NUMERICAL MODELING OF SETTLEMENT BEHAVIOR OF TREATED AND UNTREATED FOUNDATION SOILS UNDERLYING MSE WALLS FOR THE I-15 RECONSTRUCTION PROJECT, SALT LAKE CITY, UTAH

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Utah Department of Transportation
Research and Innovation Division

Submitted by:
Department of Civil and Environmental Engineering, University of Utah

Authored by:
Michelle D. Cline
Steven F. Bartlett
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7. Author
Michelle D. Cline, Steven F. Bartlett


9. Performing Organization Name and Address
Department of Civil and Environmental Engineering
University of Utah
122 South Central Campus Drive, Suite 104
Salt Lake City, Utah 84112-0561

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16. Abstract
As urban environments become more populated, it is becoming more common to construct larger highway embankments in close proximity to existing facilities. For such embankments constructed on soft, compressible soils, the design engineers must consider the settlement impacts to adjacent structures. During the recent I-15 Reconstruction Project in Salt Lake City, Utah, several large mechanically stabilized earth (MSE) walls were constructed over compressible soils. In order to increase the stiffness and strength of very soft foundation materials in some locations along the corridor, these soils were treated with prefabricated vertical (PV) drains, and in one location, foundation soils were treated with lime cement columns (LCCs). Various types of instrumentation were installed and monitored during and following construction to collect settlement data. These performance data have been used along with a suite of numerical modeling analyses as part of this research to evaluate the effectiveness of numerical modeling as a tool to predict primary consolidation settlement. Two sites along the I-15 Reconstruction Project alignment were evaluated. Performance data collected from the 200 South Site, treated with PV drains, were used to back-calculate soil parameters for numerical analysis of the I-80 Site, treated with LCCs. Linear elastic (LE) and hyperbolic non-linear elastic (HNLE) models were considered. As expected, the HNLE model is a better predictor of primary consolidation settlement than the LE model; however, this approach may only be warranted for critical facilities because a significant amount of effort is required to produce reasonably accurate results. In addition to conducting numerical analyses, this study established guidelines for settlement versus embankment height based on the performance data. As a conservative approximation, embankments should be constructed a distance of at least 1.5 times the maximum embankment height to limit settlements to 25 mm (1 in.) or less.

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lime cement column, numerical modeling, hyperbolic non-linear elastic constitutive model, MSE Wall, settlement, Young’s modulus, Sigma/W, instrumentation

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1.0 INTRODUCTION

1.1 Background

The I-15 Reconstruction Project, completed in 2001 in Salt Lake City, Utah, incorporated several innovative embankment and foundation treatments as part of a fast-paced, 4-year, $1.5 billion design-build project. In many cases, these features were required to facilitate construction over compressible clayey foundation soils present over much of the alignment. Prefabricated vertical (PV) drains were used in conjunction with preloading for a large part of the alignment to accelerate primary consolidation settlement and reduce secondary consolidation settlement, respectively. Expanded polystrene (EPS) geofoam was used as an extremely lightweight fill material in some utility corridors, so as not to trigger large and damaging settlement to underground lines. Also, at one location, lime cement columns (LCCs) were employed to improve the foundation soils of a mechanically stabilized earth (MSE) wall. At this location, the 12-m (39-ft) high MSE wall had the potential to induce unacceptable settlements in a nearby commercial building.

In conjunction with these design features, various types of instrumentation were installed at select sites to collect field performance data during and after construction. Instrumentation included horizontal and vertical inclinometers, magnet extensometers, strain gages, open and closed-ended piezometers, total pressure cells and survey settlement points. These instrument arrays were installed by the design-build contractor to monitor construction performance and by the UDOT Research Division to monitor construction and post-construction performance. The data from these instrument arrays have been reported, in part, in various publications and technical reports.

1.2 Purpose

The primary purpose of this research is to evaluate the effectiveness of numerical modeling in predicting primary consolidation settlement of the foundation soils that underlie two MSE walls. One of the MSE walls, located at approximately 200 South 800 West, supports an embankment that connects southbound I-15 to westbound I-80. The other site is located at approximately 2400 South 300 West, and consists of an MSE wall-supported embankment constructed over LCC-treated ground in the I-80/I-15/Highway 201 interchange area.

The usefulness of the numerical modeling is evaluated based on a comparison of the modeled results with the measured settlement data at the above sites. Specific attention is given to predicting primary consolidation settlements that developed beneath buildings located in close proximity to large MSE walls. Also, another objective of this research is to comment on the
usefulness of numerical modeling as a design/analysis tool for the practicing engineer. It is hoped that the findings can be used to minimize construction-related damages to important existing facilities that are founded on compressible foundation soils near new, large embankments.
2.0 SOFTWARE

Sigma/W™ is a finite element software distributed by Geo-Slope, International Ltd. that was used for the numerical analyses in this project. This graphical software interface permits computation of stresses and deformations for two-dimensional (2D) plane-strain and axis-symmetrical problems. Originally, a three-dimensional (3D) finite difference code, FLAC3D™, was considered for use, but the idea was abandoned because of the complexity, computational runtime and additional overhead required to run a 3D model. FLAC2D™ was also considered as an alternative, but was not used because at the time of the analyses it did not have the constitutive relation (i.e., hyperbolic model) that was selected to analyze the settlement behavior. Sigma/W was ultimately selected because of its ease of use and because the University of Utah Civil and Environmental Engineering Department already had a license agreement. Furthermore, Sigma/W already had the hyperbolic model as a standard, built-in feature.
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3.0 MODEL SETUP

3.1 Introduction

Sigma/W has many built-in features and capabilities; those pertaining to this study are discussed in general detail herein. The modeling procedures followed during this investigation are described briefly. However, it should be noted that there are many other ways a problem can be defined and solved in Sigma/W.

Analyses in Sigma/W are accomplished with three programs – DEFINE, to define the model; SOLVE, to compute results; and CONTOUR, to view and plot the results. For all analyses, consistent units must be used.

3.2 Define Program

The first step in a Sigma/W deformation analysis is to define the model. The initial step in defining the model uses the Set menu in DEFINE to set the page, scale, grid, and axes. As with most modeling software, it is recommended to prepare a scaled sketch of the problem before attempting to draw it in Sigma/W. Entering the problem into the software involves the use of the KeyIn, the Draw, and the Tools menus.

3.2.1 KeyIn Menu

The KeyIn menu is used to define the analysis parameters, material properties, body load, and initial water table conditions. The Analysis Settings command has several sub-commands. The analysis type is set with the Type sub-command. For this modeling study, the analysis type was set as “insitu 1” when establishing the initial stress conditions, and as “load/deformation” when modeling application of fills. In most cases, an initial conditions file and corresponding time step must be identified when a load/deformation analysis is defined. The finite element mesh has to be identical in both the initial conditions file and the load/deformation file, and the two files must reside in the same electronic directory.

The Control sub-command establishes the problem view as either axisymmetric or 2D; 2D was selected for this analysis. The Convergence sub-command is used to indicate the maximum number of iterations that SOLVE will execute in an effort to reach a solution. The Sigma/W User’s Guide (Geo-Slope, 2002) suggests that a maximum of 25 iterations are sufficient for most problems. Non-linear analyses for this study were conducted with the specified maximum number of iterations ranging from 20 to 35. The Convergence sub-command is also used to define convergence tolerances. Geo-Slope (2002) indicates that a convergence criterion of 1% to 2% is usually adequate. Convergence tolerances of 1% were used in the evaluations. The Time
The constitutive model, parameter type, and color are assigned for each soil type using the Material Properties command in the KeyIn menu. The User’s Guide gives some suggestions regarding modeling progression. It is recommended that a simple linear elastic analysis be conducted prior to a non-linear analysis to assess whether the problem has been defined satisfactorily. The linear elastic solution can be used as a benchmark for comparing the results of a more complex analysis. If reasonable results are not obtained with a linear elastic model, then it is not likely that reasonable results will be obtained with a non-linear model.

For this investigation, the linear elastic constitutive model was used for the embankment fills and half-space. When embankment fills are included in the mesh but are not to be included in stress computations during a particular run, they are modeled as “null” materials and the constitutive model is specified as “none.” Soil layers above the half-space were modeled with the linear elastic (LE) or hyperbolic non-linear elastic (HNLE) constitutive model, depending on the type of analysis being performed. Model input parameters are discussed in further detail in later sections of this report. The parameter type was designated as “effective,” and each material was assigned a color to represent it in the finite element mesh.

The Body Load command is used in initial conditions analysis to define the unit weight and $K_o$ (ratio of horizontal stress to vertical stress) of all material types. In load/deformation analysis, the unit weight and $K_o$ of only the applied fill is defined, to model stresses due to the applied loading. The final command in the KeyIn menu is Initial Water Table, which is used to input the x- and y-coordinates of the static water table.

3.2.2 Draw Menu

After supplying the information required in the KeyIn menu, the Draw menu is employed to generate elements and establish boundary conditions. Geo-Slope (2002) recommends using the Sketch command in Sigma/W to draw the primary problem components before generating elements. Elements are drawn individually or within a quadrilateral or triangular region mesh using the Elements command. The User’s Guide provides some general guidelines pertaining to mesh size, element shape, and element compatibility. Element properties, including material type, integration order, and thickness, are required.

Triangular and quadrilateral elements and mesh patterns were used for mesh construction in this study. The mesh incorporated higher order elements with secondary nodes and a default element thickness of 1. Although greater processing time is required for problems with higher order
elements, they should be used with non-linear soil models because they exhibit better behavior than ordinary elements.

Elements along the boundaries of a mesh can be defined as infinite with the Infinite Elements command; but the direction of infinity must be specified. Infinite elements should not be used in initial conditions analysis. Finite element meshes for load/deformation analyses of the 200 South and I-80 Sites had infinite elements in the positive x-infinity direction on the right boundary and negative x-infinity direction on the left boundary.

The Node Boundary Conditions command in the Draw menu is used for fixing displacements, forces, or springs at element nodes along the x- and y-boundaries of the mesh. Boundary conditions for the 200 South and I-80 models included fixed displacements in the x-direction along the left and right sides of the mesh. In addition, displacements were fixed in the x- and y-directions along the bottom boundary.

Modeling of embankment fills is performed with the Fill/Excavation Elements command, which adds or removes fill/excavation elements at the specified time steps. An element type of “permanent,” “fill,” or “excavation” must be selected. Permanent elements exist throughout an analysis, and a time step of 0 is automatically assigned. Embankment fills in this research were modeled with fill/excavation elements, as described later in this report.

3.2.3 Tools Menu

Typically, the last step in problem definition is verification and sorting of the mesh. The Verify/Sort command in the Tools menu is utilized to check the node and element data, and to sort node and element numbers horizontally or vertically. It is important to sort in the direction of smallest problem dimension to keep processing times and round-off errors to a minimum. Given that the models were of greater dimension in the horizontal direction, they were sorted vertically.

3.3 Solve Program

After the problem is defined and verified, results are computed using the SOLVE program. Convergence data for each time step and iteration are displayed in the SOLVE window while computations are performed. The convergence data may be viewed graphically as well. The graph of the unbalanced load norm is particularly useful in evaluating convergence. The plot should trend toward 0 without erratic undulations. Geo-Slope (2000) furnishes examples of graphs with acceptable and unacceptable variations of the unbalanced load norm parameter.
3.4 Contour Program

After a solution is obtained, results can be viewed and plotted using CONTOUR. CONTOUR has many capabilities and can create contour plots, displacement plots, Mohr circles, and various graphs of numerous parameters. Contour plots of \(y\)-effective stress and graphs of \(y\)-settlement versus distance were heavily relied upon during this study to evaluate analysis results.
4.0 CONSTITUTIVE MODEL

4.1 Introduction

Primary consolidation of a saturated clayey soil is a non-linear, time-dependent process, which progresses as excess pore pressures dissipate from the consolidating layer and the void ratio changes in a non-linear fashion in response to changes in the effective stress. To model the time dependent nature of the consolidation process, a visco-elastic model is often used. However, a major limitation of the visco-elastic model, in its current state, is that this theory is only applicable to linear materials (Holtz and Kovacs, 1981). In reality, soils have a highly non-linear behavior during the consolidation process; thus, visco-elastic theory, in its present state, is not very useful.

