5-2024

# Primary Consolidation Settlement of South Louisiana Clay Deposits in Marsh Environment

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Primary Consolidation Settlement of South Louisiana Clay Deposits in Marsh Environment

A Thesis

Submitted to the Graduate Faculty of the
University of New Orleans
in partial fulfillment of the
requirements for the degree of

Master of Science in Engineering
in
Civil Engineering

by

Alice Stark

B.S. University of New Orleans, 2015

May 2024

# Acknowledgements

I would like to express my deepest appreciation to my advisor, Professor Satish Bastola, for the guidance and advice on the research subject. I'm sincerely grateful to Eustis Engineering for their support and for providing the data used for this project. I would like to express my gratitude to Sean Walsh, P.E., Patrick Thurmond, P.E. and James Williams, P.E. for their invaluable discussion and suggestions. To my sister, Alexis Miano, my daughter, Ailsa Stark and my husband, Andrew Stark, I offer my sincerest gratitude for your love and patience.

# Table of Contents

# FIGURES

## TABLES



APPENDICIES

# Abstract


Accurate estimation of primary consolidation of in-situ marsh soils is inherently difficult in south Louisiana due to the soft, settlement prone soils and variable alluvial material. To investigate the accuracy of settlement analysis methods commonly utilized in local practice, estimates from numerical models were compared to the results of a settlement monitoring program at a project site during construction. Sensitivity analyses were then performed using several parameters that were used in the settlement analysis, and the results were compared to the field observations.

The disagreement of observed settlement data and settlement estimates produced with a model that did not include construction staging highlights the importance of including staged fill placement in a settlement model. Similarly, adjustment of most soil properties resulted in changes of both the magnitude and timeline of settlement, indicating the importance of the selection of appropriate model parameters.



Keywords: Consolidation settlement, Settle 3D, Settlement plate, 1D consolidation theory

# Introduction

Accurate estimation of primary consolidation of in-situ marsh soils is inherently difficult in south Louisiana due to the soft, settlement prone soils and variable alluvial material. The magnitude and rate of primary consolidation settlement of in-situ clays due to fill placement are important parameters in planning the schedule and budget of construction projects in south Louisiana. Estimation of not only the magnitude but also the timeline of this settlement is often a critical task in the design phase and can affect many variables of the project. Despite this, these values are famously difficult to predict with considerable accuracy in the area.

For large fill placement areas, the deep influence depth of the loading stress results in a large swath of soils with variable alluvial material that all contribute to this settlement. Additionally, field exploration and lab testing can only be performed on a small portion of the soil in a potential project area (from a practical time and budget perspective), so variations in this material can be missed. Laboratory testing to determine appropriate consolidation parameters (such as consolidation or odometer tests) is especially time-consuming and costly to perform and is therefore typically limited in quantity within a geotechnical exploration scope of work. Some correlations with soil index parameters may be used to estimate consolidation parameters. The advantage of this approach is that less costly testing can be performed on many more samples, which can then be used for these parameter estimates. Additionally, in-situ testing, such as cone penetrometer testing, can be used to estimate subsurface characteristics, but again, variations across a project site can be missed, and correlations to the consolidation parameters must be relied upon.

For larger and more critical projects, it is common practice to perform a settlement monitoring program to observe the settlement during construction and adjust any analyses and assumptions made during the initial geotechnical investigation. Settlement plates are a simple monitoring tool that may provide readings of settlement occurring in real-time on a construction site.

A plethora of opportunities exist to improve settlement estimates and prediction analyses in south Louisiana. Connections with industry professionals and academic contacts to investigate the results of settlement monitoring programs could be highly beneficial to the geotechnical industry in the area. Sensitivity analysis performed for this project could similarly be performed for any project with substantial fill placement in the region. Regardless of the project type or specific geology, any project where a geotechnical field exploration and a settlement monitoring program were performed could be studied and added to the local knowledge base. The model simulation may be plagued with uncertainties arising from 1) accurate record keeping of schedule and magnitude of fill placement, 2) movement of settlement plates by earthwork efforts, 3) variation in earthwork compaction, and 4) poor parametrization of parameters and coefficients. The simulation results show that the Terzaghi 1-D settlement model analyzed through Settle3 is acceptable for monitoring during construction. However, adjusting parameters as initial settlement monitoring data is available would be an important tool for improving the accuracy of predicting the remaining settlement at a project site. With an increase in certainty of a predicted settlement magnitude and timeline, construction schedules can be compressed, and cost savings can be gained.

## Research Objective

Laboratory testing and regional correlations-based methods are generally used to estimate the settlement behavior of soil. The time-consuming nature of a lab-based approach and the low accuracy of a correlation-based approach are the issues in characterizing the compressibility of soil. The objective of this study is to use existing settlement instrumentation equipment installed to monitor settlement at the project site during construction and create a numerical model to assess the reliability and sensitivity of the model-based settlement estimate. Sensitivity analyses were performed using several parameters used in the settlement analyses, and the results were compared to the field observations. In this study, immediate and secondary settlements were not estimated, and only settlements due to consolidation were analyzed.

## Literature review

Liang *et a*l. (2018) discuss the in-situ monitoring of the long-term settlement of high-fill subgrade. The authors discuss monitoring large heights (more than 4 m, or about 13 feet) of fill placement on the Lanzhou-Yongjing highway. Single-point settlement meters were placed at various depths along an approximately 5 km stretch of roadway. The meters were placed in the embankment in pairs at each depth, separated by approximately 8 m in a horizontal direction. They found that the shoulder area settlement was greater than the driveway, due to the lack of lateral restraint of the shoulder area. Also, vehicle loads had no appreciable effect on the settlement, and the settlement at the original ground surface increased with time up until about a year after placement. Greater settlement was observed at the top of the embankment due to some amount of self-weight settlement. Some heaves of the subgrade due to freeze/thaw mechanics were observed.

Additionally, the road shoulder and driveway settlement could be reasonably modeled by hyperbolic, logarithmic, power function and exponential function models. The exponential function provided the best fit for prediction of subgrade settlement, with an $R^2$ coefficient of 0.977.

Bergado *et al*. (2000), Teerawattanasuk, Youwai, and Vootipruex constructed a full-scale test reinforced (using hexagonal wire mesh) embankment on soft Bangkok clay. They used the finite element program PLAXIS to model a mechanically stabilized earth test embankment on soft Bangkok clay. Their Plaxis model included a hexagonal wire mesh reinforcement. The authors investigated the embankment loading methods during construction, soil permeability during settlement, and the model and properties at the soil-reinforcement interface. The embankment was 6.0 m high, 6.0 m long at the crest of the embankment, 6.0 m wide at the crest of the embankment, and 18.0 m wide at the base of the embankment. The embankment was constructed in three lifts over 2 months. After 405 days, an additional surcharge of 1 m of fill was placed at the top of the embankment. Half of the embankment was reinforced with zinc-coated hexagonal wire mesh reinforcement and the other half was reinforced with PVC-coated hexagonal wire mesh reinforcement. The back slope and side slopes built were 1:1, while the gabion facing of the embankment was built at a 10-degree angle. The soil profile was generally characterized as weathered clay overlying soft clay, underlain by stiff clay. The backfill material used was silty sand. Groundwater was assumed to be about 2 m below the existing ground surface. Pore water pressures, lateral movements and settlement were observed through instrumentation. The authors reported that the numerical simulation resulted in realistic overall behavior of the full-scale reinforced test embankment. The authors used one-dimensional consolidation tests and standard local correlations were used to determine the soil model parameters. The authors selected the interpolated recompression coefficient ($C_r$) values gained from the test results as the upper bound due to typical disturbances in the field during sampling, which can increase the $C_r$ results. The author noted that permeability is difficult to estimate accurately. The weathered clay layer included fissures, which increased permeability, as well as sand lenses and silt seams. South Louisiana has similar issues with commonly observed sand lenses and silt seams, as well as organic material that can be found permeating various alluvial strata. Vertical permeability values of one, two, and five times that of the laboratory tests were used and evaluated in the Bergado *et. al* (2000) paper. Additionally, the horizontal-to-vertical permeability ratio was assumed to be 2:1, although it can vary from 1 to 15. Similarly to our project's results, the horizontal permeability did not appear to have a large effect on the estimated settlement values. The authors used the variable permeability formulation proposed by Taylor (1948).

Like the research method used in this thesis project, settlement plates were installed, and the readings were compared to the settlement estimates. The readings taken at the front of the embankment were reasonably close to the settlement estimates performed, with permeability values of two times the lab results. The readings taken at the middle and rear of the embankment were reasonably close to the settlement estimates performed, with permeability values equal to the lab results. Additionally, in all estimated settlement models, the estimated settlement during the beginning of construction was greater than that observed and followed the estimates more closely after about 100 days. The authors theorize that this results from the partially drained behavior effect on the in situ soft clay material during construction.

Subsurface settlement readings at 3 m and 6 m depths below the ground surface were also taken. These readings were relatively consistent with the settlement estimates using permeability values twice that of the lab permeability values.

Ravaska and Vepsalainen (2004) introduced a method to improve the settlement computer program by relating the coefficient of consolidation and stress with input parameters. The authors discuss how the coefficient of consolidation can be expressed according to Terzaghi's consolidation theory by $Cv = k\,\frac{M}{y_w}$ (Eqn. 1), where the deformation modulus *M* and the coefficient of permeability can be assumed constants where the stress differences are small. However, in many typical construction situations the stress variance can be very large. As stress increases in a soil system the voids tend to decrease which in turn leads to a decrease in permeability.

Ravaska and Vepsalainen (2004) state that the relationship between the coefficient of consolidation and stress is complex and can vary widely with soil type. Typically, a linear relationship between the void ratio and semilogarithmic stress can be utilized through $C_c$ *(Coefficient of compression)* parameter correlations, but they discuss the differences in stress-strain behavior of young and soft post-glacial clays of Nordic Countries from most reported clay behavior in the available literature.

The Ohde-Janbu model (Janbu 1963) is commonly used in in Finland. The Ohde-Janbu model estimates the strain caused by stress in soil. The application of the Ohde-Janbu model assumes that $C_v$ rapidly decreases after exceedance of preconsolidation pressure for sensitive clays and then stays constant or increases slowly as stress increases. Ravaska and Vepsalainen (2004) evidently show that soft post-glacial clays tend to have increasing $C_v$ with increasing stress, but that $C_v$ can increase, decrease or remain constant depending on the soil material. The authors present a non-linear, stress-dependent model of strain, permeability, and coefficient of consolidation. The advantage of their method is that it will provide more accurate results with $C_v$ values calculated for various stress states. However, it requires that permeability be measured at multiple strain levels in an oedometer test, which can be time-consuming and thus costly.

Ravaska and Vepsalainen (2004) present the below equations as a suitable relationship for normally consolidated, post-glacial very soft Finnish clays:

$$k = k_0(1 - \varepsilon)^{\alpha} \qquad (2)$$

where $\varepsilon$ is the strain, $k_0$ is the coefficient of permeability at zero strain, and $\alpha$ is a dimensionless model parameter, and

$$c_v = \frac{k_0(1-\varepsilon)^{\alpha} m\sigma_v}{\gamma_w}(\frac{\sigma}{\sigma_v})^{1-\beta} \qquad (3)$$

where *m* and β are dimensionless model parameters, $\gamma_w$ is the unit weight of water, $\sigma$ is the effective vertical stress during the consolidation process and $\sigma_v$ is a reference stress of 100 kPa.The authors utilize their lab test results to produce various *k* values for their analyses.

Li and Chunlin (2014) proposed a simplified method for estimating potential settlement based on in situ data. Li, Chunlin (2014) discuss settlement prediction as an alternative to settlement estimation. The advantage of this method being that there is less reliance on estimated coefficients which can render settlement estimates ineffective due to uncertainty. They review several methods, including the Asaoka (1978) and Hyperbolic method (Tan *et al*. 1991) but both require an initial time point which can greatly affect the validity of the prediction. Li, Chunlin (2004) is based on Terzaghi's 1D consolidation theory but does not require the specification of initial time point. At any time, *t*, the predicted settlement (defined as the difference between the final settlement and settlement at time t) can be defined as (Eqn 4):

$$S_p = S_{\infty}\frac{8}{\pi^2}e^{bt} \qquad (4)$$

where *b* is a constant determined from observational data and $S_{\infty}$ is the final settlement calculated by Asaoka's (1978) method.

Based on $S_p$ estimated from Eqn (4), the plot of *t* vs $\ln(\frac{S_p\pi^2}{8S_{\infty}})$ can be plotted to estimate the slope of the plotted line. In the plot, the final settlement corresponds to the point where *x* converges to *y*. The settlement is estimated by substituting the final settlement value and *b* into Eqn. (4). The method could reproduce the observational data better than either the Asaoka (1978) or the Hyperbolic method (Tan *et al*. 1991).

