# Slope Rehabilitation at the Baltimore-Washington Parkway Using Rammed Aggregate Piers

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Word Count

| | | |
|---|---|---|
| Abstract | = | 0250 |
| Text | = | 2096 |
| Tables (3x250) | = | 0750 |
| Figures (13x250) | = | 3250 |
| **Total** | **=** | **6346** |

**Transportation Research Board**
**83rd Annual Meeting**
**January, 2004**
**Washington, D.C.**

# Slope Rehabilitation at the Baltimore-Washington Parkway Using Rammed Aggregate Piers

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**ABSTRACT :** The Baltimore-Washington Parkway is a major traffic artery traveled by daily commuters in suburban Washington, D.C., USA. Much of the parkway roadbed lies on a raised embankment constructed to allow for grade separation between the parkway and surface streets facilitating access to the parkway. After a series of rainstorms in the fall of 2002, the fill embankment at the recently reconstructed Route 197 interchange failed resulting in settlement and lateral movement of a retaining wall that supports the southbound lanes of the parkway. The slide encroached upon the traffic lanes, jeopardizing the integrity and safety of the parkway.

After analyzing a variety of possible solutions, the United States Federal Highway Administration and the National Park Service opted to stabilize the slide using *Geopier Rammed Aggregate Piers* acting in concert with a toe berm stabilized with *Tensar* high-strength structural geogrid. The rammed aggregate piers improved the stability of the slope by providing significant increases in the composite shear resistance because of their high angle of internal friction (44-52 degrees).

This paper presents a case history of the use of rammed aggregate piers to stabilize a landslide. The analytical methods used in the design solution are presented along with a description of the construction sequence. This paper is significant because it describes how a simple and cost-effective solution may be implemented to stabilize landslides.

**Keywords**: embankment stability, slope stability, rammed aggregate piers, soil improvement

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## INTRODUCTION

The Baltimore-Washington Parkway is a 29-mile, limited access, four-lane divided roadway running between Washington, D.C. and Baltimore, Maryland (Figure 1). The roadway is considered the ceremonial entrance route to the Nation's Capital from the north and currently serves as a north-south commuter route providing access to a number of government agencies and facilities along its alignment. Parkway construction began in 1942 and the roadway was opened to traffic in October, 1954. Traffic volumes in the southbound lanes in the project area now exceed 100,000 vehicles per day.

The Parkway interchange with Maryland State Route 197 has been improved to enhance safety and traffic flow. The improvements included: construction of eastbound Route 197 under the Parkway, two new Parkway bridges over eastbound Route 197, replacement of two Parkway bridges over westbound Route 197, and ancillary improvements including placement of top-of-slope precast concrete safety barrier wall (Figure 2). In the fall of 2002, after a prolonged period of heavy rains, the approximately 2(H):1(V) slope on the west face of the southbound Parkway embankment between the Parkway bridges over Route 197 began to show signs of movement. The failure led to horizontal and vertical movement of the barrier wall and threatened a wooded area at the toe of the slope (Figure 3). After a variety of alternatives were considered, the Federal Highway Administration (FHWA), responsible for design and construction of the roadway, opted to stabilize the slope using a steepened toe berm supported and stabilized by rammed aggregate piers. This option was chosen because it could stabilize the slope while preserving the National Park Service wooded area and allow the southbound lanes of the Parkway to remain open to traffic.

This paper describes the geotechnical conditions at the project site and the geotechnical analysis used to design the repair. Design methods and calculations are presented. This paper is significant to highway geotechnical engineers because it describes the design methods that may be used to implement a rapid and economical solution for slope stabilization.

## BALTIMORE-WASHINGTON PARKWAY SLOPE FAILURE

After a series of rainstorms in the fall of 2002, the north slope of the west abutment failed in the direction of the adjacent wooded area at the toe of the slope. The failure manifested itself as significant movement of the barrier wall and tension cracking in the pavement adjacent to the barrier wall (Figures 3 and 4). The slope failure extended approximately 100 m (330 feet) along the Parkway alignment. The surface of the slope was wet and had sloughed in a number of areas. Two underdrains were installed in the wettest areas. However, slope movements continued.

A geotechnical investigation was performed under the supervision of the FHWA to investigate subsurface conditions in the area of the failed slope and to provide geotechnical data for the design of the repair. The investigation included drilling three soil borings at locations shown in Figure 2; conducting direct shear, moisture content, and Atterberg Limits testing; and installing a slope inclinometer to define the depth of the failure surface. No obvious toe bulging of the slope was observed during several site inspections. Although the slope surface was soft and wet in some areas, no free ground water was observed in the borings.

Figure 5 presents a cross-section showing the geotechnical conditions encountered in the borings. The fill embankment was constructed using predominantly clayey soils. Standard penetration and classification test results are summarized in Table 1. A direct shear test was performed on an undisturbed sample taken from boring B-2 at a depth of 2.5 to 3 m (8 to 10 feet) below the ground surface. Test results are summarized in Table 2. The embankment foundation soils consisted of highly-plastic Coastal Plain soils with standard penetration test N-values ranging between 5 and 53 blows per foot (bpf).

Slope inclinometer profiles measured after the slope failure are presented in Figure 6. The inclinometer installed within boring B-2 advanced near the slope crest indicated slope movements were experienced in the upper 5 m (16 feet) of the soil profile after the failure had already occurred.

Based on the slope geometry, subsurface conditions, observed surface conditions, and inclinometer data, it appeared that the cause of the failure was softening of the plastic clay embankment soils. The loss of shear strength appeared to be linked to surface infiltration versus a rise in phreatic surface as a result of precipitation. Additionally, wetting and drying cycles of the embankment soils over time could have contributed to a loss of shear strength.

## BACK-CALCULATIONS FOR SLOPE FAILURE

After the completion of the supplemental geotechnical investigation, stability calculations were performed on the existing slope geometry using the computer program PCSTABL6H. The modified Bishop method of analysis was utilized. An existing failure surface was determined using the observed cracking in the Parkway pavement adjacent to the barrier wall and the failure surface elevation determined in the inclinometer casing. Various probable angles of internal friction were analyzed using the existing slope geometry, the probable failure surface, and a factor of safety of 1.0. Figure 7 presents the results of the analyses for failed conditions using trial shear strength parameters. Shear strength parameters and appropriate soil properties to be used in slope stabilization design were determined based on backcalculations and laboratory test results with input from FHWA geotechnical engineers assigned to the Administration's Eastern Federal Lands Highway Division. A drained angle of internal friction of 18 degrees and a cohesion intercept of 1 kilopascal (20 pounds per square foot, psf) were used in design of the slope stability system.

