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# Evaluation of a Geogrid-Reinforced Slope Subjected to Differential Settlements

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ABSTRACT: A geogrid-reinforced toe buttress was constructed in 1987 under the direction of the Environmental Protection Agency (EPA) in order to enhance the stability of the southeastern slopes of the Operating Industries, Inc. (OII) Landfill Superfund site. The landfill is located approximately 16 km east of downtown Los Angeles, in an area of high seismicity. The front of the approximately 4.6 m high, 460 m long toe buttress was founded on concrete piers. However, as the back of the reinforced slope was founded on waste, the structure has been subjected to more than 0.6 m of differential settlements since its construction. Finite element analyses were performed to evaluate the long-term integrity of the geogrid reinforcements under the loads induced by 30 years of additional differential settlements followed by the design earthquake. The calculated maximum strains in the geogrid reinforcements after the long-term static and design seismic loadings are well below the allowable strain, indicating that the integrity of the toe buttress should be maintained even when subjected to large differential settlements and severe earthquake loads.

KEYWORDS: Finite Element Analysis, Landfills, Reinforcement, Seismic Loads, Settlement Analysis.

## 1 INTRODUCTION

A geogrid-reinforced toe buttress was constructed in 1987 under the direction of the Environmental Protection Agency (EPA) in order to enhance the stability of the southeastern slopes of the Operating Industries, Inc. (OII) Landfill Superfund site. The toe buttress is immediately adjacent to a residential development. The waste slopes behind the toe buttress are up to 37 m high, with intermediate slopes between benches up to 18 m high and as steep as 1.3H:1V. The approximately 460 m long, 4.6 m high toe buttress was founded on concrete piers at the front of the buttress and reinforced using HDPE geogrids. However, as the back of the reinforced buttress was founded on waste, the toe buttress has been subjected to significant differential settlements since its construction.

A thorough evaluation was undertaken to assess the long-term integrity of the reinforced toe buttress and, consequently, the stability of the southeastern landfill slopes behind the toe buttress. Analyses calibrated on the previous performance of the toe buttress were used to predict its future performance considering 30 years of additional settlement followed by the design (maximum credible) earthquake. The analyses of the toe buttress included three distinct components: (i) interpretation of monitoring data to evaluate the history of differential settlements in the toe buttress area and to project the future differential settlements to which the structure will be subjected over the next 30 years, (ii) analysis of the global stability of the southeastern slopes of the landfill, assuming that the internal integrity of the toe buttress is maintained; and (iii) evaluation of the internal integrity of the geogrid-reinforced toe buttress, subjected to the predicted long-term differential settlements followed by the design earthquake, using nonlinear finite element analyses.

The scope of this paper is limited to some aspects of the element evaluation. Subsequent publications will present further aspects of the long-term and seismic evaluation of the geogrid-reinforced to buttress. The finite element analyses presented herein were performed in three sequential phases: (i) toe buttress construction, modeled by sequentially activating soil and bar elements in the reinforced soil zone; (ii) development of differential settlements beneath the toe buttress, simulated by imposing incremental displacements at the base of the reinforced soil mass; and (iii) earthquake loading, modeled pseudo-statically by applying horizontal body forces, representing the maximum average acceleration estimated from a finite element site response analysis, to the reinforced soil mass.

## 2 BACKGROUND INFORMATION

Schematic profiles through the toe buttress and the waste slope along the southeastern perimeter of the OII Landfill are illustrated in Figure 1. Reinforced cast-in-place concrete piers were constructed along the roadway which was located at the toe of the landfill, along the property line, in areas where the natural ground surface continued to slope downward beyond the property line (Figure 1a). Piers were not installed in areas where the ground surface was level beyond the toe of the waste slope (Figure 1b). A total of 201 piers, 0.9-m in diameter, were installed at 1.8-m center to center spacing along approximately 360 m of the 460 m long toe buttress.

