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Ground Improvement and Settlement Monitoring Program For A Power Plant Project

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ABSTRACT

CH2M HILL is providing engineering, procurement, and construction services for a power plant project in Utah. Before beginning construction on the power plant, geotechnical studies were performed to characterize the subsurface conditions for the anticipated settlement and to determine a ground improvement method to accelerate the construction period. Ground improvement with wick drains and surcharge fill placement was carried out to improve the soft soil conditions at the project site. Settlement monitoring instrumentations were installed before placing structural fill and surcharge fill. An extensive settlement monitoring program was also implemented inside and around the perimeter of the project site to monitor the settlement impact to adjacent facilities due to structure load and surcharge fill placement. The monitoring period was extended even after surcharge fill removal to observe the rebounding behavior of the foundation soil. This paper presents the challenging site conditions, such as soft soil, the design optimization implemented to accelerate the settlement period, and the comparison between predicted and measured settlement at the project site. This paper also presents finite element simulation of ground deformation and rebound behaviors observed during the surcharge loading and unloading stages.

INTRODUCTION

Design and construction of embankments and structures on soft clay deposits are one of the important challenges of geotechnical engineering. For construction of deformation-sensitive structures, such as a power plant, the magnitude of deformations and control of these characteristics are extremely important for the serviceability of structures and equipment. Excessive deformations under sensitive structures may lead to cracking, fractures, and potential structure and/or equipment failures.

Soft clay deposits usually have a low bearing capacity, lower permeability, and high compressibility. It is inevitable that the soft clay deposits have to be treated before the placement of structures. Although a variety of soil improvement techniques are available, pre-consolidation using wick drains and surcharge fill preloading is one of the most popular and effective techniques in practice. Consolidation of compressible soils involves removal of pore water from the soil. This can be done by applying a surcharge load to squeeze the water out. To accelerate the dewatering and consolidation process, wick drains can be installed to provide conduits for water flow and to shorten the flow path of the water in the low-permeability soil.

A power plant is being constructed in former marshland area near Utah Lake. Prior to beginning construction of structures, a ground improvement program using wick drains and surcharge preload was conducted with extensive field instrumentation to monitor ground deformations. This paper presents challenging subsurface conditions, a design optimization implemented to accelerate the settlement period, field monitoring data, and comparison between measured ground deformation and rebound behaviors observed during the loading and unloading stages and simulated ground deformation behavior using finite element method (FEM) analyses.

PROJECT BACKGROUND

The proposed power plant site is situated on an approximately 83-acre area near the northeast corner of Utah Lake in Utah. The proposed plant is a nominal 637-megawatt electric generating facility that uses natural gas to produce electrical energy. The plant layout contains the following three functional areas:

- Combustion turbine generators (CTGs) and generator step-up transformers (GSUs)
- A steam turbine generator (STG), water treatment buildings, and a GSU
- Cooling towers

Several other facilities, systems, and equipment are within each of these areas. The major components include water tanks, heat recovery steam generators (HRSGs), and an STG and its auxiliaries housed inside the STG building. The structures for the high-voltage transmission lines to carry the electrical energy out of the plant will be built. The proposed facilities will remain in service for a 30-year design life.

The natural topography across the plant site prior to its development was generally flat with elevations ranging between 4,495 and 4,498 feet. No significant past development appears to have taken place at the site. The site condition before construction is shown in Figure 1.

Fig. 1. Site Condition before Construction

SUBSURFACE CONDITIONS

The plant site is located approximately 0.5 mile east of the current shoreline of Utah Lake and approximate 3 miles west of the Wasatch Mountains in north-central Utah. The site is situated within the limits of historic Lake Bonneville (Solomon et al., 2009). Lake Bonneville was a large, ancient lake that existed from about 32 to 14 thousand years ago. It occupied the lowest, closed depression in the eastern Great Basin. Lake Bonneville at its peak covered about 20,000 square miles of western Utah and encroached upon minor portions of eastern Nevada and southern Idaho. According to the Geologic Map for the Pelican Point Quadrangle (Solomon et al., 2009), the site is mapped as being underlain by lake (lacustrine) deposits. The following unit description is modified from Soloman et al. (2009): Upper Pleistocene-aged, lacustrine silt and clay, (calcareous silt (marl) and clay with minor fine sand); typically laminated or thinly bedded; deposited in quiet water in moderately deep parts of the Bonneville basin and in sheltered bays. Exposed unit thickness is less than 15 feet, but total thickness may exceed several tens of feet.

The field exploration for the plant site consisted of rotary-wash borings and cone penetration test (CPT) soundings to depths of approximately 100 to 135 feet below ground surface (bgs). Sampling procedures in the rotary-wash borings generally followed ASTM International (ASTM) methods for SPT and split-barrel sampling of soils (ASTM D1586) and Shelby tubes. The CPTs were conducted in general accordance with the current ASTM D5778 specifications (ASTM, 2007) using a 15-square-centimeter (cm^2) electronic cone penetrometer.

Artificial fill was encountered in the borings drilled at the southern area of the plant site. Generally, the fill appears to be uniform in thickness averaging about 8 to 9 feet. The artificial fill is predominantly loose to medium-dense silty sand with some fine gravel. It is understood that this fill was placed to construct the lay-down area during grading of a previous construction.