## SLOPE STABILIZATION DESIGN OPTIONS

A variety of options were considered for the repair of the failed slope. These options included retaining walls, a rock buttress system, and remove/replace of some volume of existing slope material. Several factors impacted the selection process. Disturbance of the mature trees beyond the toe of the slope had to be limited. Restrictions to traffic flow in the Parkway southbound lanes had to minimized while the barrier wall sections were removed temporarily. This led to the inclusion of a working surface to allow a crawler crane to lift the barrier wall sections from the toe area of the slope. After numerous studies, cost comparisons, and aesthetic considerations, the repair option depicted in Figure 8, including a toe berm and foundation reinforcement with rammed aggregate piers, was selected for design. As part of the stabilization plan, the barrier wall sections would be temporarily removed and replaced following installation of rammed aggregate piers to limit potential future differential settlements of the wall segments.

## DESIGN AND CONSTRUCTION OF RAMMED AGGREGATE PIERS

### Shear Strength

The shear strength of the reinforced soil mass is determined by calculating the weighted average of the shear strength of the matrix soils and the rammed aggregate piers. Index test results, embankment soil direct shear test results, and shear strength parameter values of the matrix soils and rammed aggregate piers are described in Tables 1 through 3.

The shear strength of the rammed aggregate pier material has been determined from the results of full-scale direct shear tests performed on rammed aggregate pier elements at a project site near Atlanta, Georgia (Fox and Cowell, 1998) and by triaxial tests performed at Iowa State University (White, 2001). Test results, shown in Figures 9 and 10, respectively, indicate a friction angle of about 48 degrees for piers constructed from manufactured open-graded stone (no fines) and a friction angle of about 52 degrees for piers constructed from manufactured graded aggregate base material (5 to 10 percent fines) consisting of recycled concrete. A friction angle of 44 degrees was used for the recycled concrete material to be used to construct the rammed aggregate pier shafts. The 44-degree friction angle for recycled concrete was determined by testing performed at Iowa State University (White, et al., 2002).

### Composite Shear Strength

The composite shear strength of soils reinforced with rammed aggregate piers is computed using the conventional method of calculating the weighted average of the shear strength components of the rammed aggregate piers and matrix soil materials (FHWA/RD, 1983). The composite cohesion intercept ($c_{comp}$) is computed with the expression:

$$c_{comp} = c_g{*}R_a + c_m\,(1\text{-}R_a) \qquad (1)$$

where $c_g$ is the cohesion intercept of the rammed aggregate pier aggregate, $c_m$ is the cohesion intercept of the matrix soils, and $R_a$ is the ratio of the total rammed aggregate pier area to the gross footprint area of the reinforced soil zone. Because the cohesion intercept of the rammed aggregate pier elements is zero, Equation 1 reduces to:

$$c_{comp} = c_m\,(1\text{-}R_a). \qquad (2)$$

The composite friction angle ($\phi_{comp}$) is computed with the expression:

$$\phi_{comp} = \text{arc tan}\,[R_a \tan\phi_g + (1\text{-}R_a)\tan\phi_m] \qquad (3)$$

where $\phi_g$ is the friction angle of the rammed aggregate pier material and $\phi_m$ is the friction angle of the matrix soils. The composite cohesion and friction angle values (Equations 2 and 3) are used to represent the composite shear strength of the soil layers reinforced by the rammed aggregate piers. Composite angles of internal friction and cohesion intercepts for various concentrations of pier elements are presented in Table 3.

## Stability Analyses

Stability analyses were performed on sections approximately 15 m (50 feet) apart along the alignment of the failed slope. In each case, a minimum factor of safety of 1.25 was used to determine the geometry of both temporary construction, which involved the toe bench and crane surcharge, and final conditions which included a 12 kPa (250 psf) traffic surcharge on the roadway behind the barrier wall. Long-term ground-water levels were not available from the borings drilled as part of the slope failure investigation. Borings advanced in the area of the slide, as part of previous explorations for the existing ramps and bridge abutments, encountered ground-water levels at approximately +42.7 m (+140 feet), mean sea level. Therefore, a piezometric surface was set at 1.5 m (5 feet) below the ground surface at the toe of the slope which corresponds to the ground water elevation encountered in the previous borings.

In order to achieve the minimum factor of safety of 1.25 for the final conditions along the slope, it was determined that a counterbalance berm would also be required. The berm breaks the slope angle into approximately a 4(H):1(V) angle in the upper half and a 2(H):1(V) angle in the lower half. The steeper lower half is armored with riprap-sized crushed stone. The berm also resulted in increased shear strength of the rammed aggregate piers due to the additional confinement and increased normal stresses on potential failure planes. The effects of the increased composite shear strength as a result of installing piers to support the barrier wall were also considered in the analyses.

Stability analyses were performed using PCSTABL6H and a circular surface search routine. Representative results of stability analyses performed for the design solutions are presented in Figure 11.

## Construction

The construction of rammed aggregate piers is described in the literature (Lawton and Fox, 1994; Lawton, et al., 1994; Wissmann and Fox, 2000; Wissmann, et al., 2000; Wissmann, et al., 2001; Minks, et al., 2001) and shown in Figure 12. A total of 472 rammed aggregate piers for slope stabilization were installed by drilling 760 mm (30 inch) diameter holes to depths ranging between 3 and 6.5 m (10 and 21 feet), placing controlled lifts of aggregate within the cavities, and compacting the aggregate using a specially designed high-energy beveled impact tamper. The first lift consisted of clean stone and was rammed into the soil to form a bottom bulb below the excavated shaft. The piers were completed by placing additional 0.3-m (one-foot) thick lifts of aggregate over the bottom bulb and compacting the aggregate with the beveled tamper. During densification, the beveled shape of the tamper forces aggregate laterally into the sidewall of the excavated cavity. This action increases the lateral stress in the matrix soil thus providing additional stiffening and increased normal stress perpendicular to the perimeter shearing surface.

The toe berm was constructed utilizing a wire basket and geotextile wrap facing system and Tensar UX1100 uniaxial geogrid. The six-step process involving berm construction, toe pier placement, crane activity, wall pier placement, and final grading is shown in Figure 13.

## SUMMARY AND CONCLUSIONS

Rammed aggregate piers were used in conjunction with a counterbalance berm to effectively stabilize a failed slope. A precast concrete barrier wall at the crest of the slope was also supported with rammed aggregate piers. The slope area was in a location that involved limited and difficult access, high traffic volumes, and aesthetic preservation issues. The stabilization work was able to progress through the winter and spring of 2002-2003 despite unusually wet weather conditions.

