UNIVERSITY OF MINNESOTA

64th Annual Geotechnical Engineering Conference

Design and instrumentation of a widened interstate embankment constructed over soft floodplain soils

Liang Chern Chow, E.I.T.
American Engineering Testing, Inc., St. Paul, Minnesota USA

Joseph G. Bentler, P.E.
American Engineering Testing, Inc., St. Paul, Minnesota USA

ABSTRACT: This paper presents the design and stability performance of the Interstate I-90 highway embankment constructed in La Crosse, Wisconsin. Geotechnical investigations consisting of soil borings, piezocone (CPTu) soundings, and field vane (FV) shear tests indicated a thick soft organic clay at shallow depth, underlain by interbedded sand and clay layers. A design approach that involved muck-excavation, backfilling, and surcharging was preferred over the originally proposed design to install vertical drains. In order to assess stability issues during construction, undrained strength stability analyses were performed based on the shear strengths obtained from CPTu and FVs. Measurements of overburden stress, settlements, lateral deformations, and excess pore water pressure from extensive geotechnical instrumentation have shown a good agreement with predictions evaluated by using one-dimensional consolidation theory. In addition, monitoring the lateral displacement behavior during construction has provided useful information to better understand embankment behavior in floodplain soils.

1 INTRODUCTION

The I-90 Dresbach Bridge project in the La Crosse, Wisconsin and La Crescent, Minnesota region involved replacement of the existing 2500 ft (762 m) long steel bridge over the Mississippi River with a new concrete structure. Due to the construction of the new bridge a short distance north of the existing bridge, the east (Wisconsin) approach embankment required a significant widening of the maximum 35 ft (11 m) high embankment onto the adjacent marshy floodplain. During the preliminary design phase, soil borings and piezocone (CPTu) soundings were performed, but they were limited to the toe of the existing embankment. A base design of staged construction which involved vertical drains, geosynthetic reinforcements, and surcharge was subsequently recommended. The total settlements projected at end of construction were 0.7 ft to 2.2 ft (213 mm to 671 mm) at various cross sections. Following a supplemental geotechnical investigation using additional soil borings, CPTu soundings, and field vane (FV) shear tests, re-evaluation of the design showed that the required embankment stability and settlement performance could be achieved with a method of muck-excavation, backfilling and surcharging, which the contractor determined was more economical and elected to use instead of the originally proposed design. Additional piezocone soundings were conducted one month after muck-excavation and backfilling to evaluate the success of the muck-excavation process that was performed partially submerged.

Ladd and Foott (1974) and Mesri (1975) showed that for a particular natural cohesive soil deposit, undrained shear strength, S_u can be normalized with effective overburden stress, σ'_{v0} resulting in a constant S_u / σ'_{v0}. This relationship is particularly true in a homogenous clay layer, where S_u may linearly increase with depth. However, when S_u varies with depth irregularly, the soil should be considered to consist of different average mobilized undrained shear strength at different layers. An investigation of failure, e.g. embankment loading, based on the average mobilized undrained shear strength was referred to as an undrained strength stability analysis by Mesri (1983) and subsequently as undrained strength analysis by Ladd (1991). Furthermore, because undrained shear strength varies with rate of shear and time to failure, selecting which undrained shear strength measurement, e.g. field vane, unconfined compression tests etc., will be used to assess the S_u profile requires experience and judgment (Terzaghi et al., 1996). In order to demonstrate that the required factor of safety for overall stability would be maintained throughout construction, lower bound values of the average S_u values as interpreted and measured from CPTu and from FVs, respectively, were used to evaluate the embankment stability. Meanwhile, settlement analysis based on one-dimensional consolidation theory was also performed to estimate total settlement at the end of construction. In order to monitor the settlement and stability performance of the highway embankment, extensive geotechnical instrumentation including vertical and horizontal ShapeAccelArrays (SAAs), earth pressure cells (EPC), and piezometers (PZ) were installed, in conjunction with an automated remote datalogging system.

2 GEOTECHNICAL INVESTIGATIONS

2.1 *Geological and subsurface condition*

Based on Bulletin 101 of the Wisconsin Geological and Natural History Survey (Evans, 2003), the naturally occurring soils in this area consist of Holocene age alluviums. These sediments are generally described as fine- to medium-grained sand with gravel in places and are characterized predominantly by quartz grains. Extensive areas of marshlands containing organic sediment, peat, and muck are also commonly present along the Mississippi River valley.