The lacustrine deposits underlying the plant site below the artificial fills consist of uniform distinct zones within the subsurface profile throughout the site. In the upper 38 feet, the lacustrine deposits consist of stiff lean clay with occasional thin layers of dense clayey sand and become predominantly soft lean clay between depth ranges of 38 and 85 feet. Below 85 feet, the lacustrine deposits consist of two medium-dense to dense sand layers separated by a stiff lean clay layer underlain by soft to stiff lean clay. At a depth of about 130 to 140 feet, the lacustrine deposits become granular and consist of dense to very dense clayey sand, fine to coarse gravel, and possibly cobbles.

Groundwater elevations encountered at the project site varied from 4,495 to 4,498 feet. Artesian conditions exist at the site within the sand layers between 90 and 110 feet bgs and within the gravel layer below 130 feet bgs. Based on the pore pressure dissipation test performed in CPTs, a head of approximately 10 feet above the static groundwater level was estimated within the 90- to 110-foot sand layers, and a head of approximately up to 30 feet above the static groundwater level was estimated within the gravel layer between 130 and 140 feet bgs.

ENGINEERING SOIL PROPERTIES

The variation of natural water content, Atterberg limits, coefficient compression indexes, and overconsolidation ratio (OCR) are shown in Figure 2 with the generalized soil profile. The grain size distributions of the stiff clay and soft clay include 81 to 100 percent fines and 92 to 99 percent fines, respectively. The overall range of natural water content, liquid limit, and plastic limit were as follows: 9 to 58 percent, 17 to 58 percent, and 12 to 27 percent, respectively.

The coefficient of compression (C_{ce}) and coefficient of re-compression (C_{re}) were determined using one-dimensional consolidation tests. In stiff clay, C_{ce} and C_{re} ranges from 0.059 to 0.09 (with an average of 0.08) and 0.004 to 0.025 (with an average of 0.02), respectively. In soft clay, C_{ce} and C_{re} ranges from 0.133 to 0.286 (with an average of 0.2) and from 0.011 to 0.06 (with an average of 0.04), respectively. Preconsolidation pressures obtained from the one-dimensional consolidation tests indicated that the deposits are overconsolidated in the upper parts, and the overconsolidation decreases with depth to normally consolidated condition as shown in Figure 2. Coefficients of consolidation were also measured during the one-dimensional consolidation tests. As shown in Figure 2, the coefficients of consolidation generally show good agreement with *Approximate Correlations for Consolidation Characteristic of Silts and Clays,* presented in the Naval Facilities Engineering Command (NAVFAC) *Design Manual 7.01, Soil Mechanics* (NAVFAC, 1986).

PRECONSTRUCTION SETTLEMENT PREDICTION

During the design phase, potential settlements were estimated using one-dimensional consolidation theory. The calculations are presented in *Technical Memorandum for Ground Improvement* and *Geotechnical Design Report* prepared by CH2M HILL (2011a and 2011b). Settlement magnitudes were estimated for two cases—one after the structural fill placement without ground improvement and the other case with wick drains and then structural fill placement and surcharge loading. In both cases, the existing fill was removed from the project site and the unsuitable soil was overexcavated from the upper 0.5 to 1 foot in the wetland area to the elevation of 4,495 feet.

For the case without ground improvement, approximately 15 feet of structural fill was modeled on top of the native ground before constructing the power plant equipment footings. After the structural fill placement, the proposed equipment footings were modeled at specified embedment depths to estimate the settlement. Under these loads, settlement periods from 26 to 30 months were estimated to achieve 95 percent of consolidation settlement. The estimated maximum settlement was approximately 25 inches.

For the case with ground improvement, wick drains and surcharge fills were modeled in addition to the structural fill. The top elevation of the structural fill was planned approximately 3 feet above proposed finish grade to compensate the anticipated settlement. The settlements of 21 inches and 31 to 35 inches were estimated under the 15-foot surcharge fill and 22-foot surcharge fill, respectively, with surcharge period of 3 months.