An assessment of the available information on the geometry of the bottom of the waste beneath the toe buttress was undertaken to aid in the toe buttress global stability evaluation. The logs for the 201 concrete piers drilled along the toe buttress, along with historical aerial photos and limited data from borings through the waste, provided relevant information regarding the depth of the waste in the toe buttress area. This available information indicated that the bottom of the waste in the vicinity of the toe buttress area slopes down at an approximately 1H:1V inclination from the property line into the landfill.

Visual observations and survey data indicate that, since its construction in 1987, significant differential settlements have taken place over the width of the toe buttress along most of its alignment. The presence of the concrete piers under the front edge of the buttress and the increasing thickness of the waste towards the back of the buttress both contributed to the substantial differential settlements observed over the width of the toe buttress. Settlement profiles at eight stations along the toe buttress were measured in October 1992 and in April 1996. The results from the 1996 survey showed that, while the differential settlement rate at most of the stations along the toe buttress has decreased since 1992, significant differential movements were still occurring.

Because the settlement surveys were not tied to an external reference, it was assumed that the elevation of the toe buttress surface immediately above the drilled piers was fixed. The settlements monitored at the back of the toe buttress were projected forward in time to evaluate the potential for future settlements. Settlement was projected for each individual cross section as a straight line on a semi-logarithmic plot. The differential settlements projected 40 years beyond the end of construction (until year 2027) was less than 1.17 m for every cross section but one. For Cross Section 3 the projected differential settlement was 1.98 m. However, because of the inconsistency of the data for Cross Section 3, the projected differential settlement of 1.98 m for this section was considered to be an outlier. A differential settlement of 1.17 m was considered a conservative projection of the settlement at the back of the toe buttress over the next 30 years for the purpose of evaluation of the long-term integrity of the toe buttress. Nevertheless, in response to EPA comments, the performance of the toe buttress when subjected to a projected differential settlement of 1.98 m was also evaluated.

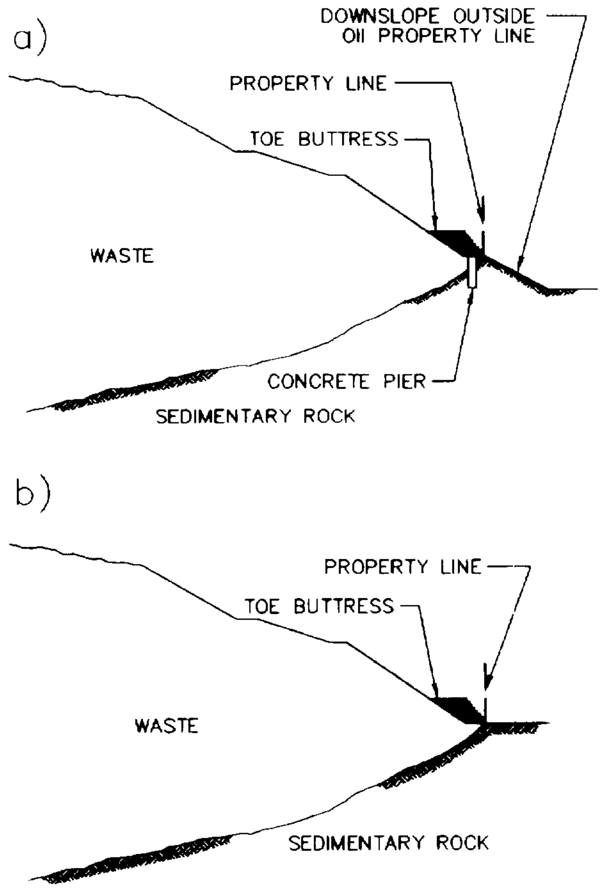


Figure 1: Typical profiles of the toe buttress at the OII Superfund Landfill.