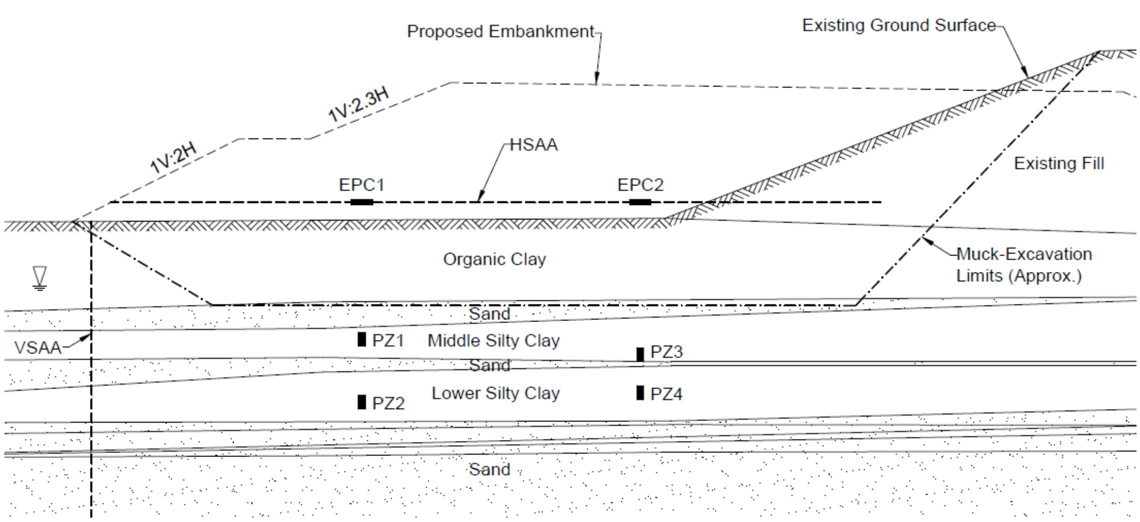

Figure 1. Schematic cross section of embankment and instrumentation. North is to the left.

Geotechnical investigations conducted at the site consisting of 18 borings and 25 CPTu soundings revealed interbedded layers of non-uniform fine and coarse alluvial soils. At the bridge abutment, subsurface soils are coarser alluviums becoming finer as the alignment moves east along the embankment. The fine alluvium is generally soft, and is described as clay loam (CH), plastic fine sandy loam (CL), and plastic silt loam (CL). These soils generally extend to depths on the order of 13 to 18 ft (4 to 5.5 m); however, these soils are thicker near the central 250 ft (76 m) long portion of the embankment alignment where they extend to a depth of about 38 ft (11.6 m). Even in that area, an intermediate alluvial sand layer was encountered between typical depths of 13 to 16 ft (4 to 5 m). The coarse alluvial soils are generally sand and loamy sand (SP). Groundwater was encountered at a depth of about 10 feet below original ground surface. Figure 1 shows the cross section of interbedded layers and instrumentation locations.

That central 250-ft (76-m) long portion of the alignment is the focus of this paper, and Figure 2 denotes results from supplemental (C105) and after muck-excavation (C211) piezocone soundings at the center. Sounding C105 shows that the soil profile consists of interbedded sand and clay layers. Starting from the ground surface at elevation 635 ft, the upper 13 ft (4 m) is a soft organic clay layer, underlain by a 4 ft (1.2 m) thick sand layer. Below the sand layer is a 19 ft (5.8 m) thick silty clay layer that extends to elevation 599 ft. Within this silty clay layer is a 1 ft (0.3 m) thin alluvial sand layer. Based on this information, it was recommended to muck-excavate 15 ft (4.6 m) then backfill to the original grade. One month after the backfilling was completed, another sounding C211 was pushed at 40 ft (12 m) away from C105 to confirm that the soil correction was successful. High tip resistance and low friction ratio from the C211 sounding indicate that the upper soft organic clay had been replaced with sandy backfill. It is interesting to note the relatively lower tip resistance of the sand fill at C211 below elevation 629 ft. Fill below that elevation was bulldozed "in the wet" into standing water within the excavation, while fill above that elevation was compacted.