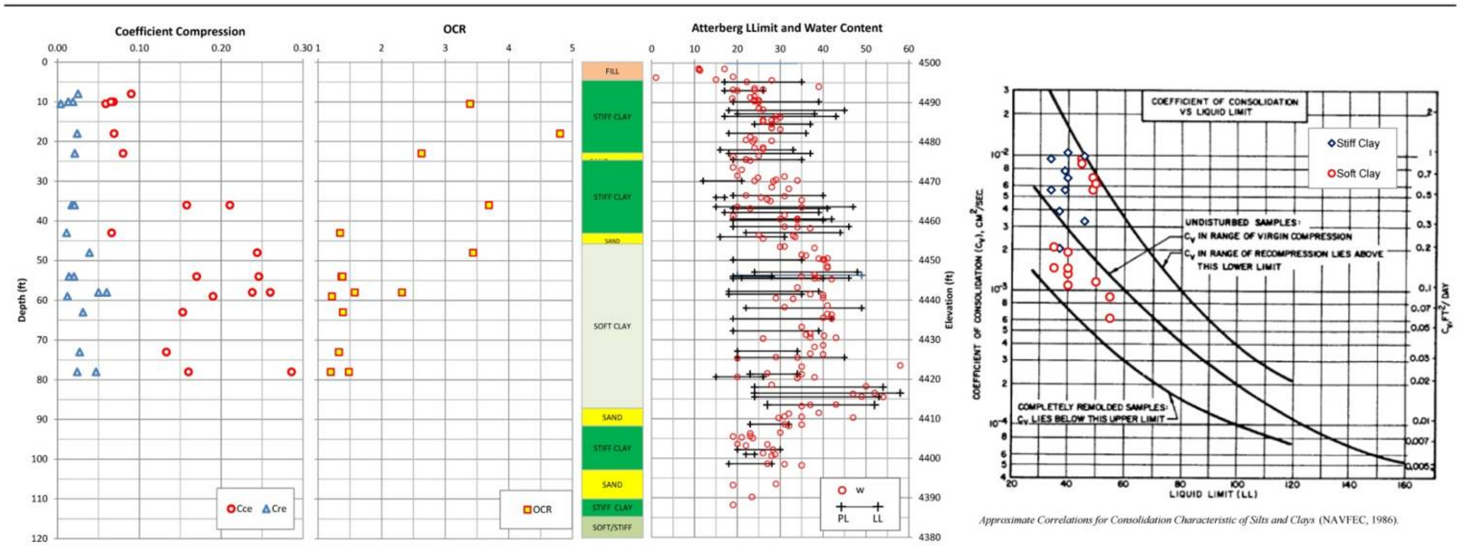

Fig. 2. Variation of Soil Property Parameters

GROUND IMPROVEMENT

Based on the preconstruction settlement prediction, a design optimization to accelerate the settlement period was selected. Because of the soft subsurface soils with low permeability at the site, the settlement induced by the structural fill and the equipment loads was estimated to be a maximum of approximately 25 inches with 26 to 30 months of settlement period to achieve its 95 percent of consolidation settlement. To expedite the settlement time at the project site, the subsurface ground was improved by placing wick drains and surcharge fill (CH2M HILL, 2011b).

Site Preparation

The existing fill, which was lay-down fill from the previous construction, was removed to the elevation of the native ground (approximately 4,495 feet). The onsite native soils are generally wet, soft, and with pumping conditions as shown in Figure 1. As such, the prepared ground surface was stabilized by placing Tensar MS 220 Geogrid to provide a firm subgrade for the access of construction equipment.

Drainage Blanket

On top of the prepared ground surface and Geogrid at an elevation of 4,495 feet, an approximately 2-foot-thick drainage blanket layer was placed to receive the flow of water conveyed through the wick drains. The drainage blanket layer consists of aggregates with maximum size of 3/4-inch. Details of the drainage blanket are presented in Figure 3.

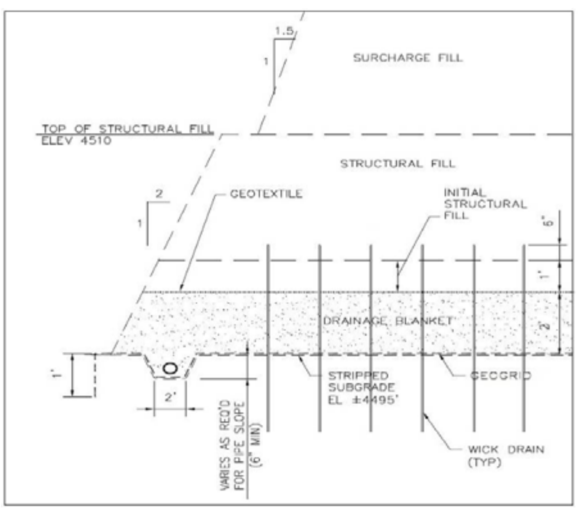

Fig. 3. Drainage Blanket and Wick Drain Installation Details

Wick Drain Installation

To facilitate the installation of the wick drains and the equipment, 1 foot of structural fill was placed on top of the drainage blanket. A Geotex 801 geotextile was placed between the drainage blanket and structural fill to prevent migrating fines from entering the structural fill.

Wick drains were installed in a triangular pattern with a center-to-center spacing of 4 feet to an installation depth of 75 feet. Because of the artesian conditions at the project site, wick drains were limited no deeper than 80 feet below subgrade. Details and limits of wick drain installation are presented in Figures 3 and Figure 5.

A Nilex Mebra-Drain 7407 wick drain was used. This wick drain consists of a corrugated polypropylene core surrounded by a non-woven polypropylene filter fabric, which has an apparent opening size equal to a US #70 sieve, or 0.0083 inch. The drain is 4 inches wide and 0.142 inch thick, which gives an equivalent wick diameter (*dw*) of 0.22 inch based on the following equation (Rollins and Smith, 2012):

$$d_w = 2(b_w + t_w)/\pi \qquad (1)$$

Where;

d_w = the equivalent diameter of the wick

b_w = wick drain width

t_w = wick drain thickness

Wick drains were typically installed by pushing a hollow-steel mandrel, generally rectangular in section, into the ground. The mandrel houses the wick material and protects it from damage as the mandrel was inserted into the ground to the termination depth. At the base of the mandrel, the wick material was looped through an anchor, which holds the drain securely in place as the mandrel was extracted. Once the mandrel has been extracted from the ground, the wick drain was cut and the next drain was installed. The wick drain installation is shown in Figure 4.