## Research Method

The settlement of fine-grain soil is commonly predicted using the 1D theory of Terzaghi. The application of Terzaghi-based 1D consolidation theory, however, requires simplification on the behavior of soil under stress. The soil compressibility and other geotechnical properties required for studying the consolidation behavior requires laboratory testing for estimating the recompression, compression, permeability vs void ratio, coefficient of consolidation, and pre-consolidation stress. The relationship between soil compressibility and stress is not always linear, and may be complex, and can vary widely depending on the soil type. The literature review above shows the use of simplified and complex numerical approaches to predict the settlement of soft soil. Laboratory testing to determine appropriate consolidation parameters (such as consolidation or odometer tests) is time-consuming and costly. It is typically limited in quantity within a geotechnical exploration scope of work. Correlations of soil compressibility with soil index parameters and in-situ tests (e.g., Cone penetrometer) may be used to estimate consolidation parameters. In this study, the observed settlement data is collected through settlement plate readings during construction and is used to calibrate and analyze the sensitivity of the parameters of the soil consolidation prediction model to better characterize the parameters of the model rather than parameterizing it based on literature values or laboratory measured soil consolidation parameters. The schematic of the method is shown in Figure 1. The methods have four components: 1) Site characterization, 2) setup of the numerical settlement estimate model, 3) Settlement measurement using settlement plates, 4) Calibration, and validation of the numerical model.

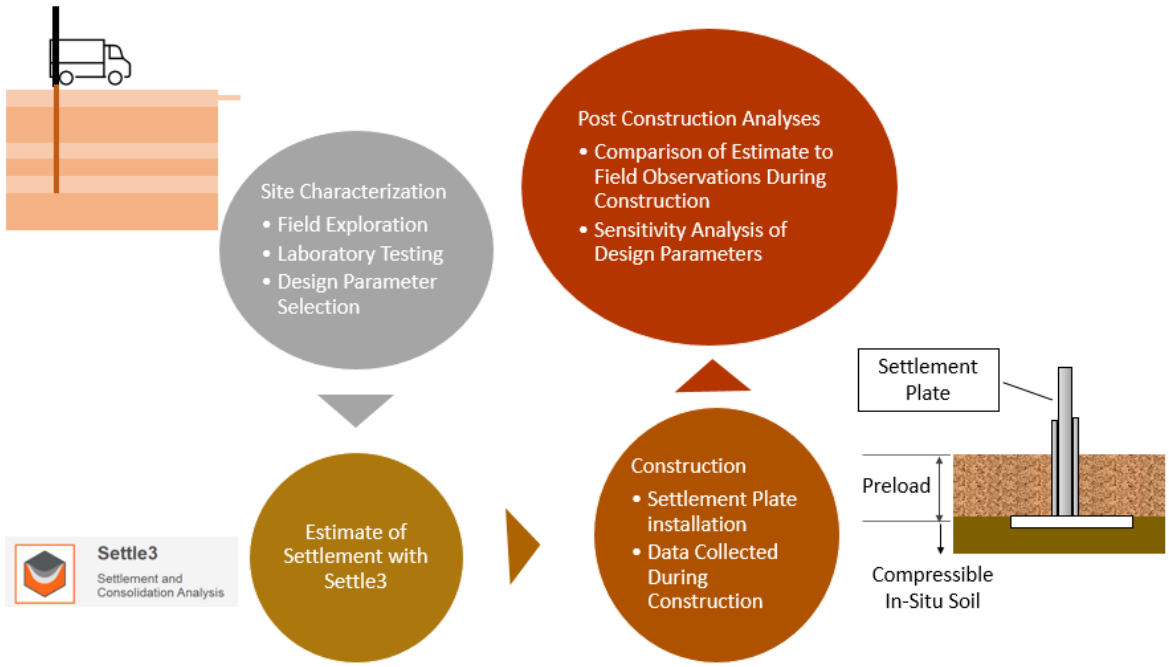


*Figure 1- Schematic of the Study Method showing the use of data from settlement plate readings to validate and analyze the sensitivity of Settle 3D model.*

## Study Region

A project in Louisiana, U.S., was selected for this study (Figure 2). The project site is located in southern Louisiana, in the Lafourche system of the Gulf Coast Delta Plain. The region is prone to soft, compressible soils. The subsurface conditions (geology and stratigraphy) of the project site are discussed below.


*Figure 2 - Study Region: Located the in the Lafourche system of the Gulf Coast Delta Plain In the state of Louisiana, U.S (Source: https://education.nationalgeographic.org/resource/miss-delta-formation/#undefined)*

## Subsurface Conditions

The soils in the project site consist of Holocene Era alluvial clays overlying Pleistocene Era clays. Natural levee, abandoned distributary, marsh and undifferentiated interdistributary deposits are all common in the general vicinity of the project area. At the project site, thick interdistributary deposits and shallow marsh deposits were observed.

Subsurface soil data was available in the project area from three 3-inch diameter soil borings and three 5-inch diameter undisturbed soil borings. This data was used to create the interpreted stratigraphy shown below in Figure 3. In general, thick deposits of soft to medium stiff clay and silty clay with organic and peat layers were observed at all boring locations. Some isolated silt and sand lenses, seams and pockets were also observed. This is to be expected due to the alluvial nature of the project site. Additionally, a water level between *el 0* ft and *el 3* ft was typically observed in the project area.

Based on the available data, specific design parameters were selected for analysis. The design parameters used in this thesis are included in Appendix I attached. These included the interpreted unit weight (y), over consolidation ratio (*OCR*), past maximum pressure ($P_c$), water content (*w*), void based compression ratio ($C_{ce}$), void based recompression ratio ($C_{re}$), coefficient of consolidation ($C_v$), and poisson's ratio. Soil drainage conditions at the boundary of each stratum was also selected.

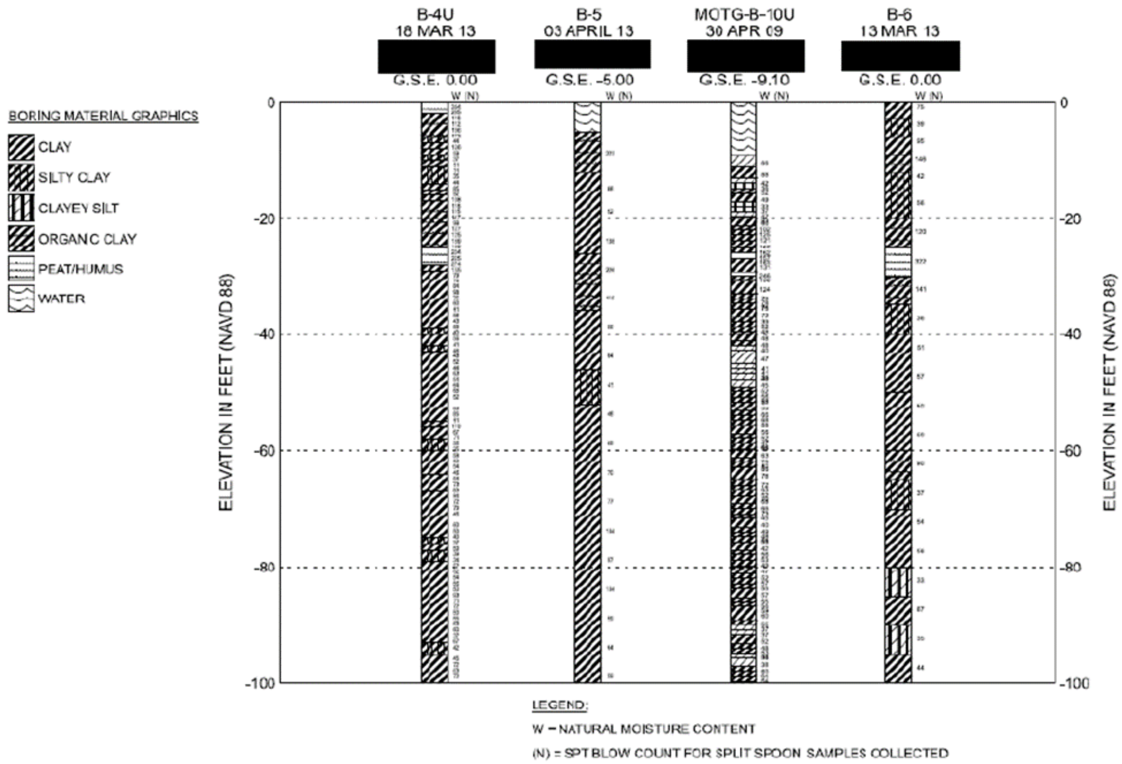


*Figure 3 – Project Site Soil Stratigraphy: The soils in the project site consist of thick deposits of soft to medium stiff clay and silty clay with organic and peat layers.*

## In Situ Monitoring and Data collection

In this study, the results of the settlement plate portion of an instrumentation program performed for a project in south Louisiana was used. The project included various instrumentation methods utilized for a monitoring program performed during construction of the project, which included a preload and final levee tie-in embankment. Overall, the project included a floodgate, floodwall, and levee embankment tie-in. The geotechnical exploration performed indicated that the site soils consisted of very thick, soft clay deposits with some organic clay and peats. Generally, these soils were very soft, compressible and settlement prone. The purpose of the preload and monitoring program was to induce settlement of the compressible clay and organic material and monitor the results until the risk of any considerable down drag settlement of nearby pile supported structures was minimal.

In early 2018 an uncompacted bridge lift was installed at the levee tie-in site to form a workable construction platform before the installation of the instrumentation. The fill placement was estimated to be about seven (7) feet thick, resulting in three (3) feet of elevation gain. It was surmised that mudwave action was induced, contributing to the reduction in elevation gain relative to the height of fill placed.

Following placement of the uncompacted bridge lift, high strength geotextile was placed over the subgrade of the project area and vertical wick drains (PVDs) were installed. The PVDs were installed to a depth of 60 feet from the existing ground surface, spaced triangularly at 5-foot centers. Instrumentation was then installed at the levee tie-in. The instrumentation included settlement plates, settlement gauges, magnet extensometers, Shape Array Accelerometers (SAAs) and piezometers. Only settlement plate data has been considered and presented for this case study.

Generally, during settlement plate installation, a square steel plate of about 2 feet by 2 feet in area, with a riser attached perpendicularly to the plate is placed on stable subgrade before fill placement. The elevation of the riser is noted before any fill placement begins, and then periodically as the fill placement and hold period progresses in the monitoring program.

## Numerical Modeling

The most common theory used in numerical modeling of soft soil foundation is the differential equation of one-dimensional consolidation behavior of soil, which Terzaghi initially proposed. The simplified analytical form requires simplifying assumptions (small deformation, constant compressibility, permeability coefficient, etc.) on soil behavior. However, numerical techniques such as the finite element method may require relaxing the simplifying assumptions. With advances in computer power, the solution of complex geotechnical problems is feasible. The specification of soil design parameters (discussed below) is key to predicting primary consolidation settlement.

### *Soil Design Parameters*

Primary consolidation settlement $\Delta\varepsilon$ can be estimated through the below Eqns. (5, 12-14):

$$\Delta\varepsilon = \frac{Cc}{1+e0} HLog(\frac{\sigma_i{'}+\Delta\sigma}{\sigma_i{'}}) \qquad (5)$$

where $C_c$ is the compression coefficient, $H$ is the height of the soil strata, $e_0$ is the initial void ratio, $\sigma'_i$ is the initial stress and $\Delta\sigma$ is the change in stress. If $\sigma'_i + \Delta\sigma > Pc$ (Pre-consolidation stress) then Eqn. (5) estimates the total settlement. If the $\sigma'_i + \Delta\sigma < Pc$ (Defined in Eqn. 9), then the coefficient of recompression is used instead of the compression coefficient in Eqn. (5) to estimate total settlement. $C_c$ is commonly estimated through an odometer test, which presents a stress vs. void ratio curve. This curve is then used to interpret an estimated compression and recompression coefficient. However, odometer tests are time-consuming and costly, so correlations with other soil property tests (such as water content, plasticity index and liquid limits) are frequently utilized to estimate compression coefficients. *Table 1* below includes some of these correlations.