## ACKNOWLEDGEMENTS

The General Contractor for the project was Concrete General of Gaithersburg, MD. GeoConstructors, Inc. of Leesburg, Virginia was the rammed aggregate pier installer for the project. The opinions, findings, and conclusions expressed in this paper are those of the authors.

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## LIST OF TABLES



## LIST OF FIGURES

**TABLE 1. Fill Embankment Insitu and Index Test Results Summary**

| Standard Penetration Test Range (bp 0.3m) | Moisture Content Range (%) | -200 Sieve Range (%) | Liquid Limit Range (%) | Plasticity Index Range |
|---|---|---|---|---|
| 3-12 | 16-27 | 64-99 | 28-55 | 9-31 |

**TABLE 2. Direct Shear and Sample Index Test Results Summary**

| Cohesion Intercept (failure) (kPa) (psf) | Angle of Internal Friction (failure) (degrees) | Moisture Content Range (initial) (%) | -200 Sieve (%) | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|
| 22 (460) | 20 | 25-27 | 98 | 43 | 21 |

**TABLE 3. Composite Shear Strength and Properties**

| Rammed Aggregate Piers | | | | | | | | Matrix Soil | | | Composite Weighted Average | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Number of Rows | Spacing (ft) | Dia. (in) | # RAP's (ft of slope) | Zone Width (ft) | Area Ratio, $R_a$ (%) | $c_g$ | $\phi_g$ | Area Ratio, $(1-R_a)$ (%) | $c_m$ | $\phi_m$ | $c_{comp}$ | $\phi_{comp}$ |
| 6 | 4.25 | 30 | 1.41 | 23.75 | 0.29 | 0 | 44 | 0.71 | 20 | 18 | 14.2 | 28.0 |
| 6 | 3.50 | 30 | 1.71 | 20.00 | 0.42 | 0 | 44 | 0.58 | 20 | 18 | 11.6 | 31.7 |
| 1 | 5.00 | 30 | 0.20 | 2.50 | 0.39 | 0 | 44 | 0.61 | 20 | 18 | 12.1 | 30.9 |