Fig. 4. Wick Drain Installation

FIELD INSTRUMENTATION

To measure the anticipated settlement at the project site, a total of 13 vibrating wire piezometers and 7 settlement sensors were installed under the structural fill and surcharge fill loading zone. In addition, a total of 10 settlement monuments and 10 settlement points were installed on existing structures and facilities around the project site, to monitor settlement influence due to the surcharge loads. The settlement monitoring instrumentations were installed as shown on the Instrumentation Plan (Figure 5).

Settlement Sensors

To measure the amount of settlement, a Geokon vibrating wire settlement system (VWS) was installed. The VWS contained two liquid-filled tubes that extend from the sensor at the settlement location to the reservoir at the readout enclosures. The pressure changes at the sensor cause a change in the frequency of the vibrating wire. The difference between any given reading and the initial reading, after accounting for temperature effects, is multiplied by a calibration factor to calculate the settlement at that time (Geokon, 2009).

The settlement sensors were installed on plates at the ground surface following the installation of wick drains (an approximate elevation of 4,498 feet). Fine sand was used to cover the sensors to protect them from construction traffic and activities. The liquid-filled tubes from the sensor were protected within high-density polyethylene (HDPE) conduits and routed to readout boxes. The readout boxes were placed at the construction perimeters to avoid conflict with construction activities at the project site. Because the readout boxes also experienced settlements along with the sensors, the elevations of the instrument readouts were surveyed periodically, and the calculated settlement was adjusted to account for differential settlement between the readout box and settlement sensor. The settlement sensor and readout box installation details are presented in Figure 6.