*Table 2 – Compression Coefficient Correlations from Southeast Louisiana (source Clay Worley, 2022 personal communication)*

| Equation # | Equation | Notes | Reference |
|---|---|---|---|
| 1 | $0.017w_n - 0.299$ | CH/CL | Brandon, et al (2011) |
| 2 | $0.012w_n - 0.163$ | CH/CL | Harris and Jafari (2018) |
| 3 | $0.014w_n - 0.12$ | CH/CL | Deubert (1982) |
| 4 | $0.012w_n - 0.06$ | CH Only | Deubert (1982) |

| | | | |
|---|---|---|---|
| 5 | $0.016w_n – 0.29$ | CL only | Deubert (1982) |
| 6 | $0.012w_n + 0.137$ | OH/PT | Brandon, et al (2011) |
| 7 | $0.008w_n + 0.375$ | OH/PT | Harris and Jafari (2018) |
| 8 | $0.673e_0 – 0.377$ | CH/CL | Brandon, et al (2011) |
| 9 | $0.611e_0 – 0.28$ | OH/PT | Brandon, et al (2011) |
| 10 | $8e^{-0.038\gamma_d}$ | CH/CL | Brandon, et al (2011) |
| 11 | $7.82e^{-0.043\gamma_d}$ | OH/PT | Brandon, et al (2011) |
| 12 | 0.009LL – 0.1 | CH/CL | Deubert (1982) |
| 13 | 0.0085(LL+9.5) | CH only | Brandon, et al (2011) |
| 14 | 0.018(LL-19.6) | CL only | Brandon, et al (2011) |
| 15 | 0.0067(LL+95) | OH/PT | Brandon, et al (2011) |
| 16 | 0.0067(LL+95) | 0.01PI + 0.06 | Deubert (1982) |
| 17 | $0.54*(2.6w_n – 0.35)$ | CH/CL (moisture content as decimal) | Nishida (1956) |
| 18 | $0.01w_n$ | CH/CL | Azzouz (1976) |
| 18 | $0.0115w_n$ | OH/PT | Azzouz (1976) |
| 20 | $0.208e_0 + 0.0083$ | CH/CL | Azzouz (1976) |
| 21 | $1.15*(e_0 – 0.35)$ | CH/CL | Nishida (1956) |
| 22 | $0.29*(e_0 – 0.27)$ | CH/CL | Hough (1957) |
| 23 | $0.35*(e_0 – 0.50)$ | OH/PT | Hough (1957) |
| 24 | 0.0046(LL – 9) | CH/CL | Azzouz (1976) |
| 25 | 0.002 + 0.014PI | CH/CL | Nacci (1975) |

The process of consolidation settlement is related to changes in pore water pressure and therefore, changes in effective stress. In the case of new fill placement on a subgrade, the pore water pressure initially increases with the applied stress. Over time the pore water pressure dissipates, and the stress is transferred to the soil skeleton as settlement occurs. Terzaghi's one-dimensional consolidation equation below illustrates this relationship, along with soil compressibility and permeability parameters:

$$\frac{\partial \mathrm{u_e}}{\partial t} = Cv\frac{\partial^2 u}{\partial z^2} = \frac{k}{\gamma_w * m_v}\frac{\partial^2 \mathrm{u_e}}{\partial t^2} \qquad (6)$$

where $C_v$ is the coefficient of consolidation, $k$ is the permeability, $m_v$ is the volume coefficient of compressibility, and $\gamma_w$ is the unit weight of water. As shown below, this equation can be numerically solved through a Fourier series expansion:

$$u = \Delta\sigma \sum_{n=0}^{\infty} f_1(z/H) f_2(Tr) \qquad (7)$$

where $Tr$ is the time factor: $Tr = C_v \frac{t}{H_{dr}^2}$ , and $H_{dr}$ is the soil drainage path length, $t$ is the time required for *r%* consolidation, and depth, $z$ is a function of the coefficient of consolidation. This parameter encompasses the permeability and compressibility properties, as shown in Eqn (6). Coefficient $C_v$ can be estimated through odometer lab testing, similarly to $C_c$. This can be done through Casagrande's fitting method or Taylor's square root of time method. Coefficient $C_v$ can also be correlated with liquid limit values. Das (2006) provides such a correlation.

### *Settle3: Soil Settlement and Consolidation Analysis*

Numerical modeling of settlement estimates for this thesis was performed with the program Settle3 version 5.012. The application of Settle3 involves 1) preparation of input data and parameters, 2) model setup, 3) defining boundary conditions, 4) analyzing the sensitivity of model parameters, and 5) interpreting the results. The input includes data on soil properties, soil profile geometry, loading conditions, and ground improvement conditions. The models are then run for analysis (sensitivity analysis and scenario simulation).

The stress change of the soil due to the preload was estimated using the Westergaard model feature (See Settle 3D reference manual; ROCscience Inc 2021). The Westergaard model (a method that calculates the stress by weighting the layer thickness) better represents stratified soils. Settle 3D also has a rigid load type option that utilizes the average properties to compute layered stress. It is common local standard practice to utilize this stress model considering the layered alluvial geology in the area.

For the purposes of this thesis, immediate and secondary settlement were not estimated, and only settlement due to consolidation was analyzed. This type of settlement is the most significant in terms of magnitude in high moisture clays, such as the types encountered in the project area. All settlement estimates were performed at the centerline of the modeled embankment.

The stratigraphy at this site was assumed to include mostly normally and slightly over-consolidated materials. The non-linear method of settlement analysis can be performed in Settle3 through specification of either preconsolidation stress, *Pc*, over-consolidation ratio, *OCR* or over-consolidation margin, *OCM* using the following formulas:

$$OCR = \frac{\mathrm{Pc}}{\sigma\,'} \qquad (8)$$

$$OCM = P_c - \sigma\,' \qquad (9)$$

The Settle3 program model's vertical consolidation is calculated through Terzaghi's 1D consolidation theory (Eqn. 10):

$$\frac{\partial \mathrm{u}}{\partial \mathrm{t}} = C_v \frac{\partial^2 \mathrm{u_e}}{\partial \mathrm{z}^2} \qquad (10)$$

In this model, $c_v$ is the coefficient of consolidation, *z* is the vertical distance below the ground surface and $u_e$ is the excess pore water pressure. Consolidation settlement progresses as pore pressures dissipate and the effective stress of the soil increases. At any point in time, the above Eqn. (10) is used to calculate the excess pressure, which is then used to calculate the effective stress:

$$\sigma\,' = \sigma - \mu \qquad (11)$$

where $\sigma$ is the total stress due to gravity and external loads (*i.e.*, a preload and in situ material above that specific height).

The strains of the sublayers are calculated from the strain-based compression parameters $C_{ce}$ and $C_{re}$ using Eqn. (12) for over consolidated soils and Eqn. (13) for normally consolidated soils:

$$\Delta\varepsilon = C_{r\varepsilon} \log\left(\frac{\sigma_f'}{\sigma_i'}\right) \qquad (12)$$

$$\Delta\varepsilon = C_{r\varepsilon} \log\left(\frac{P_c}{\sigma_i'}\right) + C_{c\varepsilon} \log\left(\frac{\sigma_f'}{P_c}\right) \quad (13)$$

And Eqn. (14) is used to calculate the settlement:

$$\delta_i = \delta_{i+1} + \varepsilon_i h_i \qquad (14)$$

Where the settlement of the $i^{th}$ point is $\delta_{i,}$, the settlement of the point below is $\delta_{i+1}$,$h$ is the original thickness of the bottom sublayer and $\varepsilon_i$ is the vertical strain in each sublayer.

Although Eqn. (14) can be solved analytically for a single stratum, and for linear stiffness, our project includes multiple strata with varying thicknesses and coefficients of consolidation. Settle3D employs a finite difference approach for this reason. This approach discretizes Eqn. 14 in time and space and then uses either an implicit or explicit solution. Explicit iterations are performed for each relevant time step, $\Delta t$, until the time exceeds the minimum required for an implicit approach. At this point, the remainder of the analysis is performed via an implicit approach with ever-increasing timesteps. Performance of the explicit solution can be lengthy, so utilization of this progression provides an efficient calculation of a solution with a relatively minor reduction in accuracy due to the decrease in pore pressure as the iterations towards a solution continue.

Additionally, to account for variations in strata height and coefficients of consolidation, the pore water pressure terms ($u_{i+1}$) are multiplied by an $\alpha_i$ factor:

$$\alpha_i = \left(\frac{k_i}{k_{i-1}}\right)(h_{i-1} h_i) \quad (15)$$

where $k$ is the permeability and $h$ is the thickness of the strata at each node calculated. On the strata boundaries, nodes can be classified by the user as drained or undrained. If the undrained option is selected, the boundary is assumed to be impermeable and generation of a dummy node below the bottom strata node (and at the same distance as the bottom node to the second to bottom node) facilitates the calculation of the finite difference calculation. The excess pore pressure of the dummy node is equivalent to the pressure of the node second from the bottom, thereby representing no change in excess pore pressure between nodes. If a drained boundary is selected, no dummy node is created.

For analyses with multiple permeability values, the material permeability ($k$) can be calculated for a non-linear material through:

$$k = (c_v C_c \gamma_w)/(2.3(1 + e_0)\sigma_{zi}') \qquad (16)$$

Where $\sigma_{zi}'$ = initial effective stress, $\gamma_w$ = unit weight of pore water. For overconsolidated material, $c_v$ = *$c_{vr}$, $C_c = C_r$, $m = m_r$ and $C_p = C'_p$*

As stress increases in soil, permeability, void ratio, and porosity often decrease as the soil compresses. Settle3D computations include some variance in permeability. The stress and resultant permeability is calculated at the beginning of each 'stage' entered into the program. However, as previously discussed, this relationship between stress and permeability can be complicated and can vary widely depending on the site-specific soil conditions. Therefore, along with varying coefficients of consolidation by increasing or decreasing the entire soil profile for the entire length of the analysis, a model was also run with *Cv* variation proportional to the increase in loading stress with the progression of fill placement. This model is discussed further in the *Coefficient of Consolidation Sensitivity Analyses* section.

For this project, horizontal and vertical drains were included in the analyses. Settle3 calculates excess pore pressure within a drain array using Eqns (17-19):

$$u_e = u_{e0}\, exp\left(\frac{-8T_r}{\mu}\right) \quad (17)$$

$$T_r = \frac{C_h}{D_e^2}, \quad (18)$$

$$\mu = \frac{n^2}{n^2 - S^2}\, ln\left(\frac{n}{S}\right) - 0.75 + \frac{S^2}{4n^2} + \left(\frac{k_h}{k_s}\right)\left(n^2 - \frac{S^2}{n^2}\right) ln\,(S) \quad (19)$$

Where $c_h$ = horizontal consolidation coefficient is a function of the drain geometry, $d$ = the drain diameter, $d_s$ is the diameter with respect to the smear zone, $S$ is a coefficient, $D_e$ is the equivalent wick drain spacing and where $n = \frac{D_e}{d}$, $S = \frac{d_s}{d}$, and $\frac{k_h}{k_s}$ = ratio of horizontal permeability in the undisturbed zone to permeability in the smear zone.

## Coefficient of Consolidation Sensitivity Analyses

As previously discussed, Ravaska *et al* (2004) provide a model for estimating the coefficient of consolidation using permeability testing performed in conjunction with odometer tests:

$$c_v = \frac{k_0(1-\varepsilon)^{\alpha} m \sigma_v}{\gamma_w}\left(\frac{\sigma'}{\sigma_v}\right)^{1-\beta} \qquad (19)$$

where *m* and *β* are dimensionless model parameters, $\gamma_w$ is the unit weight of water, $\sigma'$ is the effective vertical stress during the consolidation process and $\sigma_v$ is a reference stress of 100 kPa.

Although permeability testing was not performed for this project, some simplifications were made to this model to explore the possible use of a coefficient of consolidation estimation with consideration of a change in loading stress. Using the settle3 'factors' feature, parameters may be adjusted by a numerical factor by stage (time) and/or depth in the soil profile. Considering that a set of $C_v$ values is available, a set of factors that vary over time and depth could easily be input to adjust these $C_v$ values for the change in permeability with loading stress.

For any depth in the soil profile, the coefficient of consolidation at initial stage i is estimated using Eqn. (20) and at any other stage is estimated using Eqn. (21)

$$c_{vi} = \frac{k_{0i}(1-\varepsilon_i)^{\alpha_i} m_i \sigma_v}{\gamma_w} \left(\frac{\sigma'_i}{\sigma_v}\right)^{1-\beta_i} \quad (20)$$

$$c_{vn} = \frac{k_{0n}(1-\varepsilon_n)^{\alpha_n} m_n \sigma_v}{\gamma_w} \left(\frac{\sigma'_n}{\sigma_v}\right)^{1-\beta_n} \quad (21)$$

Assuming the model parameters remain the same over time,

$m_i = m_n, k_{0i} = k_{0n}, \alpha_i = \alpha_n, and\ \beta_i = \beta_n.$

The variable $X$ is defined as the ratio of coefficient of consolidation at any stage to coefficient of consolidation at the initial stage (Eqn. 22).

$$X = \frac{c_{vn}}{c_{vi}} \quad (22)$$

Substituting Eqns. 20 and 21 and simplifying the Eqn. (22) can be written as Eqn. (23)

$$X=\frac{(1-\varepsilon_n)^{\alpha}\left(\frac{\sigma'_n}{\sigma_v}\right)^{1-\beta}}{(1-\varepsilon_i)^{\alpha}\left(\frac{\sigma'_i}{\sigma_v}\right)^{1-\beta}} \quad (23)$$

Assuming that the difference in strain between the initial stage and stage n is very small and assuming the value of $\beta$ as 0 then Eqn. (23) simplifies to Eqn. (24).

$$X=\frac{\left(\frac{\sigma'_n}{\sigma_v}\right)^{1}}{\left(\frac{\sigma'_i}{\sigma_v}\right)^{1}} \quad (24)$$

$$\sigma'_i = \sigma'_0 + \Delta\sigma_{iz} \quad (25)$$

$$\sigma'_n = \sigma'_0 + \Delta\sigma_{zn} \quad (26)$$

with $\sigma'_0 = initial\ effective\ stress,\ \Delta\sigma_{zn}$ = increase in applied stress from loading at time $n$, and the increase in applied stress from loading at time *i*, $\Delta\sigma_{iz} = 0$ as the initial time has no applied load in the model.

