# Rapid Construction and Settlement Behavior of Embankment Systems on Soft Foundation Soils

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**Abstract:** The I-15 Reconstruction Project in Salt Lake City, Utah required rapid embankment construction in an urban environment atop soft lacustrine soils. These soils are compressible, have low shear strength, and require significant time to complete primary consolidation settlement. Because of this, innovative embankment systems and foundation treatments were employed to complete construction within the approved budget and demanding schedule constraints. This paper evaluates and compares the construction time, cost, and performance of three embankment/foundation systems used on this project: (1) one-stage mechanically stabilized earth (MSE) wall supported by lime cement columns; (2) expanded polystyrene (geofoam) embankment with tilt-up panel fascia walls; and (3) two-stage MSE wall with prefabricated vertical drain installation and surcharging. Of the technologies evaluated, the geofoam embankment had the best performance based on settlement and rapid construction time considerations, but is more costly to construct than a two-stage MSE wall with PV drain foundation treatment. The one-stage MSE wall with lime cement treated soil was the most costly, and did not perform as well as expected; thus, it had only limited use on the project.

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## Introduction

Constructing large walls and embankments over soft-soil sites can be challenging in an urban setting, as special care must be taken to ensure that primary consolidation and postconstruction secondary settlements will not damage adjacent structures and utilities. In many instances, this means that methods to minimize the amount of settlement must be employed. This can be accomplished either by using a smaller loading condition or by altering the foundation conditions to withstand the required load. In either case, the net goal is to reduce the potential settlements to an acceptable magnitude. Furthermore, contracting and construction methods that speed up the construction process are also often sought after, thus, reducing the construction time placed on the facility. The objective of this paper is to present different aspects of some nontypical embankment systems over compressible soil, using the I-15 Reconstruction Project in Salt Lake City, Utah as a case history. Furthermore, this paper illustrates how the nontypical embankment systems are used to address the challenges of constructing over soft-soil sites in an urban setting.

The I-15 Reconstruction Project utilized three basic approaches for dealing with the anticipated magnitude of settlement from the soft compressible soils that were prevalent beneath much of the project. The first approach was to use surcharging with mechanically stabilized earth (MSE) walls or earthen embankments over prefabricated vertical (PV) drain treated foundation soils and allow the primary consolidation settlements to take place. The second approach was to essentially eliminate any potential foundation settlement by using geofoam as a light-weight fill embankment and, thus, greatly minimize the loading condition imposed on the foundation soils. The third approach involved strengthening the foundation soils by installing lime cement columns prior to placing an MSE wall, thus, reducing the magnitude of settlement within the stiffened foundation soils.

This paper provides an overview of these different geotechnologies as they were utilized on the I-15 Reconstruction Project. First, a comparison of the construction costs and schedule for the geotechnologies is performed. The performance of the embankments with respect to the construction and long-term settlements is then discussed. Finally, conclusions are drawn with respect to the use of each geotechnology.

## Project Background

The I-15 Project was a fast-paced reconstruction project that began during the spring of 1998 and ended in the fall of 2001, just

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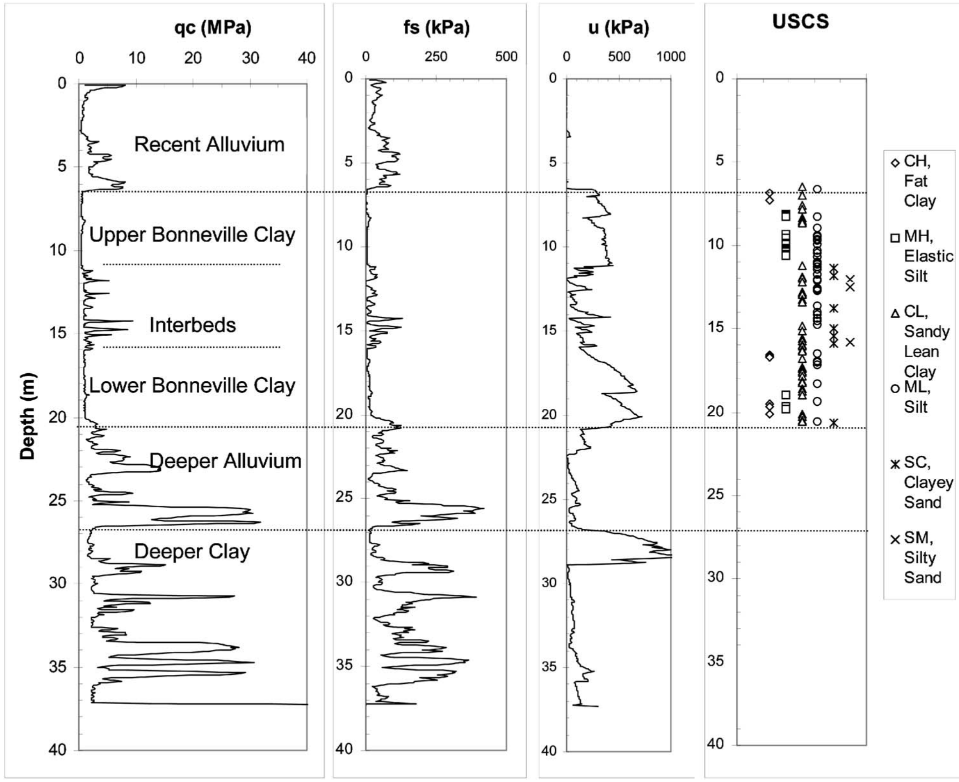


**Fig. 1.** Typical cone penetrometer log and soil descriptions for downtown segment of I-15 Reconstruction Project, Salt Lake City, Utah

prior to the 2002 Winter Olympic Games in Salt Lake City, Utah. At that time, it was the largest public highway construction project to be accomplished using a design-build project delivery system. During this 3.5-year period, the design-build consortium demolished and rebuilt 26 km of urban interstate, widening the roadway from six up to 12 lanes at a total cost of about \$1.4 billion. A large part of this cost was spent erecting 144 overpass bridge structures, constructing 160 MSE retaining walls, and placing 3.8 million $m^3$ of new embankment. The design-build contract featured a 50-year design life and an optional 10-year corrective maintenance agreement.

The strict project completion date presented unique challenges to the design-build team. Perhaps the most demanding was developing strategies to address the impacts of consolidation settlement in the northern segment of the project near the downtown area. Here, compressible, fine-grained lacustrine sediments deposited by Pleistocene-age Lake Bonneville underlie about 5 m of Holocene alluvium (Fig. 1). The lacustrine sediments are approximately 15 m thick, consist of interbedded silty clay and clayey silt (CL, ML), plastic clays and silts (CH, MH), and fine clayey and silty sands (SC, SM), and are lightly overconsolidated ($\text{OCR} \approx 1.5$). Interbedded, subaqueous silts, fine sands, and low plasticity clays are found in the middle of the Lake Bonneville sediments and separate the upper and lower Lake Bonneville clays (Fig. 1). These upper and lower clay units are compressible [compression ratio ($C_c/(1+e_o)$) from 0.1 to 0.35], have relatively low undrained shear strength (25 to 50 kPa), and require substantial time to complete primary consolidation. Settlement records from the 1960s construction of I-15 show that a typical 8 to 10 m high embankment underwent 1 to 1.5 m of primary consolidation settlement over a period of 2 to 3 years. These large magnitudes of settlement and long consolidation settlement durations can be attributed directly to the soft thick compressible Lake Bonneville clay layers. In the 1960s, the bridge foundations, bridge, approaches, and pavement were not placed until such settlement was essentially finished.

The fast-paced reconstruction from 1998 through 2001 could not accommodate these rather lengthy settlement durations. The contract required that two lanes of traffic in each direction be maintained throughout construction. This essentially meant that each direction of the interstate had to be rebuilt in a two-year period, making the reconstruction essentially a two-phased project, with each phase lasting about two years. Thus, innovative technologies and construction methods were needed to either minimize settlement (i.e., maintain stresses within the recompression range) or induce primary settlements to occur within a preload period of 6 to 12 months, so that bridge construction and paving operations could proceed on schedule. Owing to its innovative use of geotechnologies and successful implementation of a design-build project delivery system, the I-15 reconstruction project received the ASCE Outstanding Civil Engineering Achievement Award for 2002.