## 3 MATERIAL PROPERTIES

The reinforcement elements used in the toe buttress were Tensar SR2 geogrids. Manufacturing of these reinforcement products had been discontinued by the time of this investigation. Consequently, the geogrid material properties needed for the analyses undertaken in this study were evaluated primarily on the basis of information available from the literature on this type of geogrid reinforcement. This literature information was supplemented with creep tests performed on archived geogrid samples provided by the geogrid manufacturer. A 10 percent limiting strain was established from the literature as a conservative estimate of the allowable geogrid strain for long-term static loading of this reinforcement (Bonaparte and Berg, 1987). A 20 percent limiting strain was established from the literature as the allowable geogrid strain for rapid earthquake loading (McGown et al., 1984). The laboratory testing program performed as part of this investigation included wide width tensile tests and creep tests followed by rapid loading to failure. The main objective of this testing program was to address concerns expressed by EPA that sudden loading after an extended period of creep could reduce the allowable geogrid strain to a value less than that obtained from wide width testing. However, the test results verified that an allowable strain of 20 percent in the geogrid was applicable to the case of static creep followed by rapid seismic loading.

Construction records indicated that the toe buttress fill is a sandy gravel classified as GP using the Unified Soil Classification System. Specifications required a minimum relative compaction of 95 percent, based on modified Proctor compaction test, except within 0.61 m of the toe buttress face. The constitutive relationship used in the finite element analyses to model the backfill behavior is the hyperbolic model proposed by Duncan, et al. (1980). Hyperbolic model parameters for the backfill material were obtained from triaxial test results reported in the literature for a sandy gravel of similar grain size distribution and compaction characteristics (Zornberg and Mitchell, 1994). The parameters for the gravel constitutive model obtained from these triaxial test results are presented in Table 1.

As a component of the comprehensive investigation of the seismic performance of the OII landfill of which this investigation was part (GeoSyntec, 1996), a field sampling and laboratory testing program was undertaken to characterize the static and dynamic mechanical properties of the waste at the OII landfill. Direct shear test results were used to determine the shear strength properties of the waste material for the finite element analyses presented herein. Simple shear test results were used to define the hyperbolic stress-strain parameters required to characterize the behavior of the waste material in the finite element analyses. The hyperbolic parameters used in the finite element analyses to characterize the waste material are summarized in Table 1. Based upon data from the field investigation, a uniform unit weight of 15.7 kN/m$^3$ was used for the solid waste material in the analyses.

## 4 FINITE ELEMENT EVALUATION OF THE TOE BUTTRESS

The integrity of the reinforced toe buttress subjected to the projected differential settlements followed by the design earthquake loading was evaluated via finite element analysis. The analysis was performed using the finite element code GeoFEAP developed at the University of California at Berkeley for analysis of geotechnical problems (Espinoza et al., 1995). Both material and geometric nonlinearity were considered in the analysis in order to account for the constitutive behavior of the materials and for the large displacements. The strains induced in the geogrid reinforcement were modeled using three sequential analyses: (i) construction of the toe buttress, (ii) gradual increase of differential settlement, and (iii) earthquake loading.

The finite element mesh used in the analyses consisted of 1082 nodes, 1028 plane strain elements for representation of soil and waste, and 140 bar elements for simulation of the reinforcements. A relatively fine mesh discretization between reinforcement layers was found essential for the proper representation of the behavior of the soil layers.

Table 1. Hyperbolic soil parameters for the backfill and waste materials

| Parameter | Parameter definition | Backfill | Waste |
|---|---|---|---|
| $K$ | Young's modulus coefficient | 913 | 212 |
| $n$ | Young's modulus exponent | 0.6 | 0.61 |
| $R_f$ | Failure ratio | 0.64 | 0.7 |
| $c$ (kPa) | Cohesion | 0.0 | 28.7 |
| $\phi_0$ (°) | Friction angle at 1 atm. | 46.1 | 31 |
| $\Delta\phi$ (°) | Friction angle reduction parameters | 5.3 | 0.0 |
| $K_B$ | Bulk modulus number | 250 | 212 |
| $m$ | Bulk modulus exponent | 0.8 | 0.61 |
| $K_{ur}$ | Unload-reload modulus coefficient | 1485 | 428 |
| $K_0$ | At-rest lateral earth pressure coefficient | 0.35 | 0.4 |

Construction of the toe buttress was modeled by sequentially activating soil and bar elements in the reinforced soil zone, as illustrated in Figure 2. Tensile strains were induced in the reinforcement during construction by the selfweight of the backfill material. The maximum reinforcement strain estimated in the construction analysis occurs in reinforcement level 7, located 2.7 m above the base of the 4.57 m high reinforced slope. The maximum strains that develop during construction in the geogrid reinforcements are very small, with a maximum strain of less than 0.4 percent. Figure 3 shows the strain distribution computed in reinforcement level 7 during the different stages of construction of the toe buttress. The different stages indicated in this figure correspond to the placement of the soil layers during construction simulation.