Substituting for $\sigma'_i$ $and$ $\sigma'_n$ and simplifying the Eqn. (24) yields Eqn. (27):

$$X = \frac{(\sigma'_0 + \Delta\sigma_{zn})}{(\sigma'_0)} \qquad (27)$$

The value of *X,* a multiplier, was calculated for each stage in the settlement analyses and for the mid strata depth for each strata in the soil profile. The value of *X* was then defined as a *Cv* factor in Settle3. The values of *X* are included in Appendix II and can be seen in Figure 12. The analysis of the results are discussed in more detail in the results section.

# Results

In this section, the observation data collected from some of the nine settlement plates (from the project site), and the simulation results obtained from numerical models are presented.

The settlement plate reading for 9 plates (designated as SP-1 through SP-9) from the project site were collected for this study. These plates were installed throughout the site but only plates SP-2, SP-5 and SP-9 were installed within the centerline of the embankment. The settlement plate readings are shown in the below *Figure 4 – Settlement Plates and Initial Settlement Estimate* graph. The bottom panel shows the time series of settlement data from SP-1 through SP-9 settlement plates, whereas the top panel shows the elevation of reported height of fill (ft). The settlement plate readings plotted below the fill placement height generally follow the trend of increased settlement during and immediately after fill placement, with a decrease in settlement over time after fill placement is complete.

The readings for each plate generally followed the same trends in terms of an observed moderate downward slope until approximately 30 days after the settlement plate installation, and a steepened slope 30 days after plate installation.

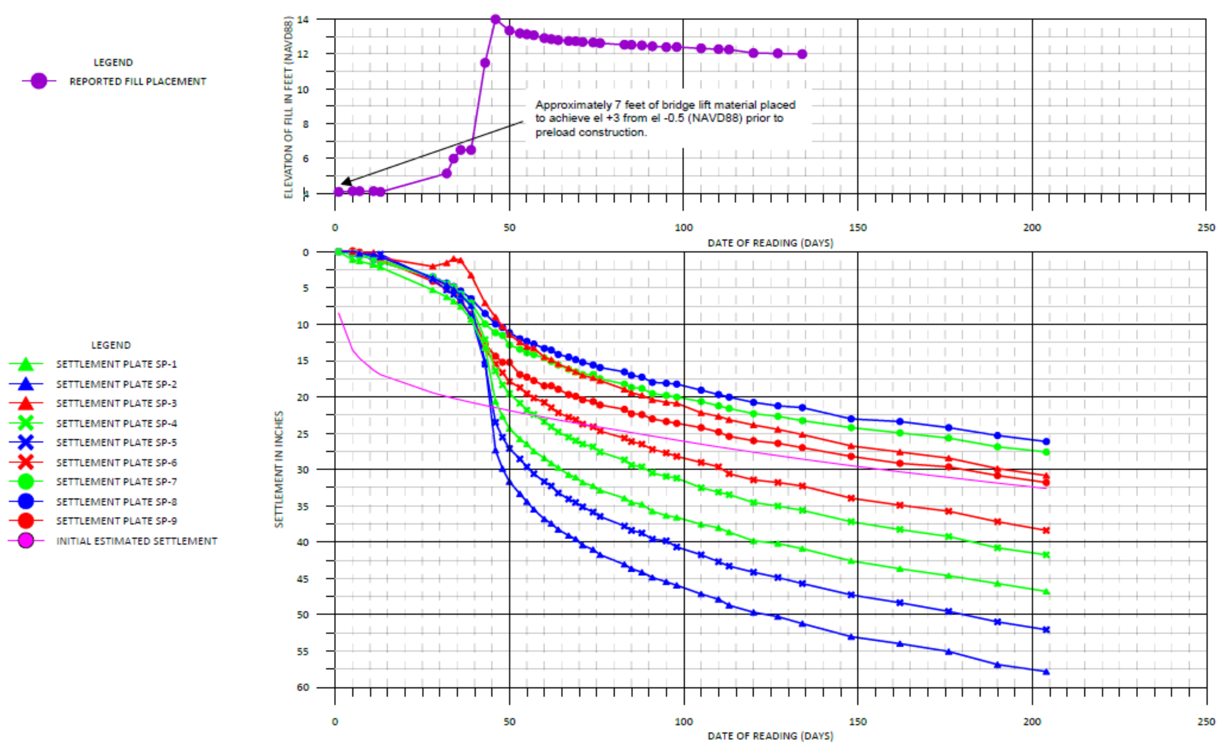


*Figure 4 – Settlement Plates and Initial Settlement Estimate: Settlement plate readings and un-staged settlement estimate plots.*

The steep slope in settlement data gradually flattened after approximately 40 to 50 days from plate installation but did not become completely horizontal within the model time frame of about 210 days. The exception to these trends was observed in the data from settlement plate SP-3, in which a brief upward slope, following a downward slope was observed. This could be due to errors caused by faulty/damage to the plate in the field. The most critical values, i.e., highest magnitude of settlement, would be of greatest concern during a construction phase of a project. Therefore, for model testing and analyzing the sensitivity of model parameters, we used data from SP-2 settlement plate observations.

The 'Initial Estimated Settlement' plotted on Figure 4 is the simulated settlement run without staged fill placement. This iteration of the analysis assumed that all fill was placed on-site on day 0. These simulated values are not in good agreement with observed readings. Notably, in the simulated result is an immediate steep downward slope, followed by a general level trend starting at approximately 20 days after day 0. To address the discrepancies, the model was run with staged fill. *Figure 5 – Staged vs Un-staged Fill Placement* includes both the 'Estimated Settlement – Not Staged Fill' line and the 'Estimated Settlement – Staged Fill' line. The simulated settlement with staged fill shows a delayed start to settlement as compared to the settlement plate readings. Settlement begins with a steep downward slope approximately 15 days after settlement plate installation and this corresponds with a reported fill height of approximately 5 feet. From about 40 days to the end of the simulation timeline, the staged fill estimate line follows the trend of the settlement plate readings but is much lower in magnitude than the highest settlement plate readings (plate SP-2).

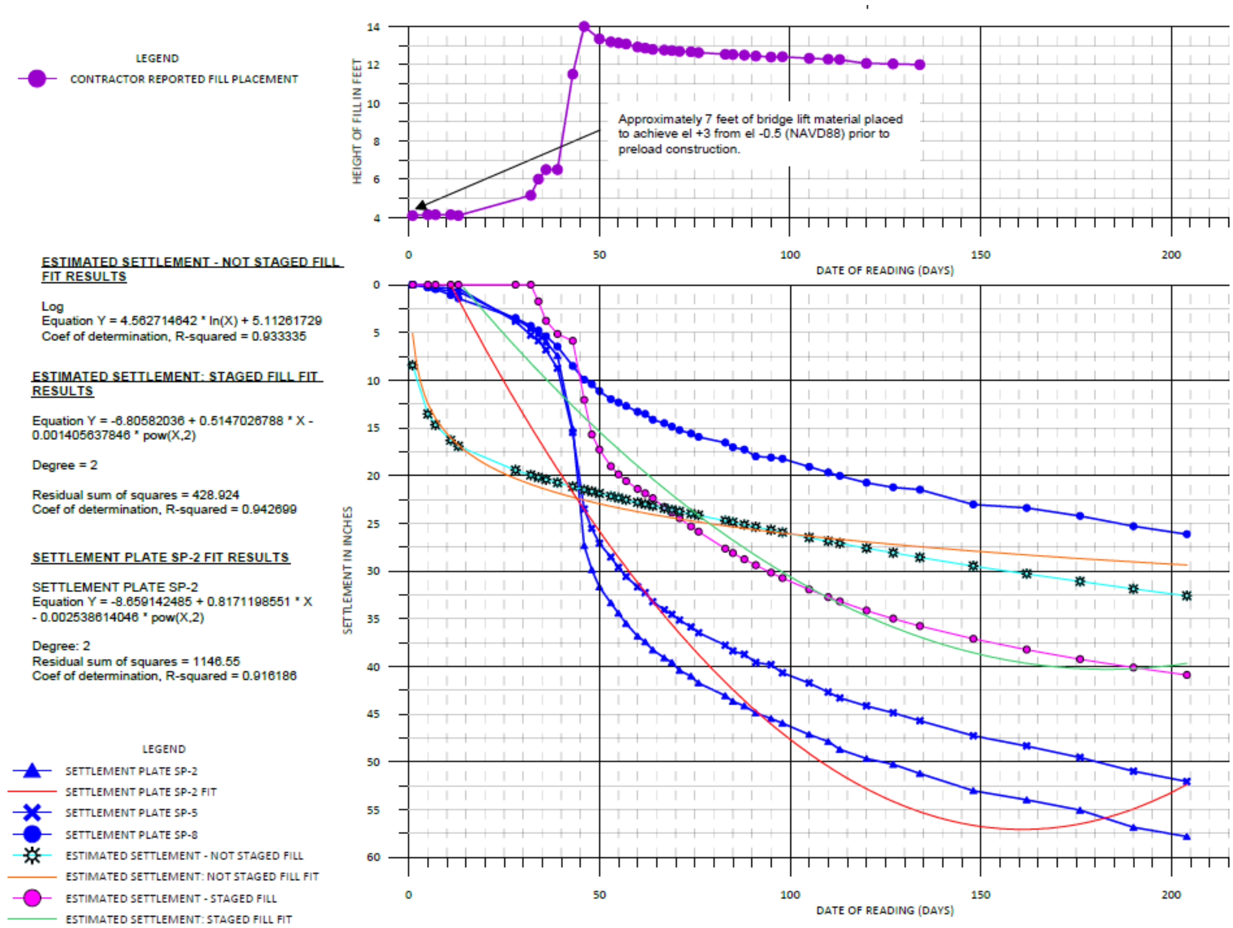


*Figure 5 – Staged vs Un-staged Fill Placement: Staged fill settlement estimates, un-staged fill settlement estimates and centerline settlement plate reading plots.*

A line of best fit was created for the estimated settlement with the staged fill line, the estimated settlement without staged fill, and for the settlement plate SP-2 readings (Summarized in Table 2 and also shown in Figure 5).

*Table 3: summary of empirical equations fitted to simulated and observed data from Plate SP-2*

| Description | Simulated_Not staged | Simualted_Staged | Observation (Sp-2) |
|---|---|---|---|
| Y=aLn(x)+b | Y=4.56*Ln(X)+5.11; $R^2$=0.93 | | |
| Y=a+bX+Cx$^2$ | | Y=-6.8+0.51X-0.001X$^2$; $R^2$=0.94 | Y=-8.6+0.81X-0.002X$^2$; $R^2$=0.91 |

Both the staged fill estimate line and the settlement plate readings line are best fit by a 2nd degree polynomial equation, each with an R-squared value of 0.94 and 0.92, respectively. The settlement estimate corresponding to not-staged fill line is best fit with a logarithmic equation (with an R-squared value of 0.93). This result highlights the importance of including staged fill placement in a settlement model, as well as the impact of an earthwork contractor's production schedule on the observed rate of settlement.

The local sensitivity method is used to analyze the sensitivity of each parameter (i.e., only one parameter at a time is changed while keeping all other parameters unchanged). Table 3 summarizes the sensitivity analysis.