## Geotechnologies

The design-build team employed fairly innovative and less common methods to successfully complete the project on time and within the initially approved budget. This paper focuses on the construction and long-term settlement performance of three I-15 embankment systems: (1) a one-stage MSE wall supported by lime cement columnsl; (2) a geofoam embankment with a tilt-up panel wall on strip footing; and (3) a two-stage MSE wall on foundation soils with PV drain installation. Details of these three embankment construction alternatives are shown in Fig. 2.

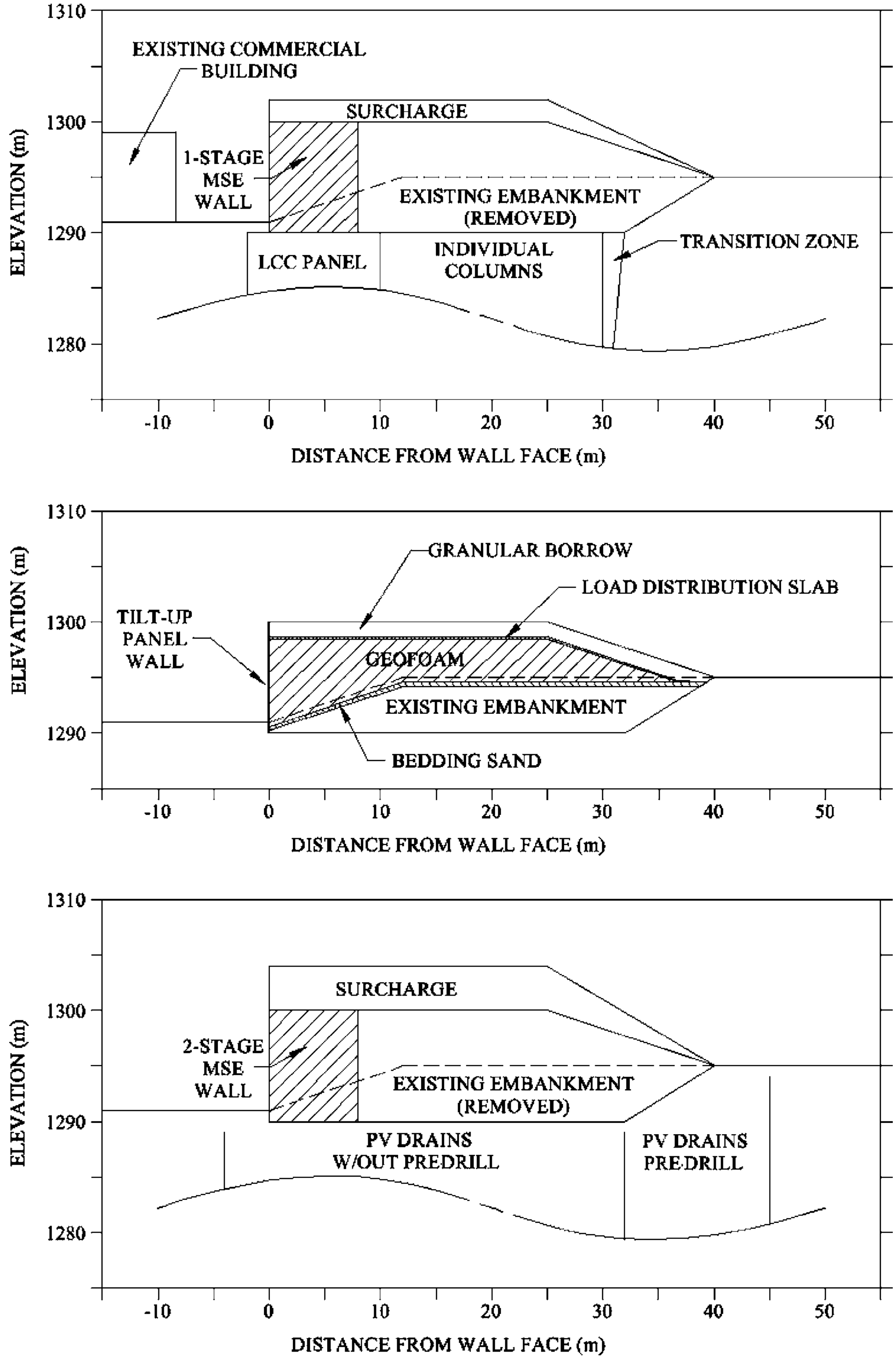


**Fig. 2.** Comparative cross sections for various geotechnologies including (a) one-stage MSE wall with LCC stabilized soil; (b) geofoam wall/embankment; and (c) two-stage MSE wall with PV drains

The implementation of these technologies is further discussed in Bartlett et al. (1999) and Saye et al. (2001a,b).

The I-15 project settlement goals and the anticipated performance and estimated construction time for each geotechnology were critical factors in the selection process. The two-stage MSE wall on foundation soils with PV drain installation was the most widely used on the I-15 project and is a baseline technology against which the performance of the two other technologies is compared. The construction cost, time, and settlement performance of the first two technologies are being highlighted because of their relatively new introduction into U.S. practice.

### *One-Stage MSE Wall and Embankment Supported by Lime Cement Columns*

Lime cement column (LCC) installation is a soil mixing technique used for soft foundation improvement. Lime and cement are mechanically mixed in situ with the foundation soils to create stiffer columns of treated soil. According to the Federal Highway Administration (FHWA 1999), the basic concept of stabilizing foundation soils by mixing lime in situ was first introduced in Scandinavia in 1975. In the 1990s, Sweden began making extensive use of mixing both lime and cement in situ for stabilization of soft soils, "mainly for the reduction of settlements and improvement of stability for the construction of new roads and railroads." The first lime column system to be used in the United States occurred in 1992 "as part of a research effort by the Florida Institute of Phosphate Research." The use of lime and cement soil mixing on the I-15 Reconstruction Project was one of the first applications within the United States.

The lime cement columns used on the I-15 Reconstruction Project were constructed with a reagent admixture of 15% lime/85% cement and injected at a mass concentration of 125 $kg/m^3$ of untreated soil. The columns were constructed by inserting a mixing tool to the target depth, and while withdrawing the tool, injecting the dry lime and cement within the soil to be mixed in situ. For this project, the columns were installed to a depth of 20 m and were either 0.8 m or 0.6 m in diameter. The overall spacing pattern of the columns was quite complex, as shown in Bartlett and Farnsworth (2002). In general, the LCC spacing consisted of either 0.8 m diam intersecting columns that created panels of reinforced soil with panels spaced 2 m apart or individual 0.6 m diam columns spaced approximately between 1 to 2 m apart in either a triangular or rectangular pattern.

One-stage MSE wall systems, on the other hand, have been commonly used throughout many countries (including the United States) since the 1970s. The use of the one-stage MSE wall is not unique to this project, other than being placed atop an uncommon foundation treatment. A one-stage MSE wall consists of attaching the horizontal reinforcement directly to the concrete facing panels in one phase of construction. This process involves several different steps. The first row of facing panels is erected and then backfill is placed and compacted to the first layer of reinforcement. The first layer of horizontal reinforcement is then placed over the backfill and connected to the facing panels. Another layer of backfill is placed and compacted, with subsequent facing panels, reinforcement and backfill being placed as the wall is constructed. Because the facing panels can be damaged by excessive differential settlement, this type of wall was only used where total primary consolidation settlement was expected to be less than 250 mm. The one-stage MSE walls on the I-15 Reconstruction Project were constructed with galvanized welded wire metallic horizontal reinforcing grids and 1 m $\times$ 1.5 m rectangular precast concrete facing panels.