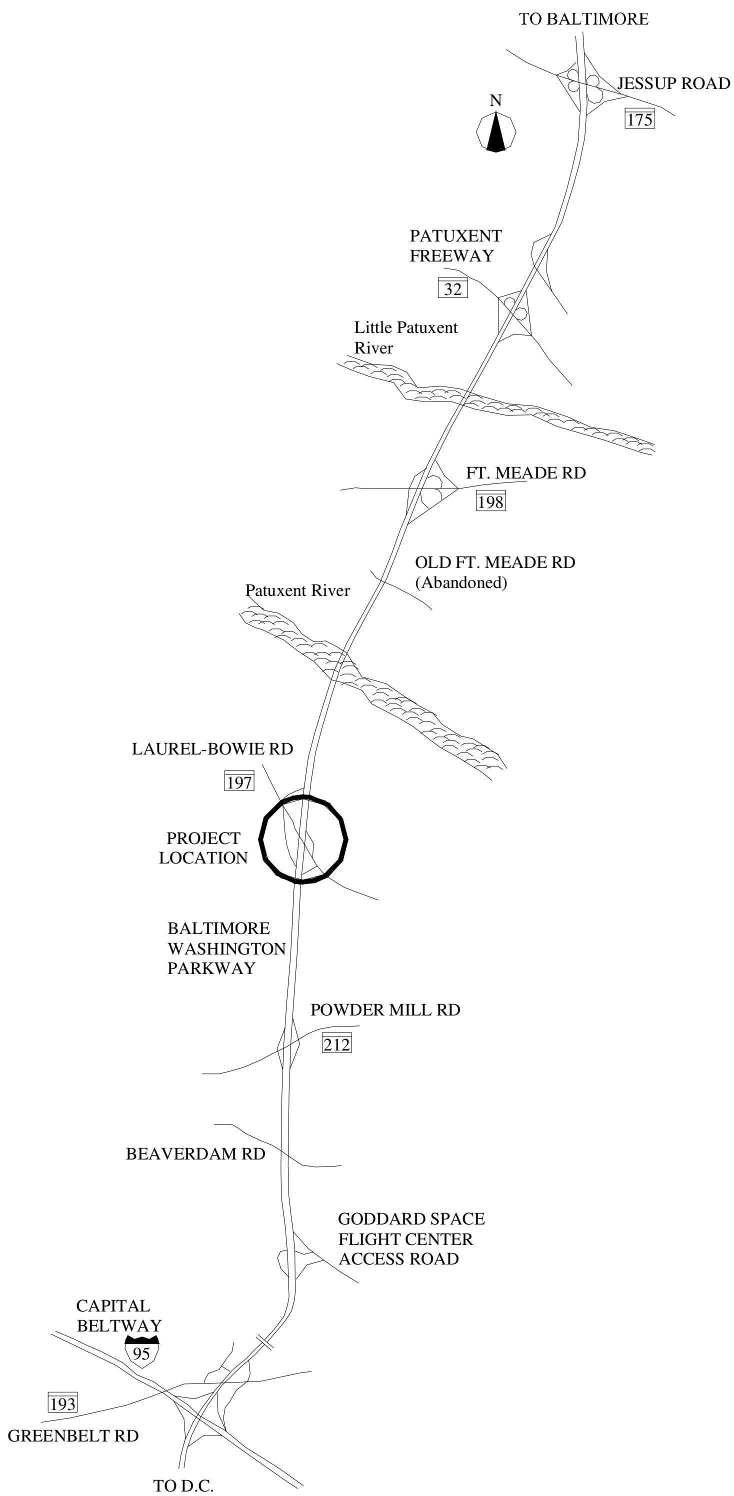


FIGURE 1. Project location map

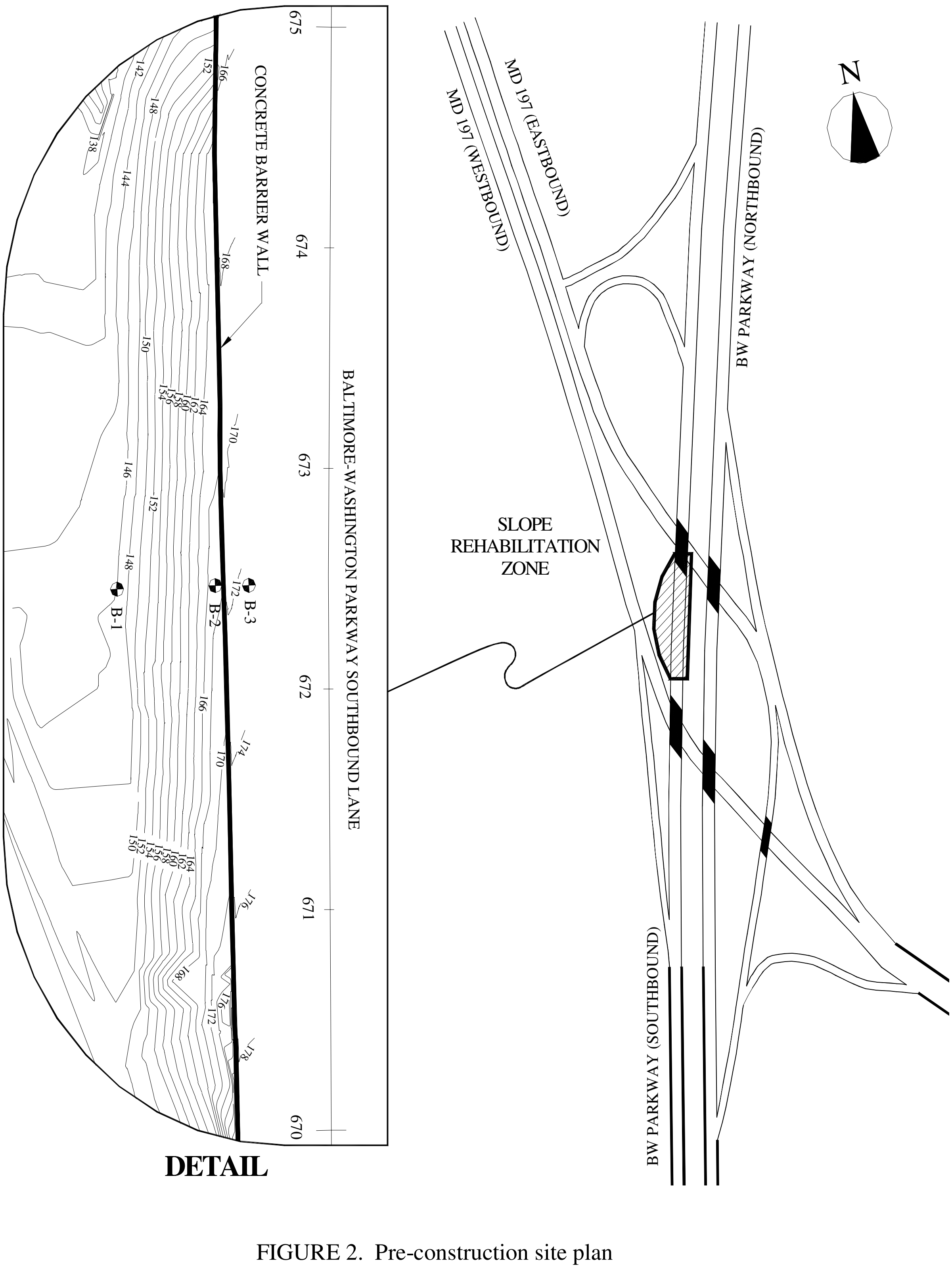


FIGURE 2. Pre-construction site plan

FIGURE 3. Photograph of failed slope area

FIGURE 4. Photograph showing distress in the roadway behind concrete barrier wall