*Table 3 Summary of the sensitivity experiment by percentage change in settlement to evaluate the sensitivity of the most important parameters of soil settlement.*

| Parameter | Change with respect to baseline parameter (One parameter at a time method) (% Change in Settlement at Day 204) | | | | | | |
|---|---|---|---|---|---|---|---|
| *OCR* | +20% ( -17%) | +50% (-49%) | -20% (+26%) | -30% (+44%) | -40% (+62%) | -50% (+71%) | |
| *Unit weight* | +5% (-11%) | -5% (+5%) | | | | | |
| *Wick Area (Width of Wick Area)* | +10ft (0%) | +50ft (0%) | +100ft (0%) | +200ft (0%) | | | |
| *Smear Zone Ratio* | 2/2 (-1%) | | | | | | |
| *Wick Horizontal Flow (*Ratio of Ch/Cv) | 4 (+7%) | 3 (+4%) | 2 (+2%) | 0.5 (-5%) | | | |
| *Drained Interfaces* | Drained Interface (+12%) | | | | | | |
| *Coefficient of Consolidation* | 50 (+20%) | 20 (+20%) | 5 (+19%) | 2 (+11%) | 1.5 (+7%) | 1.2.. 0.8, 0.5 (-10% to +3%) | X (+8%) |
| *Coefficient of Compression* | +20% (+13%) | +50% (+30%) | -20% (-13%) | -50% (-37%) | | | |

Figure 6 shows the sensitivity of the Overconsolidated Ratio (*OCR*). Several iterations of settlement analysis were run with various *OCR* values. The *OCR* parameters for each soil strata were increased by 20% and 50% and decreased by 20%, 30%, 40%, and 50% (See Table 2).

The increased *OCR* value (by 20% and 50%) both followed the staged fill placement settlement estimate line until approximately 40 days, at which point the *OCR* adjusted iterations both trended towards a horizontal plotted line with the 50% increase in *OCR* line trending towards minimal settlement after about 70 days and the 20% increase in OCR line trending towards minimal settlement after about 120 days. The final magnitude of settlement for both the 50% increase and 20% increase in *OCR* line was about 5 to 20 inches greater than the unadjusted settlement estimate.

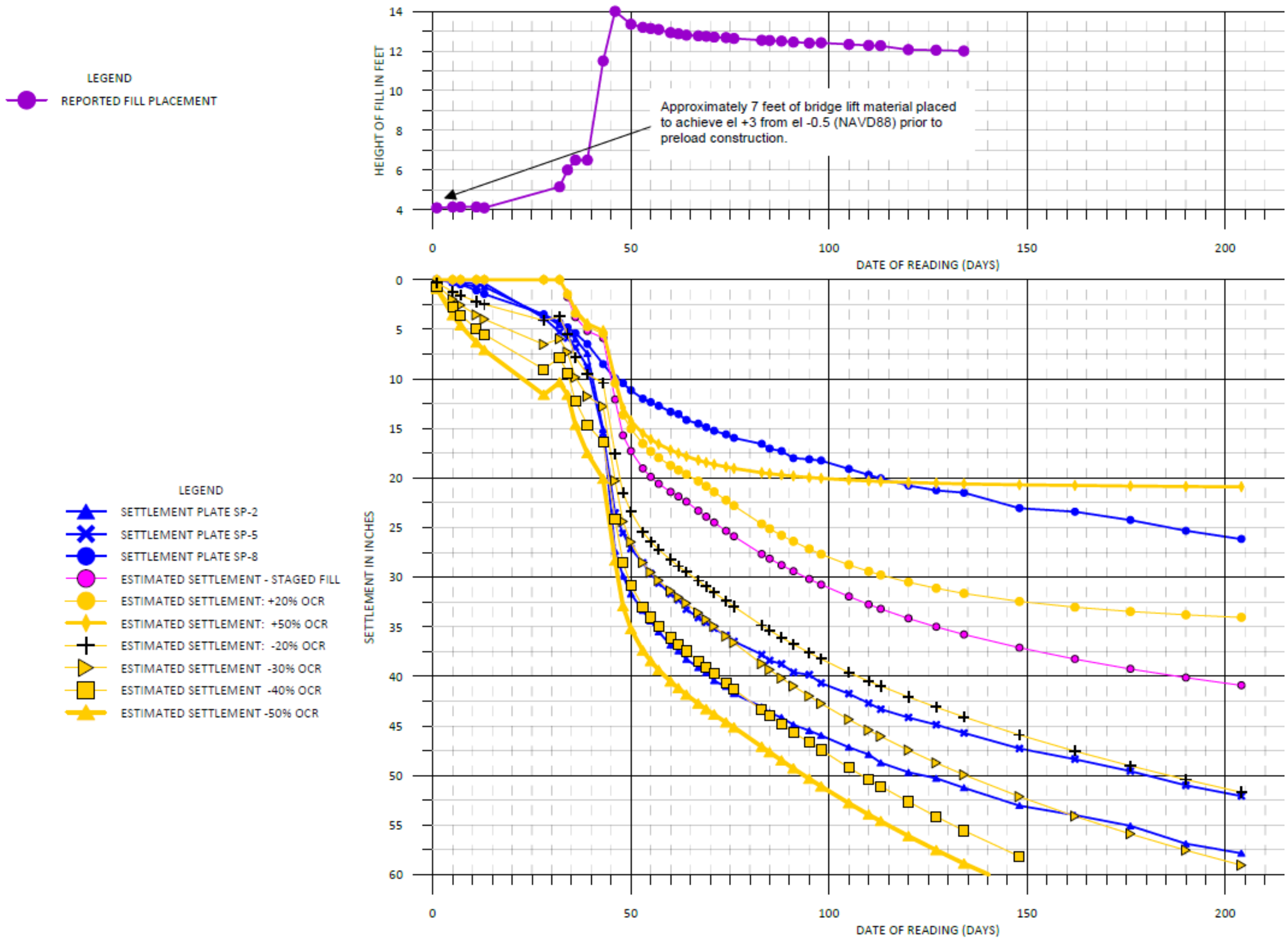


*Figure 6 – OCR Sensitivity: Settlement estimates with OCR parameters for each soil strata increased by 20% and 50%, as well as decreased by 20%, 30%, 40% and 50%.*

The decreased *OCR* value iterations (by 20% to 50%) showed a downward slope in the initial settlement estimate (up to about 20 days) which follows the trend observed in the settlement plate readings. The 20% decrease in *OCR* iteration matched plate SP-2 most closely during this period and generally followed the trend of a steep downward slope followed by a less steep slope to the end of the model time period. However, the 20% decrease in *OCR* iteration showed a generally lower magnitude of settlement. The 30%, 40%, and 50% decrease in *OCR* iterations showed steeper slopes and higher settlement up to about 20 days in the model. These iterations followed the steep downward slope of the SP-2 readings until approximately 50 days. From 50 days to about 80 days, the 40% decrease in *OCR* iteration generally matched the SP-2 readings, with the 30% and 50% decrease in *OCR* iterations plotting above and below the SP-2 readings, respectively. From about 170 days to the end of the model, the 30% decrease in *OCR* iteration most closely matched the SP-2 plate readings, with the 20% OCR decrease plotting above and the 40% and 50% decreases plotting below.

Sensitivity analysis was also performed for the unit weight parameter, as shown in *Figure 7 – Unit Weight Sensitivity*. The unit weight values were both increased by 5 % and decreased by 5% and plotted along with the settlement plate readings and staged fill settlement estimate line. Both of the unit weight value iterations generally followed the staged fill estimated settlement line. From day 0 to approximately day

45, both iterations follow the staged fill estimate fairly closely. After 45 days, the 5% increase in unit weight iteration follows the original estimate at a slightly lower magnitude of settlement, while the 5% decrease in unit weight iteration follows at a slightly higher magnitude. Both of these iterations present a much lower magnitude of estimated settlement as compared to the SP-2 settlement readings.

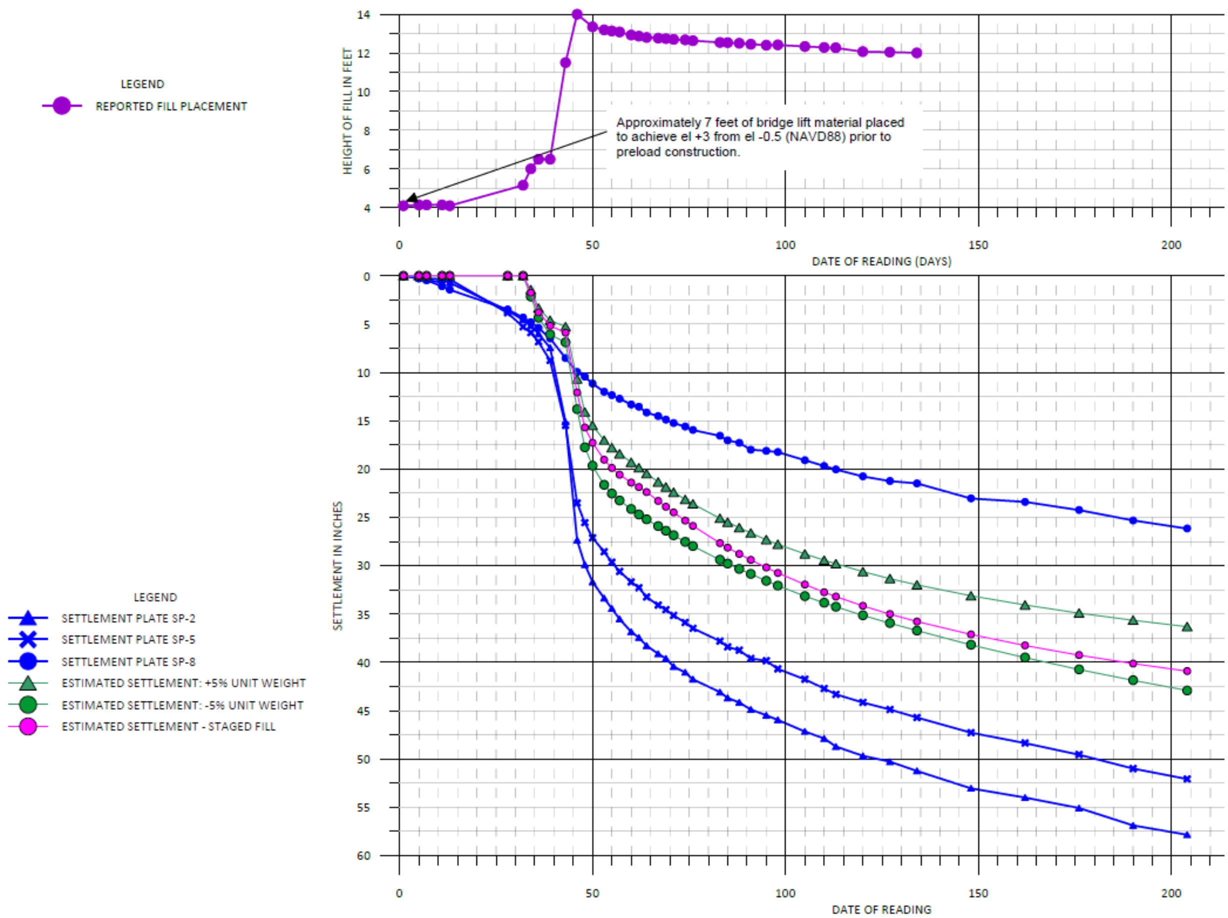


*Figure 7 – Unit Weight Sensitivity: Settlement estimates with unit weight values increased by 5% and decreased by 5%.*

In the initial analyses, the wick drain installation area was assumed to be approximately equivalent to the embankment footprint. The settlement analysis was also run with iterations of an increase in the width of the wick drain installation area. Iterations were run for 10-, 50-, 100- and 200-foot increases in the width of the wick area, but no changes in the settlement estimations were observed from these parameter adjustments. The results of these iterations are included in *Figure 8 – Wick Area Sensitivity*.