The second phase in the finite element modeling of the toe buttress consisted of imposing differential settlement at the base of the reinforced soil mass, as illustrated in Figure 4. Strain and tension in the reinforcements were induced by progressively increasing the base settlements in a triangular pattern, with zero settlement at the front of the mesh and the maximum settlement at the back of the finite element mesh. A total of 2.0 m of differential settlement was imposed at the base of the finite element mesh to simulate the long-term differential settlement of 1.20 m projected for the surface of the toe buttress. The maximum geogrid strain computed after imposing this differential settlement occurs in reinforcement level 3, located 0.9 m above the base of the toe buttress. Figure 5 shows the strain distribution estimated in the reinforcement level 3 due to increasing differential settlements. The differential settlement was imposed considering ten intermediate stages.

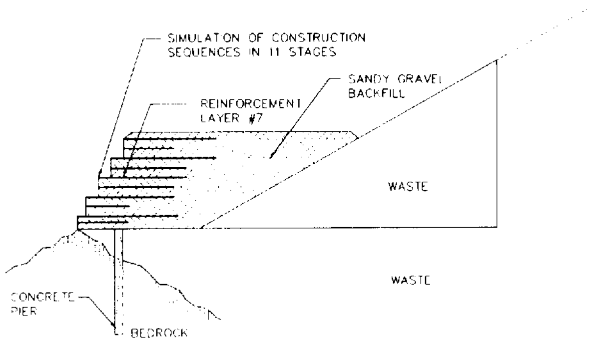


Figure 2: Finite element simulation of the construction sequence of the toe buttress.

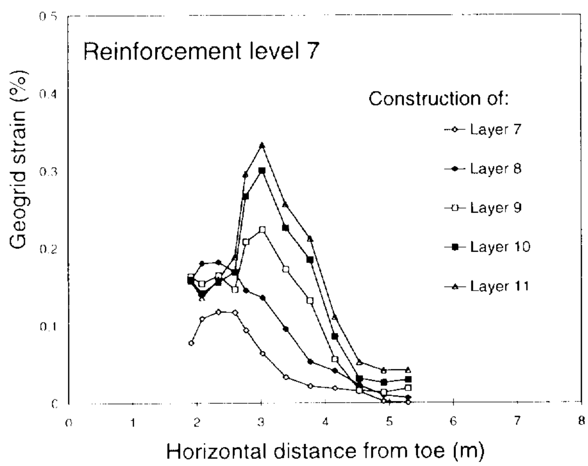


Figure 3. Estimated geogrid strains induced during construction.

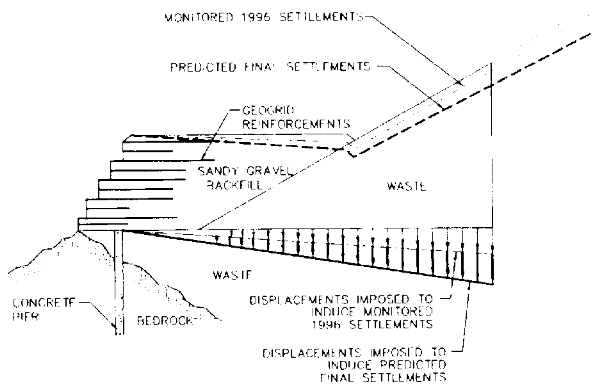


Figure 4: Finite element simulation of the differential settlements in the toe buttress.

