, estimating the settlement behavior of individual layers. Unfortunately, this method requires full-scale data on embankment settlement from initial load through primary consolidation and is therefore not always practical, especially during a routine geotechnical soil investigation. This method requires making assumptions about the smear effects of the soil and the drainage capacity of the PV drains.

Rowe cell testing provided horizontal drainage results that best matched actual field behavior when the lower-bound data were considered. As with most laboratory tests, it provides a great deal of opportunity to introduce disturbance and error between obtaining undisturbed samples from the field and performing the test in the laboratory, and caution must be exercised to minimize unnecessary error. Although the results exhibited some minor scatter, thus demonstrating the variability of the soil, in this case the lower-bound data provided the most accurate representation of the controlling behavior of the macrofabric of the entire soil layer. The standard one-dimensional consolidation test is a more commonly performed laboratory test, which can be used effectively if the relationship between the horizontal and vertical drainage characteristics is readily understood.

The pore pressure dissipation test can be performed quickly and economically during routine CPTU subsurface exploration. However, the results for this methodology were significantly larger than the values obtained in the laboratory or from actual field data. This methodology requires an assumption about the rigidity index of the soil. Although the value used in this research ($I_R = 50$) is already on the lower end of the range expected in clays, decreasing this value decreases the disparity in the results. However, the results are probably more like to be heavily influenced by adjacent subsurface drainage layers.

The ultimate goal in selecting design parameters from any of these methods is to provide realistic, practical estimates of time of settlement. This research suggests that the more conservative values obtained by using backcalculation of magnet data and Rowe cell testing best matched actual field behavior. The Rowe cell is not used much within the United States, but it is an effective laboratory tool that should get more use in obtaining horizontal drainage design parameters. The CPTU test for dissipation of pore pressure is the most common of the three methodologies because of its simplicity and availability. However, as this research has shown, the results from this test varied greatly, tended to be heavily influenced by adjacent drainage layers, and were significantly higher than actual field behavior. Users should exercise prudent judgment in selecting design values with this methodology.

ACKNOWLEDGMENT

The authors acknowledge and thank the Utah Department of Transportation for its financial contributions to and technical support of this research project.

REFERENCES

1. Bartlett, S. F., and A. T. Ozer. *Estimation of Consolidation Properties from In-Situ and Laboratory Testing.* Report UT-04.20. Utah Department of Transportation, Salt Lake City, 2004.
2. Ozer, A. T. *Estimation of Consolidation and Drainage Properties for Lake Bonneville Clays.* PhD dissertation. University of Utah, Salt Lake City, 2005.
3. Stamatopoulos, A. C., and P. C. Kotzias. *Soil Improvement by Preloading.* John Wiley & Sons, New York, 1985.
4. Elias, V., J. Welsh, J. Warren, and R. Lukas. *Ground Improvement Technical Summaries.* FHWA-SA-98-086. FHWA, U.S. Department of Transportation, 1999.
5. Lawton, E. C. Soil Improvement and Stabilization: Nongrouting Techniques. In *Practical Foundation Engineering Handbook* (R. W. Brown, ed.), McGraw-Hill, New York, 2000, p. 6.261.
6. Bergado, D. T. Asian Experiences on the Use of Prefabricated Vertical Drains (PVD) in Soft Bangkok Clay. *Proc., 1st International Symposium on Lowland Technology,* Saga, Japan, 1998, pp. 49–60.
7. Asaoka, A. Observational Procedure of Settlement Prediction. *Soils and Foundations,* Vol. 18, No. 4, 1978, pp. 87–101.
8. Farnsworth, C. B., and S. F. Bartlett. *Evaluation of Methods for Determining Horizontal Drainage Properties of Soft Clayey Soils.* Report UT-08.11. Utah Department of Transportation, Salt Lake City, 2009.
9. Bartlett, S. F., and C. B. Farnsworth. *Monitoring and Modeling of Innovative Foundation Treatment and Embankment Construction Used on the I-15 Reconstruction Project, Project Management Plan and Instrument Installation Report.* Report UT-04.19. Utah Department of Transportation, Salt Lake City, 2004.
10. Teh, C. I., and G. T. Houlsby. An Analytical Study of the Cone Penetration Test in Clay. *Geotechnique,* Vol. 41, No. 1, 1991, pp. 17–34.
11. Abu-Farsakh, M. Y., and M. D. Nazzal. Reliability of Piezocone Penetration Test Methods for Estimating the Coefficient of Consolidation of Cohesive Soils. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1913,* Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 62–76.
12. Bay, J. A., L. R. Anderson, J. C. Hagen, and A. S. Budge. *Factors Affecting Sample Disturbance in Bonneville Clays.* Report UT-03.14. Utah Department of Transportation, Salt Lake City, 2003.
13. Farnsworth, C. B., S. F. Bartlett, and E. C. Lawton. Using a Rowe Cell to Establish Horizontal Drainage Properties of Soft Soils. *Proc., Geo-Congress 2014,* Atlanta, Ga., Feb. 23–26, 2014, ASCE, Reston, Va., pp. 157–169.
14. Ozer, A. T., E. C. Lawton, and S. F. Bartlett. New Method to Determine Proper Strain Rate for Constant Rate-of-Strain Consolidation Tests. *Canadian Geotechnical Journal,* Vol. 49, No. 1, 2012, pp. 18–26.
15. Rollins, K. M., and G. M. Smith. *Reduction in Wick Drain Effectiveness with Spacing for Utah Silts and Clays.* Research Report No. UT-12.04. Utah Department of Transportation, Salt Lake City, Utah, 2012.

The Standing Committee on Transportation Earthworks peer-reviewed this paper.