4.2 Hyperbolic Non-linear Elastic Model

If it is not necessary to calculate the time-rate of settlement, then non-linear models can be used to estimate the amount of primary settlement. Many non-linear models do not require the coupling of volumetric strain with pore pressure dissipation, and in this sense, they are easier to apply to settlement problems than visco-elastic models.

The hyperbolic non-linear elastic (HNLE) constitutive model developed by Duncan and Chang (1970) and modified by Duncan et al. (1980) was used to represent the stress-strain material behavior of the soft soils underlying the 200 South and I-80 Sites. Originally, the modified Cam Clay (MCC) constitutive model was considered for use. This model, which is based on principles of critical state soil mechanics, represents more aspects of the stress-strain behavior of clay than the HNLE approach. However, after some evaluation, it was recognized that the values for the MCC input parameters could not be reliably estimated based on the limited field and laboratory data. The HNLE model requires fewer parameters and because Sigma/W contains this model as a built-in feature, the HNLE model was deemed easier to use and more accessible to practicing engineers.

The HNLE model represents the essential stress-strain behavior of soil – namely, non-linearity, inelasticity and stress-dependency – with a relatively small number of parameters. The model uses a hyperbolic stress-strain curve in deviator stress versus axial strain space. Original formulation of the hyperbolic relationships is explained in Duncan and Chang (1970), and later modifications are described in Duncan et al. (1980).

The incremental procedure described in Duncan et al. (1980) was established to allow non-linear analysis of soil deformations in conjunction with the finite element method. For each analysis increment, the stress-strain relationship is evaluated on the basis of Hooke’s law.
describes the relationship of stresses and strains in soil under linear, elastic, and isotropic conditions (Budhu, 2000). For plane-strain conditions, the principle is described mathematically as follows:

\[
\begin{bmatrix}
\Delta \sigma_x \\
\Delta \sigma_y \\
\Delta \tau_{xy}
\end{bmatrix} = \frac{3B}{9B-E} \begin{bmatrix}
(3B + E) & (3B - E) & 0 \\
(3B - E) & (3B + E) & 0 \\
0 & 0 & E
\end{bmatrix} \begin{bmatrix}
\Delta \varepsilon_x \\
\Delta \varepsilon_y \\
\Delta \gamma_{xy}
\end{bmatrix}
\]  

(1)

where \(\Delta \sigma_x, \Delta \sigma_y\) = normal stress increments; \(\Delta \tau_{xy}\) = shear stress increment; \(B\) = bulk modulus; \(E\) = Young’s modulus; \(\Delta \varepsilon_x, \Delta \varepsilon_y\) = normal strain increments; and \(\Delta \gamma_{xy}\) = shear strain increment.

### 4.3 Limitations of HNLE model

There are several significant limitations of the HNLE model. The HNLE relationships may not realistically predict soil behavior at or beyond the failure condition. The relationships are primarily applicable for stable earth masses. The HNLE model does not account for volume changes due to shear stress. Finally, the HNLE parameters represent soil behavior under a limited range of conditions; thus, they are not fundamental soil properties. When insufficient data are available for determination of the parameter values, they may be estimated using the parameter data compiled in the Duncan et al. (1980) report. Otherwise, the parameters must be found by laboratory testing under conditions that correspond to anticipated field conditions.

### 4.4 Initial Tangent Modulus

As illustrated in Fig. 1, the HNLE method, developed by Duncan and Chang (1970), employs three values of moduli to represent the inelastic stress-strain behavior of soil. The initial tangent modulus, \(E_i\), represents the initial slope of the stress-strain curve. The value of \(E_i\) is dependent on confining pressure, \(\sigma_3\), as indicated by the empirical equation

\[
E_i = K p_a \left( \frac{\sigma_3}{p_a} \right)^n
\]  

(2)

where \(K\) = modulus number; \(p_a\) = atmospheric pressure; and \(n\) = modulus exponent. In Eq. (2), \(K\) and \(n\) are unitless, and \(E_i, p_a,\) and \(\sigma_3\) are in consistent units. Duncan et al. (1980) report conservative values of \(n\) ranging from 0.25 to 0.6 for various types of soil.
4.5 Tangent Modulus

The tangent modulus, $E_t$, corresponds to the slope of the curve beyond the initial portion defined by $E_i$. The tangent modulus is computed as

$$E_t = \left[ 1 - \frac{R_f(1-\sin\phi)(\sigma_1 - \sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi} \right] E_i$$

where $R_f =$ failure ratio; $\phi =$ soil friction angle; $(\sigma_1 - \sigma_3) =$ deviator stress; and $c =$ soil cohesion intercept. The failure ratio symbolizes the ratio between the asymptote to the hyperbolic curve and the maximum shear strength, and typically ranges from 0.5 to 0.9 for most soils. Eq. (3) indicates that $E_t$ is dependent on confining pressure and the percentage of shear strength mobilized.

4.6 Unload-Reload Modulus

When the soil is unloaded from a higher stress state and then reloaded, the stress-strain relationship is described by the unload-reload modulus $E_{ur}$, calculated as
where $K_{ur}$ = unload-reload modulus number. Eq. (4) accounts for the fact that strains occurring during primary loading are only partially recoverable on reloading, a common characteristic of inelastic soil behavior. For primary loading, $K_{ur}$ is always greater than $K$. Often, unloading data are not available and thus $K_{ur}$ is assumed. For stiff soils, such as dense sands, $K_{ur}$ is approximately equal to $1.2K$. For softer soils including loose sands, $K_{ur}$ is approximately equal to about $3K$.

### 4.7 Bulk Modulus for HNLE model

The non-linear volumetric strain of the HNLE model is accounted for by using a $B$ parameter that is dependent upon confining pressure as

$$B = K_b p_a \left( \frac{\sigma}{p_a} \right)^m$$

(5)

where $K_b$ = bulk modulus number; and $m$ = bulk modulus exponent. For most soils, $m$ is in the range of 0.0 to 1.0.

### 4.8 Bulk Modulus for LE Model

For LE analyses, $B$ values do not vary as a function of confining pressure, but are constant for a given material type. $B$ values are related to Poisson’s ratio, $\nu$, by the theory of elasticity as follows:

$$B = \frac{E}{3(1-2\nu)}$$

(6)

Values of $\nu$ are restricted to lower and upper limits of 0.1 and 0.5, respectively, for most finite element analyses. A value of 0.5 indicates that there is no volume change (i.e., no volumetric strain) and is used to model “undrained” soil conditions in most LE analyses. (Note that a $\nu$ of 0.49 must be used in Sigma/W to model “undrained” conditions. Sigma/W will not reach a solution if 0.5 is used.) Values of $\nu$ less than 0.5 allow for volume change during strain and are called “drained” values.
4.9 Phi Angle

For a large range of confining pressures, it is appropriate to use a value of $\phi$ that varies with confining pressure in accordance with the following equation:

$$
\phi = \phi_o - \Delta \phi \log_{10} \left( \frac{\sigma_3}{p_a} \right)
$$

(7)

where $\phi_o$ = value of $\phi$ when $\sigma_3$ is equal to $p_a$; and $\Delta \phi$ = change in $\phi$ for a 10-fold increase in $\sigma_3$.

4.10 Estimation of HNLE Model Parameters from Laboratory Tests

In summary, nine parameters – $K$, $K_w$, $K_b$, $n$, $m$, $c$, $\phi_o$, $\Delta \phi$, and $R_f$ – are used in the HNLE stress-strain relationships. These parameters can be readily obtained from unconsolidated undrained (UU) triaxial compression tests to represent total stress conditions, or consolidated drained (CD) triaxial tests to represent effective stress conditions. Techniques for estimating the parameter values based on laboratory test results are summarized in Duncan et al. (1980). In addition, parameter values obtained from triaxial testing of approximately 150 soils are presented.
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5.0 OVERVIEW OF ANALYSIS APPROACH

Following the selection of a modeling software package and the soil constitutive model, numerical analyses were conducted in two phases. The first phase focused on the MSE wall and embankment at 200 South Street (“200 South Site”). The intent was to use measured settlement data available for this site to calibrate the numerical model parameters for the subsurface soils. The second phase concentrated on the LCC-treated MSE wall and embankment at I-80 and 300 West Street (“I-80 Site”). The calibrated soil properties for the 200 South Site were modified to include consideration for varying soil conditions, embankment geometry, and inclusion of LCCs. An analysis was then conducted to determine if the numerical model could reasonably predict settlements at the I-80 Site based on available performance data.

For each site, the analysis approach included review of available test data, development of a generalized soil profile, construction of a model mesh, numerical modeling of the site, and comparison of results to the measured settlement data.

The two MSE wall sites modeled in this study are more completely described in Bartlett and Farnsworth (2004), a report that outlines the site locations, configurations, and installation of various monitoring devices. Details of the sites and the instrumentation installed will therefore not be repeated in this report.

Unless noted otherwise, drained HNLE parameters were used for all numerical analyses. Settlements were computed based on the simplified assumption that excess pore pressures had fully dissipated at the end of primary consolidation; thus pore-water pressure effects were not considered. Using drained parameters in the HNLE constitutive model permitted modeling of effective stress conditions.
6.0 ANALYSIS OF THE 200 SOUTH SITE

6.1 Site Description

As part of the I-15 Reconstruction Project, an 8-m (26-ft) high embankment was constructed for the I-15/I-80 alignment at approximately 200 South and 800 West Streets. At this location, new embankment fills were placed atop and outside the footprint of the existing 6-m (20-ft) high embankment that was constructed during the original I-15 project in the mid-1960s. An older, wood-frame house on shallow foundations is located approximately 11 m (36 ft) northwest of the new MSE wall that is retaining the new embankment. Based on observations of the house in December 2003, the structure has experienced settlement-related damage. However, it is not known when the damage occurred relative to construction activities. A photograph of the site, taken about three years after construction, is presented in Fig. 2. The damaged house is shown on the far right side of the figure.

6.2 Construction Sequencing

Reconstruction of the interchange was completed in two phases (Fig. 3). Phase 1 consisted of construction of the southeastern half of the alignment. The embankment was constructed to a maximum design height of 8 m (26 ft), and additional surcharge fill (about 4 m, or 13 ft) was placed over the embankment fill to pre-compress the foundation soils and minimize secondary settlement. The original embankment was about 6 m (20 ft) high and narrower than the new embankment. Also, PV drains were used to accelerate primary consolidation settlements. After about 98% completion of primary settlement, the surcharge fill was removed and pavement construction commenced.

Phase 2 consisted of a similar construction of the northwestern half of the alignment, where a two-stage MSE wall was erected to retain the widened embankment. The maximum fill height was approximately 12 m (39 ft), which included about 4 m (13 ft) of surcharge fill supported with a temporary welded wire wall. Upon removal of the surcharge fill and temporary wall, the finished concrete panels were attached to the permanent welded-wire face of the MSE wall and pavement construction was completed.

6.3 Subsurface Soil Profile

Foundation soils below the embankment at the 200 South Site consist of about 5 to 8 m (16 to 26 ft) of interbedded, alluvial clays, silts and sands. This upper alluvium was deposited during the Holocene epoch by stream channels extending from the canyons of the nearby Wasatch Mountains and from the floodplain of the Jordan River. The upper alluvium is underlain by a 10-m (33-ft) to 13-m (43-ft) sequence of soft, compressible lacustrine soils commonly referred to
**Fig. 2.** Embankment and MSE wall after construction at 200 South Site (view towards the south)

**Fig. 3.** Typical embankment profile at 200 South Site (view towards the northeast)
as Lake Bonneville deposits. This Pleistocene sequence consists of interbedded clayey silt and silty clay, with thin beds of silts and fine sand near the middle of the unit. Beneath the Lake Bonneville deposits are about 3 m (10 ft) of interbedded Pleistocene alluvial and lacustrine sediments consisting of sands, silts, and clays. These interbedded deposits are in turn underlain by the Cutler Dam Lake sequence. These Pleistocene lacustrine deposits are about 4-m (13-ft) to 9-m (30-ft) thick and consist of clay with occasional seams and layers of silt and sand. Dense, alluvial sands and gravels underlie the Cutler Dam Lake sediments. Typically, groundwater is encountered within a depth of about 3 m (10 ft) below the ground surface in this area. For purposes of this study, groundwater was assumed to be located at a depth of 2 m (7 ft).