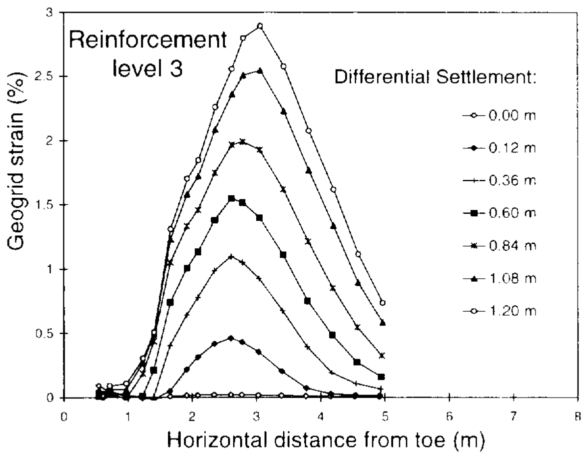


Figure 5. Estimated geogrid strains induced by increasing differential settlements.

The current (1996) average differential settlement at the back of the toe buttress is approximately 0.60 m. From the results shown in Figure 5, the maximum tensile strain in the geogrid reinforcements computed for the current condition is approximately 1.5 percent. Moreover, Figure 5 shows that the maximum tensile strain in the geogrid reinforcements computed for the long-term condition (i.e. after reaching 1.20 m of differential settlements on the surface of the toe buttress) is approximately 2.9 percent. Both the current and long-term geogrid strain levels predicted in the finite element analyses are well below the maximum static strain level of 10 percent established for the geogrid reinforcements. Extrapolation of the finite element results to larger strain levels indicates that it would require approximately 3.9 m of settlement between the crest of the toe buttress and the drainage ditch at the back of the structure to induce the maximum allowable static strain of 10 percent in the geogrids. This exceeds by a factor of almost two the maximum long-term settlement of 1.98 m considered for Cross Section 3. As discussed previously, this magnitude of settlement was considered an outlier, but was addressed in response to EPA concerns.

To model the impact of seismic loading on the performance of the toe buttress, horizontal body forces corresponding to the maximum average acceleration estimated for the toe buttress area were applied to the active reinforced soil wedge, as shown in Figure 6. The design earthquake was a magnitude 6.9 earthquake on a blind thrust fault immediately below the site. A pseudo-static acceleration of 1.0 g, estimated in a finite element site response analysis as the maximum average acceleration of the toe buttress in the design earthquake, was used for the analyses presented herein. The earthquake-induced strains are most significant in the upper reinforcement layers of the toe buttress, in contrast to the results of the previous static phases of the analysis. Reinforcement level 9, located 3.66 m above the base of the 4.57 m high toe buttress, shows the maximum estimated tensile strain when the structure is subjected to the design pseudo-static seismic loading.

Figure 7 shows the strain distribution estimated in reinforcement layer 9 during application of the seismically-induced horizontal body forces. The strain distributions that correspond to the end of construction and to the long-term differential settlement are also shown in the figure (the 0.0 *g* cases). The final stage shown in the figure corresponds to the results obtained after applying the design earthquake loading (1.0 *g*). The magnitude of the maximum tensile strain in the reinforcement at this stage of the analysis is approximately 8.5 percent, considerably lower than the 20 percent allowable strain for combined static and dynamic loads. The 1.0 *g* pseudo-static seismic load induced a 6.7 percent strain increase in the reinforcement. Extrapolation of these results indicates that a seismic coefficient of more than 1.5 *g* would be required to induce an incremental strain of 10 percent in the geogrids (10 percent is the difference between the rapid and the creep limited allowable strains).

The numerical results obtained in the three phases of the finite element analyses show that the maximum geogrid strain estimated after each phase of the study does not occur at the same elevation. The maximum strain due to construction loading occurs at midheight of the reinforced toe buttress, while the maximum strain due to differential settlement occurs towards the base of the structure and the maximum strain due to earthquake loading occurs towards the top of the slope. The results of the finite element analysis presented herein show that the integrity of the toe buttress should be maintained even when the toe buttress is subjected to the projected long-term differential settlement followed by the design earthquake loads. The predicted strain level in the geogrid reinforcement for the combined effect of these anticipated loadings is well below the allowable strains for combined long-term static and earthquake loading.

