Geocells Reinforced with Geogrids is the Winning Solution for Repairing Landslides in Hawaii

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ABSTRACT
Six landslides occurred on military service roads during a 40-day rain storm in 2006. Surface water runoff on the roads triggered embankment failures in clayey silt fill soils and saprolite. When the USACE advertised the design-build project, their conceptual repair only considered structural walls supported with vertical piles.

The selected design-build team determined that mechanically stabilized earth (MSE) walls could be constructed less expensively, quicker, and more safely than cast-in place walls resting on steel piles. The project saved $1.5 million (approximately 50% of the project cost) by constructing MSE walls with cellular confinement (geocell) facing and geogrid reinforcement, instead of the conceptual design provided with the Request for Proposal (RFP) documents.

Geocells were selected as the MSE wall facing due to their light weight, ease of installation with relatively small construction equipment, tolerance to differential settlement, and cost compared with traditional pre-cast products and cast-in-place construction methods Additionally, the green geocell facing blended into the lush tropical vegetation of the ravine.

1. INTRODUCTION

This design-build contract managed by the US Army Corp of Engineers (USACE) included repairing six landslides along two roads that contour down both sides of a ravine within the Wheeler Army Airfield, located in the middle of the island of Oahu, Hawaii. The six landslides occurred during the winter of 2006, during what was known as the “40 day rainstorm.” During the storm, surface water from the runway and adjacent areas flowed down the roads and overfilled the storm drain system. Uncontrolled surface water flowed over the roads and down into the ravine below the roads causing severe erosion at the six sites as well as other locations.

The RFP documents included site plans with limited topography, cross-sections, photographs, and some limited geologic information. The requested repairs included constructing retaining walls, restoring the paved driving surface, adding guard rails, replacing and improving stormwater drainage controls, and replacing damaged electrical lines.

2. SITE CONDITIONS

The landslides appear to have been debris avalanches that developed into debris flows (Turner and Schuster, 1996). The failures occurred in road fill, existing colluvium on the slopes, and erosion into the native soils. The primary cause of the damage was due to uncontrolled surface water runoff causing severe erosion and hydraulic excavation. Each of the landslides were located where surface water flowed over the side of the road causing severe erosion in the embankment fill. The landslide scarps at all six sites were similar. The vertical scarps were 1.5 meters (m) to 3.7 m deep directly below the road. Hydraulic excavation had created steep gullies that drained to the main ravine. The bottoms of the gullies were 6 m to 12 m below the road and had accumulations of colluvium and slide debris within them. Photograph 1 shows the landslide scarp at Site 4.



Photograph 1. Landscape Scarp at Site 4.

2.1 Exploratory Methods

The RFP documents provided preliminary topography, borings drilled near two of the six landslides, laboratory test results of soils collected from the borings, photographs of each site, and a geologic cross-section of each site based on visual observations during a site reconnaissance. Four additional explorations were drilled on the road near the landslides that had not been drilled previously.

Drilling at the toe of the slopes was considered, but it was determined to be too difficult to drill below the steep landslide scarps in a safe manner. The main goal of drilling below the scarps was to determine the thickness of loose slide debris that would need to be removed. Since this could not be determined prior to construction, the volume of excavation and export, as well as the subgrade elevation of the walls would need to be determined during construction. This, however, was acceptable, since this was a design-build project.

2.2 Soil Conditions

The site vicinity is characterized by pahoehoe lava flows from the Koolau Volcano, known as the Koolau Basalt. Weathering of the Koolau Basalt has developed two soil types classified as saprolite and residual soil. Saprolite is a soil that exhibits relic structure of the basalt. Vesicles and joints may be visible due to color differences in the soil. The saprolite consists of stiff to hard clayey silt with variations in color and mottling. Residual soils are the result of a greater degree of weathering of the basalt so that the rock structures are no longer present. The soil is stiff to hard, brown and red, clayey silts. The Koolau Basalt and the saprolite and residual soils derived from it are known for high shear strengths (Sherrod et al., 2007).

The borings drilled at each site and the soils observed in the scarps documented road fill, residual soils, and saprolite derived from vesicular basalt. Geotechnical laboratory tests revealed that the residual soils and saprolite are generally high plasticity silt with moisture contents near their plastic limit. The laboratory strength tests results showed large variations, at least partially due to the difficulty of collecting and testing very stiff soils. Based on observations of exposed soils, drilling and sampling results, and laboratory testing, the soils possess high shear strengths. Figure 1 is a geologic cross-section from Site 4 that illustrates AMEC's stratigraphic interpretations, which was similar to all six sites.