During the design phase of the I-15 Reconstruction Project, data were compiled within a design segment that extended from approximately 600 North Street to 1300 South Street. (This section of the I-15 alignment is referred to as the “600 South Design Segment” by project personnel.) The idealized 200 South Site subsurface profile developed herein was generalized using available laboratory and field test data for the 600 South Design Segment.

Layer depths for the generalized 200 South Site soil profile are summarized in Table 1. The idealized profile consisted of seven layers: Upper Alluvium, Upper Bonneville, Interbedded Bonneville, Deeper Alluvium, Cutler Dam, and Half-Space. The thickness of the soil layers above the half-space was established based on review of available Cone Penetration Test (CPT) data. The thickness of the half-space was assumed to be equal to 40 m (130 ft), which extended the subsurface profile to a depth of 70 m (230 ft), or approximately one width of the Phase 2 embankment. (Based on results of preliminary numerical analyses, realistic settlement estimates can be obtained in a reasonable amount of time when the subsurface profile is extended to a depth of at least one times the new embankment width.)

Estimated general soil properties in Table 1 include unit weight $\gamma$, drained friction angle $\phi'$, and drained Poisson’s ratio $\nu'$. Unit weights were computed as averages of laboratory test values for the 600 South Design Segment as presented in Gerber (1995). Values of $\phi'$ were estimated based on CPT correlations and past experience with the site soils. Values of $\nu'$ were assumed based on typical values published in Budhu (2000). Also, values of $K_o$, defined as the ratio of horizontal to vertical stresses, were estimated for purposes of evaluating in situ stress conditions. Values of $K_o$ were calculated using the following equation, which accounts for overconsolidation effects (Meyerhoff, 1976):

$$K_o = 1 - \sin \phi' (OCR)^{0.5}$$  \hspace{1cm} (8)
### Table 1. 200 South Site Layer Depths and Generalized Soil Properties

<table>
<thead>
<tr>
<th>Description</th>
<th>Top Depth (m)</th>
<th>Bottom Depth (m)</th>
<th>Thickness (m)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$\phi'$ (deg)</th>
<th>$\nu'$</th>
<th>OCR</th>
<th>$K_0$ (kPa)</th>
<th>$E'_{\max}$ (kPa)</th>
<th>$E'_{\max}$ b Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>0</td>
<td>5</td>
<td>5</td>
<td>19.2</td>
<td>37</td>
<td>0.3</td>
<td>3.5</td>
<td>0.74</td>
<td>108,400</td>
<td>1.2</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>5</td>
<td>12</td>
<td>7</td>
<td>18.2</td>
<td>31</td>
<td>0.3</td>
<td>2.1</td>
<td>0.70</td>
<td>90,600</td>
<td>1.0</td>
</tr>
<tr>
<td>Interbeds</td>
<td>12</td>
<td>16</td>
<td>4</td>
<td>18.8</td>
<td>31</td>
<td>0.3</td>
<td>1.7</td>
<td>0.64</td>
<td>187,600</td>
<td>2.1</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>16</td>
<td>22</td>
<td>6</td>
<td>18.2</td>
<td>31</td>
<td>0.3</td>
<td>1.6</td>
<td>0.62</td>
<td>188,300</td>
<td>2.1</td>
</tr>
<tr>
<td>Deeper Alluvium</td>
<td>22</td>
<td>25</td>
<td>3</td>
<td>19.5</td>
<td>40</td>
<td>0.35</td>
<td>1.4</td>
<td>0.43</td>
<td>393,200</td>
<td>4.3</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>25</td>
<td>30</td>
<td>5</td>
<td>18.4</td>
<td>37</td>
<td>0.25</td>
<td>1.3</td>
<td>0.46</td>
<td>294,400</td>
<td>3.3</td>
</tr>
<tr>
<td>Half-Space</td>
<td>30</td>
<td>70</td>
<td>40</td>
<td>21.2</td>
<td>-- a</td>
<td>0.35</td>
<td>-- a</td>
<td>-- a</td>
<td>455,400</td>
<td>5.0</td>
</tr>
</tbody>
</table>

1 m = 3.28 ft  
1 kN/m$^3$ = 6.366 pcf  
1 kPa = 0.145 psi

* a Not required for analyses.

* b This column was calculated by dividing the $E'_{\max}$ value for each layer by the $E'_{\max}$ value for the Upper Bonneville clay (i.e., the layer with lowest $E'_{\max}$).

where $OCR = \text{overconsolidation ratio}$. Estimated $OCR$ values listed in Table 1 were determined using effective overburden stresses calculated by the equilibrium method, and assumed preconsolidation stresses based on available consolidation data for the site (Woodward-Clyde, 1997).

Values of the low-strain, Young’s modulus $E'_{\max}$, can be estimated from elastic theory, if geophysical measurements of the shear wave velocity $v_s$ data are available. The low-strain, Young’s modulus is related to the low-strain shear modulus $G_{\max}$ by

$$E'_{\max} = 2G_{\max} (1 + \nu')$$

(9)

The relationship between $G_{\max}$ and $v_s$ is described mathematically as
\[ G_{\text{max}} = \frac{\gamma (v_s)^2}{g} \]  

(10)

where \( g \) is acceleration due to gravity. Units must be consistent in Eq. (9) and Eq. (10). The \( v_s \) profile presented in Gerber (1995) for the 600 South Design Segment was used in developing a \( E'_{\text{max}} \) profile for the 200 South Site. Based on evaluation of the \( E'_{\text{max}} \) profile, weighted averages were estimated for the subsurface layers. The values are exhibited in Table 1. The ratios of \( E'_{\text{max}} \) were used for back-calculating elastic and initial tangent moduli for other numerical analyses, as discussed in later sections.

It should be noted that \( E'_{\text{max}} \) and \( G_{\text{max}} \) calculated from geophysical tests (\( v_s \)) are valid for very low levels of strain (approximately \( 1 \times 10^{-4} \) percent). This strain level is very much lower than that used in the laboratory (approximately \( 1 \times 10^{-2} \) to \( 1 \times 10^{-1} \) percent) or in the field (about 1 percent). Young’s modulus, \( E \), is truly non-linear (higher at low levels of strain) and can vary substantially, depending on the level of strain over which it was measured.

### 6.4 Measured Settlement Data

The UDOT Research Division installed six survey points at distances ranging from approximately 4.4 to 33.5 m (14 to 110 ft) northwest of the MSE wall to monitor settlements during the construction and post-construction period. In addition, a horizontal inclinometer was installed near the base of the wall to measure settlements at the base of the wall. The inclinometer extended a distance of about 12 m (39 ft) into the base of the MSE wall. These points and instruments were surveyed on a periodic basis during construction to monitor the progression of primary settlement.

Review of the measured settlement data indicates that the majority of primary consolidation settlement occurred during spring and summer months of 2000, when embankment construction was completed. Survey and inclinometer data from July 2000 were used to plot the surface settlement profile of Fig. 4. At this time, primary consolidation settlement was essentially finished and the surcharge had been removed and panel placement for the MSE wall had begun. (There are no inclinometer settlement measurements made after July 2000, because the wall panel covered the access to the inclinometer.)

In Fig. 4 and subsequent figures in this chapter, the left side corresponds to the northwest direction. It is noted that the inclinometer data was originally interpreted with a downward-trending settlement-distance curve extending from the wall face into the embankment. This curve indicated that more settlement occurred near the center of the embankment than at the wall at the base of the wall.
Based on conversations with UDOT, it was determined that the data had inadvertently been inverted during inclinometer data reduction and that the settlements should be plotted as an upward-trending curve, as shown in Fig. 4.

The maximum settlement was recorded near the MSE wall face, where the full 12-m (39-ft) height of Phase 2 fill was placed on the northwest side of the alignment. Fig. 4 shows a measured primary settlement of approximately 1.1 m (3.6 ft) at the wall face. Settlements on the order of 0.06 m (0.2 ft) were measured below the southeast wall of the adjacent house, with decreasing settlements trending toward the northwest side.

6.5 Rules of Thumb from Settlement Measurements

As reported in Bartlett and Farnsworth (2002), maximum primary consolidation settlements of about 10% of the embankment fill heights can be expected in the Salt Lake Valley for locales where embankment is placed over virgin ground. The measured primary settlement at the 200
South Site at the face of the MSE wall corresponds to about 9% of the 12-m (39-ft) fill height, which is in good agreement with the “10% rule.”

However, just as important is estimating the zone of significant settlement for new embankments and MSE walls constructed near adjacent facilities. Although different structure and foundation types can tolerate differing amounts of settlement, a reasonable goal might be to limit the total settlement incurred by an adjacent facility to 25 mm (1 in.). Thus, another rule of thumb might be established to give guidance regarding how close a MSE wall can be constructed to an adjacent structure without causing settlement of the structure that exceeds 25 mm (1 in.). Based on the measured settlement data for the 200 South Site, structures adjacent to large MSE walls should be located a horizontal distance of at least 1.3 times the maximum fill height (i.e., embankment plus surcharge) to limit settlements to 25 mm (1 in.), or less.

6.6 Janbu’s Modulus Method

Prior to numerical modeling, a one-dimensional (1D) deformation analysis based on Janbu’s modulus method was performed. Janbu’s elastic model was considered because it is a non-linear 1D model that is relatively easy to use. Also, design engineers for the I-15 Reconstruction Project had used this model to make settlement predictions at the 200 South Site and other important sites.

The Janbu model accounts for the non-linear, 1D load-compression behavior of soils by assuming a hyperbolic relationship between stress and strain (Fang, 1991). This approach is similar to that followed for the 2D HNLE constitutive model. Axial strain $\varepsilon$ is computed as a function of effective stress and constrained modulus $M_t$ with

$$
\varepsilon = \int_{\sigma'_{vo}}^{\sigma'} \frac{1}{M_t} \, dt
$$

(11)

where $\sigma'_{vo} =$ initial vertical effective stress, and $\sigma' =$ vertical effective stress. The constrained modulus is evaluated as

$$
M_t = \frac{m}{a+1} \left( \frac{\sigma'}{p_o} \right)^{1-a}
$$

(12)

where $m = $ dimensionless modulus number; and $a = $ stress exponent. Typical values of $a$ for rock, sand, and clay are 1, 0.5, and 0, respectively.
Janbu’s modulus method was incorporated in unpublished Excel™ spreadsheets created by Woodward-Clyde Consultants (1997) during the I-15 Reconstruction Project. The spreadsheets can calculate primary and elastic settlements for plane-strain conditions. Additionally, the influence of surcharge removal and secondary consolidation settlements may be estimated. The spreadsheet uses a 2D Boussinesq stress distribution and handles the embankments as flexible loads of constant or linearly increasing intensity applied at the original horizontal ground surface. Consequently, the stress distribution that develops within the embankment above the ground surface is not taken into account.

For this report, the Woodward-Clyde spreadsheet was utilized to estimate primary and elastic settlement for the Phase 2 fill at the 200 South Site. Required input parameters include soil type, layer depths, unit weight, compression ratio, modulus number, recompression modulus number, preconsolidation pressure, and depth to water. For the analysis, the soil type, layer depths, unit weight, and depth to water were obtained from the idealized soil profile for the 200 South Site. The remaining input parameters were computed as averages of the input data used during the design phase of I-15 Reconstruction Project.

The spreadsheet is designed to model three construction conditions: (1) embankment placed on a free-field condition, (2) new embankment placed over existing embankment, and (3) a final embankment with surcharge placed over free-field or existing embankment condition. To simulate embankment construction at the 200 South Site, the new construction Phase 1 fill with surcharge was modeled as the existing embankment (condition 2). The new Phase 2 fill with surcharge, in conjunction with removal of the Phase 1 surcharge, was modeled as the final embankment with surcharge (condition 3). Thus for this case, only the effect of the Phase 2 fill with surcharge is included in producing the resulting settlement pattern (Fig. 4).

The resulting settlement-distance curve is plotted with the measured settlement data in Fig. 4. The shapes of the measured data curve versus the predicted curve based on the Woodward-Clyde spreadsheet vary in significant ways. The magnitude of the maximum deformation computed by the spreadsheet is about 0.9 m (3 ft), which is about 20% less than the maximum measured deformation. This difference in maximum settlement could be caused by slight underestimation of the Janbu parameters inputted into the spreadsheet.