Lime cement column (LCC) treated foundation soil was used at one location on the I-15 Reconstruction Project to reduce consolidation settlement and improve the shear strength of the Lake Bonneville clays (Fig. 2). A 200 m long, 10 m high one-stage MSE wall was needed to form the bridge approach for a pile-supported overpass structure at the I-80 intersection with I-15. Surcharge was also placed atop the LCC treated MSE wall to overconsolidate the foundation soils and reduce the amount of postconstruction settlements (Fig. 2). Saye et al. (2001a) and Bartlett and Farnsworth (2002) further discuss the design and construction of the LCC treated area.

### *Expanded Polystyrene (Geofoam) Embankment*

EPS block geofoam has been used as a light weight embankment fill since at least 1972, where it was used for a roadway project in Norway (NCHRP 2004a). Subsequently, use continued throughout Scandinavia and began to spread to the rest of Europe and Japan. In Japan, the first lightweight fill project using geofoam occurred in about 1985, but after 10 years, Japan's use comprised approximately 50% of worldwide usage. Some of the earliest documented applications of geofoam being used for settlement mitigation within the United States include construction of the Carousel Mall in Syracuse, New York in 1990 (Negussey and Sun 1996) and for an emergency truck ramp at the Kaneohe Interchange in Oahu, Hawaii in 1995 (Mimura and Kimura 1995). The use of geofoam on the I-15 Reconstruction Project was the largest settlement-related application to date within the United States. Geofoam embankment design and performance from the I-15 Reconstruction Project are further discussed in Bartlett et al. (1999), (2001a), Negussey et al. (2001), Negussey and Stuedlein (2003), and Stuedlein (2003). General design and construction considerations for geofoam embankments are also found in NCHRP (2004a,b).

Constructing with geofoam blocks as lightweight fill embankment is a fairly straightforward process. The site is first leveled and a layer of bedding sand is placed. Geofoam blocks are then stacked with additional bedding sand filling the gap between the geofoam and the backslope. A load distribution slab consisting of reinforced concrete is constructed atop the geofoam, followed by a small layer of fill, and finally the pavement section. A tilt-up panel wall is placed to cover and protect the exposed face.

EPS geofoam with a nominal density of 20 $kg/m^3$ was used for the lightweight embankment construction on the I-15 Reconstruction Project. The contract specifications did not require trimming of the geofoam block by the manufacturer. As necessary, individual geofoam blocks were cut on site to desired shapes and sizes. The average unconfined compressive strength at 10% strain of standard 50 mm cube samples was 110 kPa. A working stress of 40% of the average strength at 10% strain was allowed for the overlying fill, pavement pressure, and transient loading.

Approximately 100,000 $m^3$ of geofoam embankment was placed on the I-15 Project at several locales. The primary use of geofoam on the I-15 Project was as lightweight embankment over existing buried utilities to minimize settlements. At many locations, buried water, sewer, gas, and communication lines either traversed or paralleled the roadway alignment where the embankment was to be widened, or where the roadway grade needed to be raised. However, if conventional embankment were placed atop these utilities, they would be damaged from the primary consolidation settlement of the underlying Lake Bonneville sediments. Thus, these utilities could either be relocated, which was costly and time consuming, or other methods had to be employed to protect them in situ.

Ultimately, the design team selected geofoam embankment for buried utility corridors, due to its extremely light unit weight, 20 $kg/m^3$. The use of geofoam enabled construction of 8 to 10 m high embankments over existing utilities without causing a significant increase in vertical stress in the foundation soils; hence, damaging primary consolidation settlement did not develop. An increase in vertical stress could have been completely avoided by subexcavating and removing the same weight of foundation soil required to compensate for the combined weight of the geofoam, load distribution slab, granular borrow, roadbase, and concrete pavement placed atop the geofoam. For the I-15 project, this required about 2 m of subexcavation; however, only 1 m of subexcavation was done in most areas due to shallow groundwater. Thus, the vertical stress was slightly increased in the foundation soils, such that all settlement was in recompression and was acceptably small, partly on account of slight overconsolidation of the foundation soils due to prior aging and desiccation effects.

Geofoam embankment was also used to expedite the construction in a few critical locations where the project schedule did not allow for conventional embankment construction and the requisite 6 to 12 month waiting period for accelerated primary consolidation settlement with PV drains. The use of geofoam at these locations completely eliminated the settlement time associated with placement of conventional embankment.

### *Two-Stage MSE Wall and Embankment with PV Drains and Surcharge*

The use of two-stage MSE walls is quite common in practice today. However, two-stage walls are generally used for applications where large magnitudes of settlement are anticipated. For the I-15 Reconstruction Project, two-stage MSE walls were prescribed for locations where total settlements were expected to be larger than 250 mm.

The first stage of a two-stage MSE wall is constructed much like the one-stage MSE wall previously described. However, in the one-stage MSE wall where the reinforcing straps are attached directly to the precast concrete facing panel, in the two-stage MSE wall a galvanized welded wire metallic grid and geofabric backing are used as the wall face. The wall is constructed, including any surcharge, and then the majority of the primary consolidation settlement is allowed to occur. The second stage then consists of removal of the surcharge and attaching precast concrete facing panels to the welded wire face via threaded couplers. The welded wire wall face used in the first stage can withstand much more deformation from the primary consolidation settlement than if the precast concrete facing panels had been used. The two-stage MSE walls on the I-15 Reconstruction Project were also constructed with galvanized welded wire metallic horizontal reinforcing grids and $1\ \text{m} \times 1.5\ \text{m}$ rectangular precast concrete facing panels.

Prefabricated vertical (PV) drains, in their present form, have been used worldwide (including the United States) since the 1970s (FHWA 1999). PV drains are installed through thick soft soil layers to expedite settlement by providing a shorter horizontal drainage path for which the excess pore water pressures can dissipate. This greatly decreases the settlement time of the soft foundation soils, which in turn also accelerates the rate of strength gain of the foundation soils.

Where the construction schedule allowed and buried utilities were not present, the I-15 Reconstruction Project made extensive use of two-stage MSE walls and staged embankment construction (Saye et al. 2001b). Prior to constructing the MSE walls, most of the existing embankment was removed and PV drains were installed in the foundation soils at 1.5 to 1.75 m triangular spacing to accelerate the duration of primary consolidation to about 90 days per each embankment stage. In addition, surcharging was used atop the MSE wall and adjoining embankment to reduce the amount of secondary settlement using the technique of Stewart et al. (1994) and site-specific testing of the Lake Bonneville clays by Ng (1998). Approximately 3 to 4 m of surcharge was added for typical 8 to 10 m high embankments. The surcharge was designed to reduce secondary settlements to about 76 mm in a 10-year postconstruction period.

## Comparison of Construction Costs

Construction cost was an important factor leading to the selection and implementation of each geotechnology. Table 1 provides a cost comparison using typical I-15 unit costs (year 2000 values) for 10 m length of embankment/wall and the typical cross sections shown in Fig. 2. (These cross sections are based on typical construction and layout details for a half-width or one direction cross section of I-15.) The LCC treated cross section [Fig. 2(a)] is an actual cross section from the project. Based on this section, comparative cost estimates for the other two technologies were developed using similar geometries. The costs and time of the road base and concrete pavement construction were the same for each alternative and have not been included in Table 1.

The total cost of the LCC treated soil and one-stage MSE wall was about $160,000 per 10 m length of wall/embankment. Prior to the LCC treatment, the existing embankment was removed. The column installation pattern was somewhat complex and included 20 m long columns of two diameters (0.8 m and 0.6 m). The columns in the panel and individual column zones were installed to a depth of 20 m and to shallower depths in the transition zone (Fig. 2). Based on the actual installation pattern (Bartlett and Farnsworth 2002), a total of 2,580 and 3,260 linear m of 0.8 m and 0.6 m columns, respectively, were installed for 10 m length of wall/embankment. A 2 m surcharge fill was also placed atop the one-stage MSE wall and later removed before constructing the pavement section.