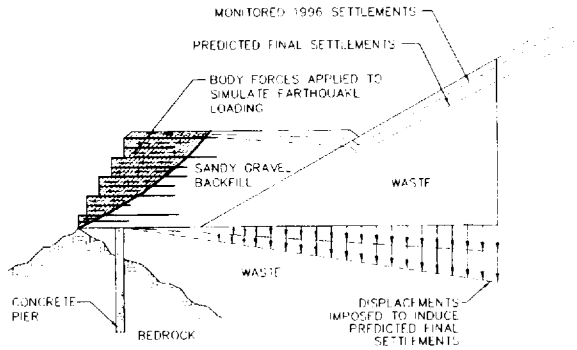


Figure 6: Finite element simulation of earthquake loading in the toe buttress.

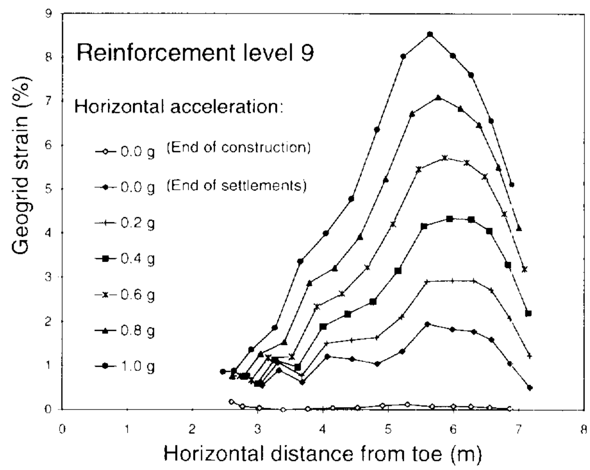


Figure 7. Estimated geogrid strains induced by seismic loads.

## 5 SUMMARY AND CONCLUSIONS

A geogrid-reinforced toe buttress was constructed in 1987 under the direction of the EPA in order to enhance the stability of the southeastern slopes of the OII Landfill Superfund site. The toe buttress is immediately adjacent to a residential development. The waste slopes behind the toe buttress are up to 37 m high with intermediate slopes between benches up to 18 m high and as steep as 1.3H:1V. The landfill is located 16 km east of downtown Los Angeles, in an area of high seismicity.

The approximately 4.6 m high, 460 m long toe buttress was built using sandy gravel as backfill material.

The front of the structure was founded on concrete piers. However, as the back of the toe buttress was founded on waste, the structure has been subjected to more than 0.6 m of differential settlements since the end of its construction. In response to concerns regarding the internal stability of the reinforced soil structure, finite element analyses were performed to evaluate the long-term integrity of the geogrid reinforcements under static and seismic loads. The analyses considered 40 years of settlement followed by the design earthquake. The finite element modeling evaluated the strains induced in the geogrid reinforcement considering both material and geometric nonlinearity. The analyses were performed in three sequential phases: (i) toe buttress construction, modeled by sequentially activating soil and bar elements in the reinforced soil zone; (ii) gradual increase in differential settlements, simulated by imposing incremental displacements at the base of the reinforced soil mass; and (iii) earthquake loading, modeled by applying horizontal body forces representing the maximum average acceleration estimated in a finite element site response analysis.

A total of 2.0 m of differential settlement was imposed on the base of the finite element mesh to simulate long-term differential settlement. The maximum strain in the geogrid reinforcements calculated after this long-term static loading is less than 3.0 percent, well below the allowable static strain of 10 percent. The calculated maximum geogrid strain induced by construction, long-term differential settlement, and earthquake loading is approximately 8.5 percent, well below the allowable strain of 20 percent established for rapid loading. The results of this study indicate that the integrity of the geogrid-reinforced toe buttress should be maintained even when subjected to large differential settlements and severe earthquake loads.

## ACKNOWLEDGMENTS

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