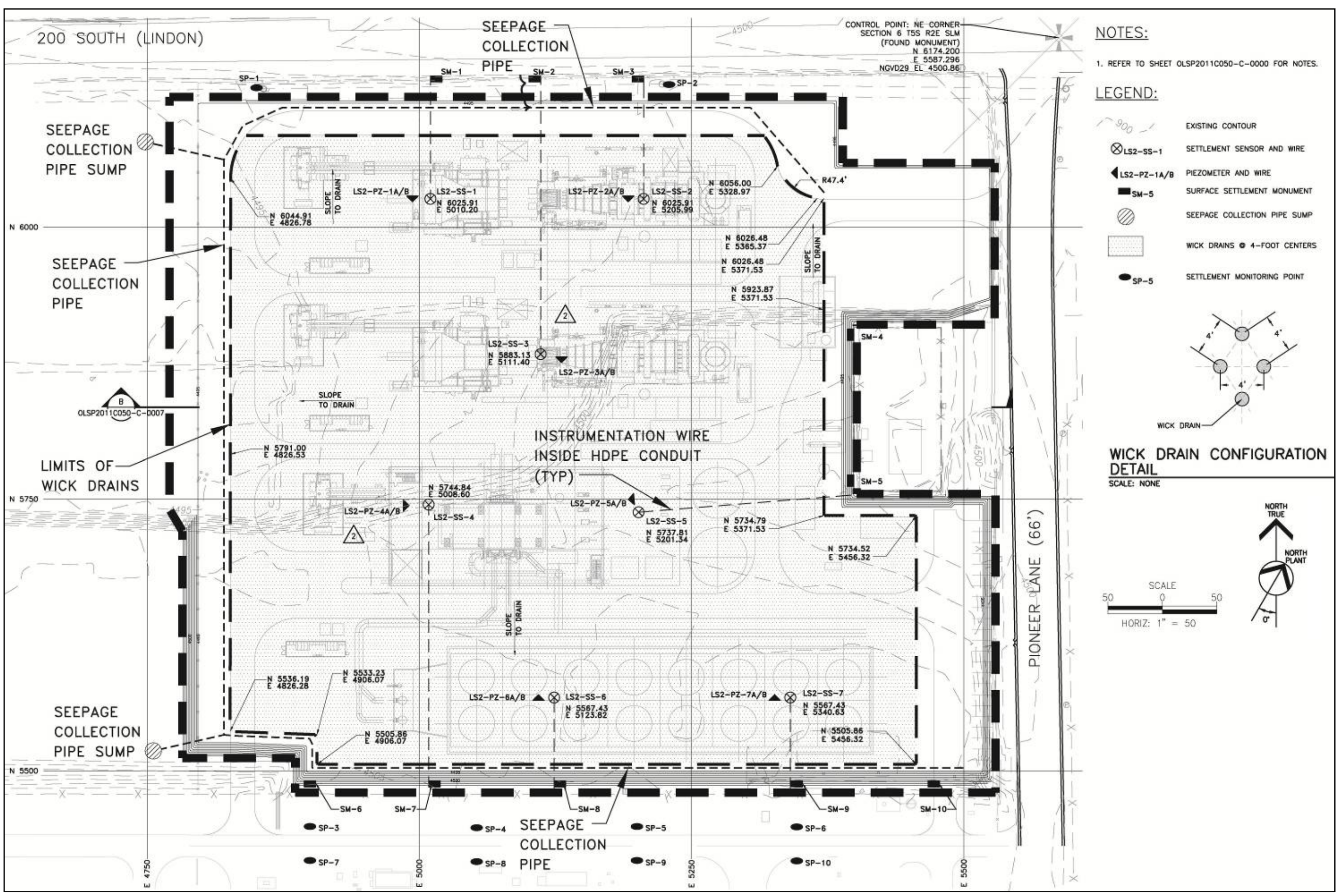

Fig. 5. Ground Improvement and Instrumentation Plan

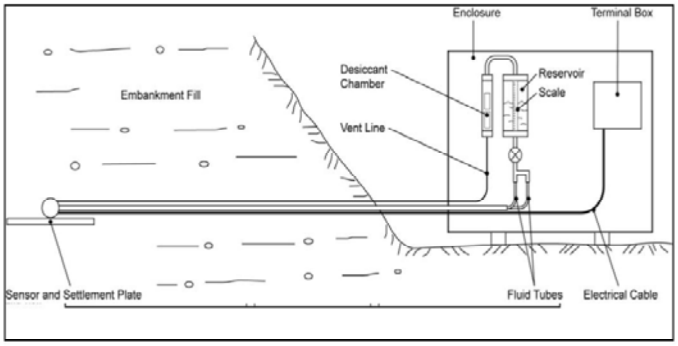

Fig. 6. Typical Settlement Sensor and Readout Box Installation Details (Geokon, 2009)

Piezometers

To measure the pore water pressure and degree of settlement, 13 Geokon Model 4500 vibrating wire piezometers were installed at the same locations of the settlement sensors. The piezometers were intended primarily for long-term measurements of fluid and pore pressures in the subsurface. The instrument uses a sensitive stainless-steel diaphragm to which a vibrating wire element is connected. Changing pressures on the diaphragm causes it to deflect, and this deflection is measured as a change in tension and frequency of vibration of the vibrating wire element. A portable readout unit was used to obtain excitation, signal conditioning and readout of the instrument (Geokon, 2011).

Piezometeres were installed after wick drains had been placed but prior to placing structural or surcharge materials. The piezometers were generally installed at depths of 25 feet and 70 feet bgs at six locations. Only one piezometer was installed at a depth of 40 feet at the location of LS2-SS-1. The piezometer wires were also protected in HDPE conduits and routed to the readout boxes with the settlement sensor tubes.

CONSTRUCTION SCHEDULE AND SETTLEMENT MONITORING

Structural fill was placed on top of the initial 1 foot of structural fill after the completion of the installation of the wick drains to the elevation of 4,510 feet across the site, as indicated in Figure 3. The existing fill, which was lay-down fill from the previous construction, was utilized as a portion of the structural fill. In addition, imported granular fill was placed as the structural fill. The fill was placed and compacted to a minimum of 95 percent of the maximum dry unit weight as determined in accordance with ASTM D1557-09 Test Procedure. The average dry unit weight and moisture content of the compacted structural fill were 129.5 pounds per cubic foot (pcf) and 5.4 percent, respectively.

To accelerate the settlement period, 15- to 22-foot-high temporary surcharge fills were constructed above the finished grade of the structural fill. High-density steel slag material was used as the surcharge fill. The surcharge material has an average unit weight of 150 pcf when compacted to 90 percent relative compaction in accordance with ASTM D1557-09. This surcharge fill, along with the wick drains, expedited consolidation prior to the foundation construction. The surcharge fill was removed after the target settlement was reached. The surcharge fill heights and limits are presented in Figure 7.

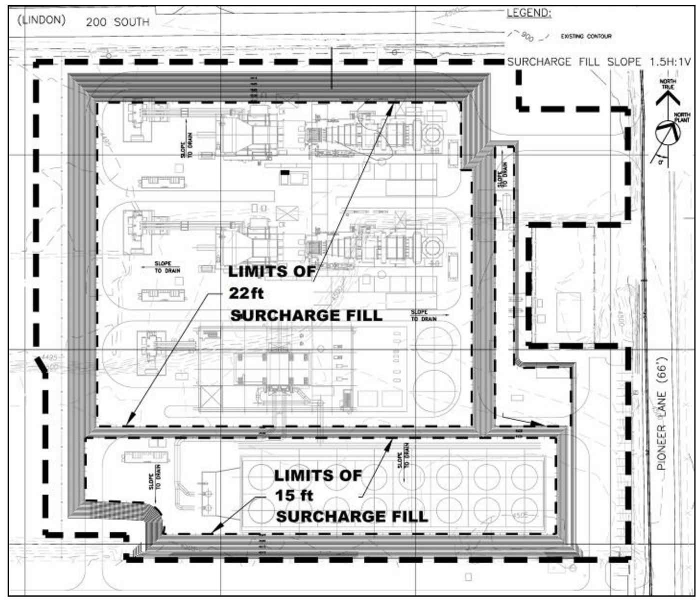

Fig. 7. Surcharge Fill Heights and Limits

The fill height and monitored settlement data are presented with time history in Figure 8. The structural fill was placed first at the northern portion of the site, where the settlement sensors of LS2-SS-1 to 3 are located, and followed by the southern portion, where the settlement sensors of LS2-SS-4 through 7 are located. The surcharge loading periods were approximately 115 to 120 days at the 22-foot surcharge area and approximately 110 days at the 15-foot surcharge area. During the surcharge loading periods, the top of surcharge fill elevations were monitored by periodically surveying three settlement monuments. The changes of the top elevations are shown in Figure 8. After completion of the surcharge periods, the removal of the surcharge fill was completed in 5 to 10 days. Subsequently, foundation excavations were conducted to accommodate mat foundations for the proposed structures. The elevations for those foundation excavations varied from 4,502.5 to 4505.7 feet.