If geofoam embankment had been used at this location, the total cost of the embankment/wall would have been about $120,000 per 10 m length of wall (Table 1). Site preparation for geofoam installation included subexcavation and placement of approximately 0.3 m of bedding sand as a leveling surface for block placement. A 0.15 m reinforced load distribution slab was poured atop the geofoam and a tilt-up panel wall on strip footing was erected to protect the vertical face (Fig. 2). In addition, up to 1 m of granular borrow and/or scoria was typically placed atop the load distribution slab to establish the final subgrade elevation.

If a two-stage MSE wall with PV drain treatment had been used at this site, the cost would have been about $100,000 per 10 m length of wall (Table 1). Foundation preparation for this system included removal of much of the existing embankment to allow for PV drain installation and construction of the MSE wall. In addition, predrilling of pilot holes for the PV drain installation was required in some areas where the embankment was not removed. A 0.3 m sand layer was placed to serve as a drainage layer for the PV drains. In addition, 4 m of surcharge placement and removal have been included in this cost estimate.

When comparing the relative costs for each geotechnology, the one-stage MSE wall with LCC treated foundation soil costs around 60% more than conventional construction (i.e., the two-stage MSE wall with PV drains) and 30% more than the geofoam embankment system. Much of that cost (approximately 60%) is from the LCC foundation treatment alone. The one-stage wall itself is actually cheaper to construct than a two-stage MSE wall. Furthermore, the LCC foundation treatment costs alone are nearly identical to the total cost of constructing a two-stage MSE wall with PV drains. The geofoam system costs are about 20% higher than those of conventional construction, but the unit price of geofoam block has significantly risen in the last few years due to increases in petroleum prices. Geofoam embankments were often utilized in locations where utilities would have otherwise needed to be relocated. Utility relocation costs have not been included in this cost analysis. It is recognized that the cost analysis presented

**Table 1.** Cost and Time of Construction Comparison for 10 m of Wall/Embankment Length Using Typical Cross Sections (Fig. 2) from the I-15 Reconstruction Project

| Geotechnology | Various construction activities (*With typical unit cost*) | Associated costs (Year 2000) | Time (Months) |
|---|---|---|---|
| Lime cement columns | Existing embankment removal ($6/m$^3$) | $9,500 | 0.25 |
| | Lime cement column installation (0.8 m *column*—$17.5/m, 0.6 m *column*—$16/m) | $97,000 | 2 |
| | One-stage MSE wall/embankment construction ($200/m$^2$ *wall face*) | $43,500 | 1 |
| | One-stage embankment construction, surcharging, settlement, and removal (*placement*—$9/m$^3$, *removal* $6/m$^3$) | $10,000 | 8.75 |
| | **Total=** | **$160,000** | **12** |
| Geofoam | Existing embankment removal ($6/m$^3$) | $1,500 | 0.25 |
| | Bedding sand ($7/ton, *with 1 crew 1* week) | $5,500 | 0.25 |
| | Geofoam embankment ($45/m$^3$) | $65,000 | 2 |
| | Tilt-up panel wall ($200/m$^2$ *wall face*) | $20,000 | 0.75 |
| | Load distribution slab ($60/m$^2$ *surface area*) | $23,000 | 0.5 |
| | Embankment above geofoam ($9/m$^3$) | $5,000 | 0.25 |
| | **Total=** | **$120,000** | **4** |
| Two-stage MSE wall | Existing embankment removal ($6/m$^3$) | $9,500 | 0.5 |
| | Bedding sand ($7/ton, *1 crew 2* days) | $2,500 | |
| | PV drain installation (1.5 m triangular spacing) ($1.5/m *without predrilling*, $3/m *with predrilling*) | $14,000 | 1.5 |
| | Wall/embankment construction and settlement time ($300/m$^2$ *wall face*, $9/m$^3$ *embankment*) | $54,000 | 2 |
| | Three-stage embankment construction, surcharging, settlement time, and removal (*placement*—$9/m$^3$, *removal* $6/m$^3$) | $20,000 | 10 |
| | **Total=** | **$100,000** | **14** |

above also neglects the costs associated with the time of construction. If costs associated with utility relocations can be avoided, or if rapid construction is required, the geofoam system may be cost competitive with conventional construction. It should be noted that the relative costs provided in this cost analysis are from this specific case study, and that they are intended only to provide a relative comparison of the cost for each geotechnology system. They were put together with the unit costs that were specific to this project. Additionally, some of the costs are dependent upon the site conditions. For example, the cost of the LCC foundation treatment was dependent upon the exact amount of columns that were required at this location.

## Comparison of Construction Schedule

The time required to construct each geotechnology was a very important selection criterion. Table 1 also presents representative construction times for a typical reach of wall/embankment from the I-15 Reconstruction Project. Although these durations are for a major project, where several walls and embankments were being constructed simultaneously, the relative construction time for each geotechnology may be similar for smaller projects.

The following conclusion can be made regarding typical construction times; geofoam embankments can be constructed much more rapidly (around 3 to 3.5 times faster) than the other two technologies. MSE walls with LCC or PV drain treated soils require much longer construction times as a consequence of the time lost waiting for completion of consolidation settlement. To minimize this impact, two-stage walls with PV drains were constructed during the spring and summer months, so that consolidation settlement could take place during the fall and winter, thus, allowing for pavement to be placed during the next construction season. Unexpectedly, the LCC treated soil required about 8 to 9 months to complete consolidation settlement (Table 1). A more rapid deformation of the treated soils was anticipated, but the LCC columns appear to have induced consolidation settlement in the deeper clays below the 20 m deep treated zone (Fig. 1). The design-build contractor did not use LCC treatment at other project locations due to equipment problems, installation rates, and treatment costs that were not as favorable as originally anticipated.

## Settlement Criteria and Performance Monitoring

Project requirements and settlement performance goals played a vital role in selecting, designing, and constructing each geotechnology. The settlement criteria and performance observations for each method of embankment construction are presented below.

A team of UDOT and design-build personnel established the performance goals and design criteria. In regard to settlement performance of earthen embankments and MSE walls, these were: (1) potentially damaging settlement to adjacent structures and facilities should not extend beyond the UDOT right of way; (2) existing utilities located within zones of significant settlement should either be relocated or protected in place; and (3) the total postconstruction settlement of the embankments, MSE walls, and bridge approaches should be limited to a maximum 76 mm during a 10-year postconstruction period. For the I-15 project, the postconstruction period started once the concrete pavement was placed. The bridge foundations were designed for 25 mm of postconstruction settlement and the 50 mm of differential settlement was to be accommodated by a 15 m long bridge approach slab.

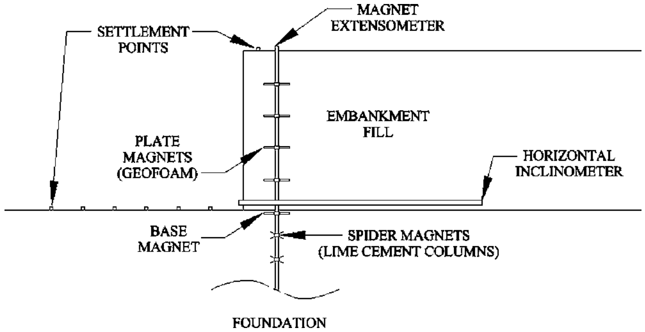


**Fig. 3.** Typical setup for UDOT long-term settlement monitoring arrays, Salt Lake City, Utah

The first criterion was established to protect adjacent structures and facilities beyond the project limits, but in a few cases where settlements became excessive, adjacent properties were purchased, repaired, or the owner was compensated. The design-build contractor proposed the third criterion to UDOT as an achievable goal based on the accelerated construction schedule, cost of surcharging embankments, and the anticipated performance of the foundation soils. UDOT via the design-build contracting mechanism encouraged innovative construction techniques and practices to meet these performance goals and criteria.