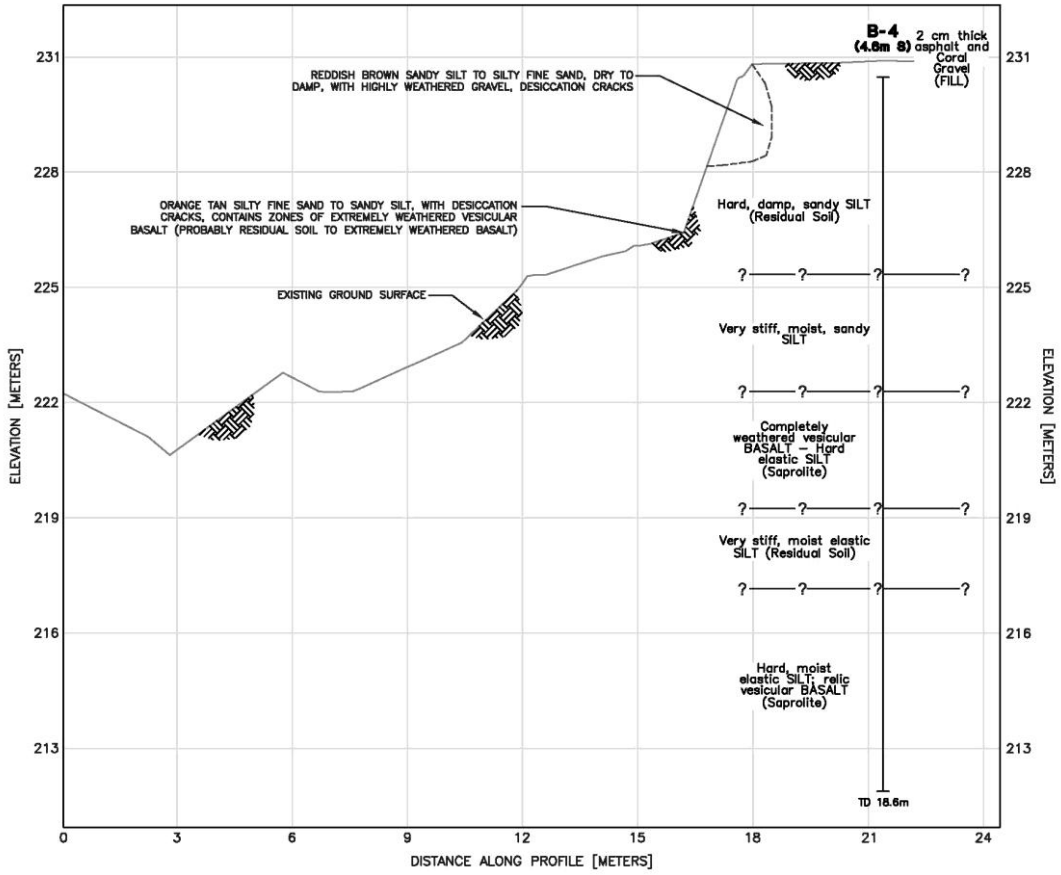


Figure1. Site 4 Geologic Cross-Section.

2.3 Groundwater Conditions

Explorations were drilled during both wet weather (January and February 2010 by others) and dry weather (October 2010) and none of the borings encountered groundwater. Groundwater was not encountered during construction (December 2011 and January 2012).

3. DESIGN

Three conceptual retaining wall options were included in the RFP for restoring the roads. The first concept used a cast-in-place concrete wall supported with micropiles; the second option used micropiles, a shotcrete wall face, and a cast-in-place concrete drive slab; and the third option used drilled shafts, steel H-beams as soldier piles, and pre-cast concrete panels for lagging (Figure 2).

The selected design-build team determined that pile structures were too expensive and too difficult to construct safely near the edge of the landslide scarps. Access would have been difficult for the large equipment needed for the steel pile and concrete wall concepts. The team decided that mechanically stabilized earth (MSE) walls could be constructed less expensively, quicker, and more safely. Rock slopes and reinforced soil slopes were also considered; however, the toe of the slopes would have extended beyond allowable construction limits into wetland areas for most of the sites. A rock slope was used to restore Site 6, the smallest of the landslides, with minimal site impacts. Gabion basket gravity walls were considered, but the baskets were not readily available on the island and they would have been susceptible to corrosion in the highly humid tropical environment. Stacked geocell facing with geogrid reinforcing layers were selected for their design advantages, constructability, and availability on the island for five areas.