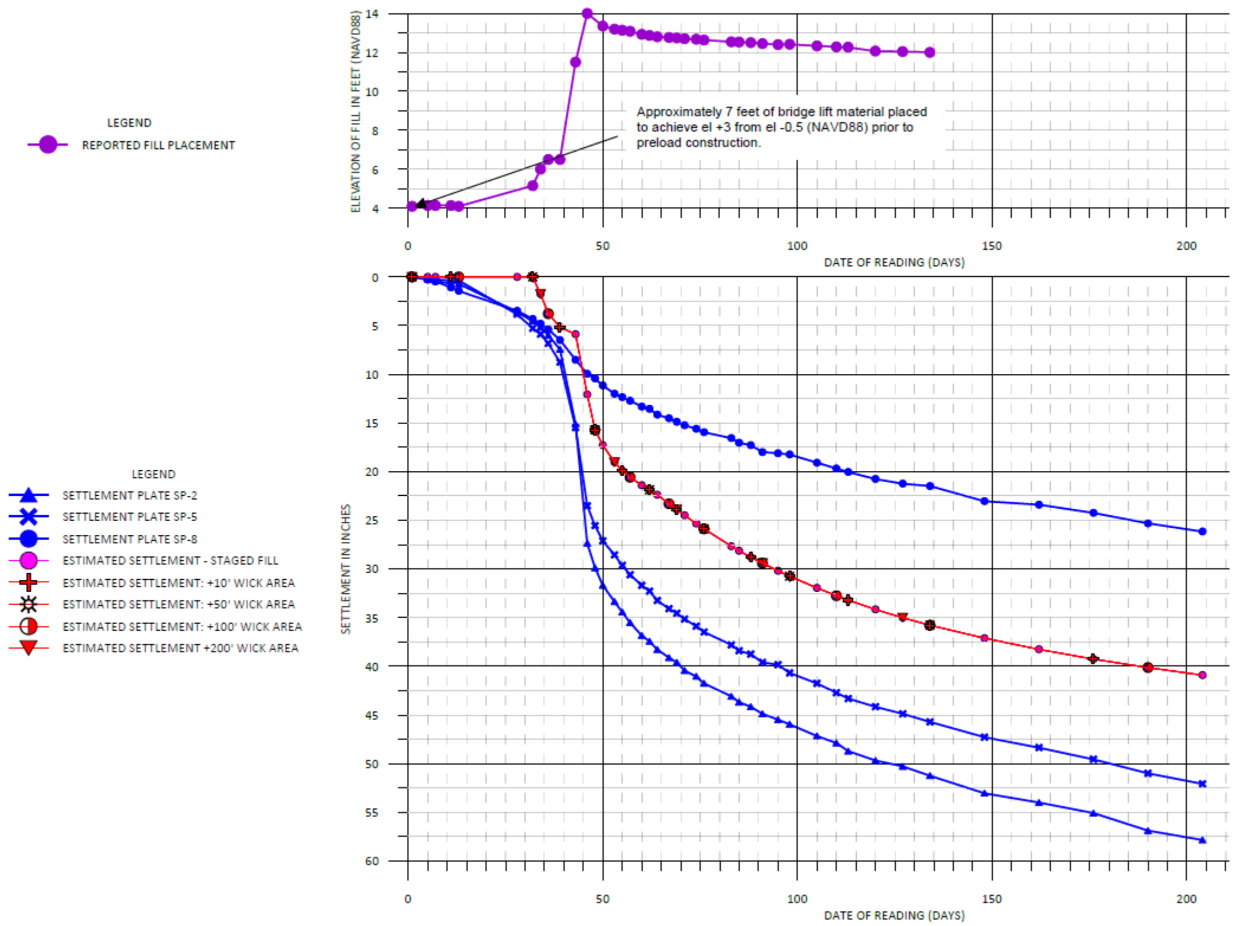


*Figure 8 – Wick Area Sensitivity: Settlement estimates with 10-, 50-, 100- and 200-foot increases in the width of the wick areas.*

The initial analyses included assumptions regarding the wick drain smear zone ratios. The ratio of the diameter of the smear zone to the diameter of the drain, and the ratio of undisturbed to smear zone permeability, were both assumed to be 1. A settlement estimate iteration was run with both ratios designated as 2. *Figure 9 – Smear Zone Ratio Sensitivity* show the results of this iteration. The adjusted smear ratio estimates plot very closely to the original estimate, with a slight decrease in magnitude of settlement from about 80 days to the end of the model timeline.

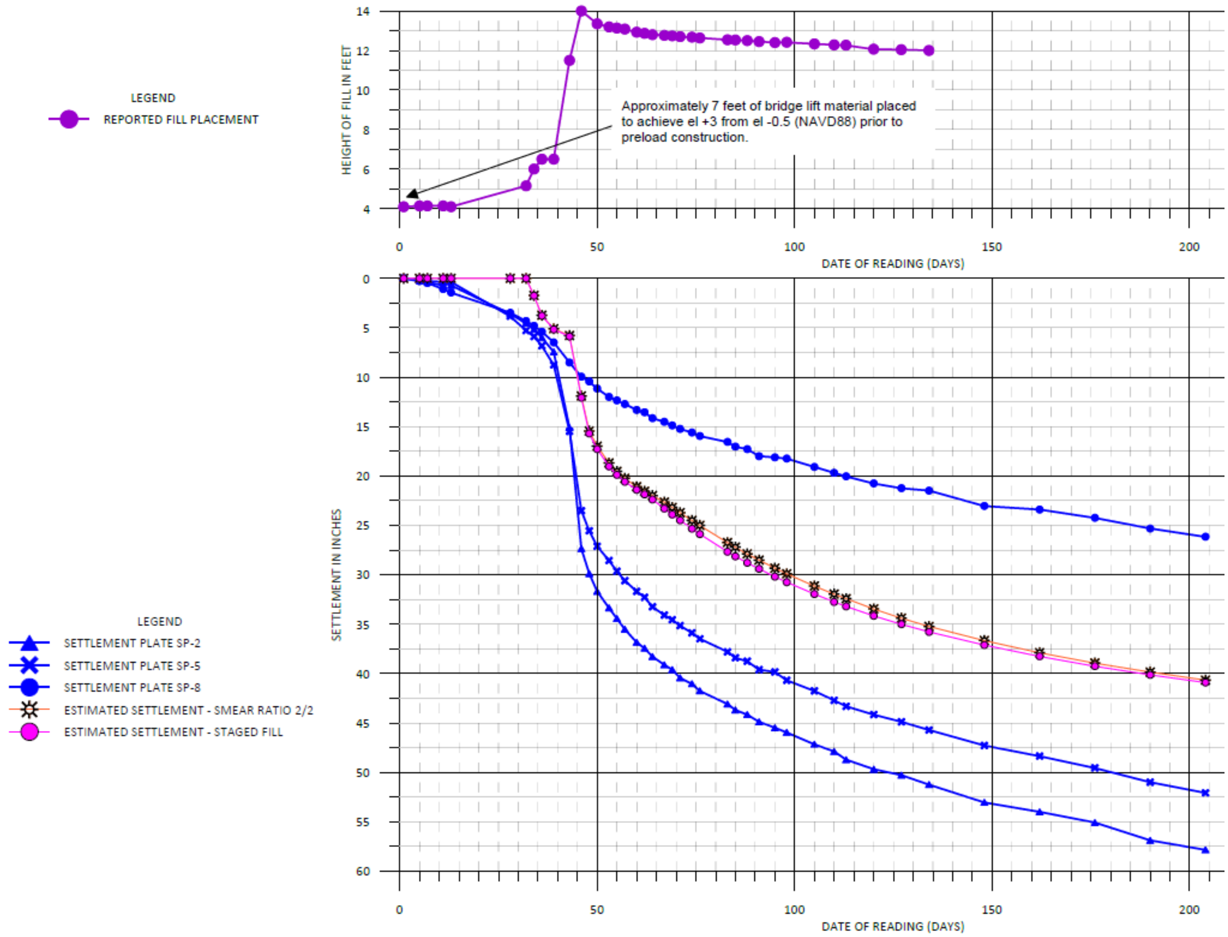


*Figure 9 – Smear Zone Ratio Sensitivity: Settlement estimate with ratio of the diameter of the smear zone to the diameter of the drain, and the ratio of undisturbed to smear zone permeability assumed to be 2.*

The Settle3 program has the option to select a horizontal to vertical flow ratio to adjust the horizontal flow for wick drains ($C_h/C_v$). The initial settlement estimate analyses were performed with this ratio set to 1. Settlement analysis iterations were performed for a *Ch/Cv* ratio of 4.0, 3.0, 2.0 and 0.5, as shown on *Figure 10 – Wick Horizontal Flow Sensitivity*. All iterations followed the initial settlement estimate until approximately 45 days, at which point the 0.5 ratio iteration represented a smaller magnitude of settlement and the 2.0, 3.0 and 4.0 iterations showed a larger magnitude of settlement, all while following the general downward trend of the initial estimate. While all iterations produced settlement estimates much smaller in magnitude than the settlement plate SP-2 readings, the iteration with a ratio of 4.0 produced estimates closest to the plate readings.

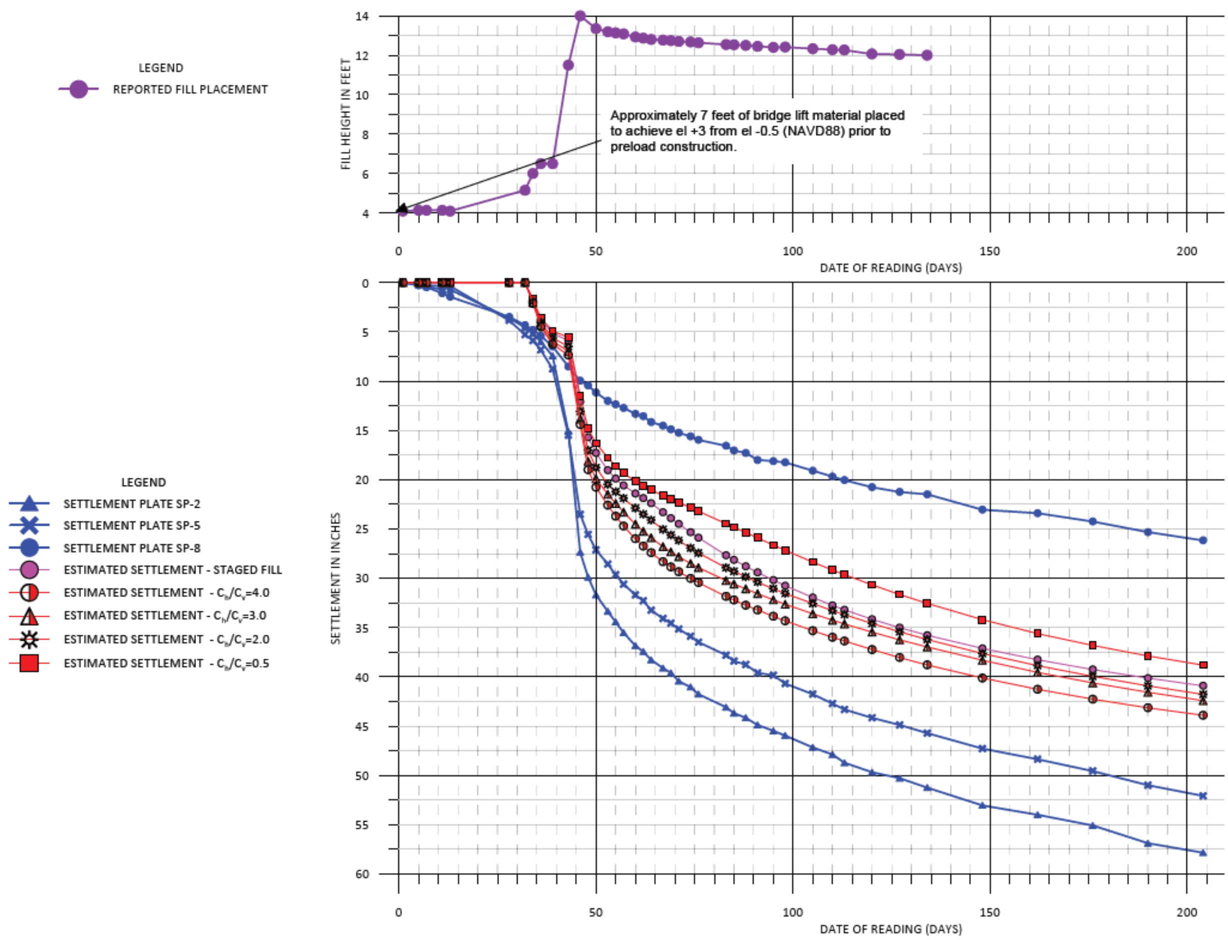


*Figure 10 – Wick Horizontal Flow Sensitivity: Settlement analyses with horizontal to vertical flow ratios (Ch/Cv) of 4.0, 3.0, 2.0 and 0.5.*

When entering the soil parameters for each stratum into Settle3, the interfaces between each stratum can be designated as drained or undrained. As shown on the soil parameters table in Appendix I, some interfaces were designated as drained. To explore this parameter further, a settlement estimate iteration was run with all interfaces designated as drained. This iteration is presented in *Figure 11 – Drained Interfaces Sensitivity*. This iteration followed the initial settlement estimate until approximately 25 days, where it continued to follow the initial settlement, albeit at a larger magnitude of settlement. This settlement estimate iteration shows settlement magnitudes closer to the SP-2 settlement plate readings, but the estimates were still much smaller than the plate observations.