As opposed to the measured settlement profile, which shows a maximum settlement near the wall face, the results based on the Janbu modulus with a 2D Boussinesq stress distribution suggest that maximum settlements occur near the center of the Phase 2 embankment. In other words, the largest stresses and resulting settlements were predicted by the spreadsheet where the fill was at its greatest height and extent. However, this does not match the observed settlement pattern very well. The observed settlement is greatest near the face of the wall and diminishes into the wall as the footprint of the old embankment is encountered. Thus, the influence of the
old I-15 embankment and its affect on the preconsolidation stresses in the underlying soils is playing an important part in changing the settlement pattern underneath the MSE wall. Also, the settlement pattern away from the face of the MSE wall varies significantly from that predicted by the spreadsheet approach. The spreadsheet results suggest a settlement of about 0.05 m (0.2 ft) beneath the existing house, which somewhat underpredicts the actual settlements that were measured along the southeastern edge of the house.

The spreadsheet results indicate an apparent heave of about 0.06 m (0.2 ft) beneath the southeast embankment, where the surcharge was removed. Although some measurable heave may have occurred during construction, the heave calculated with the Woodward-Clyde spreadsheet may be somewhat overstated.

The mismatch between the modeled and observed settlement profile for this site is probably attributed to three factors: (1) the 2D Boussinesq stress distribution used by the spreadsheet to model the actual stress distribution that develops in the fill is inadequate; (2) the 2D Boussinesq distribution is for a homogenous, isotropic medium and the actual condition is a multi-layered system with varying stiffness; and (3) the stiffness for a given soil layer probably varies in the horizontal direction due to varying degrees of preconsolidation caused by the new Phase 1 fill and the original I-15 embankment. All of these limitations can be more fully addressed by using the HNLE model.

6.7 Finite Element Mesh

The finite element mesh for the numerical modeling used a cross-section that extended perpendicularly through the MSE wall toward the southeast corner of the existing house. This cross-section was obtained from the I-15 Reconstruction Project plans (UDOT, 1998) and represents the nearest cross section to the house location. Construction cross-sections were developed on 20-m (66-ft) project stationing and are shown perpendicular to the main-line alignment.

Three stages of embankment fill were represented in the finite element mesh (Fig. 5). The first stage consisted of the original embankment, approximately 6 m (20 ft) high and 102 m (335 ft) wide along the cross-section. The second stage consisted of the Phase 1 fill placed over the southeastern portion of the roadway alignment to a maximum fill height of 12 m (39 ft). The final stage consisted of the Phase 2 fill placed over the northwestern portion of the alignment to a maximum height of 12 m (39 ft). The geometries of the embankment fills were obtained from plan sheets for the I-15 Reconstruction Project (UDOT, 1998).

The initial water table was placed at a depth of 2 m (7 ft) below the ground surface. For the numerical model boundary conditions, displacement was fixed in the x-direction along the
Fig. 5. 200 South Site generalized finite element mesh

vertical axes and in the x- and y-directions along the horizontal axis. Infinite elements were used at the left and right model boundaries for load/deformation analyses.

6.8 Modeling Progression

Numerical analyses were performed in four sequential steps: (1) establishment of initial conditions, (2) placement of original embankment, (3) placement of Phase 1 fill, and (4) placement of Phase 2 fill (Figs. 4 and 5). For all cases, the LE constitutive model was used to represent the half-space.

6.8.1 Initial Conditions

The initial conditions analysis is required to calculate the initial state of stress in the foundation soils. In this analysis, the embankment fills were modeled as null materials (i.e., no constitutive model was assigned and the embankment has no body load). The subsurface layers were modeled as LE materials. When the LE constitutive model is assigned, values of $E$ and $\nu$ are required for each layer (Table 1). However, the computed initial stresses are independent of the values of $\nu$ in initial conditions analyses. Values of $\gamma$ and $K_o$ from Table 1 were input as body load parameters for the subsurface materials. Values of $E'_{max}$ given in Table 1 are “low strain”
values or maximum values obtained from geophysical testing and hence are not representative for the strain range that will develop in the foundation soils underneath large embankments. These values will be adjusted to more representative values, as the model is calibrated, as discussed later.

6.8.2 Placement of Original Embankment

From the results of the initial conditions file, a load/deformation analysis was performed to simulate placement of the original embankment fill. The subsurface layers were assigned drained material properties applicable to the analysis being performed, as described in later sections of this report. In addition, the fill was modeled as an LE material; parameters $E'$ and $\nu'$ of the fill were assumed equal to 98,700 kPa (14,300 psi) and 0.35, respectively. In research conducted by others, these values were used to represent embankment fills placed at or near the 200 South and I-80 Sites (Alcorn, 2003). The values fall within the ranges of published data in Bowles (1996). As required to compute stresses imposed by the fill weight, the body load parameters $\gamma$ and $K_o$ for the fill were assumed equal to 21.2 kN/m$^3$ (135 pcf) and 0, respectively, based on review of available data (Alcorn, 2003). The original embankment fill was modeled with 2-m (7-ft) lifts placed in 4 time steps.

6.8.3 Placement of Phase 1 Fill

After solving the original embankment model, a load/deformation analysis of the Phase 1 fill was conducted. The solution file for the original embankment fill was specified as the initial conditions file. The subsurface layers and original embankment fill were modeled with “permanent” fill/excavation elements. The Phase 1 fill was modeled with the properties assumed for the original embankment in the previous analysis, and was placed in 2-m (7-ft) lifts over 6 time steps.

6.8.4 Placement of Phase 2 Fill

Finally, the Phase 2 fill was analyzed with initial conditions established from solution of the Phase 1 fill analysis. All materials other than the Phase 2 fill were characterized as “permanent,” and upon removal of the Phase 1 surcharge, the fill was modeled with 2-m (7-ft) lifts in 6 time steps. It should be noted that the Sigma/W model can be modified to perform these steps in a single analysis. However, using separate files for different stages of construction is convenient for modifying and organizing the results.
6.9 Results of Analyses

6.9.1 LE Analysis

A LE analysis was conducted for the cases listed above and compared with the field performance data. The LE modeling provided a working familiarity with the finite element code and a basis to evaluate the reasonableness of the modeled geometry and the phasing of the embankment construction in the numerical model.

The values of $\nu'$ listed in Table 1 and back-calculated values of $E'$ shown in Table 2 were used as the elastic properties for the LE analysis. Values of $E'$ were back-calculated from the $E'_{\text{max}}$ ratios given in Table 1 using an iterative process. In the back-calculation, it was deemed reasonable to assume that the $E'_{\text{max}}$ ratios for the various layers could be used to define the relationship between $E'$ for the respective layers in the LE model. In other words, it was assumed that the $E'_{\text{max}}$ ratios of the various layers would remain approximately the same for the LE analysis and these ratios could be used to approximate the $E'$ ratios for the LE analysis. Once the $E'$ ratios are known for the multi-layered system, it is possible to assume an $E'$ value for one layer in the system, then calculate the corresponding $E'$ values for all other layers.

<table>
<thead>
<tr>
<th>Description</th>
<th>$E'_{\text{max}}$ Ratio</th>
<th>$E'$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>1.2</td>
<td>4,800</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>1.0</td>
<td>4,000</td>
</tr>
<tr>
<td>Interbeds</td>
<td>2.1</td>
<td>8,300</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>2.1</td>
<td>8,300</td>
</tr>
<tr>
<td>Deeper Alluvium</td>
<td>4.3</td>
<td>17,350</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>3.3</td>
<td>13,000</td>
</tr>
<tr>
<td>Half-Space</td>
<td>5.0</td>
<td>20,100$^a$</td>
</tr>
</tbody>
</table>

$^a$ Used for the LE analysis
$^b$ Used for the HNLE analysis

1 kPa = 0.145 psi
Assuming an $E'$ for the softest layer (i.e., the Upper Bonneville), the $E'_{\text{max}}$ ratios were used to calculate $E'$ for each of the remaining layers. From this the total settlement and the settlement below the Cutler Dam unit were calculated. Estimates of $E'$ for the Upper Bonneville unit were then adjusted, until the modeled results indicated that about 0.12 m (0.4 ft) of deformation occurred below the Cutler Dam unit. This constraint was placed on the model based on magnet extensometer data for other instrumented sites in the Salt Lake Valley. These data suggest that approximately 10% to 15% of the total primary settlements results from consolidation of the soils below the Cutler Dam sediments. In the case of the 200 South Site, having a maximum primary settlement of about 1.1 m (3.6 ft), it was assumed that settlement below the Cutler Dam was about 0.12 m (0.4 ft).

The $E'$ values for the layers above the half-space generally fall within the ranges given in Bowles (1996) and Budhu (2000) for similar material types. However, it is noted that the ranges of published values are large and the corresponding levels of strain are not identified.

The results of the Sigma/W analysis are illustrated in Fig. 6. Based on comparison of the LE analysis profile with the measured settlement profile, the LE analysis overpredicted the settlement beneath the house by a factor of about 2, and underpredicted the maximum settlement beneath the MSE wall by a factor of about 0.5. The bulge in the curve on the right side of the model, beneath the Phase 1 fill, indicates an apparent heave of about 0.03 m (0.1 ft). Results of all the numerical analyses confirm this bulging effect to some degree, and this behavior is attributed to possible numerical instabilities occurring during model solution. Given that the primary focus of this research is the prediction of settlement near MSE walls, the apparent slight heave indicated by the model was not investigated further.

It should be noted that it is possible to further calibrate the LE model until it gives better predictions of settlement at the wall face, but this was not done. The authors believe that modeling a non-linear stress-strain process with a LE model is a crude approximation and not very useful; thus, non-linear model parameters were developed and calibrated to the performance data, as discussed in the next section.

6.9.2 Calibrated HNLE Analysis

A non-linear analysis was performed to calibrate the HNLE parameters for subsurface layers above the half-space. Laboratory triaxial test data were not available; therefore for the initial computer runs, drained HNLE properties were assumed from typical published values. These assumed values were adjusted using an iterative back-calculation process constrained by field measurements and other considerations, as discussed below.
The Sigma/W dialog box for input of non-linear model parameters has 15 input fields; however, not all fields are required for HNLE analyses. Default values were utilized for atmospheric pressure $p_a$, cohesive strength $c$, minimum minor principal stress $\sigma_{3,min}$, minimum Poisson’s ratio $\nu_{min}$, minimum initial tangent modulus $E_{i,min}$, and matric suction angle $\phi_b$. The default value for $c$ is zero, which was used both for granular soils (sand and silts) as well as clays. For the clay soils, a zero cohesion intercept is reasonable because the HNLE analysis used “drained” parameters; hence, the $c$ value requested by Sigma/W is really a $c'$ value (i.e., the cohesion intercept for an effective stress failure envelope). Values of $c'$ for normally to slightly overconsolidated clayey soils are usually quite small and typically can be neglected.

The generalized values used for $\phi'$ in the HNLE analysis are listed in Table 1. It was found that the deformation analysis for the HNLE model is not very sensitive to the values of $\phi'$ for a reasonable range of this variable. Also, it should be noted that in the Sigma/W dialog box, zero was specified for $E_i$ and $\nu$. This indicates that Sigma/W will calculate these parameters from the inputted modulus numbers, modulus exponents, and failure ratio, i.e., $K_b$, $K_{ur}$, $n$ and $R_f$, respectively.

---

**Fig. 6.** 200 South Site settlement profile from LE analysis
For these analyses, the modulus exponents $n$ and $m$ and the failure ratio $R_f$ were assumed based on the conservative parameter values provided in Duncan et al. (1980) for the appropriate soil type. The assumed parameter values are presented in Table 3. The Upper Bonneville, Lower Bonneville, and Cutler Dam layers were assigned $m$ and $n$ values of 0.45 and 0.2, respectively, as suggested for clays (CL). The Upper Alluvium and Interbeds were modeled with $m$ and $n$ of 0.6 and 0.5, respectively, as recommended for silty to clayey sands (SM-SC). Parameters $m$ and $n$ for the Deeper Alluvium were assumed to be 0.25 and 0, respectively, as published for silty sands (SM). The suggested value of $R_f$ equals 0.7 was used for all soil classifications (Duncan et al., 1980).