The proposed walls were located close to the previous edge of the road shoulder due to the construction limits and access limitations. Furthermore, due to the depth of the landslides, a large amount of temporary excavation was necessary to access the base of the wall, allow room for the geogrid reinforcements and create a safe slope for workers (Photograph 2).

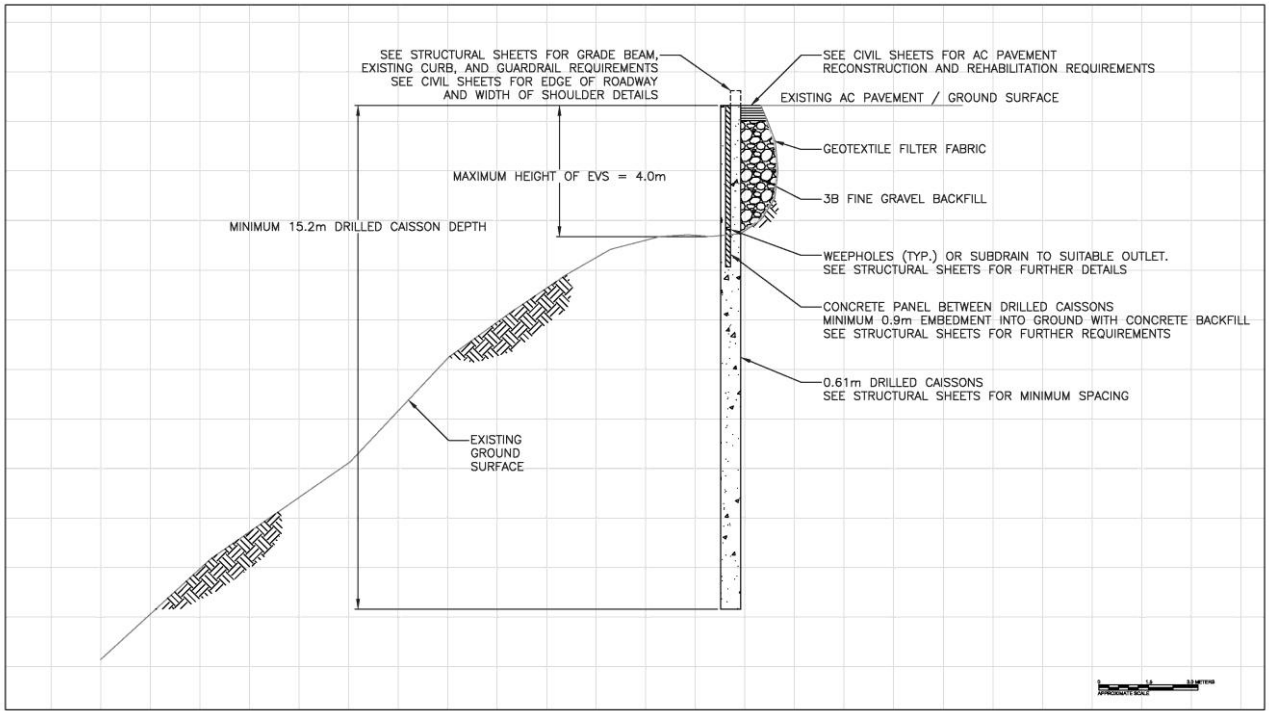


Figure 2. Conceptual Walls.



Photograph 2. Site 1 Excavation, Total Depth of 6.1 meters.

3.1 Slope Stability Analyses

The safety factors against sliding for trial failure surfaces were high for any deep-seated failures when the failed slopes were analyzed. Based on this evaluation, it appeared that the landslides that occurred at each of the six sites was the result of erosion, and there was no indication of deep-seated slope instability. This analysis gave the team confidence that the original concept of structural walls supported by piles was not necessary and an excessively costly repair given the site conditions. Our estimated values of internal friction angle, cohesion, and density for each soil layer are listed in Table 1 below. These conservative values are based on exploration information, laboratory testing, and slope stability calibration.

Table 1. Estimated Values of Internal Friction Angle, Cohesion, and Density for Each Soil Layer.