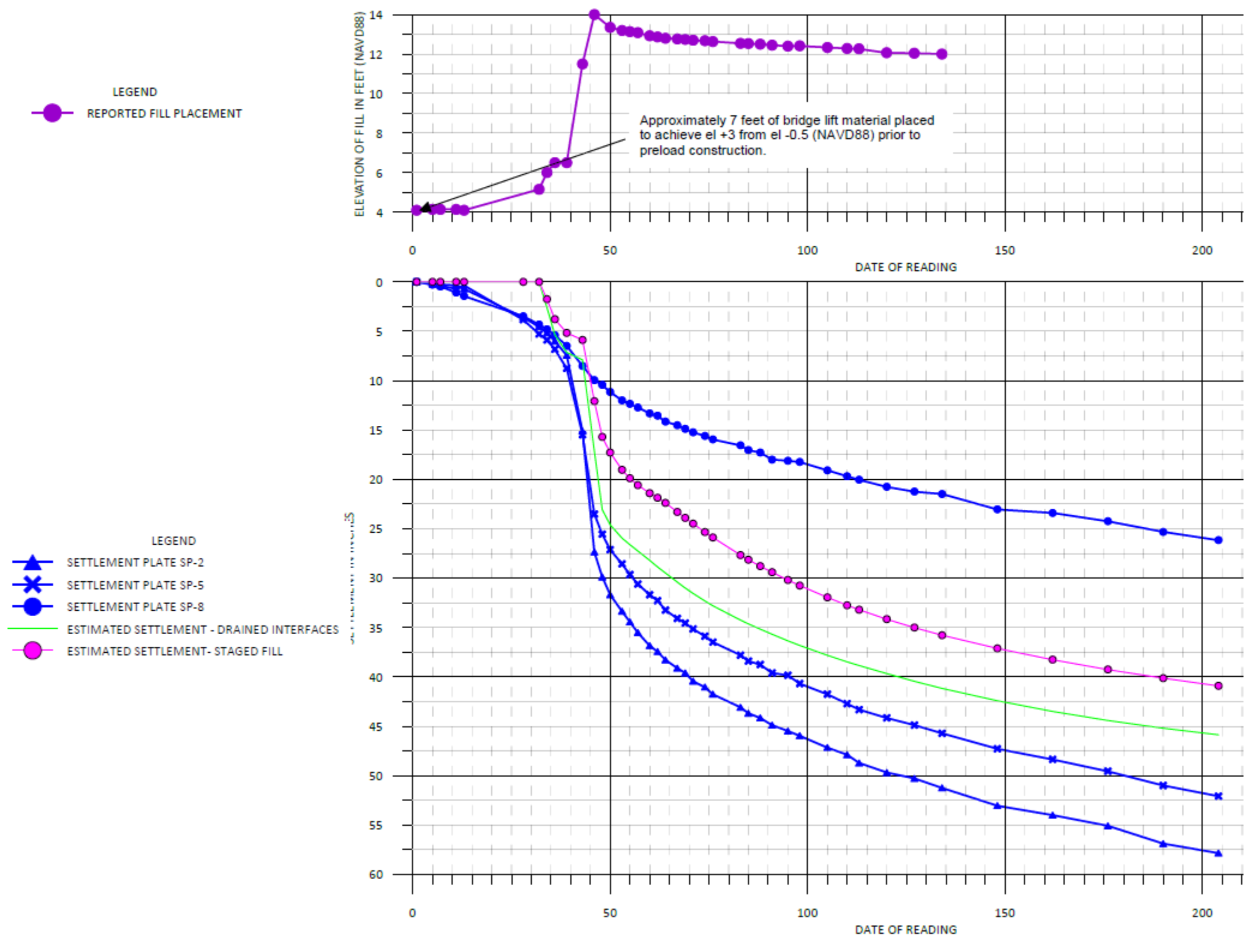


*Figure 11 – Drained Interfaces Sensitivity: Settlement analysis with all interfaces designated as drained.*

Several iterations of settlement analysis were run with various coefficient of consolidation values and plotted in *Figure 12 – Coefficient of Consolidation Sensitivity*. The $C_v$ parameters were adjusted to 50, 20, 5, 2, 1.5, 1.2, 0.8 and 0.5 times the original values. All the *Cv* iterations followed the initial settlement estimate to approximately 40 days, with the exception of the 50 and 20 multiplier iterations, which had a slight increase in settlement from about day 20 to day 40 from the initial settlement estimate plot. From day 40 to the end of the model, the 0.5 and 0.8 multiplier iterations presented as trending generally along the initial estimate with a slightly lower magnitude. The 1.2, 1.5 and 2 multiplier iterations also presented as trending generally along the initial estimate, albeit with a slightly higher magnitude estimate. The 5, 20 and 50 multiplier iterations presented a peak of increasingly rapid settlement to about 40 to 50 inches of settlement, followed by a rapid decrease in settlement to the model completion. This peak does not fit well with either the initial settlement estimate, or the SP-2 plate readings.

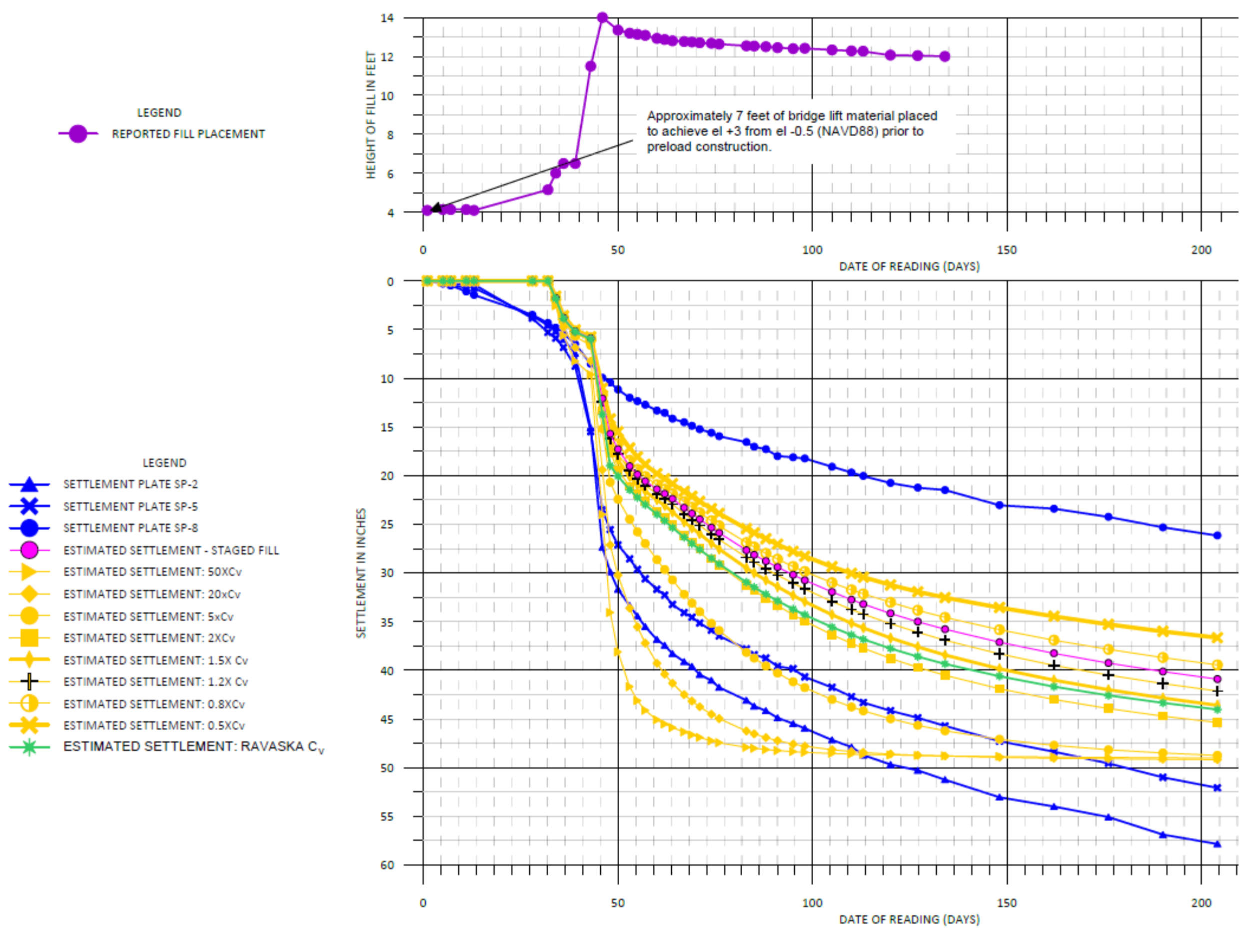


*Figure 12 – Coefficient of Consolidation Sensitivity: Settlement estimates with coefficient of consolidation, Cv parameters adjusted to multiples of 50, 20, 5, 2, 1.5, 1.2, 0.8 and 0.5 and X times the original values where X is a variable of applied loading stress as previously described.*

Additionally, an iteration of settlement analysis was performed with *Cv* values factored by *X,* a variance of applied loading stress changes in both time (stages) and depth (soil strata) as described in the Numerical Modeling section. These factors were calculated based on a derivation of the Ravaska *et al* model described previously. This iteration is plotted as the 'Estimated Settlement: Ravaska Cv' line in Figure 12. This iteration follows the initial settlement estimate to about 40 days, where the magnitude of settlement increases and follows the 2 multiplier iteration to approximately 80 days, where it falls between the 1.5 and 2.0 multiplier iterations in terms of settlement magnitude and slope trends.

Finally, several iterations of settlement analysis were performed with coefficient of compression, or $C_c$ values. Iterations were run with a 20% and 50% increase, as well as a 20% and 50% decrease. These iterations were plotted on *Figure 13 – Coefficient of Compression Sensitivity*. The iterations all plotted similarly to the initial settlement estimate until approximately 45 days, where the divergence from that initial estimate begins. The 20% and 50 % increase follow this initial settlement estimate line but show an increasing magnitude of settlement from it. The 20% and 50% decrease iterations also follow this initial settlement estimate line, but show a decreasing magnitude of settlement. The 50% increase in Cc iteration is observed to match most closely to the SP-2 plate readings, but all of the estimates have a much lower magnitude of settlement than the plate readings.

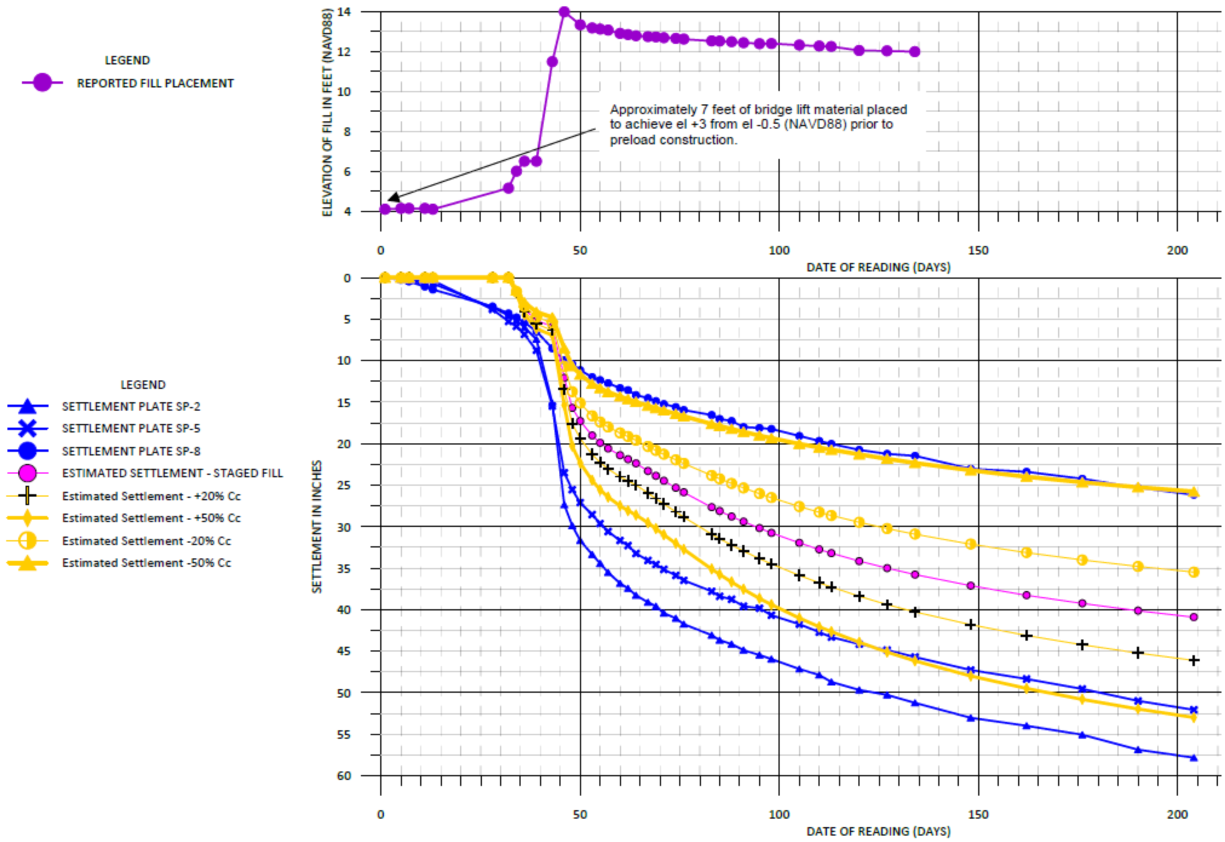


*Figure 13 – Coefficient of Compression Sensitivity: Settlement analyses with a 20% and 50% increase, as well as a 20% and 50% decrease in the compression index, or Cc values.*

The sensitivity results are summarized in the table below. *OCR*, unit weight of soil, $C_v$ and $C_C$ are all highly sensitive. To quantify the results of the sensitivity analysis, the impact of the changes to input parameters is represented by the change in the estimated settlement value after 204 days of simulation. Detailed results of each parameter on the time dependent behavior of settlement is shown in Figs (6-13). The settlement for the default parameter is 40.9 inches (@ 204 days).