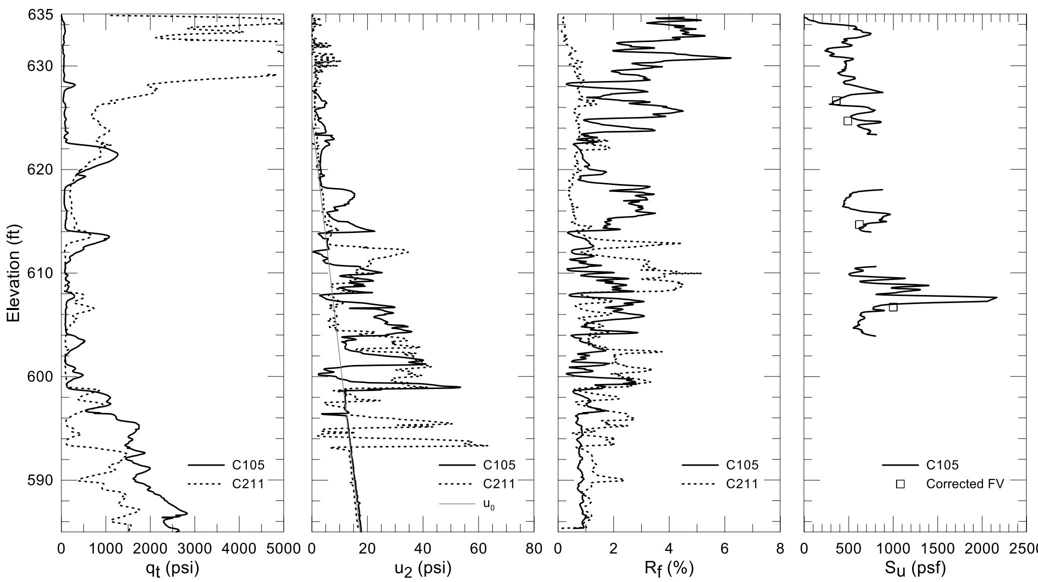

Figure 2. Piezocone sounding results. From left to right: tip resistance, pore pressure, friction ratio, and undrained shear strength.

2.2 *Compressibility and consolidation parameters*

The compressibility indices, C_c and C_r, and preconsolidation pressure, σ'_p values used to estimate consolidation settlement were determined from oedometer tests. Three thin-walled tube samples were obtained from the soft soil layers in the central portion of the alignment, at a location offset about 36 ft (11 m) from C105, for laboratory characterization. The resulting parameters are presented in Table 1. Correlations based on cone resistance and overburden stress have been used to verify preconsolidation pressure of the soft soils. The ratio C_r/C_c is useful and convenient because it is comparable with an empirical correlation with natural water content. For most low to medium compressibility soils, C_r/C_c is in the range of 0.02 to 0.2.

Table 1. The oedometer test results during preliminary geotechnical investigation.

Depth (ft)	Depth (m)	USCS Soil Type	ω_0 (%)	LL (%)	σ'_p (psf)	σ'_p (kPa)	OCR	C_c	C_r	C_r/C_c	c_v* (cm^2/sec)
7	2.1	CH	44	56	1960	93.8	2.7	0.41	0.08	0.20	-
21	6.4	CL	39	37	1800	86.1	1.4	0.32	0.04	0.13	0.003
27	8.2	CL	35	30	1600	76.6	1.0	0.20	0.03	0.15	0.002

*Computed from c_h via CPTu dissipation tests and assumed ratio k_v/k_h = 0.1.

When Terzaghi's theory of one-dimensional consolidation is used, the coefficient of consolidation, c_v is required. The values of c_v can be determined from oedometer time-deformation data, by the classical "curve-fitting methods" developed by Casagrande and Taylor. However, obtaining a c_v value to represent field conditions can be misleading because soil borings can potentially miss thin sand layers, especially in the encountered soil profile (see Figures 1 and 2). Those thin sand layers, if relatively interconnected, can significantly affect the average drainage boundary conditions for the clay layers. Only by performing a significant number of borings and CPTu soundings was it apparent that the thin sand layers were widespread within the clay foundation soils. Furthermore, by performing CPTu dissipation tests, the coefficient of consolidation in horizontal direction, c_h and an anisotropy permeability ratio, k_v / k_h can be used to estimate more representative c_v values (see Table 1).

2.3 *Undrained shear strength*

It is commonly known that undrained shear strength, S_u is not a unique property of cohesive soils because it depends on the way which the soil is brought to failure. Soil disturbance is one of the many factors that affects the measurement of undrained shear strength, whether *in situ* or in laboratory. Although disturbance cannot be completely eliminated, field vane shear testing provides reliable measurements of *in situ* strength because it is the most comprehensively calibrated *in situ* test for mobilized undrained shear strength, S_u(mob) in soft clays and silts (Terzaghi et al., 1996). The corrected values of S_{u0}(FV) with plasticity are provided in Figure 2. Note that subscript "0" in "S_{u0}(FV)" represents measured vane strength without any waiting period for strength gained. The overall S_u profile was interpreted from CPTu by using Equation 1. These S_u values along with corrected S_{u0}(FV) were used in the subsequent stability analyses.

$$S_u = (q_t - \sigma_{v0}) \big/ N_{kt} \tag{1}$$

where q_t = cone resistance corrected for pore pressure at u_2 location; σ_{v0} = total overburden stress; and N_{kt} = cone factor. A number of studies have shown that cone factor, N_{kt} varies with plasticity of soils and typically ranges between 10 and 18 (Aas et al., 1986; Powell and Quarterman, 1988; Lunne et al., 1997). Most of these correlations are based on plasticity index or liquid limit.

As already mentioned, the undrained strength values used for the undrained strength stability analyses were the lower bound values from CPTu and field vane results. Accordingly, the S_u values for the upper soft organic clay, middle silty clay, and lower silty clay layers were 400 psf (19.2 kPa), 500 psf (23.9 kPa) and 750 psf (35.9 kPa), respectively.

