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# DESIGN AND PERFORMANCE OF THE FOUNDATION STABILIZATION TREATMENTS FOR THE RECONSTRUCTION OF INTERSTATE 15 IN SALT LAKE CITY, UTAH

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ABSTRACT: This paper summarizes the design and initial performance of the foundation stabilization treatments adopted for the reconstruction of Interstate 15 in Salt Lake City, Utah that began in 1997. The roadway designs required extensive high fills over soft foundation soils to raise and widen existing embankments and to construct new embankments. The foundation treatments included prefabricated vertical drains, surcharge fills, high-strength geotextile reinforcement, stability berms, staged embankment construction, light weight fills, and lime cement columns in order to improve foundation stability, avoid damage to existing structures, and to reduce postconstruction pavement settlements. The relative success of the design is evaluated from geotechnical instrumentation results obtained during the first phase of the reconstruction in 1997 and 1998.



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Contractor ______ Witness Bartlett

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Ladd (1989), as presented in Stewart et al. (1994) and shown in Figure 25, was used to estimate the design surcharge heights using the "average" relationship. Special testing was completed for the final design by Ng (1998) at MIT to develop the specific design relationship for Lake Bonneville Deposits presented in Figure 26. These data show a much larger reduction in $C_\alpha{}' / C_\alpha(NC)$ at the lower surcharge levels than reported by Ladd (1989) for other cohesive soils, as illustrated in Figure 25. Figure 27 combines the MIT data from both Figures 25 and 26 in the semi-log format proposed by Ng (1998). The maximum reduction curve shown in Figure 27 for $C_\alpha{}' / C_\alpha(NC)$ versus the adjusted amount of surcharge (AAOS) was selected for final design. Without the special testing at MIT, the mean reduction line in Figure 25 would have been used for design resulting in an increased thickness of surcharge fills. The mean line for $t_s / t_t$ vs AAOS in Figure 27 was used in final design.

## 6 Stability Analyses

Stability was a major design concern at most locations along the alignment in Figure 1. Large surcharge fills were needed at most locations, further increasing the stability problems. Many loading conditions involved the widening of existing embankments with staged filling, and vertical drains, as shown in Figure 2. The 35% design reports also identified severe stability problems along the portion of the project shown in Figure 1. Examples of sections requiring stability analyses are illustrated in Figure 28.

The loading conditions associated with staged embankment construction required prediction of both the initial undrained shear strength ($s_u$) profile and increases due to consolidation. The SHANSEP method (Ladd and Foott 1974) was used to calculate these $s_u$ profiles using the following relationship:

$$s_u = \sigma'_{vo} (S)(OCR)^m \tag{4}$$

where: $\sigma'_{vc} = \sigma'_{vo}$ for virgin ground (stage I) and the calculated vertical effective stress with consolidation (under embankment for subsequent stages), $S = s_u / \sigma'_{vc}$ at OCR = 1, OCR = the overconsolidation ratio ($\sigma'_p / \sigma'_{vo}$), and m = an experimental exponent.

The SHANSEP parameters were developed for undrained shear in plane strain compression (PSC) and extension (PSE) and direct simple shear (DSS). These values were derived from $CK_oU$ triaxial compression and extension, and DSS tests run at MIT using the SHANSEP reconsolidation technique to reduce sample disturbance effects and included the following "corrections" described by Ladd (1991): 1) increase triaxial strengths for plane strain (Section 4.6); 2) decrease peak strength to account for strain compatibility (Section 4.9); and 3) define $s_u$ as the shear stress on the failure plane (i.e. $\tau = 0.5 (\sigma_1 - \sigma_3) \cos \phi$). The values of S and m selected for the cohesive layers of the Lake Bonneville Deposits are:

| | | |
|---|---|---|
| Shear in PSC | $S_c = 0.3$ | $m_c = 0.8$ |
| Shear in DSS | $S_d = 0.24$ | $m_d = 0.8$ |
| Shear in PSE | $S_e = 0.18$ | $m_e = 0.8$ |

The design initial undrained strength profiles for a wide variety of site conditions were calculated from the effective overburden stress, which was adjusted for artesian pressures with depth, and values of the preconsolidation

stress estimated with the approach described in Section 5.1. Beneath the existing embankments, the present, higher, undrained strengths were obtained using the embankment height, with appropriate influence factors, to calculate the change in $\sigma'_{vc}$ for addition to the free-field effective overburden stress. The same approach was used for staged embankment loading to predict the improved strengths beneath the new embankment after the consolidation period, except that the values of $S_c$ and $S_d$ were reduced by 10 % since no aging effects would occur during the consolidation period following staged loading.

Figures 29a and 29b illustrate the development of undrained strengths for different sections of the embankment for: a) a wide extension of the embankment (virgin ground initial stress conditions) and b) a narrow widening of the existing embankment where improved strengths beneath the existing embankments increase the strengths. By separating the strengths in this manner, undrained strength profiles were calculated for a wide range of loading and embankment configurations.

The instrumentation observations during the first phase of construction in 1997 indicated that the alluvial soils encountered to depths of 5 to 6 m below original grade that were penetrated with prefabricated vertical drains developed limited, if any, excess pore water pressure. Hence stability assessments for the second phase of construction were made with strengths computed for drained shear in the upper 6 m of the natural soils, although this may overestimate the actual resistance during a rapid failure.

Reinforcement of the embankments with high-strength geotextile and staged embankment construction with prefabricated vertical drains were the primary methods to improve stability conditions. High-strength geotextile was used extensively in the 2400S area of the alignment where the embankments were constructed over virgin ground conditions and where embankment heights up to 18 m were needed. Representative stability calculations from the 2400S area, based on undrained loading, are shown in Figure 30 for 2H to 1V slopes with staged filling. Without reinforcement, an initial embankment height of 7 m was calculated at a factor of safety of 1.3, increasing to a height of 10 m for Stage 2 and only 10.5 m for Stage 3. The declining incremental increase in embankment height with successive stages was a significant factor in the geotechnical design that limited the usefulness of staged construction. Hence, installation of high-strength geotextile (GT No. 1) near the bottom of the embankment was used to improve global stability. Figure shows that three layers of reinforcement gives design heights about 20 percent higher than unreinforced embankments.

Global stability calculations were made using the Modified Bishop method with the UTEXAS3 or SLOPE/W programs. In many instances stability was a limiting condition for construction of walls and embankments. The UTEXAS3 program described by Wright (1991) was used to evaluate the reinforcement effects of geotextile in the global stability calculations. The geotextile provides an additional resisting moment at the intersection with the critical failure surface. The high-strength geotextile also helps the embankment to act as a unit that can produce a squeezing type failure of the foundation soils below the reinforced mass. Bonaparte and Christopher (1987) describe the methodology used to assess this type of failure for embankments with geotextile reinforcement.