*Table 4 - Summary of Sensitivity Experiment Results by Magnitude of Settlement at Day 204*

| Simulation | Parameters | Change | Settlement in Inch @ 204 days | Simulation | Parameters | Change | Settlement in Inch @ 204 days |
|---|---|---|---|---|---|---|---|
| 1 | | 20% | 34.1 | 19 | | 0.5*Cv | 36.6 |
| 2 | OCR | 50% | 20.9 | 20 | Coefficient of Consolidation Cv | 0.8*Cv | 39.4 |
| 3 | | -20% | 51.7 | 21 | | 1.2*Cv | 42.1 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 4 | | -30% | 59.0 | 22 | | 1.5*Cv | 43.6 |
| 5 | | -40% | 66.4 | 23 | | 2*Cv | 45.4 |
| 6 | | -50% | 69.9 | 24 | | 5*Cv | 48.7 |
| 7 | Unit wt. | 5% | 36.3 | 25 | | 20*Cv | 49.2 |
| 8 | | -5% | 42.9 | 26 | | 50*Cv | 49.0 |
| 9 | Wick area (feet) | 10 | 40.9 | 27 | Coefficient of Compression (Cc) | 20% | 46.1 |
| 10 | | 50 | 40.9 | 28 | | 50% | 53.0 |
| 11 | | 100 | 40.9 | 29 | | -20% | 35.5 |
| 12 | | 200 | 40.9 | 30 | | -50% | 25.8 |
| 13 | Smear ratio | 2/2. | 40.7 | 31 | SP-2 Plate | | 57.8 |
| 14 | Wick horizontal flow | 4 | 43.9 | 32 | SP-5 Plate | | 52.1 |
| 15 | | 3 | 42.4 | 33 | SP-8 plate | | 26.2 |
| 16 | | 2 | 41.8 | | | | |
| 17 | | 0.5 | 38.8 | | | | |
| 18 | Drain Interfaces | | 45.9 | | | | |

Both increasing and decreasing the wick area had minimal effect on the magnitude of the estimated settlement. As shown in Eqn (19), the wick drain diameter has an effect on the change in pore pressure that would affect the settlement estimate but the results of the parametric analysis indicate that an extremely large increase in the number of wicks would be required to see a significant change in the estimated settlement.

Adjustment of the coefficient of consolidation yields significant changes in estimated settlement magnitude. This follows the relationship between dissipation of pore water pressure (related to loading stress) and the coefficient of consolidation as shown in Eqn. (6). It is also interesting to observe that increasing the coefficient of consolidation yields diminishing returns on the final model settlement estimate, as the coefficient is related to the rate of change of pore water pressure, which decreases towards the end of the model, rendering any increase in the coefficient of consolidation less impactful.

Similarly, adjustment to the coefficient of compression also yields significant changes in estimated settlement magnitude. As seen in Eqn. (5), the magnitude of estimated settlement changes with the logarithm of the change in stress, factored by the coefficient of compression.

Changes in the *OCR* values also yield significant changes in estimated settlement magnitude. It is important to note that *OCR* is the only parameter to yield significant changes to the settlement magnitude as the loading stress increases from approximately day 5 to day 45, as seen in Figure 6. As shown in Eqn. (6), the *OCR* is a ratio of the preconsolidation stress to effective stress. The initial relationship (from day 5 to day 45) between *OCR* and estimated settlement would indicate that the original estimated preconsolidation pressure values may have been overestimated. However, for any singular *OCR* adjustment from day 5 to the end of the model, the accuracy of the estimate severely declines. For example, the iteration with *OCR* values decreased by 20%, matches the settlement plate 2 line fairly well from day 5 to 45 but from day 45 to the end of the model, the settlement estimate is lower than the settlement plate readings. This indicates that there are possibly multiple parameters in which accuracy could be improved.

## Discussion and Conclusion

Comparison between this Settle3 model and a simple Terzaghi 1D solution using a tool such as the method described in Fang Xu *et al* (2019), which assumes one homogenous layer, shows the benefits of the Settle3 program. The following input was utilized with Xu *et al*'s model assuming a uniform surcharge and one homogenous soil strata:

```
surcharge = 100 #kPa
duration = .30 #years of which surcharge is applied
length = 0.1 #m, PVD strip long side
width = 5e-3 #m, PVD strip short side
spacing = 1.5 #m c/c spacing
pattern = 'square' #arrangement
smear = 0.8 #smearing factor

#Strata thickness, initial and final effective stresses
#and double drain condition validity
H = 1 #m
Po, Pf = 1,93 #kPa
double_drain = True #if mudstone/stiff clay base, set to False

#Soil consolidation properties
Pc = Po #kPa, Assumed NC in this case
e0 = 2.0
Cc =0.3
Cr = 0.05
cv = 0.678 #m^2/year
ch = 0.678 #m^2/year
```

*Figure 14 – Simplified Settlement Estimate Input Parameters*

The simplified analysis resulted in the below settlement estimate:

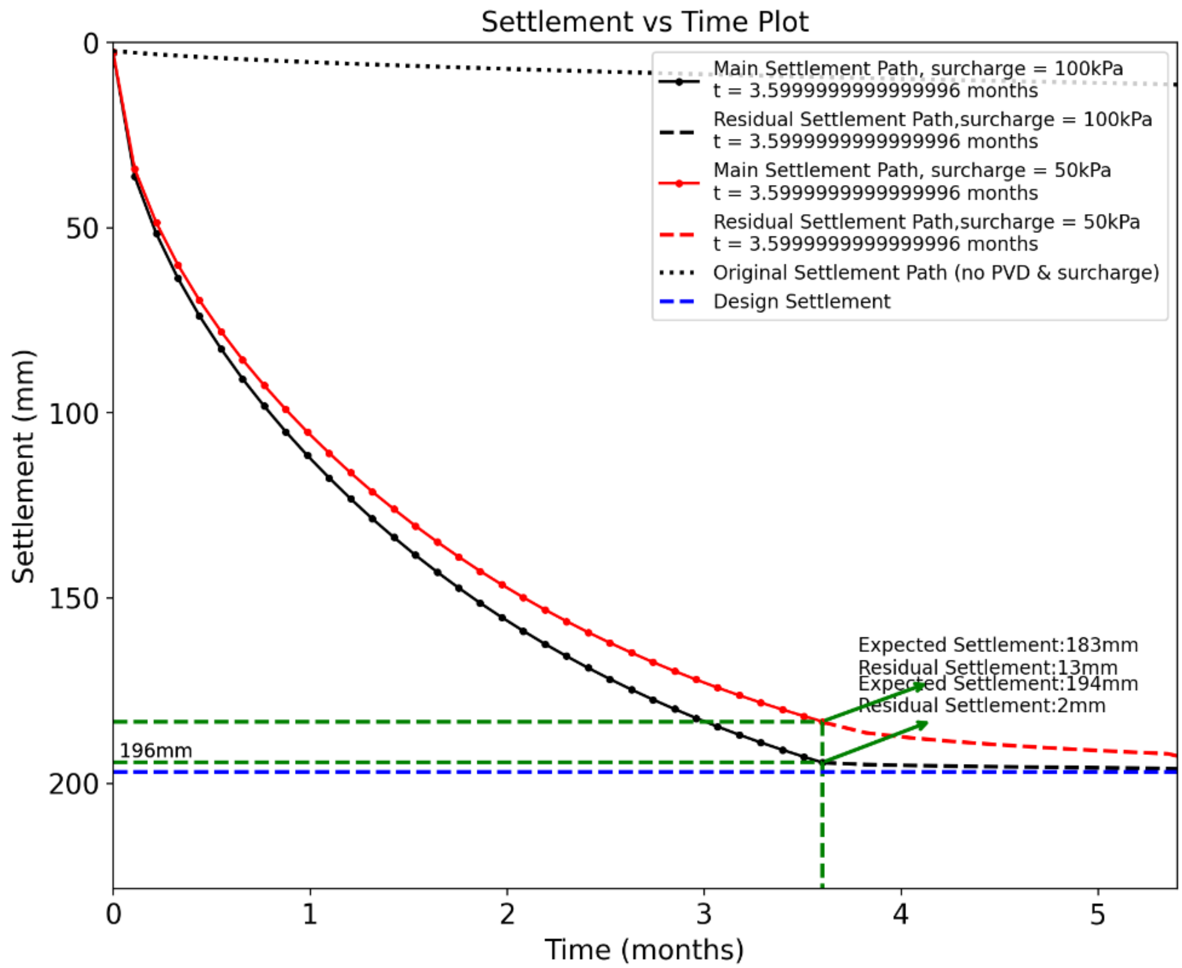


*Figure 15 - Settlement vs time plot with Simplified consolidation model*

To compare the performance with the Settle 3D program, the study also used a simplified soil settlement model for a time of 105 days. The simplified model uses both Terzaghi's and Barron's analytical solutions to simulate expected and residual settlement magnitudes given general soil properties and duration. Even though computation approaches like Settle 3D provide robust solutions, analytical solutions (if calibrated properly) also provide reasonable estimates (comparable to FDM, and FEM) and cross check results obtained from numerical models. The simplified model resulted in an expected settlement of 183 mm or 7.2 inches (Fig 15). The un-staged fill settlement estimate performed with Settle3 resulted in about 25 inches of settlement over the same time period, and the staged fill settlement estimate resulted in about 32 inches of settlement. The readings for settlement plate SP-2 produced about 47 inches of settlement at 105 days. This simplified model has much lower accuracy than the other methods previously explored.

One important aspect of all data gathered and utilized in these calculations is the possibility of inaccuracies. There are an almost endless number of ways in which the input data and model parameters can be affected by these potential inaccuracies. The schedule and magnitude of the fill placed for the embankment was reported by a general contractor and those reports could very well be inaccurate. Settlement plates are very commonly disturbed by earthwork equipment and any movement after the initial plate readings could affect the subsequent readings. The compaction of the embankment could have some variation, affecting the assumed applied loading stress per foot of fill placed. Any of the soil parameters selected from laboratory testing results could be affected by soil disturbance, or variations between exploration locations could have been missed during the geotechnical field exploration. At this particular site, it is estimated that about 7 feet of fill was placed before the settlement plates were installed to construct a stable working platform. Installation of settlement monitoring equipment would not have been possible before installation of this initial fill, but it is very likely that there was some resulting effect on the soil system.

Considering all these difficulties, the typical Terzaghi 1-D settlement model analyzed through Settle3 is acceptable for the purposes of construction monitoring. However, use of adjustments of parameters as initial settlement monitoring data is available would be an important tool for increased accuracy in predicting the remaining settlement at a project site. With an increase in certainty of a predicted settlement magnitude and timeline, construction schedules can be compressed and cost savings can be gained. Additionally, more extensive initial geotechnical field exploration may result in cost savings over the total life of the project if confidence in settlement estimates can be increased.

## Recommendations for future research

The coefficient of consolidation estimates outlined in Ravaska *et al*'s research is a promising avenue for further research. The ideal next step to continue local correlations would be to perform stepwise permeability testing in conjunction with odometer (consolidation) testing. This would provide a baseline for modeling how south Louisiana clays vary in permeability with strain. Beyond permeability testing, the beta value could be correlated from multiple sites with available settlement monitoring data.

Additionally, the settlement monitoring program for this project site included additional instrumentation such as settlement gauges, magnet extensometers, piezometers and Shape Array Accelerometers (SAAs). The settlement gauges provided continuous settlement readings, the magnet extensometers provided observation of settlement at various depths, piezometers provided estimates of pore water pressure over time and the SAAs provided lateral deflection observations. All of this instrumentation could be compared with the parametric studies previously discussed.

There is certainly a plethora of opportunity for continued improvements of settlement estimate and prediction analysis in south Louisiana. Connections with industry professionals and academic contacts to investigate results of settlement monitoring programs could be extremely beneficial to the geotechnical industry in the area. The same type of sensitivity analysis performed for this project could similarly be performed for any project with substantial fill placement in the region. Regardless of the project type of specific geology, any project where a geotechnical field exploration and a settlement monitoring program was performed has the potential to be studied and to add to the local knowledge base.

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# Vita

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