Calibrated modulus numbers $K$, $K_b$, and $K_{ur}$ (Table 3) were estimated through an iterative back-calculation procedure, similar to that followed for the LE analysis. The constraint placed on the HNLE analysis was that the ratio of the initial tangent moduli, $E_i$, for the various layers modeled by the HNLE model, must be consistent with the $E'_{max}$ ratios given in Table 1. The steps for doing this are: (1) calculating an $E_i$ for the Upper Bonneville layer using Eq. (2) with the $K$ and $n$ parameters in Table 3 and the confining pressure from Eq. (13); (2) from this $E_i$ value and the appropriate $E'_{max}$ ratios from Table 1, calculating the $E_i$ values for all other layers; and (3) from these $E_i$ values, estimate the appropriate $K$ for all other layers using Eq. (2) and the typical $n$ values from Table 3. For the first step, the confining pressure was estimated from

$$\sigma'_{s} = K_o \sigma'$$

(13)

<table>
<thead>
<tr>
<th>Description</th>
<th>$K$</th>
<th>$K_b$</th>
<th>$K_{ur}$</th>
<th>$m$</th>
<th>$n$</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>60</td>
<td>40</td>
<td>240</td>
<td>0.6</td>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>30</td>
<td>20</td>
<td>120</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Interbeds</td>
<td>50</td>
<td>40</td>
<td>220</td>
<td>0.6</td>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>50</td>
<td>40</td>
<td>190</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Deeper Alluvium</td>
<td>110</td>
<td>120</td>
<td>430</td>
<td>0.25</td>
<td>0.0</td>
<td>0.7</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>70</td>
<td>50</td>
<td>300</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
</tbody>
</table>
where: \( \sigma' \) is the average effective vertical stress at the middle of the layer for the free-field condition (i.e., no embankment present).

It should be noted that it is improper to use \( E'_{\text{max}} \) values from Table 1 to represent \( E_i \) values in the HNLE model. Although the former are determined from geophysical tests and are truly “low-strain initial tangent moduli,” they are much higher than the typical \( E_i \) values published by Duncan et al. (1980). The latter values come from triaxial shear test results, and hence are valid for a strain range that is initially higher than the strain produced by the geophysical tests. Notwithstanding, the latter are referred to as “initial tangent moduli” by Duncan et al. (1980), and it should be remembered that this definition is based on laboratory test results.

The bulk modulus number, \( K_b \), is also needed for the HNLE model, which can be calculated using Eq. (5). To apply this equation, however, estimates of \( B \) are needed. Estimates of \( B \) must be consistent for the appropriate stress level. Hence, \( B \) values were calculated from Eq. (6), using the \( E_i \) values obtained in the previous paragraph and the typical values of \( \nu' \) from Table 1. Then Eq. (5) was applied to calculate \( K_b \) for the appropriate stress calculated from Eq. (13). This procedure guaranteed that all of the HNLE model parameters were internally consistent and agreed with typical values of \( \nu' \) for low levels of applied confining pressure.

For all layers, \( K_{ur} \) was input as four times \( K \). This approximation is based on settlement measurements made during the original embankment construction in the 1960s. A plot of measured settlement versus fill height was recorded for sites near the 200 South Site. Based on this plot, the settlement in recompression is about 4 times less than the settlement in virgin compression, for the same increase in vertical stress. This implies that the soil profile is, on average, about 4 times stiffer in recompression than virgin compression over the strain range induced by the placement of the new fill at this site. Furthermore, the settlement versus fill height plots for this area show that the settlement for recompression and virgin compression is not highly non-linear, at least for the stress range imposed by the embankments at this locale.

Because the above approximation is made based on measured field data, the ratio is an average for all layers undergoing settlement; thus, \( K_{ur} \) equal to \( 4K \) was used for all layers in the model. However, in reality, the \( K_{ur}/K \) ratio probably varies somewhat according to material type and variations in stiffness. However, there was not enough information to make this refinement in a layer-by-layer fashion, thus the average of 4 was used. It should be noted that this value is slightly higher than the value of 3 suggested by Duncan et al. (1980) for soft soils. However, use of the slightly higher value was considered appropriate, because it is based on field performance data.
The initial $E_i$ values and corresponding modulus numbers were adjusted in an iterative fashion, preserving the corresponding $E'_{\text{max}}$ ratios, until the Sigma/W analysis produced a settlement profile that reasonably matched the measured settlement profile at the 200 South Site. In determining the goodness of fit, both the shape and the magnitude of the calculated settlement profile were considered during the calibration process. The aim of this calibration was to predict the maximum settlement below the MSE wall face to within about 5% of the measured value and closely match the settlement pattern underneath the adjacent house. Throughout the initial calculations, the half-space was modeled with an $E'_{1/2sp}$ of 20,100 kPa (2,900 psi), as back-calculated during the LE analysis. However, the calibration was further refined using an $E'_{1/2sp}$ of 30,000 kPa (4,400 psi) (Table 3). This latter value produced a settlement of about 0.12 m (0.4 ft) below the Cutler Dam (a constraint placed on the model), and yielded a good match with the measured settlement profile as shown in Fig. 7.

It is noted that problems with achieving convergence were encountered sometimes during the finite element modeling. The problem was circumvented when the modulus numbers were rounded to the nearest 10 in the input file. The HNLE properties used in the final computer run are given in Table 3.

---

**Fig. 7.** 200 South settlement-distance curve, HNLE calibration
As indicated in Fig 7., the calibrated settlement profile matches the measured profile quite well for the zone between the face of the MSE wall and the house. It is this zone that requires the best prediction, because this is the zone of potential settlement damage to adjacent structures. The HNLE model has adequately captured both the magnitude and settlement pattern in this critical zone.

Beyond the MSE wall towards the center of the embankment, the calibrated curve shows slightly greater deformation than the measured data curve. However, the match in this area was viewed as less important because this study concentrated on predicting the settlements beneath adjacent structures, rather than settlements beneath embankments.

### 6.9.3 Comparison of HNLE Analysis Results with Typical HNLE Model Parameters

The back-calculated values (Table 3) were compared with the recommended Duncan et al. (1980) values at 85% compaction (Table 4). This was done to judge the reasonableness of the back-calculated parameters.

Duncan et al. (1980) propose conservative values of $K$ and $K_b$ for compacted soils with relative compactions (standard AASHTO method) ranging from 85% to 105%. Although a direct comparison of values for compacted soils (e.g., Duncan et al. (1980)) and uncompacted soils (i.e., 200 South Site foundation soils) is not strictly appropriate, it is useful in assessing whether or not the back-calculated values are realistic. The back-calculated $K$ values of 30 and 50 for the Upper and Lower Bonneville layers, respectively, are somewhat lower than the value of 60 recommended for CL soils.

<table>
<thead>
<tr>
<th>Description</th>
<th>$K$</th>
<th>$K_b$</th>
<th>$K_{ur}$</th>
<th>$m$</th>
<th>$n$</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>100</td>
<td>50</td>
<td>200</td>
<td>0.6</td>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>60</td>
<td>50</td>
<td>120</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Interbeds</td>
<td>150</td>
<td>75</td>
<td>300</td>
<td>0.6</td>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>90</td>
<td>80</td>
<td>180</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Deeper Alluvium</td>
<td>300</td>
<td>250</td>
<td>600</td>
<td>0.25</td>
<td>0.0</td>
<td>0.7</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>120</td>
<td>110</td>
<td>240</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
</tbody>
</table>
The back-calculated $K$ for the Cutler Dam deposits is 70, which is slightly greater than the recommended value for CL soils. However, this is a stiff, overconsolidated clay layer and the higher value is probably justified. Back-calculated values for the Upper Alluvium and Interbed layers, ranging from 50 to 60, are substantially lower than the suggested value of 100 for SM-SC soils. However, both of these units are not strictly SM-SC soils and have not been compacted. Sediments in the Upper Alluvium have considerable silt and clay content and are locally quite variable, and the Interbeds of the Bonneville deposits are a mix of fine-bedded clay, silt and fine sand. Thus, a SM-SC material type does not completely reflect the highly variable and interbedded nature of this unit. Also, the $K$ value of 110 for the Deeper Alluvium is somewhat lower than the value of 150 proposed for SM soils, but is not unreasonable. Similar trends are observed when comparing the back-calculated $K_b$ values to those recommended in Duncan et al. (1980), who suggest conservative $K_b$ parameters of 50, 50, and 150 for CL, SM-SC, and SM soils respectively. Based on this comparative assessment, the back-calculated $K$ and $K_b$ values seem reasonable, or at least significant differences can be reasonably explained.

For comparison with results of the back-calibration analysis, non-linear analysis was performed using the typical parameters in Table 4. The $K_u/K$ ratio was assumed equal to 2, which falls within the range suggested in Duncan et al. (1980). The analysis results are illustrated in Fig. 8.

The predicted settlement pattern using the typical HNLE parameters was significantly less, by a factor of about 2, than the measured settlement beneath the MSE wall and also in the zone between the MSE wall and the house. Near the house, the predicted values were in good agreement with measured values, but this may be somewhat fortuitous.

The above example demonstrates the importance of the calibration process and how it improved the model’s predictive capability. It is recommended that the calibrated parameters in Table 3 be used for future HNLE settlement analyses for sites underlain by soft, Lake Bonneville deposits.
**Fig. 8.** 200 South settlement-distance curve, analysis with typical HNLE model parameters
7.0 ANALYSIS OF THE I-80 SITE

7.1 Site Description

During the I-15 Reconstruction Project, lime cement columns (LCCs) were used to stabilize a 12-m (39-ft) to 13.5-m (44-ft) high MSE embankment on I-80 between 300 West and 200 West Streets, where a new ramp was being constructed to convey westbound I-80 traffic to northbound I-15. The LCC stabilization was used below the northern half of the ramp, where the roadway was widened by about 15 m (49 ft) to the north to support westbound traffic. The design of the LCC treatment at the I-80 Site is discussed by Saye et al., (2001). The primary purpose of the treatment was to improve the stability, bearing capacity and settlement behavior of the MSE wall by increasing the shear strength and stiffness of the foundation soils. Fig. 9 presents a plan view of the I-80 Site and the LCC installation pattern. Each circle in this figure represents the top of a lime cement column.

A commercial building (Fig. 10) is located approximately 7 to 12 m (23 to 39 ft) north of the face of the MSE wall, which supports the widened embankment (Figs. 9, 11). It was the design-build contractors’ desire to minimize the construction and post-construction settlement at this site, so that no significant settlement damage was incurred by this building supported on shallow foundations. The instrumentation and settlement performance at this site has been well documented by Bartlett and Farnsworth (2002).

Prior to construction, the existing 7-m (23-ft) to 7.5-m (25-ft) high, sloped embankment fill was removed from the footprint of the treatment area. The original embankment fill on the southern half of the alignment was left in-place so that traffic could be maintained during construction. LCCs of 0.6-m (2.0-ft) and 0.8-m (2.6-ft) diameters were installed in a relatively complex pattern of intersecting panels, triangular spacings, and rectangular spacings. Most columns were installed to a depth of 20 m (66 ft) below ground surface, except in the transition zone in the extreme southern edge of the treatment area (Fig. 11). Installation depths of LCCs in this zone were decreased to form a vertically stepped transition to tie into the existing embankment to the south. It was hoped that this stepped zone would minimize the differential settlement that developed between the treated and untreated zones.

Upon installation of the LCCs, a one-stage MSE wall, embankment, and surcharge were constructed to a maximum height of about 13.5 m (44 ft) in the bridge abutment and instrument array area. Temporary surcharge, on the order of 1.5 to 3 m (5 to 10 ft) in height, was placed over the roadway footprint to pre-compress the soils at depth. This surcharging was deemed necessary to minimize the post-construction settlement at this site, especially in the soils below the installation depth of the LCCs. A temporary welded wire wall, with horizontal
Fig. 9. I-80 Site plan and lime cement column installation pattern
Fig. 10. Embankment and MSE wall during construction at I-80 Site (view to the southeast)

Fig. 11. Typical embankment profile at I-80 Site
reinforcement, was erected atop the 1-stage MSE wall to retain the surcharge fill during construction. After completion of primary consolidation, the surcharge and temporary wall were removed and final roadway construction was completed for the west-bound side of the alignment. A photograph of the site during construction of the MSE wall is given in Fig. 10. At this time the MSE wall construction has essentially been completed and no surcharge fill remains in place. Also, a generalized cross-section of the embankment is provided in Fig. 11. Construction of the south side of the alignment, which ensued after completion of the north side, was not included in this analysis.