Under the loading criterion established for EPS geofoam, end of construction settlements of up to 1% strain in the geofoam and postconstruction settlements of up to 2% strain after a period of 50 years in the geofoam were anticipated. Deformations measured by magnet extensometers placed in the geofoam fill included elastic compression of the geofoam, gap closure between geofoam block layers, and seating of extensometer plates. These deformations resulted primarily from placement of the overlying load distribution slab, subbase and base materials, and pavement.

### *Instrumentation*

Two instrumentation and monitoring programs were implemented for the I-15 Reconstruction Project. The first was developed by the design-build contractor and expedited embankment construction by assessing foundation/embankment stability and monitoring the progression of primary consolidation (Bartlett et al. 2001b; Saye and Ladd 2004). It was also used to ensure that the surcharged fill was left in place for sufficient duration so as to meet the 76 mm in 10 years postconstruction settlement criterion for embankments. A second program was implemented by UDOT to monitor and evaluate the construction and postconstruction performance of innovative embankment/MSE wall construction at 12 array sites for a 10-year postconstruction period (Bartlett and Farnsworth 2004). This paper discusses results for some of these arrays for an approximate 5- to 7-year postconstruction monitoring period. Additional details and results from the UDOT evaluations can be found in Bartlett and Alcorn (2004), Bartlett and Farnsworth (2002), Bartlett et al. (2001a), Negussey et al. (2001), Negussey and Stuedlein (2003), and Stuedlein (2003).

The UDOT program used three basic types of instrumentation technologies for measuring settlement: (1) monuments with high-precision surveying; (2) magnet extensometers; and (3) horizontal inclinometers. Fig. 3 shows a typical wall array that includes settlement points placed within the footing and in the adjacent ground away from the base of the wall, and survey plugs placed in the pavement atop the wall. The intent of the settlement points was to provide, in conjunction with a horizontal inclinometer, a complete settlement profile cross section through the embankment and away from the wall. The settlement points placed in the ground were 900 mm long and partially cased with an oversized pipe to prevent movement from frost heave. All settlement points and monuments were surveyed with a self-reading digital level with submm precision. The survey circuits were closed on stable off-site benchmarks and adjusted so as to have accuracy of about 1 mm, or less. Additionally, the ends of the extensometer and inclinometer casing were surveyed and the data were adjusted for their movements. Magnet extensometers were installed within the foundation soils in embankment areas, inside and outside of the lime cement column foundation treated area, and within the geofoam embankments. Plate magnets and/or spider magnets were placed at strategic levels within the foundation soils or the embankment. The locations of the magnets were targeted for boundary conditions (top and bottom of embankment and bottom of instrument) and changes within the subsurface stratigraphy such as the interface between clay and granular layers, thus, bracketing the soft compressible clay layers. The positions of the magnets are periodically measured with a probe to record the relative compression between the detector magnets. The position of each magnet can be read with an accuracy of about 3 mm. Additionally, horizontal inclinometers were placed at the top of the foundation soils and within the geofoam embankment to provide a continuous settlement profile through the embankment. The horizontal inclinometer has a system accuracy of about 6 mm per 25 m of length. Vibrating-wire total pressure cell plates were installed at many arrays, but these data will not be discussed herein. All instrumentation was placed at full height embankment areas away from transition zones (i.e., geofoam/MSE wall transitions or bridges) to avoid complex edge effects and at locations that provided accessibility, long-term protection of the instrumentation, and safety of those gathering the data.

Additional settlement reference points and magnet extensometers were also installed within relatively large earthen embankments (2400 South, 900 West, and 400 South Streets) to monitor postconstruction settlements. These embankments were generally 8 to 10 m high after surcharge removal and were constructed with 2H:1V side slopes on PV drain-treated foundation soils. In addition, these embankments were constructed in areas of new alignment or where a significant amount of preexisting embankment had not preloaded the foundation soils. The cross section and foundation conditions of these embankments are more like the original construction of I-15 in the 1960s rather than the reconstruction cases for which alternative technologies were used. Settlement observations for these embankments have been included herein for reference and comparison.

### *Settlement Performance of LCC Treated Soil*

The LCC stabilized one-stage MSE wall was the first area that UDOT instrumented and was selected because of the wall proximity to a commercial building (Fig. 2). The installation of the columns caused the nearest side of the building to heave about 25 mm, which resulted in minor cracking in some of the building interior walls (Saye et al. 2001a; Bartlett and Farnsworth 2002). At its closest point, this building is located about 8 m from the wall face and 6 m from the edge of the LCC treated zone.

Sensor arrays at this site were installed after column installation, but before MSE wall/embankment construction. Fig. 4 shows the end-of-construction and 7-year settlement profiles measured at this location using settlement points placed around the

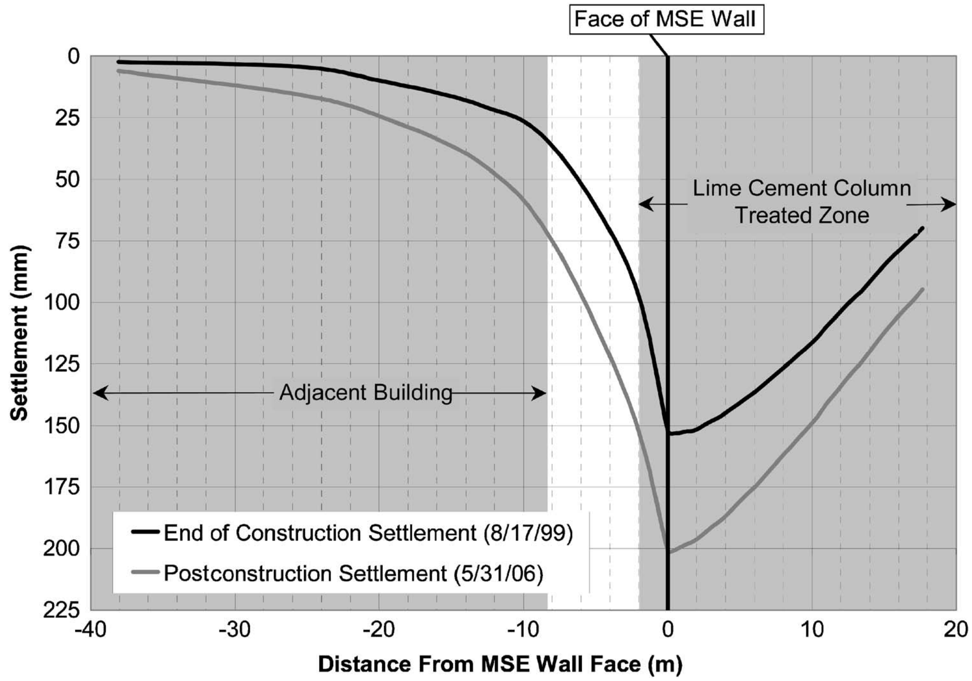


**Fig. 4.** Construction and postconstruction settlement profile for lime cement column area, I-15 Reconstruction Project, Salt Lake City, Utah

building and horizontal inclinometers that extended into the wall near the bottom of the reinforced zone. The largest settlement occurred at the MSE wall face, which was about 150 mm at the end of the construction period. The amount of settlement measured by the horizontal inclinometer decreased with increasing distance into the wall. This occurred because the preexisting I-15 embankment preloaded the foundation soils in this zone. Settlement measurements from an adjacent magnet extensometer placed in the MSE reinforced zone showed that approximately 50% of the construction settlement occurred from compression of the LCC treated zone and the remaining 50% occurred in the soils beneath the columns (Bartlett and Farnsworth 2002). This extensometer was destroyed during paving operations and an additional extensometer with spider magnets was placed at the toe of the wall into the foundation soils. This second magnet extensometer confirmed that about 50% of the postconstruction settlement is occurring beneath the LCC treated zone. In a 7-year postconstruction period, settlement points at the wall face show that the MSE footing has undergone about 50 mm of additional settlement, resulting in a total of about 200 mm settlement at the wall face. The total settlement is still within the design recommendation of using one-stage MSE walls where total settlements do not exceed 250 mm. However, the settlement at the wall face has caused minor cracking in some of the concrete facing panels.