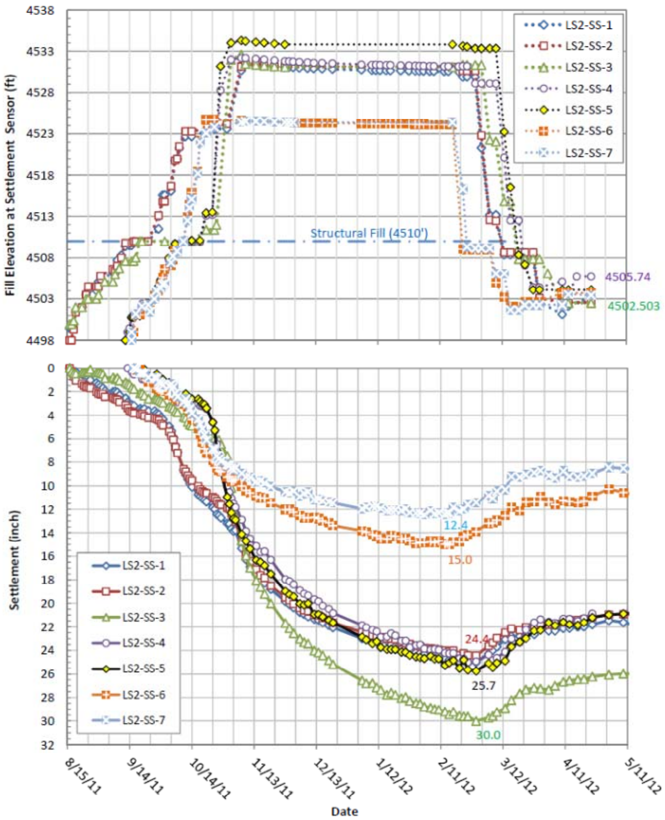

Fig. 8. Fill Placement and Settlement Monitoring Data

The monitoring of the settlement sensors and piezometers was performed from August 15, 2011 to May 11, 2012. Survey readings of the settlement monuments and points were taken by a surveyor from August 26, 2011 to April 24, 2102. The frequency of monitoring was conducted daily at the time of fill placement and reduced to three times in a week readings after completion of the surcharge fill placement. As presented in Figure 8, the maximum settlement of 30 inches was observed at LS2-SS-3, which is located at the middle of the site. The settlements at the perimeter of the 22-foot surcharge area were observed uniformly in ranges of about 24 to 26 inches. Settlements under the 15-foot surcharge area ranged from about 12 to 15 inches. The ground deformation monitoring continued during the surcharge unloading and foundation excavation stages with about 3.5 to 4 inches of rebound being observed.

Based on the monitoring data from the settlement monuments and points, settlements at the perimeter and outside of the project site were monitored. At the northern perimeter of the project site, maximum settlements ranging from 2 to 2.5 inches were observed. At the southern perimeter of the site, where an existing power plant is located, maximum settlements of about 2.5, 1.2, and 0.3 inches were observed approximately at the toe, 40 feet away, and 70 feet away from the fill, respectively. The furthest settlement observation point located approximately 70 feet away from the toe of the fill rebounded to the original elevation after the removal of the surcharge fill. The closest facility is located at least 80 feet away from the toe of the fill. This monitoring data indicated the structures near the surcharge fill experienced negligible settlements.

NUMERICAL ANALYSES

To analyze the behavior of the improved soft soil under the staged embankment loading and unloading, a finite element computer program, PLAXIS Version 2010.01, was used. The analysis allows the simulation of the nonlinear and time-dependent behavior of soils, including the hydrostatic and excessive pore pressure development in the soil.

The numerical analyses of the cohesive layers were performed using the soft-soil model (SSM) in PLAXIS, which is based on the modified Cam-clay (MCC) model (Roscoe and Burland, 1968). In SSM, two main parameters to define deformation are the modified compression index (λ^*) and the modified swelling index (κ^*). These parameters can be obtained from an isotropic compression test. When plotting the logarithm of stress as a function of strain, the slope of the normal consolidation line is used to develop the modified compression index (λ^*), and the slope of the unloading or recompression line can be used to compute the modified swelling index (κ^*). There is a difference between the modified indices λ^* and κ^* and the original Cam-Clay parameters λ and κ. The later parameters are defined in terms of the void ratio (e) instead of the volumetric strain (PLAXIS, 2010). Apart from the isotropic compression test, the parameters can be obtained from the one-dimensional consolidation test. The relationships of SSM parameters and one-dimensional compression (C_c) and recompression (C_r) indexes are written as (PLAXIS, 2010)

$$\lambda^* = \frac{C_c}{2.3\,(1+e)} \tag{2}$$

$$\kappa^* \approx \frac{2\,C_r}{2.3\,(1+e)} \tag{3}$$

The sandy layers, structural fill, and surcharge fill were modeled with the Hardening Soil model (HSM). The HSM is an advanced model for simulating both soft and stiff soils (PLAXIS, 2010). In HSM, compression hardening is used to model irreversible plastic strains due to primary compression in oedometer and isotropic loading. The HSM supersedes the hyperbolic model (Duncan and Chang, 1970) by using the theory of plasticity rather than the theory of elasticity, by including soil dilatancy and by introducing a yield cap (PLAXIS, 2010). The material parameters for the cohesive and granular materials used in the finite element modeling are shown in Table 1.