7.2 Subsurface Profile

Subsurface materials below the original embankment in the proximity of the I-80 Site are similar in nature to those at the 200 South Site. The thickness of the upper alluvium ranges from about 5.5 to 7 m (18 to 23 ft). The upper alluvium is underlain by approximately 7 to 8 m (23 to 26 ft) of Lake Bonneville deposits, with a layer of interbedded silts and sands near the middle of the stratum. Interbedded alluvial and lacustrine sediments beneath the Lake Bonneville series are about 6 to 9 m (20 to 30 ft) thick. These highly interbedded deposits are underlain by the Cutler Dam Lake sequence, which is typically 4.5-m (15-ft) to 6-m (20 ft) thick. Dense sands and gravels underlie the Cutler Dam Lake sediments. The groundwater table is typically encountered at about 1 to 2 m (3 to 7 ft) below the ground surface in this area, and was assumed to be located at a depth of 2 m (7 ft) for the purpose of this study.

Layer depths and general soil properties used for the numerical analyses are summarized in Table 5. Estimates of the thickness of the untreated soil layers above the half-space are based on CPT data collected at the LCC array. The half-space thickness and general soil properties are the same as those estimated for the 200 South Site. The soil sequence is similar to that used for 200 South, except the I-80 Site was modeled with two alternating layers of the Deeper Alluvium and Cutler Dam layers to better represent the interbedding encountered below a depth of about 13 m (43 ft). Based on review of available shear wave velocity data and measured tip resistances, both sequences of these layers have similar stiffnesses. Therefore, the layers within each sequence were assigned the same general soil properties and constitutive model parameters.

The HNLE model parameters for the soil layers above the half-space are summarized in Table 6. In developing these layer parameters, the shear wave velocity profile and CPT data used for the 200 South Site were compared with the data presented by Gerber (1995) for a site in the I-80/I-15 interchange area. Based on this review, the Upper Bonneville and Interbeds layers at I-80 are approximately 15% to 20% stiffer than the layers at 200 South. The HNLE properties of these layers were adjusted accordingly to account for this slightly greater stiffness. The stiffnesses of the other layers at the I-80 Site are generally similar to those at the 200 South Site, and the
Table 5. I-80 Site Layer Depths and Generalized Soil Properties

<table>
<thead>
<tr>
<th>Description</th>
<th>Top Depth (m)</th>
<th>Bottom Depth (m)</th>
<th>Thickness (m)</th>
<th>(\gamma) (kN/m(^3))</th>
<th>(\phi') (deg)</th>
<th>(\nu')</th>
<th>(K_0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>0</td>
<td>5.5</td>
<td>5.5</td>
<td>19.2</td>
<td>37</td>
<td>0.3</td>
<td>0.74</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>5.5</td>
<td>9.5</td>
<td>4</td>
<td>18.2</td>
<td>31</td>
<td>0.3</td>
<td>0.70</td>
</tr>
<tr>
<td>Interbeds</td>
<td>9.5</td>
<td>11</td>
<td>1.5</td>
<td>18.8</td>
<td>31</td>
<td>0.3</td>
<td>0.64</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>11</td>
<td>13</td>
<td>2</td>
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<td>13</td>
<td>14</td>
<td>1</td>
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<td>0.35</td>
<td>0.43</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>14</td>
<td>21</td>
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<td>0.46</td>
</tr>
<tr>
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<td>22.5</td>
<td>1.5</td>
<td>19.5</td>
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<td>0.43</td>
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<tr>
<td>Cutler Dam</td>
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<td>27</td>
<td>4.5</td>
<td>18.4</td>
<td>37</td>
<td>0.25</td>
<td>0.46</td>
</tr>
<tr>
<td>Half-Space</td>
<td>27</td>
<td>70</td>
<td>43</td>
<td>21.2</td>
<td>--(^a)</td>
<td>0.35</td>
<td>--(^a)</td>
</tr>
</tbody>
</table>

1 m = 3.28 ft, 1 kN/m\(^3\) = 6.366 pcf
\(^a\) Not required for analyses.

Table 6. I-80 Site HNLE Model Parameters for Untreated Soils

<table>
<thead>
<tr>
<th>Description</th>
<th>(K)</th>
<th>(K_b)</th>
<th>(K_{ur})</th>
<th>(m)</th>
<th>(n)</th>
<th>(R_f)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>60</td>
<td>40</td>
<td>240</td>
<td>0.6</td>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>40</td>
<td>25</td>
<td>140</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Interbeds</td>
<td>60</td>
<td>50</td>
<td>240</td>
<td>0.6</td>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>50</td>
<td>40</td>
<td>190</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Deeper Alluvium</td>
<td>110</td>
<td>120</td>
<td>430</td>
<td>0.25</td>
<td>0.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>70</td>
<td>50</td>
<td>300</td>
<td>0.45</td>
<td>0.2</td>
<td>0.7</td>
</tr>
</tbody>
</table>
HNLE properties estimated from calibration of the 200 South model were used for the I-80 model.

The half-space was modeled as an LE material with $\nu' = 0.35$. Analyses incorporated an “uncalibrated” value of $E'_{1/2sp}$ of 30,000 (4,400 psi) or a “calibrated” value of 68,000 kPa (9,900 psi); the calibration is discussed later in this chapter.

### 7.3 Measured Settlement Data

Field measurements of primary settlement (Bartlett and Farnsworth, 2002) were used to plot the settlement curve in Fig. 12. (The left side of Fig. 12 and subsequent figures corresponds to the north direction.) Measured settlement data were collected by means of 21 surveyed settlement points (LC1 through LC 21) and 2 horizontal inclinometers (Inc 301 and 302) (Fig. 9).

The settlement points were located on the ground surface outside the east and west walls of the existing building. The southern settlement points were located along the UDOT right-of-way fence, about 6 m (20 ft) from the southern side of the building. The settlement data plotted in Fig. 12 are from survey measurements made in late June 1999, after primary consolidation was

![Fig. 12. I-80 Site measured settlement-distance curve](image-url)
complete and just prior to surcharge removal. Because settlement points were located only on the west and east sides of the building, these points were averaged to project the settlement at the centerline of the building. To determine the settlement near the south wall of the building, the measured data were plotted along a line extending through points LC1 through LC8. The settlement was established as the point of intersection of the plot of the measured data and the centerline profile.

Inclinometer 301 was used to show the settlement pattern beneath the MSE wall. Inclinometer 301 is located at the base of the new MSE wall, and extends beneath the embankment a horizontal distance of approximately 17.7 m (58 ft). Inclinometer data collected in July 1999 were used to produce the portion of the curve beneath the embankment, beginning at the wall face. A smoothed line was drawn between the settlement point data and the inclinometer data to construct the measured profile. The profile shows measured primary settlements of approximately 0.15 m (0.5 ft) at the MSE wall face and 0.02 m (0.1 ft) near the south wall of the adjacent building. Negligible settlements were measured below the northern half of the building.

7.4 Rule of Thumb from Settlement Measurements

Like was done for the 200 South Site, it is useful to use the measured settlement data to develop a “rule of thumb” that describes the zone of significant settlement at the I-80 Site. The zone of significant settlement – i.e., settlements greater than 25 mm (1 in.) – extended about 14 m (46 ft) beyond the MSE wall face at the I-80 Site. This was estimated by using settlement points LC10 and LC17 from the LCC instrument array (Fig. 9). Settlement points LC10 and LC17 are located 17 m (56 ft) and 13 m (43 ft), respectively, from the face of the MSE wall. At the end of primary consolidation settlement, settlement point LC10 had settled approximately 20 mm (0.8 in.) and settlement point LC17 had settled 30 mm (1.2 in.) (Bartlett and Farnsworth, 2002). Thus, the 25 mm (1 in.) settlement contour is approximately midway between these locations, or at a distance of about 15 m (49 ft) beyond the face of the wall.

The embankment plus surcharge height is about 13.5 m (44 ft) at this location, which gives a settlement to fill height ratio of 15:13.5, or 1.1:1. Based on these measurements, structures adjacent to large MSE walls should be located a horizontal distance of at least 1.1 times the maximum fill height to limit settlements to 25 mm (1 in.), or less, for the case where the foundation soil has been treated with lime cement columns. This is slightly less than the 1.3:1 ratio that was found at the 200 South Site. Thus, the LCC treatment appears to have played a part in reducing this ratio, when compared to sites with untreated foundation soils. However, it is somewhat surprising that the settlement to fill height ratio is only slightly lower for treated ground as for untreated ground.
7.5 Finite Element Mesh

Fig. 13 presents a generalized representation of the mesh used to simulate the original embankment fill, new fill, soil layering, and LCC-treated zone at the I-80 Site. The finite element mesh was constructed for a cross-section extending through the north-south centerline of the existing 30-m (98-ft) wide building. The mesh simulated two phases of embankment fill. The first phase consisted of the original embankment that was approximately 7 m (23 ft) high and 74 m (240 ft) wide. The second phase consisted of new fill placed over the northern half of the original embankment to a maximum height of 13.5 m (44 ft). This includes a final embankment height of 12 m (39 ft) and a surcharge fill of 1.5 m (5 ft). The LCC-treated area was modeled with a width of 36 m (118 ft) and a depth of 21 m (69 ft). It is noted that the LCCs were actually installed to a maximum depth of 20 m (66 ft), and 1- to 2-m (3- to 7-ft) of fill was placed over the tops of the columns. As discussed later, this fill did not have a significant effect on stress distribution and settlement patterns. Thus, for the purpose of this analysis, the LCC zone was assumed to extend from the ground surface to the bottom installation depth.

Fig. 13. I-80 Site generalized finite element mesh
The initial water table, node boundary conditions, element properties, and infinite elements were identical to those specified for the 200 South Site finite element mesh.

7.6 Modeling Progression

The general modeling methodology developed for the 200 South Site was followed for analysis of the I-80 Site. Deviations are described in this section. Three sequential analysis steps were taken: (1) establishment of initial conditions, (2) placement of original embankment fill, and (3) placement of new embankment fill. The original embankment fill was modeled with 1- to 2-m (3- to 7-ft) lifts placed in 4 time steps. The new embankment fill was placed in 1- to 3-m (3- to 10-ft) lifts in 6 time steps over the north side of the original embankment fill. In conjunction with installation of the new embankment, the toe of the original embankment outside the north face of the embankment was removed. The load-deformation analyses incorporated the same LE properties for embankment fill as were used for the 200 South Site. The untreated soils were assigned the HNLE material properties summarized in Table 6.

7.7 Results of Analyses

7.7.1 Untreated Ground

Initially, a non-linear analysis was performed for the I-80 Site for the case without ground treatment. This was done to predict the amount of settlement that might have occurred at this locale, if the ground had not been treated with LCCs. The linearly elastic half-space was modeled with $E'_{1/2sp} = 30,000$ kPa (4,400 psi), based on the model used for the 200 South Site.

The settlement profile from the untreated case is a v-shaped curve with the lowest point corresponding to the MSE wall face (Fig. 14). This settlement pattern seems reasonable given that the greatest height of new embankment fill was placed at the face of the MSE wall. Also, there is less original embankment fill in this zone, thus the foundation soils did not experience as much preloading as occurred near the centerline of the embankment.

Although there is no way to verify the prediction in Fig. 14, it is possible to compare these modeling results with the computed settlements reported in Bartlett and Farnsworth (2002) for the I-80 Site without LCC stabilization (Fig. 15). The profile for Fig. 15 was computed using Janbu’s modulus method and an embankment loading represented by a 2D Boussinesq stress distribution, as discussed previously. Also, the full embankment (both eastbound and westbound lanes) is modeled in Fig. 15, whereas only the first phase of construction (i.e., westbound lanes) is modeled in Fig. 14. Some variation between the two settlement profiles can be expected due to differences in the way the embankment profile was modeled. Thus, the comparison will be restricted to comparing the settlement pattern on the westbound lanes (i.e., left hand) side.
Fig. 14. I-80 Site settlement profile using HNLE model for case with untreated ground

Fig. 15. Predicted settlement profile for I-80 Site based on Janbu’s modulus method
Fig. 15 shows a u-shaped settlement profile having its lowest point near the center of the fill embankment. Consolidation settlement at the face of the MSE wall was estimated to be about 0.4 m (1.3 ft) using the Janbu spreadsheet. The HNLE Sigma/W analysis predicts a maximum settlement of about 0.87 m (2.9 ft) at the wall face.