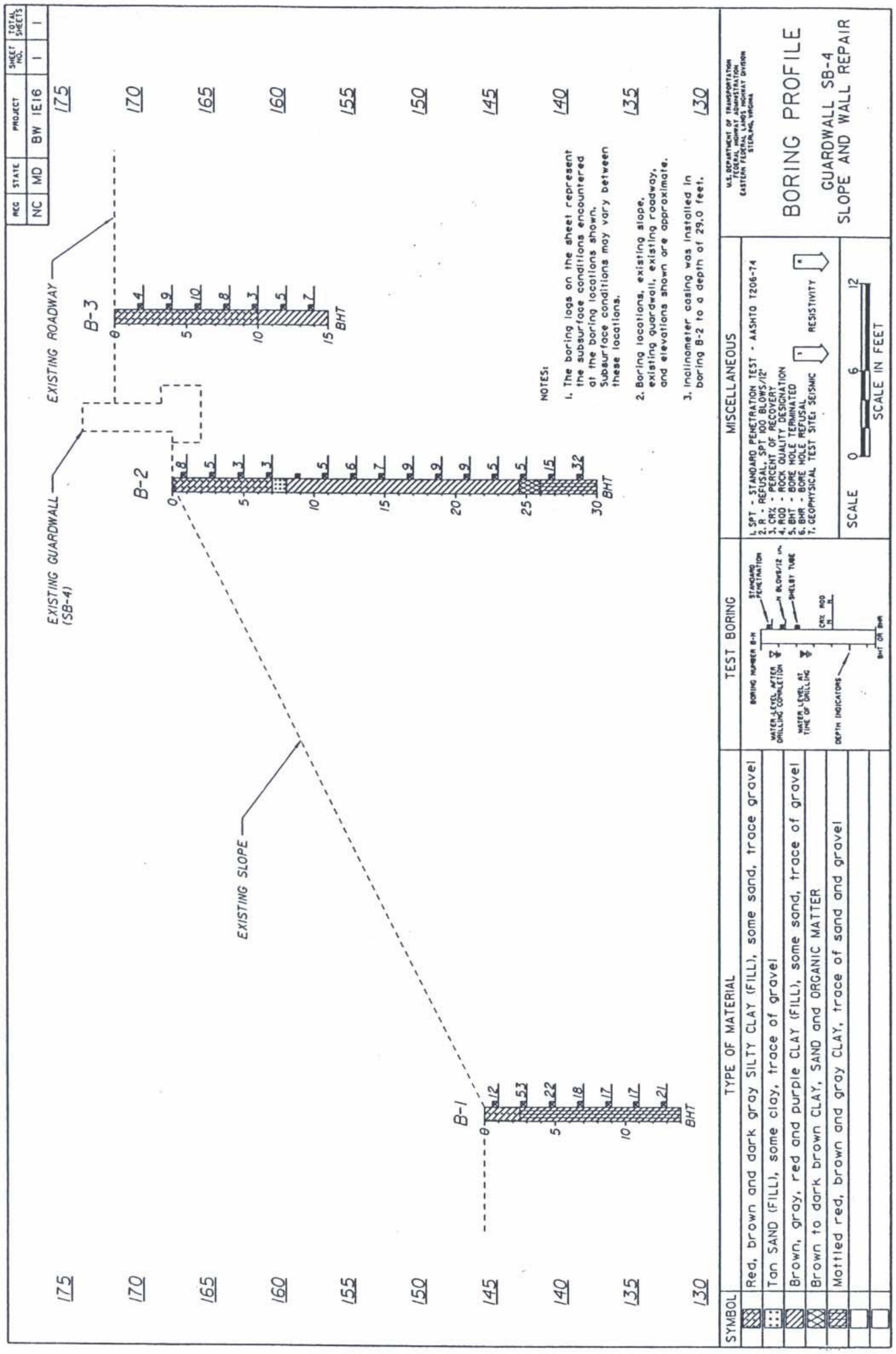


FIGURE 5. Cross-section showing slope geometry and conditions in soil borings

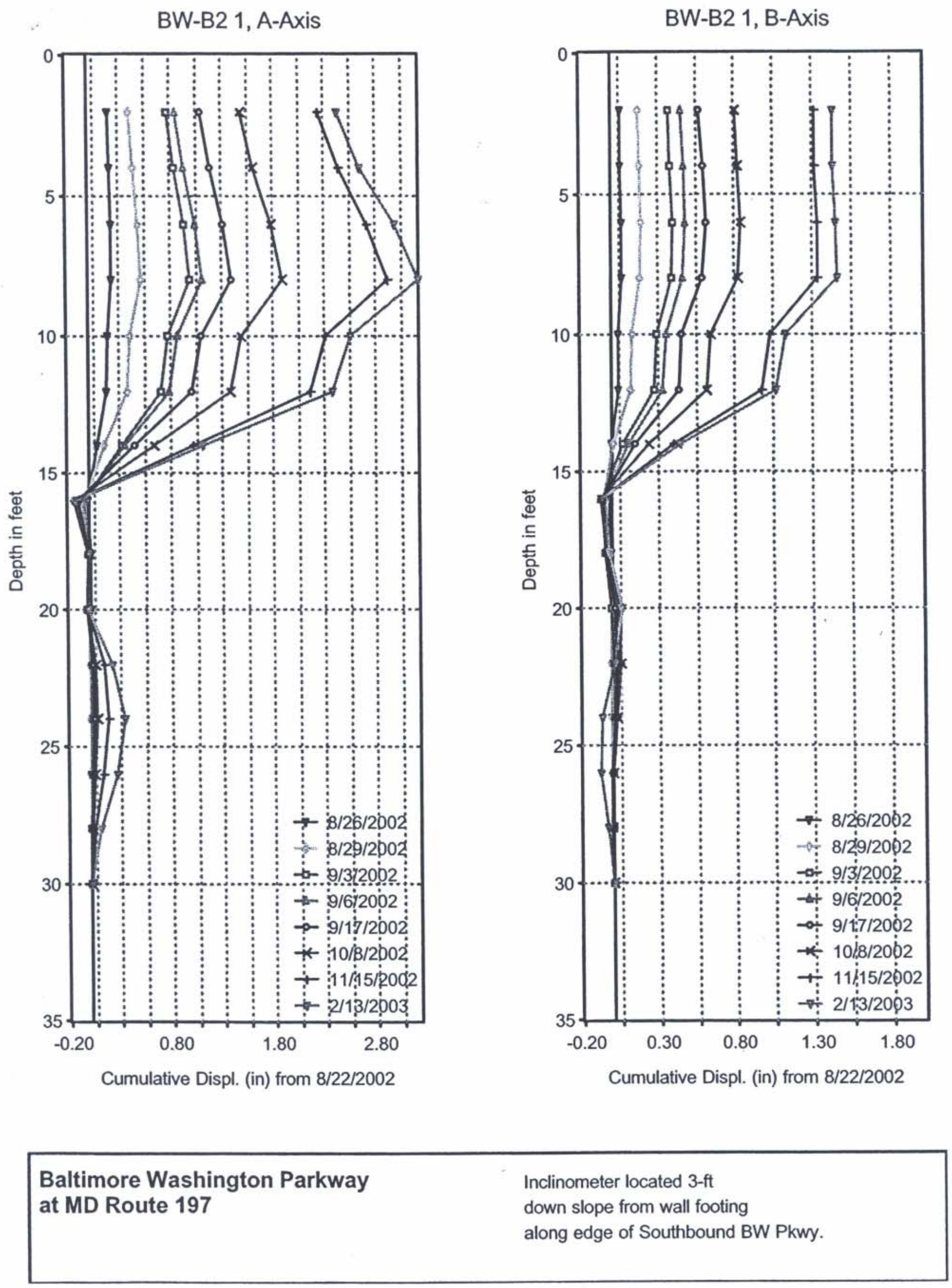


FIGURE 6. Slope inclinometer profiles measured after slope failure

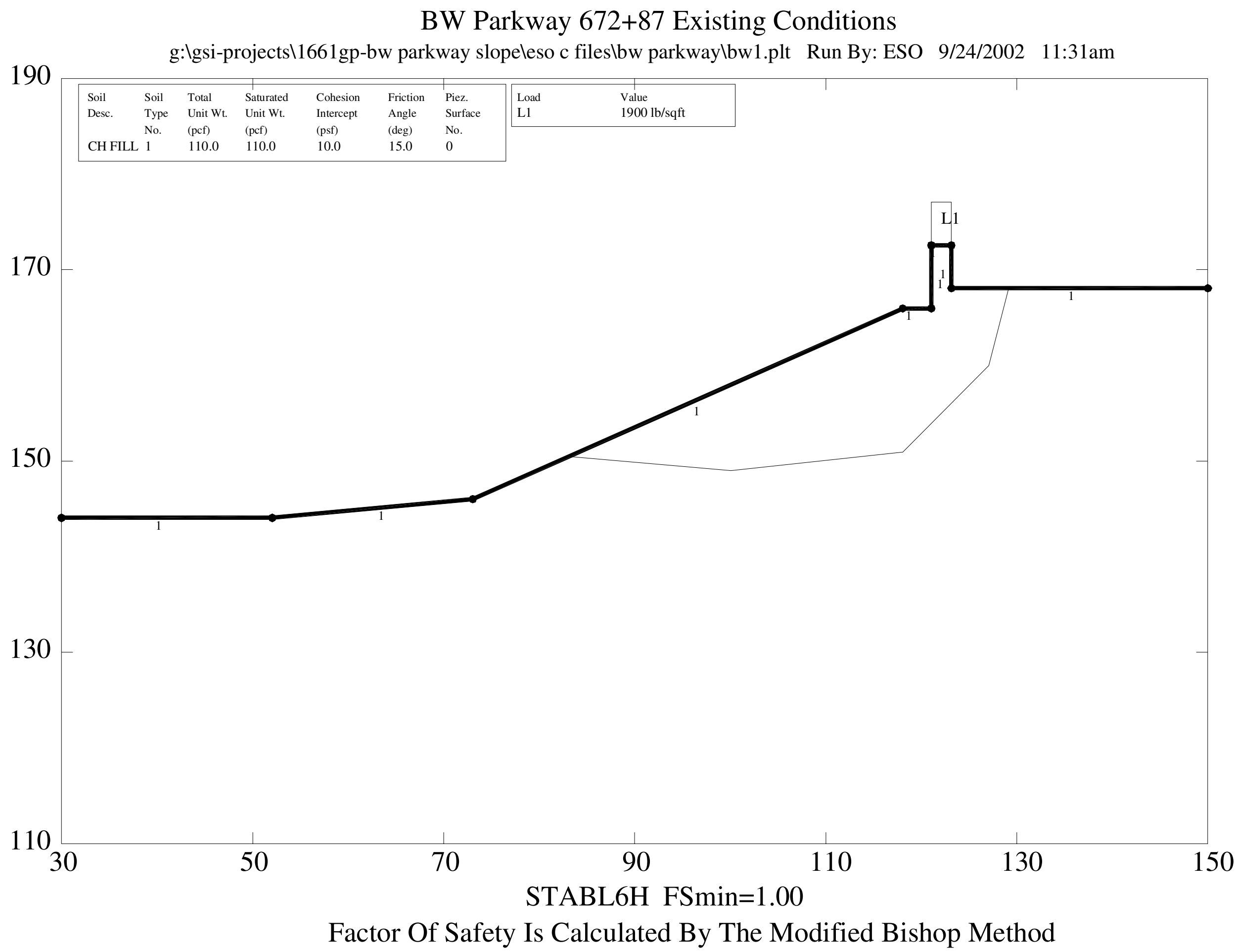