It should be noted that the wall and nonreinforced embankment behind the wall have undergone some angular distortion, as shown in Fig. 4. The angular distortion across this zone is approximately 1/210 and 1/160 after construction and 7 years of postconstruction settlements, respectively. However, these values are measured at the base of the wall. The surface fill was leveled after removal of the surcharge so that the pavement was placed at the appropriate elevation and drainage slope. Taking this into consideration, the 7-year postconstruction angular distortion of the surface pavement is considerably smaller at around 1/720. Additionally, the MSE wall reinforced zone extends only about 8 m behind the wall face with unreinforced embankment fill placed behind that (Fig. 2). The settlement profile in Fig. 4 shows that there is not any noticeable settlement difference across this transitional zone. It does not appear that the angular distortion has had any significant impact on the performance of the MSE wall or the surface pavement.

Fig. 4 also shows that the zone of measurable settlements extends about 35 m from the wall face. Construction related settlement of the adjacent building was about 35 mm at a distance of 8 m from the wall face. Seven years of postconstruction settlement at this point has produced an additional 40 mm for a total of 75 mm of settlement. Such settlement is potentially damaging to sensitive structures and is more than was anticipated in the design.

It should be noted that the west end of the MSE wall serves as an MSE wall bridge abutment on pile foundations. However, the instrumentation for this project was targeted to monitor behavior of the foundation treatment and was placed away from the bridge to avoid any complex edge effects. Thus, the instrumentation did not provide any details about the interaction of the pile foundation with the LCC treated foundation soils or the MSE wall.

### *Settlement Performance of Geofoam Walls/Embankments*

Settlement arrays were installed at two large geofoam embankment/walls located at 3300 and 100 South Streets in Salt Lake City. The first location is a large approach fill for an interstate bridge that crosses a railroad line, and the second location is a buried utility corridor that crosses perpendicular to I-15. At 3300 South Street, the design-build contractor selected geofoam embankment to expedite construction. By working a day and a night shift at this location, the geofoam embankment construction, including placement of pavement, was completed within about 3 months. At 100 South Street, the geofoam embankment was selected to minimize settlements of existing buried utilities across the I-15 alignment. The geofoam embankments at both

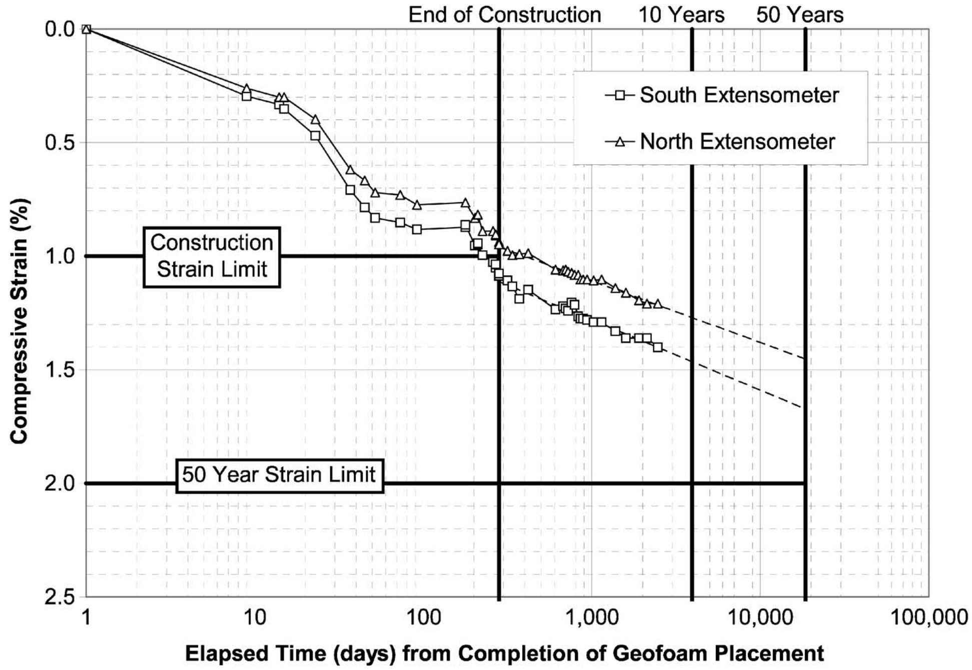


**Fig. 5.** Compressive strain within geofoam embankment from magnet extensometer data with magnets located at the top and bottom of the geofoam, at 100 South Street, Salt Lake City, Utah

locations were about 7.5 m in height, but more material was placed above the load distribution slab at 100 South Street. At this location, the subbase material over the load distribution slab consisted of scoria, and at 3300 South Street, the subbase material consisted of conventional fill. In all cases, the design required that the bearing pressures on the geofoam from the load distribution slab, subbase, base, and pavement section be below the 40% working stress criterion. This loading caused approximately 70 and 80 mm compression of the geofoam fill during construction, at 3300 and 100 South Streets, respectively, as measured by magnet extensometers located about 2.4 m from the vertical wall face. The corresponding construction induced strain is approximately 1% (Fig. 5).

A minimal amount of foundation settlement was expected at the geofoam embankment locations because the weight of the lightweight fill did not induce stresses at depth that exceeded the preconsolidation stress of the Lake Bonneville Clays. In addition, to further reduce the net loading, about 1 m of the subgrade was excavated and replaced with geofoam.

Fig. 6 shows that about 15 mm of foundation settlement occurred from placement of the roadway materials and pavement atop the geofoam at 3300 South Street. At about 300 days after completion of the pavement structure at 3300 South Street, the design-build contractor placed a 1.5 m high toe berm at the base of the geofoam wall (Fig. 6 inset). This new load produced an additional 25 mm of postconstruction foundation settlement (Fig. 6). With this additional loading at the toe, the total foundation settlement (construction and 10-year postconstruction) is expected to be about 45 mm. However, if the toe berm had not been placed, the expected foundation settlement would have been about 20 mm for the same period (Fig. 6). Nonetheless, this additional settlement was not consequential at this location because there were no nearby utilities. The construction of a large toe berm such as this was not typical for other I-15 geofoam walls.

The postconstruction cumulative compression of the geofoam embankment itself at 3300 South Street array is about 25 mm to date and is projected to reach about 30 mm after a 10-year postconstruction period. When this geofoam compression is combined with the postconstruction foundation settlement and influence of the toe berm previously described, the total 10-year postconstruction settlement of the roadway surface is estimated to be about 60 mm, which is less than the 76 mm 10-year criterion.

### *Settlement Performance of Two-Stage MSE Walls with PV Drains and Surcharging*

Construction and postconstruction settlement performance of a two-stage MSE wall was also monitored at 200 South Street. At this location, an 8 m high wall and embankment were constructed and surcharged with an additional 4 m of temporary fill as shown in Fig. 7. To expedite primary consolidation settlement, PV drains were installed at 1.5 m triangular spacing to a depth of 25 m.

The two-stage MSE wall and surcharge fill at 200 South Street, shown in Fig. 7, induced 1,100 mm of consolidation settlement at the wall face over the construction period. The amount of consolidation settlement that this wall underwent was typical of what the two-stage MSE walls over PV drain treated soil throughout much of the project experienced during the I-15 Reconstruction Project. Furthermore, these values echo the settlement values of 1 to 1.5 m that were typical of embankment construction during the original I-15 construction in the 1960s. The large magnitudes of settlement can be directly attributed to building large embankments over the soft thick Lake Bonneville clay layers. Furthermore, it should be noted that by using the PV drains, the settlement time was shortened considerably, from around 2 to 3 years to about 8 months to construct the embankment/wall and allow for the majority of the consolidation settlement to occur.