Thus, near the face of the MSE wall, the results from the two methods vary substantially. This is probably a result of how the induced stresses from the new embankment fill are modeled by each approach. As previously discussed, the HNLE Sigma/W analysis accounts for the presence of the old embankment and its influence on the newly applied load and the resulting stress distribution, whereas the 2D Boussinesq stress distribution formulation cannot. This is a substantial limitation of the 2D Boussinesq model; hence, based on the modeling of both the 200 South and I-80 Sites (see also Fig. 4), this model cannot be recommended for cases where new embankment is being placed atop old embankment.

7.7.2 Treated Ground with One Area Ratio

The initial analysis of the I-80 Site with LCC treated soil was based on the assumption that the treated zone is a composite, homogeneous, linearly elastic material. For this analysis, the drained Poisson’s ratio of the LCC zone, \( \nu_{\text{comp}} \), was assumed to be 0.3, which falls within the range of 0.25 to 0.35 reported in the literature reviewed (Baker, 2000). An equivalent composite drained Young’s modulus, \( E'_{\text{comp}} \), was back-calculated through numerical modeling using the same design assumptions made by the I-15 Reconstruction Project team for the LCC treated soil.

The I-15 Reconstruction Project team used a simplified approach to estimate settlement of the treated area. This assumed that the axial deformations of the columns were equal to the axial deformations of the soil surrounding the columns (i.e., strain compatibility between the column and the untreated surrounding soil). The simplified method is based on design principles described in Broms (1999) and Broms and Bowman (1979). Settlement of the column group, \( s_{\text{group}} \), was computed as the sum of the settlement of \( n \) layers using the following equations:

\[
M_{\text{comp}} = aM_{\text{col}} + (1 - a)M_{\text{soil}}
\]

\[
(M_{\text{comp}} = \sum_{i=1}^{n} \frac{h_{i}q}{M_{\text{comp}}})
\]

where \( h_{i} \) = thickness of the layer; \( q \) = stress increase from the applied load; \( M_{\text{comp}} \) = equivalent composite constrained modulus; \( a \) = area ratio; \( M_{\text{col}} \) = column constrained modulus; and \( M_{\text{soil}} \) = soil constrained modulus. For a column group, the area ratio is computed as
\[ a = \frac{A_{col}}{A_{group}} \]  

where \( A_{col} \) = total column area and \( A_{group} \) = area of column/soil group \((BL)\), as sketched in Fig. 16.

In order to estimate the drained composite constrained modulus, \( M'_{comp} \), in Eq. (15), the drained Young’s modulus for each soil layer, \( E'_{soil} \), must be known. Using the HNLE properties from the 200 South Site calibration, a numerical analysis was performed to represent the idealized deformation behavior of the 200 South Site after placement of the original embankment fill. An iterative procedure was followed to estimate \( E'_{soil} \) for each layer so that a LE analysis produced a settlement profile that gave the best match to the predicted HNLE settlement profile. During this calibration, the ratios of \( E'_{max} \) for the layers were maintained, so that \( E'_{soil} \) was assumed for only one soil layer, the Upper Bonneville unit. The matched curves are shown in Fig. 17.

The back-calculated \( E'_{soil} \) values were used to represent the stiffness of the soil layers within the 21-m (69-ft) deep zone of LCC treatment at the I-80 Site. To be consistent with the simplified approach, \( E'_{soil} \) was converted to \( M'_{soil} \) using elastic theory:

\[ E = \frac{M(1+\nu)(1-2\nu)}{(1-\nu)} \]  

(17)

The back-calculated moduli \( E'_{soil} \) and \( M'_{soil} \) are summarized in Table 7. The back-calculated \( E'_{soil} \) values fall within the ranges of published values for similar types of soil (Bowles, 1996; Baker, 2000). Using the \( M'_{soil} \) values, a thickness-weighted average soil modulus, \( M'_{soil,avg} \), of approximately 6,100 kPa (880 psi) was computed for the soil surrounding the columns in the treatment zone.

During construction, a pressure cell was placed on top of one of the columns (pressure cell PC2) and on the surrounding ground surface (PC1), in the approximate locations shown in Fig. 9 (Bartlett and Farnsworth, 2002). The pressure cell data indicate that the columns carried approximately nine times more load than the surrounding soil at the time of maximum fill placement. Assuming that the applied load is shared between the column and surrounding soil in proportion to the ratio of the material stiffnesses, \( M'_{col} \) was estimated to be \( 9M'_{soil,avg} \), or about 55,000 kPa (8,000 psi). This value is somewhat higher than design value of 30,000 (4,400 psi) used by the I-15 Reconstruction Project team, but it does not seem unreasonable.
**Fig. 16.** Area of column/soil group

**Fig. 17.** Curve-matching to estimate LE properties of soil layers at 200 South Site
Table 7. I-80 Site LE Parameters Assumed for Soil in LCC Treatment Zone

<table>
<thead>
<tr>
<th>Description</th>
<th>Top Depth (m)</th>
<th>Bottom Depth (m)</th>
<th>$E'_{soil}$ (kPa)</th>
<th>$\nu'$</th>
<th>$M'_{soil}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Alluvium</td>
<td>0</td>
<td>5.5</td>
<td>2,580</td>
<td>0.3</td>
<td>3,470</td>
</tr>
<tr>
<td>Upper Bonneville</td>
<td>5.5</td>
<td>9.5</td>
<td>2,150</td>
<td>0.3</td>
<td>2,890</td>
</tr>
<tr>
<td>Interbeds</td>
<td>9.5</td>
<td>11</td>
<td>4,450</td>
<td>0.3</td>
<td>5,990</td>
</tr>
<tr>
<td>Lower Bonneville</td>
<td>11</td>
<td>13</td>
<td>4,470</td>
<td>0.3</td>
<td>6,020</td>
</tr>
<tr>
<td>Deeper Alluvium</td>
<td>13</td>
<td>14</td>
<td>9,330</td>
<td>0.35</td>
<td>14,970</td>
</tr>
<tr>
<td>Cutler Dam</td>
<td>14</td>
<td>21</td>
<td>6,990</td>
<td>0.25</td>
<td>8,390</td>
</tr>
</tbody>
</table>

$1\text{ m} = 3.28\text{ ft}$  $1\text{ kPa} = 0.145\text{ psi}$

The area ratio, $a$, for the treatment zone was estimated as 0.292. The area ratio was computed using Eq. (16) after dividing the plan view into 14 sections, measuring the column and column/soil group areas in each section, and summing these values to determine $A_{col}$ and $A_{group}$ for the entire treatment area.

With estimates of $M'_{soil,avg}$, $M'_{col}$, and $a$, Eq. (15) was used to calculate $M'_{comp}$ as approximately 20,300 kPa (2,900 psi). The corresponding $E'_{comp}$ was evaluated as 15,100 kPa (2,200 psi) for an assumed $\nu'_{comp}$ of 0.3. The estimated values were included in the model to represent the zone of treatment and the treated zone was modeled as an elastic material. The HNLE properties of the surrounding soils and the LE properties of the half-space remained unchanged (Table 6). Typical values of $M'_{col}$ and $E'_{comp}$ were not found in the available literature for comparison with the back-calculated values.

The settlement profile resulting from the HNLE Sigma/W analysis is illustrated in Fig. 18 and is labeled “treated, uncalibrated.” This designation refers to the curve for treated ground with $E'_{1/2sp} = 30,000$ kPa (4,400 psi). As can be seen, the shapes of the measured data and treated, uncalibrated curves are generally similar. However, computed settlements are significantly larger than the measured values; computed values at the MSE wall face are greater by a factor of about 2, and values at the south wall of the building are greater by a factor of about 3. Near the center of the new embankment, at a distance of 60 m (200 ft), the difference is even more pronounced. These discrepancies are likely attributed to inconsistencies between the estimated constitutive model parameters and the actual subsurface material properties.
Of particular note, $M'_{\text{comp}}$ was estimated using a stress concentration ratio of 9, based on the assumption that the LCCs and surrounding soil experience an equivalent strain. It is likely that strains within the surrounding soils are larger than in the LCCs, and thus, the actual stress concentration ratio could be substantially greater than that assumed. If so, the estimated $M'_{\text{comp}}$ used in the numerical analysis could be significantly underpredicted. It is also noted that the variation of stress concentration ratio with depth was not considered. This effect is mainly caused by the increased confining pressure with depth that produces a subsequent increase in stiffness in the column. Also, the potential stiffening effect of the MSE wall reinforcement was not considered. Measured settlements may be comparatively low due, in part, to this stiffening effect, which could inhibit transfer of stresses to the foundation soils. Interestingly, the model with ground treatment predicted greater settlement beneath the building than the analysis without ground treatment. This suggests that the treated ground model with a stiffened zone predicted increased lateral stresses outside the zone of treatment, which in turn produced more settlement.

A similar HNLE analysis was completed for a subsurface profile with 1 m (3 ft) of LE fill directly beneath the embankment. The resulting settlement profile, when compared with the “treated, uncalibrated” curve, exhibited approximately the same settlement at the wall face. This model also showed slightly more settlement left of the face and less settlement beneath the MSE.
wall. However, it is concluded that the overall settlement prediction does not vary substantially with or without the upper 1 m (3 ft) of fill.

The “treated, calibrated” curve in Fig. 18 depicts the analysis results with a stiffened half-space. This step was included in the modeling progression to further constrain the HNLE model results to match the available magnet extensometer data. Through an iterative process, a value for $E'_{1/2sp}$ of 68,000 kPa (9,900 psi) was found to produce a maximum settlement of about 12 mm (0.5 in.) in the soils below the columns. This value corresponds to the actual settlement measured just prior to surcharge removal by the magnet extensometer that was installed at this locale (Fig. 9). The bottom two magnets of this magnet extensometer (i.e., M-SS-2-1) were positioned just below the bottom of the LCCs (Bartlett and Farnsworth, 2002). Of the 23 mm (0.9 in.) of total settlement measured at this location, 12 mm (0.5 in.) or approximately 50% of the settlement occurred at a depth below the bottom of the LCCs (Bartlett and Farnsworth, 2002). Thus, the “calibrated” HNLE model was forced to match this constraint, which was done by stiffening $E'_{1/2sp}$ to 68,000 kPa (9,900 psi).

As indicated by the two “treated” curves in Fig. 18, a calibrated analysis with $E'_{1/2sp}$ about 100% greater than the uncalibrated value resulted in about 10% less settlement at the wall face and about 30% less settlement below the south side of the building. Compared with the measured profile, the “treated, calibrated” profile settlements are greater by a factor of about 2, except towards the center of building where settlements begin to diminish. Thus, although the half-space calibration did lead to somewhat lower settlements, the reduction is not substantial.

7.7.3 Treated Ground with Three Area Ratios

As a final model, it was decided to evaluate the impact of dividing the treated zone into three areas, according to the differing treatment that was used in each zone. To this end, an evaluation was performed with the LCC-treated area modeled as three different linearly elastic zones along the cross-section to represent the three general installation patterns used at the site (Fig. 9). The panel zone in the finite element mesh extended from 2 m (7 ft) beyond the wall face into the embankment about 12 m (39 ft). The triangular spacing pattern extended a horizontal distance of 16 m (53 ft) from the southern edge of the panel zone, and the transition zone extended a horizontal distance of 8 m (26 ft) from the southern boundary of the triangular zone. Composite moduli for each treatment zone were back-calculated using a different area ratio for each zone and assuming $v'_{comp}= 0.3$. Pertinent model and composite material properties used for numerical analyses are listed in Table 8.

Results of the analysis with three different treatment zones and conducted with $E'_{1/2sp}$ = 68,000 kPa (9,900 psi) are plotted in Fig. 19. Compared with the same analysis performed with one area
Table 8. I-80 Site Composite LE Parameters for Three Treatment Zones

<table>
<thead>
<tr>
<th>Zone</th>
<th>$a$</th>
<th>$M'_{\text{comp}}$ (kPa)</th>
<th>$E'_{\text{comp}}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel</td>
<td>0.431</td>
<td>27,100</td>
<td>20,150</td>
</tr>
<tr>
<td>Triangular Spacing</td>
<td>0.247</td>
<td>18,100</td>
<td>13,500</td>
</tr>
<tr>
<td>Transition</td>
<td>0.118</td>
<td>11,850</td>
<td>8,800</td>
</tr>
</tbody>
</table>

1 kPa = 0.145 psi

Fig. 19. I-80 Site settlement-distance curves, three area ratios (ARs) for treated zone
ratio, the analysis with three area ratios did not produce dramatically different results. As was expected, slightly less settlements were predicted in the stiffer panel zone, and somewhat greater settlements were estimated in the softer transition zone. However, it should be noted that the curve for one area ratio is more similar in shape to the measured data curve. In addition, deformations beneath the south wall of the existing building do not appear to be affected by modeling with three treatment zones rather than one.