FIGURE 7. Slope stability analysis results – failure conditions

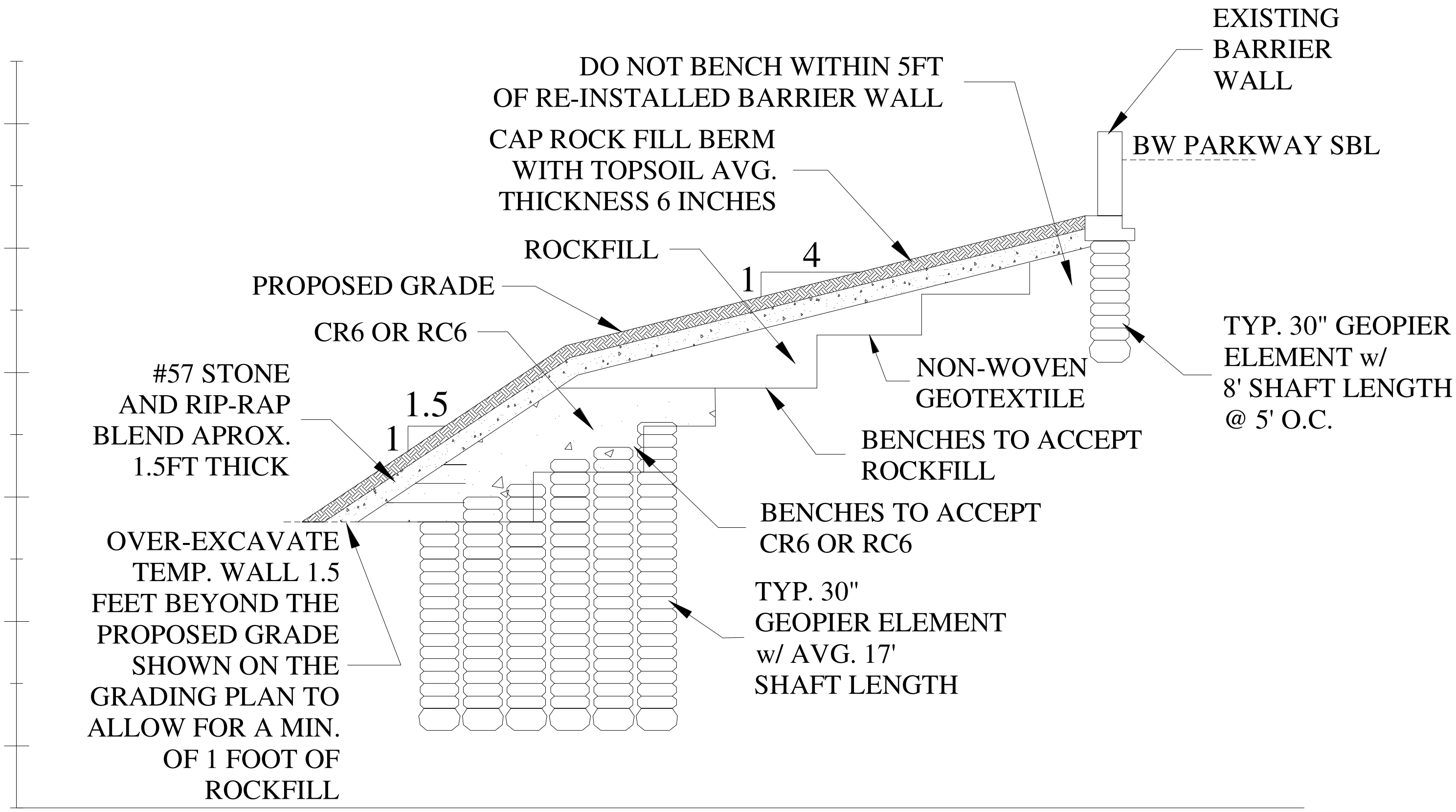


FIGURE 8. Cross-section of selected slope stability option

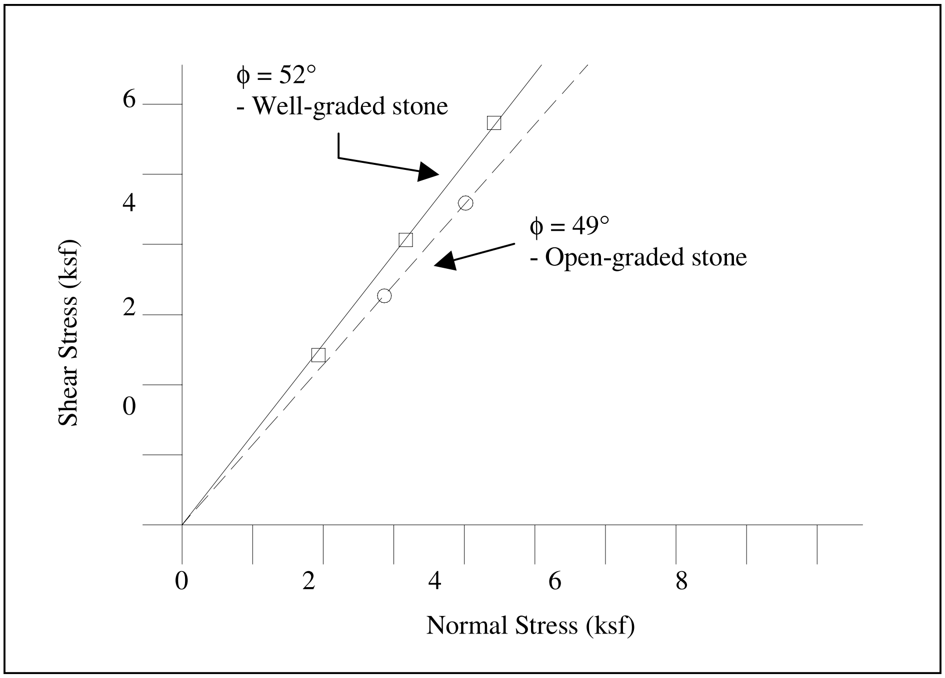