Vertical drains were modeled in the subgrade to simulate depths and spacing as installed in the field. The distance between two consecutive drains was modeled at 4 feet.

Table 1. Soil Model Parameters used in PLAXIS Analysis

Soil Property	Symbol (units)	Stiff Clay 1	Soft Clay	Sand	Soft/Stiff Clay	Stiff Clay 2
Material Model	Model	Soft Soil	Soft Soil	Hardening Soil	Soft Soil	Soft Soil
Unsaturated unit weight (pcf)	γ_{unsat}	120	115	120	115	120
Saturated unit weight (pcf)	γ_{sat}	125	120	125	120	125
Initial void ratio	e_{init}	0.685	1.06	0.55	1.0	0.68
Secant Stiffness in standard drained triaxial test (ksf)	E_{50}^{ref}	--	--	750	--	--
Tangent stiffness for primary oedometer loading (ksf)	E_{oed}^{ref}	--	--	750	--	--
Unloading /reloading stiffness (ksf)	E_{ur}^{ref}	--	--	2300	--	--
Power for stress –level dependency of stiffness	m	--	--	0.5	--	--
Modified compression index	λ^*	0.0348	0.0870	--	0.0870	0.0348
Modified swelling index	κ^*	0.0174	0.0348	--	0.0261	0.0174
Over-consolidation ratio	OCR	3.5	2	1	1.1	1.2
Cohesion (psf)	c_{ref}'	400	400	0	400	400
Friction angle (degree)	φ'	28	25	33	25	28
Dilatancy angle (degree)	ψ	0	0	3.0	0	0
Permeability (feet/day)	k	5.39×10^{-2}	2.53×10^{-3}	15.6	2.53×10^{-3}	5.39×10^{-2}
Change in permeability	c_k	0.2	1	10^{15}	1	0.2

Comparison of Field Measurements and Computed Results

The FEM analysis simulated all stages of the complex loading and unloading history for the entire fill section with the wick drain ground improvements. The simulation includes (1) removal of the existing fill, (2) loading of structural fill and surcharge fill, and (3) unloading of surcharge fill as the fill history shown in Figure 8. The settlements obtained from the numerical analysis and the recorded settlements are presented and compared in Figure 9 at four different location involving different loading histories. As shown in the settlement-time curves in Figure 9, the numerical analysis generally provided a good simulation and comparison of both the magnitude and rate of settlement with applied fill loads and unloading rebound at all locations.

Near the middle of the fill, the highest settlement of 30 inches was recorded at Sensor LS2-SS-3. However, settlements of approximately 25 inches were recorded at Sensors LS2-SS-4 and 5. This lower settlement was caused by the higher stress history from the previously placed 8-foot-high lay-down fill located over the Sensors LS2-SS-4 and 5 areas. This stress history and corresponding deformations were well simulated in the FEM analysis. However, settlements during the early stage fill placement at Sensors LS2-SS-1, 2, and 3 were overestimated in the numerical analysis compared to the monitored settlements. Artesian pressure impacts or variable OCR ratios could be the explanation of this soil behavior. Further studies are suggested.

The rebounds at the outer areas (LS2-SS-1, 2, 6, and 7) generally showed a good comparison in magnitude and time.

However, the FEM modeled rebounds in the middle part of the fill (LS2-SS3, 4, and 5) were overestimated by approximately double than what was observed.

The representative recorded and computed hydrostatic pore water pressures are shown in Figure 10. Although the excessive pore pressure was somewhat greater in the simulations than the monitored data, the numerical analysis simulated the pore pressure in close magnitude to the monitored data under the embankment loading and unloading conditions.

CONCLUSIONS

This study presents a case history of the ground improvement and the structural fill and surcharge fill constructed on soft lacustrine clay deposits. Based on the field data and FEM analysis results, the following conclusions can be drawn:

- An adequate amount of subsurface explorations, samplings, and CPT soundings with conventional laboratory testing provided relevant engineering soil parameters used in the one-dimensional consolidation and numerical analyses to estimate and simulate reasonable predictions of the ground deformation behavior.