It is interesting to note that the magnitude of the predicted settlement underneath the adjacent building is not very sensitive to the assumed composite stiffness of the LCC-treated zone. In fact, the “treated, calibrated model” predicts about the same amount of settlement underneath the building as the “untreated, uncalibrated” model (Fig. 19), even though these two models vary greatly in stiffness in the treated zone. Further, for all modeled cases using a treatment zone, the predicted settlements beneath the building and MSE wall exceed the measured values by about a factor of 2.

This suggests that the HNLE model or its inputted parameters, or the simplifications made in modeling the treated ground, are not completely describing the interaction between the MSE wall, LCC treatment zone, and native soil at this site. At this point, there was no further attempt to calibrate the HNLE model, because the best estimate of the model parameters had been produced based on the available data and settlement constraints imposed by the magnet extensometers. Any calibration beyond this would simply be guessing, which is dangerous in a multi-parameter model.

However, the results shown in Fig. 19 and the HNLE model parameters that produced them may still be useful from a design perspective, in that the actual settlement is being overpredicted by the model, which is acceptable from a design standpoint. Nonetheless, there is no guarantee that the model will be “conservative” (i.e., underpredict settlement) in all cases, based on only the results of this single case history. This will need to be verified by additional case histories and modeling.
8.0 CONCLUSIONS

8.1 Summary

Numerous linear elastic (LE) and hyperbolic non-linear elastic (HNLE) modeling exercises were conducted for the 200 South Site and I-80 Sites. In all cases, the HNLE model was a better predictor of the settlement profile that develops beyond the MSE wall face.

Other non-linear methods, such as the Janbu modulus method, did not make reliable predictions of the deformations beyond the face of the MSE wall. This is probably not a limitation of the Janbu modulus method per se, but is likely due to the way the stress distribution is implemented in the spreadsheet developed by Woodward-Clyde Consultants (1997) for the I-15 Reconstruction Project. In this spreadsheet, a 2D Boussinesq stress distribution is used to estimate the vertical stress increase in the foundation soils. This simplified distribution does not appear to give reliable stress predictions for cases where new embankment is being placed on existing embankment, nor does it match the settlement pattern that develops beyond the face of the MSE wall. Perhaps the Janbu modulus method will give more reliable predictions if coupled with a more appropriate stress distribution method.

This study shows that the HNLE model produced reasonably accurate predictions of the measured settlement profile at the 200 South Site. The settlements predicted by the HNLE model match both the magnitude and settlement pattern in a fashion that was superior to any other method that was evaluated. Based on the results shown in Fig. 7, the HNLE model appears to be sufficiently accurate to be used for design purposes in estimating settlement that results from large embankments and MSE walls constructed over untreated, soft, Lake Bonneville deposits.

The HNLE model was also used to model the lime cement treated soil at the I-80 Site. This model used HNLE parameters for the untreated Lake Bonneville deposits, and elastic properties for the LCC treated area. The modulus for the LCC treated area was calculated using a composite modulus that was based on the area ratio of the treated and untreated soil. For this site, the HNLE analysis overpredicted the measured settlements by a factor of about 2, both at the face of the MSE wall and underneath the adjacent building. The reason for the overestimation is not entirely clear, but is possibly attributed to the following factors: (1) uncertainty in using the hyperbolic deformation model to model a consolidation settlement process, (2) uncertainty in the inputted hyperbolic model parameters, (3) simplifications made in modeling the treated ground as a composite, linearly elastic material, and (4) other complex interactions between the MSE wall reinforcement, LCC treatment zone, and the native soil.
8.2 Implementation

It is expected that during future highway construction, several large embankments and MSE walls will be constructed in the Salt Lake and adjacent valleys over soft, Lake Bonneville sediments. Because of alignment constraints, it is also expected that these works will have to be constructed in close proximity to settlement sensitive structures or facilities. For such cases, it is recommended that due consideration be given to the guidelines described herein. If warranted, HNLE numerical modeling may be beneficial.

8.2.1 Guidelines Based on Performance Data

Based on performance data collected for the I-15 settlement arrays, the zone of significant settlement – i.e., total settlement greater than 25 mm (1 in.) – for MSE wall construction is about 1.3 times the maximum height of the MSE wall (i.e., maximum embankment height, including surcharge). Thus, adding a slight factor of safety, it is recommended that all structures located in a zone that is less than 1.5 times the maximum height of the MSE wall be considered as falling within the zone of significant settlement. This zone is measured as the horizontal distance from the finished face of the MSE wall to the point of interest.

For buildings, structures, or facilities located in the settlement zone, other evaluations and construction measures may be required to ensure that these structures remain undamaged, or that their performance or function is not compromised. Possible design alternative are: (1) alignment modification, (2) embankment geometry changes, (3) use of lightweight embankment fill (e.g., scoria), (4) use of geofoam, and (5) other ground treatment and ground modification techniques.

It should be noted that the above settlement guidelines apply for MSE wall construction on soft soils, where there is to be a widening of the embankment beyond the existing roadway footprint. Also, this project did not evaluate any cases of sloped (i.e., 2H:1V) embankment, or cases where the new wall is to be constructed on the slope of the existing embankment. The UDOT Research settlement arrays do not include any such cases, thus there are no guidelines to offer for these cases based solely on the performance data. However, from a screening standpoint, it is expected that the above recommendations will conservatively bound the zone of significant settlement for sloped embankments, if the measured horizontal distance is taken from the toe of the new slope. Variations in slope and embankment geometry and their position relative to existing embankments should be further explored using the HNLE model, as appropriate.
8.2.2 HNLE Numerical Modeling

This research indicates that a HNLE model can produce a reasonable match to the settlement performance data, as was done for the 200 South Site. For this analysis, the hyperbolic parameters given in Table 3 were used. These parameters were calibrated for the various layers at the 200 South Site and in aggregate provide a satisfactory match. These parameters make available a reasonable starting point for prediction of settlement at other sites underlain by soft, Lake Bonneville deposits. For such sites, the following process is recommended.

1. Use the screening criterion discussed above to decide if further settlement evaluations are wanted for the building, facility or structure.

2. Obtain the required geotechnical design data for the site. These should include oedometer testing, Atterberg limits, soil unit weights, moisier content, and CPT and Seismic CPT (SCPT) data. Also, for large projects or for evaluations adjacent to sensitive facilities where economic impacts are considerable, it is recommended that the HNLE model parameters be developed from laboratory testing using consolidated drained (CD) triaxial testing as described by Duncan et al. (1980). For smaller projects, CD triaxial testing may not be warranted, and estimates may be made from other sources as has been done in this report. It is also recommended that the project team obtain and evaluate any past settlement performance available at or near the site for similar loading conditions. These types of data are invaluable in calibrating the HNLE model parameters to site-specific conditions. Sources of historic settlement data may include: settlement plates, manometer and survey data points from previous construction, and/or performance monitoring.

3. Use the CPT data to stratify the subsurface soil profile and calculate a thickness for each layer. If applicable, divided the layered system into a scheme that is similar to that used in this report. (However, this system may have to be somewhat altered, depending on the location and the subsurface geology present at the site.) Once the system has been stratified, assign the appropriate HNLE parameters to each layer, using as a starting point the properties given in Table 3 for similar soil types.

4. Adjust the K values for each layer using the iterative process described in this report. The constraint placed on the HNLE model is that the ratio of the initial tangent moduli, $E_i$, for the various layers must be consistent with the $E'_{max}$ ratios calculated from the shear wave measurements obtained from the SCPT. Also, calculate the bulk modulus number, $K_b$, for each layer in the model using Eq. (5). To apply this equation, estimates of $B$ are needed and must be consistent for the appropriate stress level. Hence, $B$ values
should be calculated from Eq. (6) using the estimated values of $E_i$ and typical values of $\nu'$ (Table 1). Eq. (5) can then be applied to calculate $K_b$ for the appropriate stress level calculated from Eq. (13). This procedure guarantees that all HNLE model parameters are internally consistent and agree with typical values of $\nu'$ at the appropriate stress level.

5) Calculate the unload-reload modulus, $K_{ur}$, for each layer. It is recommended that a value that is four times greater than $K_b$ be used. This approximation is based on settlement measurements made during the original embankment construction in the 1960s.

6) If settlement performance data are available for the site or from a nearby locale, it is recommended that the initial $E_i$ values and corresponding modulus numbers be adjusted in an iterative fashion, preserving the corresponding $E'_{max}$ ratios, until the Sigma/W analysis produces a settlement profile that reasonably matches the end of primary settlement at the point of measurement using the loading conditions that produced the measured settlement. Generally, the point of measurement from historical UDOT settlement data is the centerline of the embankment.

7) If settlement performance data are not available for calibration, it is recommended that the end of primary settlement be estimated using laboratory oedometer data and conventional 1D consolidation theory for the centerline of the embankment, where the assumption of 1D loading conditions is generally valid. The HNLE model can then be calibrated to match the calculated 1D settlement at the center of the embankment. For this case, it is not essential that the modeled embankment geometry match the to-be-constructed embankment geometry. The purpose of this calibration is to ensure that the analysis with HNLE parameters can reasonably predict the amount of settlement obtained from the laboratory oedometer tests for a 1D loading case. For these calculations, the actual embankment geometry modeled is irrelevant as long as a 1D loading condition is a reasonable assumption for the centerline of the embankment. The reasonableness of this assumption can be verified by inspecting the increase in vertical stress produced by the numerical model to verify that it approximates a 1D loading case.

8) Once calibrated, the model should be used to estimate the settlement pattern that develops away from the embankment or MSE wall for the actual loading conditions and geometries. The model can also be applied as a design tool for preliminary design to assess potential impacts caused by embankment geometry changes, the use of lightweight fill, or the use of geofoam in the design. The authors believe that the use of the HNLE model to explore these “what if” questions is one of the most powerful uses of the model.
In conclusion, results of this research indicate that numerical analysis with the HNLE constitutive model can be employed by practicing engineers to reasonably predict primary settlements below and beyond embankments and MSE walls, provided that realistic parameters are used in the model. The criticality of the project must be assessed to determine if the cost and time required for these analyses are justified.

8.3 Limitations

The HNLE model appears to be a reliable design tool, but it carries with it significant modeling overhead. It is a challenge to derive the inputs, which are interrelated, from data typically gathered by practicing engineers. Analyses with typical HNLE properties must be approached with some caution, unless these have been calibrated by site-specific field performance measurements. However, the authors believe the best approach is to obtain the HNLE parameters from laboratory tests using site-specific undisturbed soil samples. Because of this, additional laboratory testing should be conducted as part of any future analysis.

Also, many assumptions were required throughout the course of this study. Although every attempt was made to incorporate reasonable assumptions, based on review of available subsurface and performance data, published typical values, and past experience, it is expected that some uncertainty has been incorporated into the analyses. Consequently, the developed models may not fully represent all field conditions and their variability, but it is believed that all “first order” effects have been captured.

Other limitations of this research are identified below.

(1) It was assumed that subsurface profiles at the 200 South and I-80 Sites consisted of a few discrete layers. In actuality, the profiles are highly heterogeneous, with numerous interbeds of clay, silt, and sand. These interbedded layers could not be defined in the model to a high level of detail because of the spacing of $v_s$ measurements and limitations of the subsurface data. Furthermore, a highly stratified model would begin to compromise the usefulness of the modeling approach to practicing engineers.

(2) Effects of PV drains were not included in analyses of the 200 South Site, where they were used. PV drains alter drainage patterns, affect consolidation rates, increase stiffening of the consolidating layers, and hence change the stress distributions within the subsurface soils. Such effects require a coupled, effective stress versus strain model that is generally beyond the state-of-practice analyses for most embankment projects. Early on in this project, it was decided to model consolidation settlement as a “deformation problem,” rather than a consolidation process. This was a conscious decision, in order to simplify the analyses.
(3) The potential stiffening effects of the MSE wall horizontal reinforcement were not considered in the analyses. Perhaps, further investigation is needed to determine the magnitude of these interactions and an appropriate means of including these interactions in the numerical model.
9.0 REFERENCES