3 GEOTECHNICAL ANALYSIS AND DESIGN

3.1 *Settlement and stability assessments*

Based on preliminary geotechnical investigations, which did not include pore pressure dissipation testing, the originally proposed design was to install vertical drains and geosynthetics in addition to surcharging the underlying compressible layers. This approach was considered flexible in that if the soil conditions were determined to be worse at the limits of the widening, the depth and spacing of the vertical drains; the configuration of the geosynthetics; and the fill placement sequence could all be adjusted as needed to develop a stable design.

The magnitude of total settlement of a layer thickness, L_0 due to primary consolidation or presumed end of construction is estimated from:

$$S = \frac{C_c}{1+e_0} L_0 \left(\frac{C_r}{C_c} \log \frac{\sigma'_p}{\sigma'_{v0}} + \log \frac{\sigma'_{vf}}{\sigma'_p} \right) \tag{2}$$

where C_c = compression index; C_r = recompression index; L_0 = initial thickness of the consolidating layer; e_0 = initial void ratio; σ'_p = preconsolidation pressure; σ'_{v0} = effective vertical stress; and $\sigma'_{vf} = \sigma'_{v0} + \Delta\sigma'_v$. Equation 2 is useful because the ratio C_r/C_c can be correlated with *in situ* water content. The long-term or secondary settlement is sometimes taken into account (with an additional term in Equation 2), but based on the soil properties and estimated consolidation parameters (see Table 1) of the lower two clay layers, it was determined that secondary settlement would be negligible (muck-excavation of the slightly organic upper layer also had the benefit of removing the material most susceptible to significant secondary consolidation settlement).

Slope stability analyses for short- and long-term conditions were performed using a two-dimensional slope stability program by analyzing circular (Bishop's method) and translational (three-part wedge Spencer's methods) slip surfaces because thick layers of soft soil and presence of potential weak layers were encountered. A linear phreatic surface was assumed at the original ground surface. For short-term stability analysis, drained friction angle, ϕ' and S_u were specified for cohesionless and cohesive soils, respectively. The values of S_u ($\phi = 0°$) used were the aforementioned lower-bound values while ϕ' values associated with embankment fill, backfill and sand layers were selected between 30° and 34°, depending on tip stress and relative density obtained from the CPTu soundings. On the other hand, a combination of ϕ' and drained cohesion, c' for effective stress envelope ($s = c' + \sigma' \tan \phi'$) was applied to the clay layers to compute the long-term minimum factor of safety. In particular, $\phi' = 28°$ and c' = 200 psf (9.6 kPa) was used for both the upper soft organic clay and middle silty clay layers; and $\phi' = 28°$ and c' = 400 psf (19.1 kPa) for the lower silty clay layer. An additional surcharge load of 100 psf (4.8 kPa) was also applied at the final embankment to account for live loading during construction. Searches were performed to determine the critical slip surface for each condition.

Table 2. Slope stability analysis results.

Analysis condition	Stress envelope*	Method	Side force inclination	Factor of safety
Short-term	Effective & Total	Bishop	-	1.30
Short-term	Effective & Total	Spencer	5.36°	1.36
Long-term	Effective	Bishop	-	2.25
Long-term	Effective	Spencer	7.51°	3.20

*Effective stress envelope, ϕ' and c′; total stress envelope, $\phi = 0$ and S_u from CPTu and FV.

Table 2 shows the results of slope stability analyses for the conditions described above. As was expected, the governing short-term condition barely satisfies the minimal required factor of safety of 1.30, which is comparable with acceptable error of less than ±6% between the two methods (Duncan, 1996). It is worth mentioning the short-term condition analyzed here neglects the beneficial effect of strength gained due to partial consolidation from the muck-excavation and backfilling process, which would result in higher shear strengths than had been estimated. This is discussed further in Section 4.2. The computed long-term factor of safety values exceed 2, indicating that the critical condition for slope stability would be during construction, as opposed to long-term.

3.2 *Stability failure criteria*

In addition to the generally accepted assumption of an undrained response of clay foundations subjected to embankment loadings, it has been suggested that development of lateral displacements in clay foundations is a good indicator of foundation stability (Ladd, 1991). It has also been demonstrated that the development of lateral displacements is essentially related to the state of consolidation in clay foundations, especially during construction when underlying clay becomes normally consolidated and accumulates large lateral displacements. This is the most critical case as the clay foundation exhibits an undrained response, and approaches the critical state limit. In order to evaluate embankment stability, Tavenas et al. (1979) compiled data from 21 case histories of different embankments on soft clays and concluded a relationship between maximum lateral displacement and settlement; Ladd (1991) subsequently called this a deformation ratio:

$$\textit{Deformation Ratio} = \Delta Y\max / \Delta S \tag{3}$$

where ΔY_{max} = maximum lateral displacement and ΔS = maximum settlement. During construction, $\Delta Y_{max}/\Delta S \approx$ +/- 1.0 indicates that the clay foundation becomes normally consolidated, or in other words, the clay exhibits undrained response under constant vertical effective stress. Whereas for early and end of construction stages, the deformation behavior is characterized by drained response, in which lateral displacement increases linearly with applied load, but at a relatively small magnitude. Notably, Tavenas et al. (1979) derived this relationship from clay foundations with OCR of less than 2.5 (i.e. slightly overconsolidated clays). This condition is applicable to the soil conditions at the project site (see Table 1) discussed in this paper. With the aid of the automated datalogging system and geotechnical instrumentation (see Figure 1), changes in lateral displacement and settlement from the horizontal and vertical SAAs, respectively, could be remotely monitored to evaluate embankment stability during and after construction.

4 STABILITY PERFORMANCE

4.1 *Construction Sequence and Geotechnical Monitoring*

Based on the results of the supplemental geotechnical investigation and review, the contractor decided to muck-excavate the upper soft organic clay, backfill "in the wet," place embankment fill plus 5 feet of surcharge, then allow the surcharge to sit for 7 months. Stability analyses had shown that no staging (i.e. intermediate "hold" periods) would be needed. The muck-excavation would be performed during winter months, and it was backfilled to original grade plus 5 ft (1.5 m) to elevation 640 ft in order to build a platform above the normal spring flood elevation.

Geotechnical instrumentation was required by the project specifications, in order to monitor slope inclination, settlement, and pore pressures. Extensive geotechnical instrumentation including ShapeAccelArrays (SAAs), earth pressure cells (EPC), and piezometers (PZ) were installed, in conjunction with an automated remote datalogging system. It may have been possible to rely on conventional inclinometers, but even though analyses had indicated the embankment would remain stable during construction (cf. Section 3.1), it was judged prudent to continuously monitor the stability during construction, due to the presence of the underlying silty clay layers. Hence a vertical SAA was installed to monitor lateral displacement in addition to a horizontal SAA installed to measure settlement. Taken as a whole, the instrumentation system provided the information necessary to evaluate both embankment stability and consolidation progress. Instrumentation was installed about 6 months prior to the majority of fill placement (Day 0); this was by chance, in that the focus of construction switched to other parts of the project (such as bridge construction) during the time between February 2014 and August 2014.

4.2 *Settlement*

Shown in Figure 3, a large amount of fill was placed in a rapid manner over the course of 2.5 weeks, from about Day 170 to 187. Initially, the vertical overburden pressure indicated by the EPCs illustrates that fill was placed quite rapidly, e.g. approximately 4 to 6 ft (1.2 to 1.8 m) of fill was placed from Day 169 to 171; followed by another 4 ft (1.2 m) of fill placed from Day 176 to 180; before the maximum fill height of 12 to 16 ft (3.7 to 4.9 m) was reached on Day 190. During this period, pore pressure responded in a similar "step-up" manner with dissipations clearly seen after each fill placement. The most noticeable increase in pore pressure occurred during the second fill placement, between Day 176 and 180. This resulted in maximum excess pore pressure of 300 psf (14.4 kPa) measured at PZ2 (installed at lower silty clay layer) on Day 180, suggesting the effective stress is the lowest since fill placement. Settlement data also shows an immediate change on Day 169, at an almost constant rate of 0.24 in./day (6 mm/day) until

Day 195. The large settlement rate is the result of clay layers being in virgin compression range after the muck-excavation and backfilling process.

One noteworthy point is that the settlement had already accumulated 1.1 in. (28 mm) on Day 163, then prematurely increased before the fill was placed on Day 169. The accumulated settlement is caused by the consolidation of the silty clay layers or "unexcavated" organic soft clay due to the backfilling process. However, the response in which the settlement increased prematurely and abruptly on Day 163 is not readily apparent if the embankment and filling process is only thought of in two-dimensions. It is likely that the premature settlement was due at least partly to the three-dimensionality of the filling process. The cross-section shown in Figure 1 runs north-south, but filling along the embankment proceeding mostly from east to west. On Day 163, placement of significant fill immediately to the east of this cross-section would have increased stresses at depth, with embankment settlement "shadowing" the SAA before the EPCs would register any changes in vertical overburden pressure. This postulation is difficult to confirm unless a detailed stress analysis is performed.