FIGURE 9. Results of in-situ direct shear tests on rammed aggregate piers

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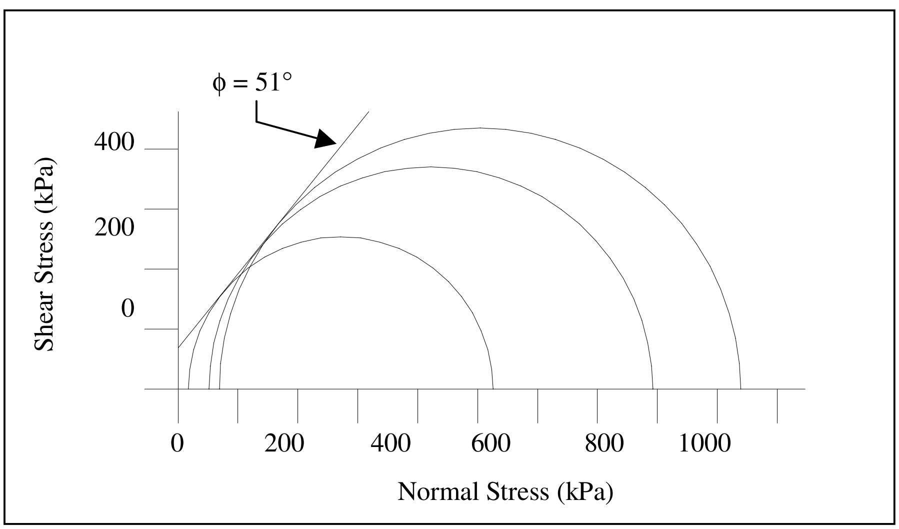


FIGURE 10. Results of triaxial shearing tests performed on reconstituted samples of rammed aggregate pier material