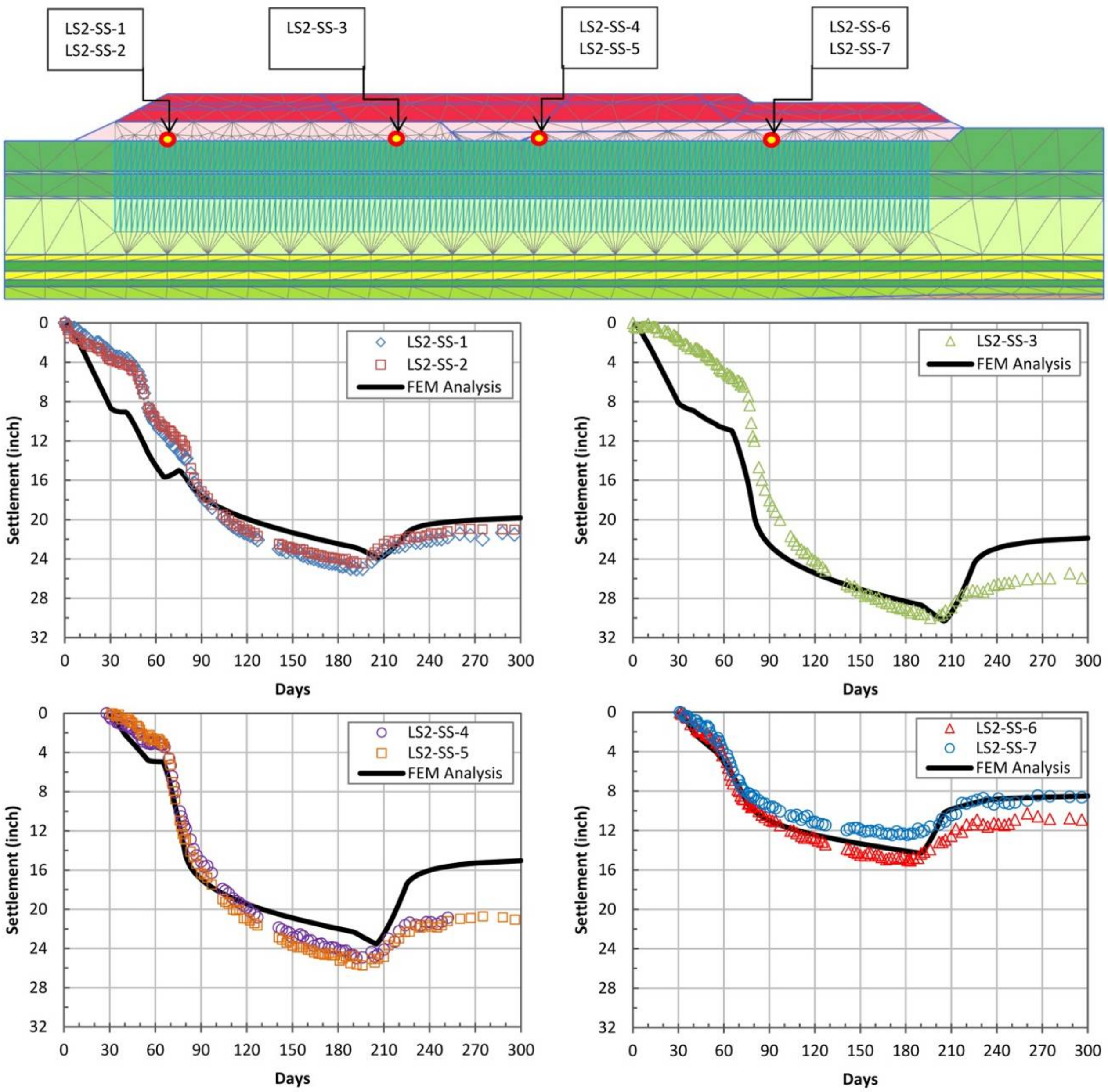

Fig. 9. Finite Element Modeling and Results comparing to Field Monitored Data

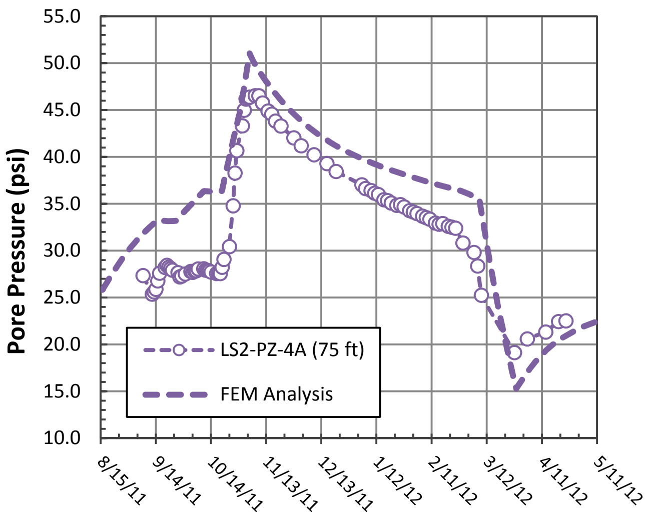

Fig. 10. Monitored and Simulated Pore Water Pressures

- In this project, relatively high fill was rapidly placed using wick drains, which dissipated excessive pore water pressure in a relatively short time period. The wick drain ground improvement reduced the preloading period and advanced the construction duration.

- Field instrumentation and monitoring of displacements and pore pressure build up in the foundation layers provided useful information, such as degree of consolidation, during construction and surcharging periods.

- The settlement points and monuments, which were installed around the project site, provided effective indications of any excessive settlements beyond the fill area and early warning to protect the existing structures and facilities adjacent to the fill.

- The numerical analysis could effectively simulate soil stress and strain behavior with complex staged construction. In the numerical analysis, the SSM presented settlement and pore water pressure predictions that are comparable to the observed field monitoring data. However, it was not equally successful in predicting the rebound conditions.

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