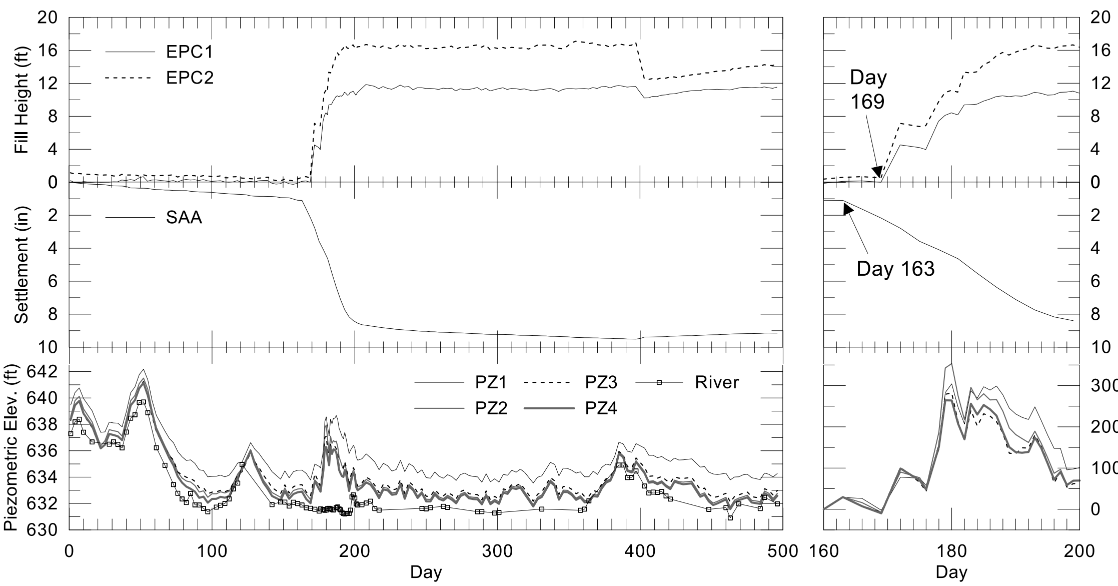

Figure 3. Field measurement results. From top to bottom: earth pressure cells, settlement sensors, and piezometers.

The total settlement estimated during the design phase was 7.0 in. (177.8 mm), based on assumed C_c, C_r/C_c, and c_v values of 0.26 to 0.31, 0.15 to 0.19 and 0.001 cm^2/sec, respectively. Compared with the measured settlement of 9.6 in. (244 mm) on Day 399 (i.e. before the removal of surcharge), the settlement was underestimated by less than 30%. The difference in predicted and measured settlements could be caused by several reasons. One of the more commonly accepted reasons is that the c_v value was assumed to be constant and was selected from the expected final effective stress under embankment from oedometer tests. As mentioned previously, c_v is not a unique parameter due to the change in void ratio and drainage or permeability during different construction stages. The fact that six months elapsed between backfilling of the muck-excavation and full-height embankment filling would likely mean that the *in situ* c_v values were "faster" than predicted during much of the filling process. Therefore, it is important to recognize that one-dimensional consolidation theory only gives an approximation for settlement. Immediate vertical movement (with no soil volume change) due to fill placement is also difficult to distinguish from consolidation settlement (with soil volume change due to pore pressure dissipation). Although the above reasons could contribute to the difference between predicted and measured settlements, correctly interpreting test results and carefully selecting the indices could better compensate for the prediction errors associated with *in situ* soil behavior.

4.3 *Lateral displacements*

As already mentioned, Tavenas et al. (1979) showed that lateral behavior in clay foundations consists of three successive stages (early, during, and end of construction), which exhibit different behavior, resulting in different deformation ratios. Figure 4 compiles the 3-day interval settlement and lateral displacement data measured by vertical and horizontal SAAs, respectively, over the period of 400 days (until the removal of surcharge). From Day 169 to 178, total accumulation of settlement was 1.9 in. (49 mm) when 6 to 8 ft (1.8 to 2.4 m) fill was placed. However, lateral deformations only developed by 0.4 in. (11 mm), resulting in $\Delta Y_{max}/\Delta S$ of 0.2. This response is similar to that of relatively stiff clay that allows significant drainage to occur during initial loading (Tavenas et al., 1979; Ladd, 1991). The second fill placement from Day 178 to 181 caused a considerable increase in lateral displacement, resulting in $\Delta Y_{max}/\Delta S$ = 1.4. During this fill placement, excess pore pressure was also found to be the highest (cf. Figure 3) or $\bar{B} = \Delta u/\Delta\sigma_v \approx 0.6$ at PZ2 location. The high $\Delta Y_{max}/\Delta S$ value of well over 1.0 may signal embankment instability or impending failure due to large displacement, yet no evident distress was observed or reported during the construction. Furthermore, Tavenas et al. (1979) reported that $\Delta Y_{max}/\Delta S$ values between 0.4 and 1.3 have been observed from studied case histories of homogenous clay foundations. Under undrained conditions, $\bar{B}$ should be close to 1.0 for normally consolidated clay and prior to failure, $\bar{B}_f$ >1.0 because of the large increase in pore pressure. Therefore, although $\Delta Y_{max}/\Delta S$ was relatively high, the lower $\bar{B}$ suggests the embankment had not reached an undrained condition but had partially normally consolidated and exhibited a stable condition. This could be the interbedded sand layers that serve as drainage boundaries for pore pressure dissipation, permitting higher $\Delta Y_{max}/\Delta S$ without failure. Also, the partial muck-excavation and backfilling with sand is thought to be an important difference from the homogenous clay foundations in Tavenas et al. (1979). Beyond Day 181, the clay layers were allowed to consolidate with average $\Delta Y_{max}/\Delta S$ estimated to be 0.5; lateral displacements developed at a decreasing rate accompanied by decreasing rate of settlement and pore pressure dissipation (see Figure 4).