Geologic Name	Soil Type	Unit Weight (kN/m^3)	Internal Friction Angle (degrees)	Cohesion (kPa)
Fill, Colluvium and Slide Debris	Silt with gravel	16.5	28	0
Residual Soil	Highly plastic Silt	18	36	0
Saprolite	Highly plastic Silt	18	34	4.8

3.2 MSE Wall Design

Project specific external loads on the walls included a traffic live load on the road above the walls and seismic horizontal acceleration coefficient, as well as the lateral earth pressure from the retained soils. The traffic live load was modeled as an additional 0.61 m of soil (12 kPa) placed on the roadway, as specified by AASHTO (2002, 2010) and FHWA (2001) design manuals. A peak ground acceleration of 0.18g was used for the seismic stability analysis, as recommended by Hawaii Department of Transportation (HDOT, 2005). Global stability of the retaining walls determined the embedment depth of the wall and the minimum length of the geogrid reinforcement. Internal and external stability analyses determined the geogrid strength and spacing for the walls.

3.3 Wall Components

The geocells consisted of polyethylene, stabilized for ultraviolet light and corrosion, forming cells that were 27 cm long by 33 cm wide by 15 cm high when expanded. The geocells were delivered in sections 0.80 m wide by 2.6 m long. Each course of geocells were stacked with a 2.5 cm setback from the course below, to achieve a batter of 1H:6V. The geogrids consisted of polyester multifilament yarns woven in tension and finished with a PVC coating and had a long-term design strength (LTDS) of 45 kN/m. The LTDS is determined by reducing the ultimate tensile strength to account for potential material degradation. Both the geocell infill and the wall backfill material was a crushed and screened gravelly sand, meeting the HDOT 703.18 Filter Material specification (2005), which had a maximum grain size of 4 cm and less than 5 percent fines (Photograph 3).

Geogrid lengths were approximately 80 percent of the total wall height and spaced every 0.61 m (four stacked geocells), except for walls greater than 6 m tall, in which case the geogrid spacing was less for the lower portion of the wall.

The walls were embedded a minimum of 1 m into the intact soils. This embedment is deeper than typical installations due to the falling foreslope and desire to ensure global stability. The typical design cross-section (Figure 3) displays the wall components.



Photograph 3.Site 1 Backfilling.