The settlement profile for the 200 South Street MSE wall (Fig. 8) shows that the amount of the consolidation settlement decreased with increasing distance into the wall, due to the in-

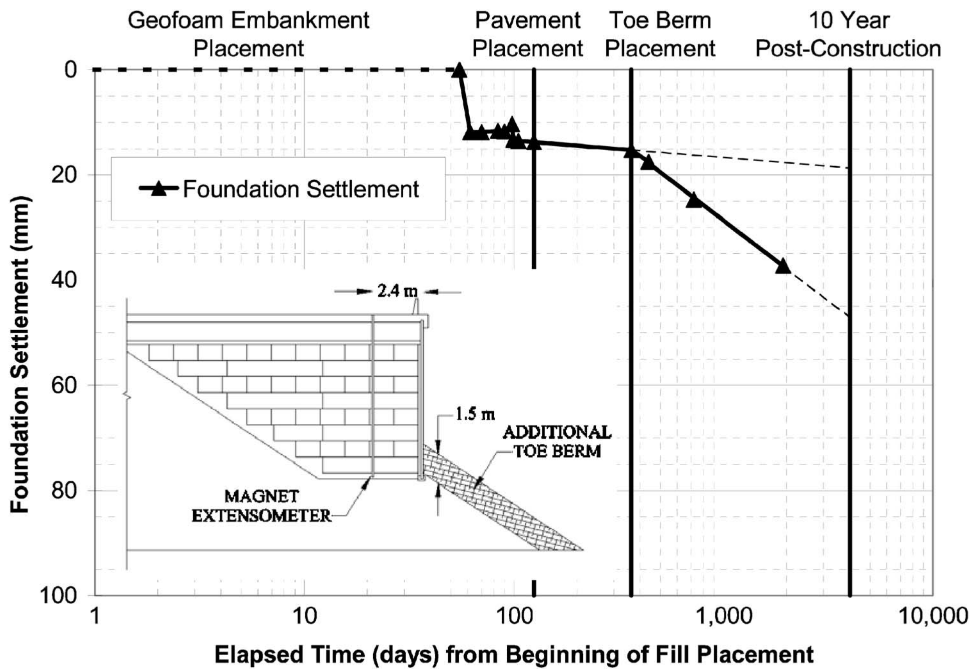


**Fig. 6.** Foundation settlement versus time for the geofoam embankment, from magnet extensometer at 3300 South Street, Salt Lake City, Utah

fluence of the existing embankment, similar to the LCC array (Fig. 4). As with the one-stage MSE wall at the lime cement column location, this wall also experienced some angular distortion due to the preexisting embankment. The angular distortion across this zone is approximately 1/50 and 1/40 after construction and 6 years of postconstruction settlements, respectively. Again, these values are measured at the base of the wall. The surface fill was leveled after removal of the surcharge so that the pavement was placed at the appropriate elevation and drainage slope. Taking this into consideration, the 6-year postconstruction angular distortion of the surface pavement is considerably smaller at around 1/260. Furthermore, the precast facing panels were placed in the second construction stage following consolidation settlement. Therefore, the welded wire wall facing would be affected by the full angular distortion. According to the Federal Highway Administration (FHWA 2001), walls with welded wire facings should have limiting differential settlements of 1/50. The construction settlements essentially reach this target value, with the postconstruction settlements exceeding it. At this time, there are not any visible signs that the behavior of the wall is being negatively affected by this angular distortion.

The zone of measurable settlement extended about 30 m beyond the wall face toward an adjacent house. The nearest edge of the house is at about 15.5 m from the wall face. A concrete driveway located between 5 and 10 m from the wall face experienced more than 100 mm of differential settlement, cracked, and was replaced. In addition, during 6 years of postconstruction monitoring, the wall face and nearest edge of the house have undergone approximately 70 and 20 mm of additional settlement, respectively. UDOT has compensated the home owner for settlement induced damages.

The large settlement magnitudes that resulted from the conventional construction of two-stage MSE walls with PV drains illustrates why the other geotechnologies (LCC foundation treat-

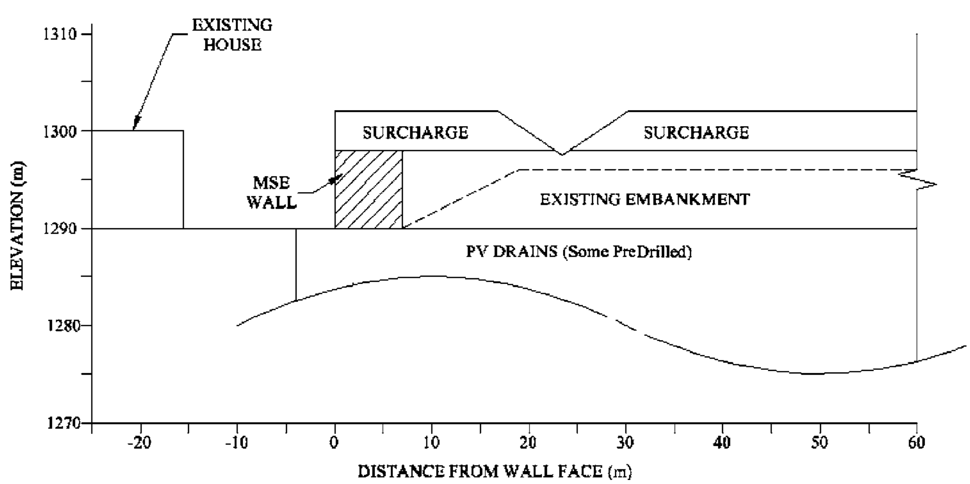


**Fig. 7.** Embankment construction at the 200 South Street settlement array

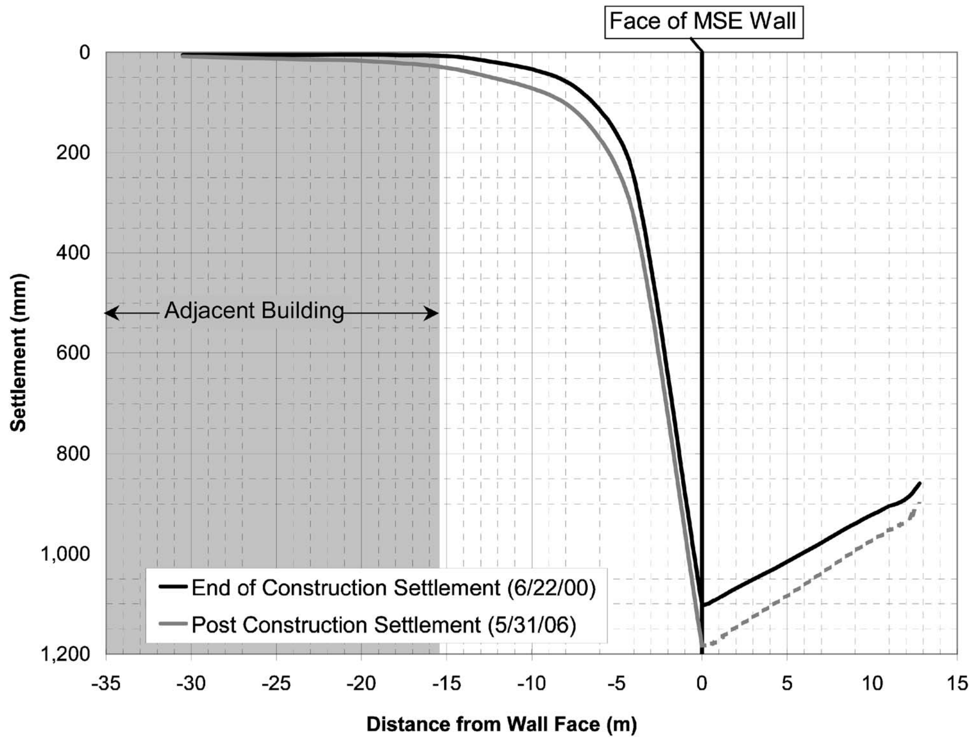


**Fig. 8.** Construction and postconstruction settlement profile for two-stage MSE wall at 200 South Street, Salt Lake City, Utah

ment and geofoam embankment) were employed in locations where settlements of that magnitude could not be facilitated. Settlement magnitudes for the other two geotechnologies were substantially smaller, on the order of 15 and 2% of the two-stage MSE wall with PV drains total settlement value for the one-stage MSE wall over LCC treated foundation and geofoam embankments, respectively. Although these values are specific to these three locations, they do show the relative potential reduction in overall settlement magnitude.