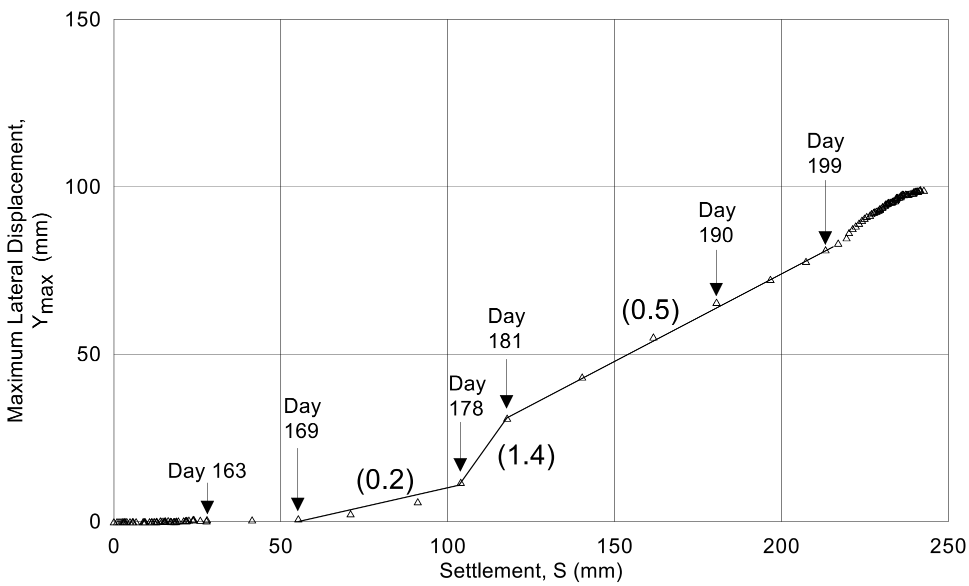

Figure 4. Relationship between maximum lateral displacement and settlement during construction (each data point shown is separated in time by 3 days).

A vertical SAA, when integrated with automated datalogging system, is able to continuously monitor the deformation profile in three dimensions. Applications of SAA can be found elsewhere, e.g. Danisch et al. (2008) and Dasenbrock (2010). Figure 5 shows both the distribution and normalized distribution of lateral displacement profiles with depth, indicating that the largest lateral displacements are located at elevations 630 and 622 ft, where the organic clay and upper sand layers were located, respectively. It is obvious that lateral displacements started to develop on Day 169 (i.e. once fill was placed directly at the cross-section) and were relatively greater at those two elevations. By normalizing each displacement profile with its largest lateral displacement, Y/Y_{max}, it now becomes clear and interesting that the largest displacement occurred at elevation 622 ft, or the top of the upper sand layer, rather than the underlying clay layers. Moreover, by referring to Figure 1, the vertical SAA (VSAA) was installed mostly beyond the muck-excavation limit. This large localized displacement could be caused by lateral squeeze of the unexcavated organic clay and adjacent backfill soil during the fill placement on Day 169. An advanced stress-deformation analysis would be required to investigate this particular scenario. Nevertheless, the lateral displacement profile generally agrees with the partially normally consolidated foundation layer as depicted in Tavenas et al. (1979).

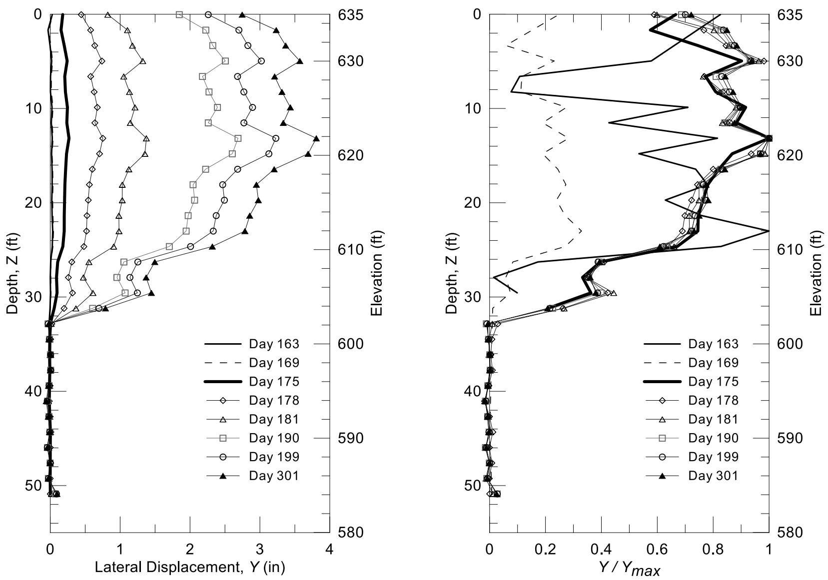

Figure 5. Left: Lateral displacement profile with time. Right: Normalized lateral displacement profile with respect to time.