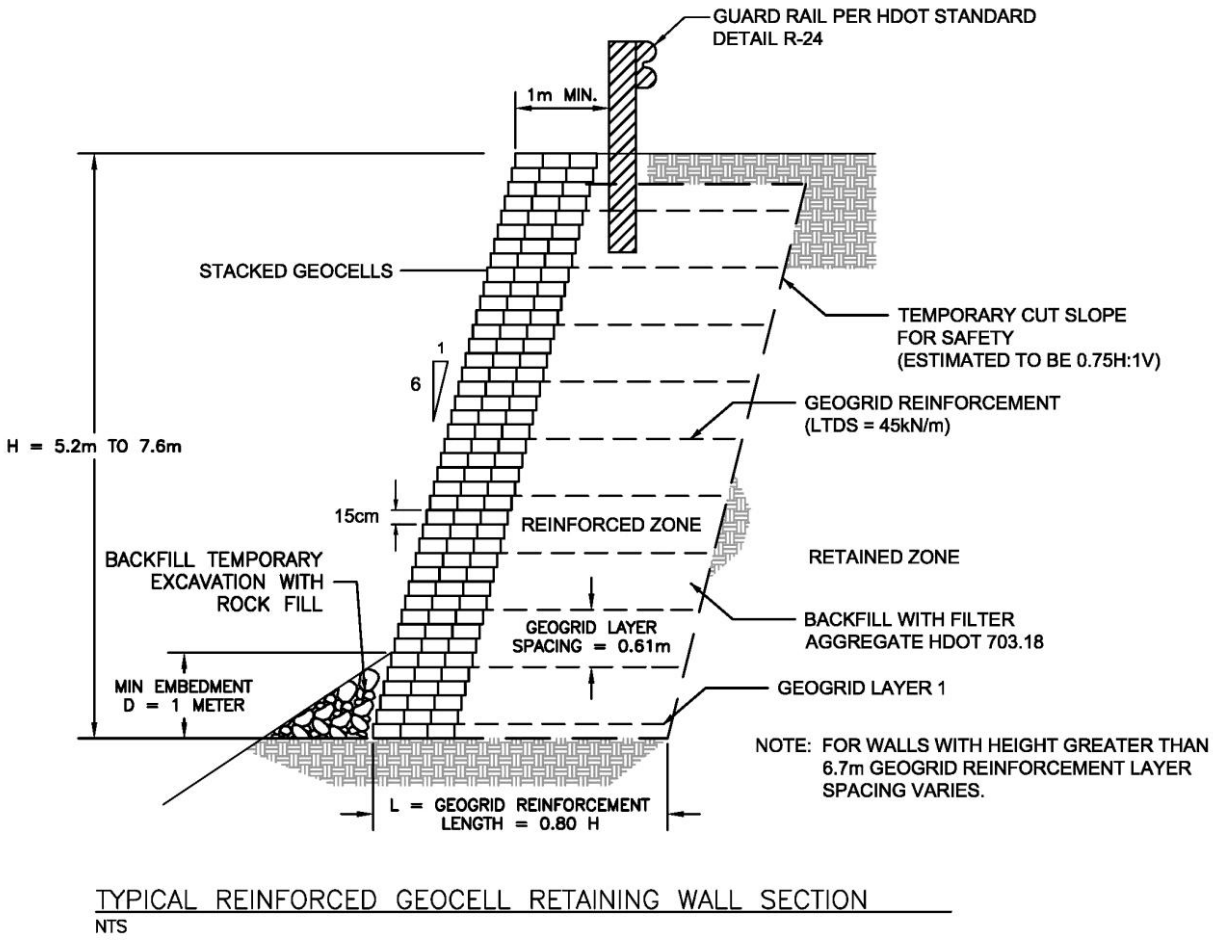


Figure 3. Design Cross-Section.

3.4 Other Design Considerations

This paper is focused on the use of geogrid reinforced geocell retaining walls, however it should be noted that there were other design and construction tasks. Surface water drainage improvements were an important component of the project in order to reduce the risk of future erosion. The project also included repairing electrical service that had been lost due to the landslides.

4. CONSTRUCTION

The advantages of geogrid reinforced geocell walls quickly became evident during construction. The light-weight materials are easier to transport; less expensive to ship to Hawaii, and easier for laborers to carry and assemble at the wall location than typical precast blocks or cast-in-place construction. The light-weight, flexible facing tolerates settlement better than heavier, rigid facing elements. It is also very easy to adjust the facing during construction to accommodate actual site conditions and the specified wall position tolerances. Since the geocells are 15 cm high, the backfill lifts are always 15 cm thick, never more. The thin lifts ensured that sufficient compaction (95%) could be achieved with relatively small equipment.

There were several advantages over the conceptual steel pile and concrete walls including:

- All of the materials could be easily and cost effectively transported;
- No need for a drill rig and a crane, and the contractor did not need to provide access for a concrete truck. The heaviest equipment near the edge of the landslides was the bucket of the tracked excavator.
- Geosynthetics are not corrosive, which was an important consideration in a tropical environment.

- The outer edge of the geocells will eventually become vegetated blending the wall into the tropical environment. In addition, the green colored geocells initially blended in with the lush vegetation of the ravine, even without becoming vegetated (Photograph 4).



Photograph 4. Site 4 Near the End of Construction.

5. COMPARING COSTS

The engineer's cost estimate for the design-build RFP was approximately $5 million, based on the conceptual steel pile and concrete retaining walls. The selected design-build team completed the project for approximately $2.9 million by constructing geocell faced walls with geogrid reinforcing and drainage improvements instead of the conceptual walls presented in the RFP. Excavation costs were about $970 per square meter of wall face, and the wall construction with imported gravel backfill was $3,200 per square meter of wall face. These unit costs will appear high compared with typical reported construction costs for MSE walls, which is due to the following factors:

- The project was on Oahu, an isolated island with limited competition, a high cost of living, and where all supplies and equipment must be shipped ahead of time.
- The project was competed under a federal contract that required contractors to be experienced with additional procedures and reporting requirements.
- Site access constraints allowed only one wall to be constructed at a time resulting in reduced economies of scale and concurrent activity scheduling instead of simultaneous,.
- The five walls were tall, ranging from 5.2 m to 7.6 m deep, but short in length. The total wall face area for each wall ranged from 49 m^2 to 93 m^2, for a total of 330 m^2 of wall facing.

6. CONCLUSIONS

This case history demonstrates the advantages of using geosynthetics for retaining wall components in difficult access conditions, and the advantage of the design-build contracting process. The design-build team repaired the landslides using safer, lighter, greener and less expensive materials than the original concept of steel pile supported concrete walls, saving approximately $1.5 million in the process.

Geocells reinforced with geogrids were the winning solution for repairing the landslides at these difficult access sites.

- The lightweight materials were easy to transport to the project and were easy to carry and place in position.
- Only small, light-weight equipment was needed to construct the walls.

- The geosynthetic materials allowed for flexibility in the layout of the walls and tolerance to differential settlement.
- Geosynthetics were non-corrosive, important in tropical environments.
- The green geocell facing blended into the tropical vegetation surrounding the walls.
- The materials and construction methods allowed for a cost effective method of repairing the failed slopes.

ACKNOWLEDGEMENTS

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