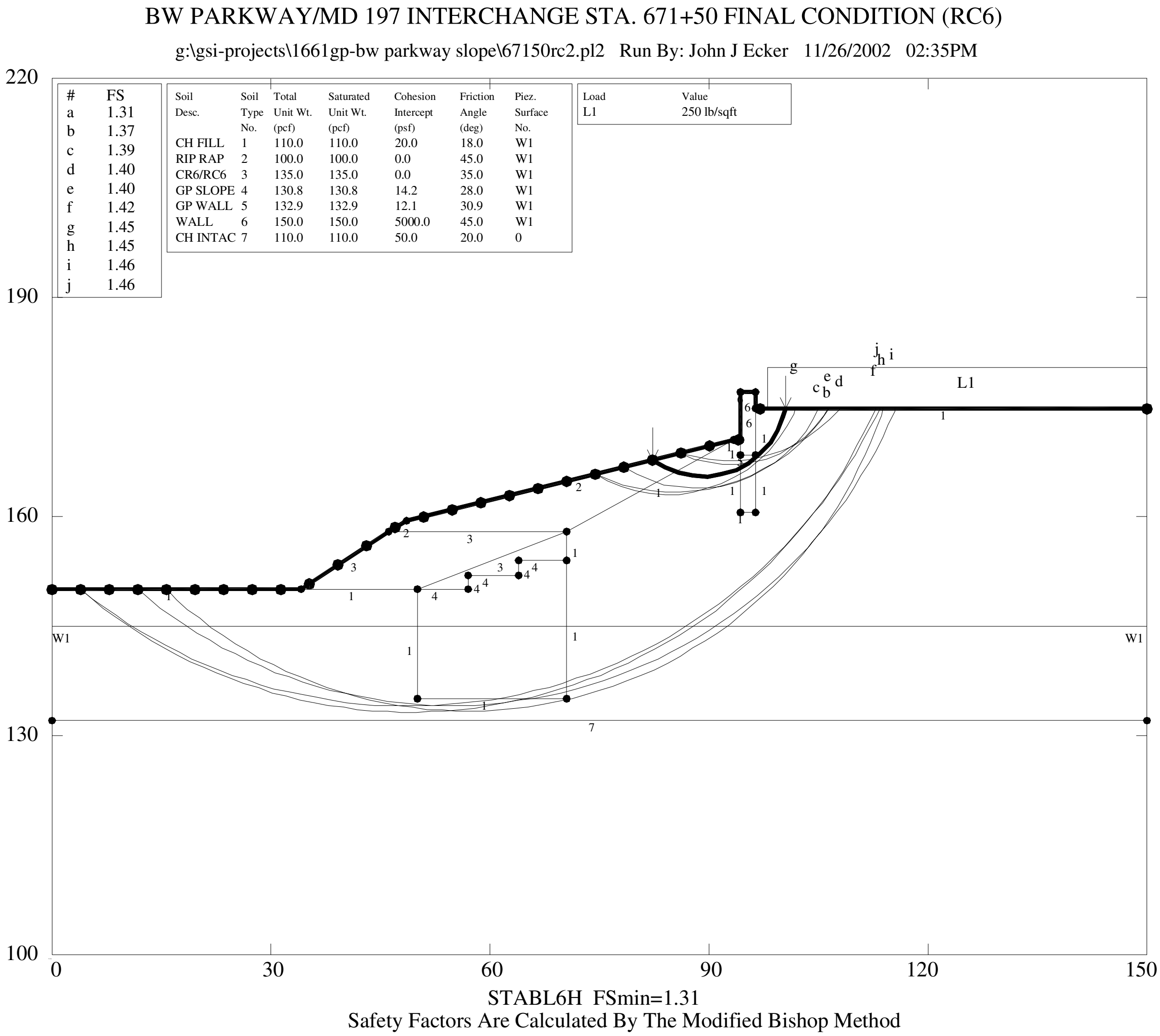


FIGURE 11. Slope stability analysis results – design conditions

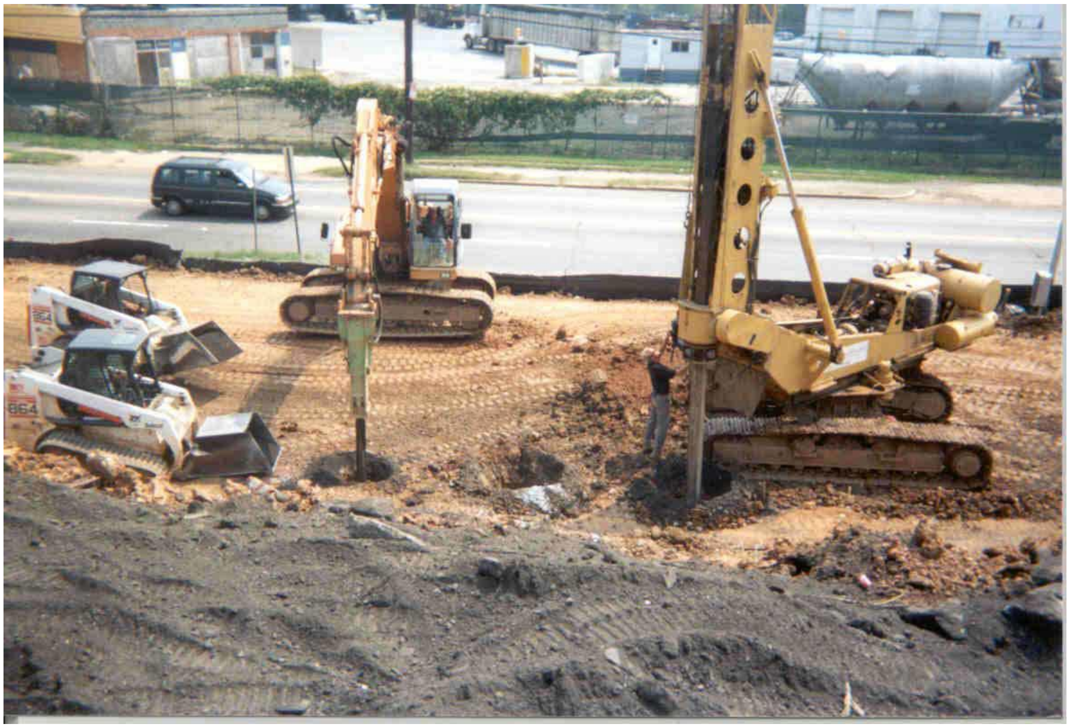

FIGURE 12. Rammed aggregate pier construction

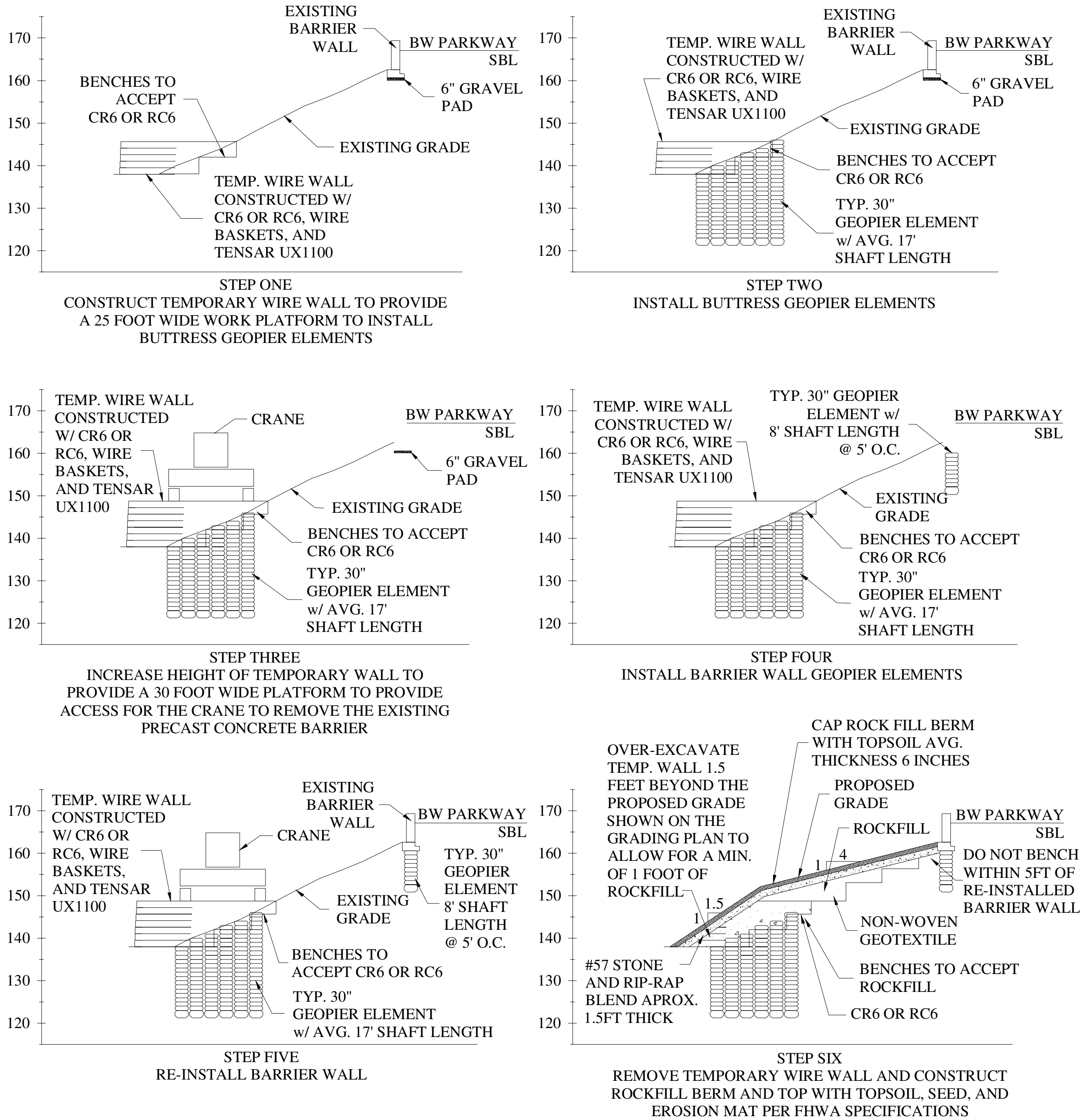


FIGURE 13. Six-step stabilization construction sequence