5 CONCLUSIONS

The construction of the new I-90 bridge over the Mississippi River required the widening of an existing highway embankment in La Crosse, Wisconsin. Followed by geotechnical investigations consisting of soil borings, piezocone soundings and field vane shear tests, an alternative method which involved muck-excavation, backfilling and surcharging was adopted instead of the originally planned vertical drains. Based on the investigation results, settlement and undrained strength stability analyses were performed in conjunction with extensive geotechnical instrumentation and monitoring system to confirm embankment stability during construction. The following conclusions can be drawn from the project experience:

(1) Piezocone soundings and field vane shear tests are highly favorable in embankments constructed on soft foundations to provide accurate relevant information for design and stability assessments. An adequate geotechnical instrumentation and monitoring system provides means to evaluate embankment behavior and stability during construction.

(2) Muck-excavation and backfilling over a stratified soil profile has been successfully implemented in this project. However, drainage conditions of interbedded sand layers should be carefully examined to validate that settlement predictions realistically consider consolidation characteristics of the soil profile.

(3) Deformation ratio, defined as a ratio between settlement and lateral displacement can be used in conjunction with pore pressure measurement to understand and evaluate embankment stability. The results of this project suggest that interbedded sand layers, which act as drainage boundaries, allow a higher $\Delta Y_{max}/\Delta S$ for normally consolidated condition under threshold fill placement.

6 ACKNOWLEDGEMENTS

The authors gratefully acknowledge MnDOT, SRF Consulting Group, and Ames Construction for the opportunity to work on the I-90 Dresbach Bridge project, and their support for use of instrumentation. The efforts of Luke Thompson, P.E., Jim Meyer, and Neil Ramdhan for field work associated with instrumentation installation and troubleshooting is much appreciated.

7 REFERENCES

Aas, G., Lacasse, S., Lunne, T., & Høeg, K. 1986. Use of in situ tests for foundation design in clay. *Proceedings of the ASCE Specialty Conference In Situ '86: Use of In Situ Tests in Geotechnical Engineering*, Blacksburg, 1-30.

Dasenbrock, D.D. 2010. Automated landslide instrumentation programs on US Route 2 in Crookston, MN. *Proceedings of the University of Minnesota 58th Annual Geotechnical Engineering Conference*, University of Minnesota, St. Paul, 165-185.

Danisch, L., Chrzanowski, A., Bond, J., & Bazanowski, M. 2008. Fusion of geodetic and MEMS sensors for integrated monitoring and analysis of deformations. *13th FIG International Symposium on Deformation Measurements and Analysis*, Lisbon, Portugal.

Duncan, J.M. 1996. State of the art: limit equilibrium and finite-element analysis of slopes. *Journal of Geotechnical Engineering*. 122(7): 577-596.

Evans, T.J. 2003. Geology of La Crosse County, Wisconsin. *Bulletin 101*.Wisconsin Geological and Natural History Survey.

Ladd, C.C. & Foott, R. 1974. New design procedure for stability of soft clays. *Journal of Geotechnical Engineering Division*. 100(7): 763-786.

Ladd, C.C. 1991. Stability evaluation during staged construction. *Journal of Geotechnical Engineering*. 117(4): 540-615.

Lunne, T., Robertson, P.K., & Powell, J.J. 1997. *Cone penetration testing in geotechnical practice*. London: Blackie Academic & Professional.

Mesri, G. 1975. Discussion of "New design procedure for stability of soft clays." *Journal of Geotechnical Engineering Division*. 101(4): 409-412.

Mesri, G. 1983. Discussion of "Stability analysis with the sample and advanced ϕ=0 method for a failed dike." *Soils Foundations*. 23(4): 133-137.

Powell, J.J.M. & Quarterman, R.S.T. 1988. The interpretation of cone penetration tests in clays, with particular reference to rate effects. *Proceedings of International Symposium on Penetration Testing*, ISPT-1, Orlando, 2, 903-10, Balkema Pub., Rotterdam.

Tavenas, F., Mieussens, C., & Bourges, F. 1979. Lateral displacements in clay foundations under embankments. *Canadian Geotechnical Journal*. 16: 532-550.

Terzaghi, K., Peck, R.B., & Mesri, G. 1996. *Soil mechanics in engineering practice*. New York: John Wiley and Sons.