### *Comparison of Postconstruction Settlement Performance*

Fig. 9 presents a summary of postconstruction settlements at the various embankment locations and for the alternative geotechnologies. The rate of secondary settlement for the LCC array is shown at two locations: One set of data shows the rate of settlement at the wall face and the other shows the rate occurring 13.5 m inside the wall using the horizontal inclinometer observations. These data show that the LCC treated soil technology will likely meet the 10-year postconstruction settlement goal.

The postconstruction settlement for the geofoam embankment is projected to meet the 10-year postconstruction settlement goal. The postconstruction settlement of the geofoam embankment is comprised of both foundation settlement and geofoam creep. Fig. 9 shows the rate of postconstruction movement at 3300 South Street with the placement of the 1.5 m toe berm and a projected rate had the toe berm not been placed. The postconstruction settlement of geofoam is highly dependent upon the loading placed at the base and the top of the geofoam wall.

The two-stage MSE wall with PV drains at 200 South Street is projected to slightly exceed the 10-year postconstruction settlement goal at the wall face, where the most pronounced settlement is occurring. Unfortunately, the horizontal inclinometer within the wall was damaged at the end of construction, thus postconstruction settlements within the wall footprint are not available. However, the end-of-construction profile (Fig. 8) shows that settlement was diminishing with increasing distance into the wall. Assuming this trend continued, much of the profile within the wall will likely meet the 10-year postconstruction settlement criterion.

Postconstruction settlement performance trends at 2400 South, 900 West, and 400 South Streets suggest these large earthen embankments will exceed the 76 mm in 10-year postconstruction settlement goal (Fig. 9). These embankments were primarily constructed in areas of new alignment or where preexisting embankment had not significantly preloaded the foundation soils. Locations over preexisting embankment may not see postconstruction settlement rates of this magnitude.

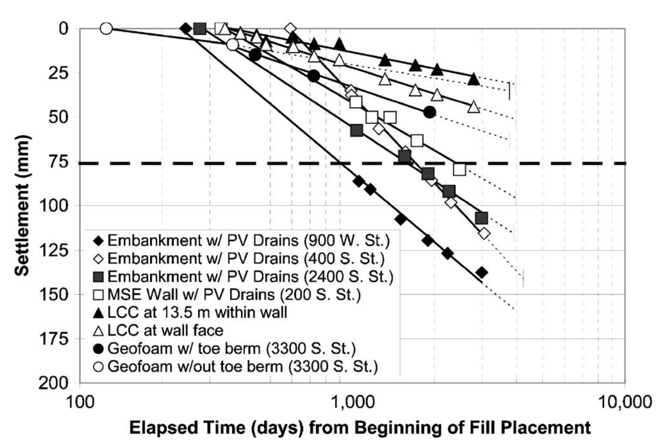


**Fig. 9.** Rate of foundation creep extrapolated to 10 years postconstruction (dashed lines) compared to the design criteria of 76 mm of postconstruction settlement over 10 years for lime cement columns, geofoam fill, two-stage MSE wall at wall face, and large earth embankments with PV drains at full height

## Conclusions

The I-15 Reconstruction Project provides a good case history illustrating the challenges of constructing large embankments and MSE walls over soft soil sites in an urban setting. The construction method utilized throughout much of the project for soft soil locations consisted of constructing two-stage MSE walls over PV drain treated soils. However, due to the large primary consolidation settlements (often greater than 1 m) induced in the underlying Lake Bonneville clay deposits by such construction, other technologies were used at locations where utilities and/or adjacent structures could not tolerate such large settlements. These consisted of LCC foundation treatment and lightweight geofoam embankment, which were designed to reduce both primary and secondary consolidation settlement. This paper has highlighted the settlement performance, cost, and time of construction of these technologies as applied to the I-15 Reconstruction Project.

The use of LCC foundations, geofoam embankments, and two-stage walls with PV drains all played an important role in the timely completion of the I-15 reconstruction project. The design-build team selected a particular technology based on cost, construction time, and settlement performance goals at each location. For a typical I-15 cross section, cost comparisons (year 2000 value) indicate that geofoam embankment and one-stage MSE walls over LCC treated soil cost about 1.2 and 1.6 times more, respectively, than conventional construction (i.e., two-stage MSE walls with surcharging and PV drain foundation treatment). However, when construction time is considered, geofoam embankments have a distinct advantage, requiring only about 3 months of construction time. The other two technologies required a year, or longer, mainly due to the lengthy time waiting for completion of primary consolidation settlement of the foundation soils before bridge foundations, bridges, and approach pavement could be constructed.

Our long-term monitoring shows that construction and postconstruction settlement performance of each technology varied widely. Two-stage MSE walls with PV drains and surcharging created the most settlement impacts to adjacent facilities and produced the largest amount of postconstruction settlement. Primary consolidation settlement at a typical two-stage MSE wall face exceeded 1 m and the zone of significant settlement (i.e., 25 mm) extended a distance of up to 1.5 times the full wall height, including the height of surcharge. Thus, we recommend that alternatives to this technology be considered at locations where settlement sensitive infrastructure falls within this zone of significant settlement. Additionally, based on 10-year projections of postconstruction settlements at four locales (1 MSE wall and 3 sloped embankments), the surcharging strategy used by the design-build team appears to limit the 10-year postconstruction settlement in the foundation soils to about 100 to 150 mm. Because these values exceed the postconstruction settlement goal established by the project of 75 mm, we recommend that further evaluations be made regarding the surcharge design, construction practices, and the feasibility of achieving this performance goal using conventional embankment construction and surcharging for the lacustrine sediments in the Salt Lake Valley.

The LCC treated system developed about 150 mm of construction settlement due to the placement of the construction of the MSE wall, embankment, and surcharge. In addition, an adjacent commercial building, located about 8 m from the MSE wall face, was slightly damaged from LCC column installation. We project an additional 50 mm of postconstruction settlement at the face over a 10-year postconstruction period. Our monitoring shows that the south side of this building, which is located nearest to the MSE wall face, has undergone about 75 mm of construction and postconstruction settlement in 7 years, resulting from the placement of the adjacent MSE wall. Survey points around this building show that the zone of significant settlement (i.e., 25 mm) extends about 20 m from the wall face, or about 1.7 times the full height of the wall, including surcharge. Thus, the LCC treatment has effectively reduced primary consolidation settlement near the wall face, but a significant zone beyond the wall face has been exposed to potentially damaging settlements. This zone is broader than what is typical for a nontreated site and may be a result of consolidation in deeper clay layers caused by a partial stress transfer from the overlying columns.

Geofoam embankments had the best overall settlement performance of the technologies monitored. Gap closure and deformation of the geofoam embankment due to placement of the load distribution slab and overlying roadway materials was about 1% of the embankment height, or about 80 mm at our array locations. In addition, the foundation soil settled about 15 mm due to the placement of the embankment and overlying loads and the face of the embankment settled an additional 25 mm in a 5-year period due to the placement of a 1.5 m toe berm at the toe of the fascia wall. Total postconstruction settlement (foundation settlement and geofoam creep) is expected to be about 60 mm at the wall face for a 10-year postconstruction period. The trend of postconstruction settlements suggest that geofoam embankments will most likely meet the 50-year postconstruction deformation limit of 1% axial strain.

The use of LCC foundations, geofoam embankments, and two-stage walls with surcharging and PV drains has been successfully employed on the I-15 Reconstruction Project. The decision to use these geotechnologies varied from location to location. However, the primary contributing factors included cost, construction time, and settlement tolerances of adjacent facilities as each location. This paper serves as a case history providing a relative comparison of these associated factors, which in turn can be used by others to explore the use of these geotechnologies in other projects.

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