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REMEDIATION OF RIVER DES PERES SLOPE FAILURE A CASE HISTORY

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ABSTRACT

This paper describes the remediation of a 300-foot long slope failure on the north bank of River Des Peres along Ellendale Avenue near the intersection with Wellington Avenue in St. Louis, Missouri. The slope failure threatened the east bound driving lane of Ellendale Avenue and a 24-inch high pressure gas line under a concrete swale along the south shoulder, and damaged the concrete channel side and bottom lining and a foulwater interceptor sewer in the River Des Peres channel. The failure occurred within a soil stratum that consists of soft to very soft, gray silt, clayey silt, silty clay and sandy silt with a trace of organics and wood. As an emergency measure, to protect Ellendale Avenue and the high pressure gas pipeline from the advancing slide scraps, The Metropolitan Saint Louis Sewer District (MSD) the project owner, approved an immediate installation of approximately 330 linear feet of PZ27 and PLZ23 sheet pile sections that were driven to bedrock refusal along the top of the slope about 12 feet south of the road pavement. Data obtained from several inclinometers, installed to monitor the slope during and after the emergency remedial work, indicated no further significant movement of the slope behind the sheet pile wall but movement continued to occur at the toe of the slope. Analysis of the in place sheet pile system revealed the need for one row of rock anchored tiebacks to render an acceptable factor of safety against the sheet pile failure. In order to stabilize the failed slope utilizing the in place sheet pile wall and maintaining the same slope geometry to satisfy the hydraulic requirement of the channel, the slope was stabilized with a grouted rock buttress. The sheet pile wall and the stabilized slope have been performing satisfactorily since the completion of the remedial work.

KEYWORDS

sheet piles, tiebacks, slope stability, slope stabilization, slope instrumentation

INTRODUCTION

An approximately 300-foot long slope failure occurred on June 9, 1995 on the north bank of River Des Peres along Ellendale Avenue near its intersection with Wellington Avenue in St. Louis, Missouri (Figure 1). A site plan showing the main feature of the project is presented on Figure 2. In the vicinity of the current slope failure, the channel walls were at approximately 1V:2H and about 25 feet high. The lower 7.5 feet of the embankment and channel were reinforced concrete paved; the upper part of the slope was covered with hand placed stone revetment (riprap). The foulwater interceptor sewer, a 4-foot deep semi-circular brick or masonry structure, exists near the center of the approximately 40 foot wide channel. In the vicinity of the slope failure, the top of the sewer was covered with 3 to 4 feet wide removable concrete panels which are in contact with the concrete channel of the north and south sides of the River Des Peres. A 24-inch steel high pressure gas pipeline, owned by Laclede Gas Company, exists under a concrete swale along the south shoulder of the east bound lanes of Ellendale Avenue, adjacent to and within the influence of the slope failure.

The slope failure was relatively well defined on the western half, with an exposed slide scarp of 6 to 10 feet adjacent to Ellendale Avenue and the 24-inch high pressure gas line underlying the concrete swale along the shoulder. At the toe, in the same general vicinity, the sloped concrete channel walls were pushed laterally into the channel and were heaved upward. The removable concrete panels appeared to have been pushed 2 to 3 feet under the concrete pavement of the south side of the channel. On the east end of the failed area, the slope failure was not as pronounced but, prior to commencement of emergency repairs, a significant tension crack was observed at the south edge of the eastbound lanes of Ellendale Avenue; lateral displacement of the removable concrete panels over the foulwater interceptor in the channel bottom was also observed. The slope failure, as will be shown by the analysis, appeared to be related to the rapid draw down of water within the channel that occur after a major rain.

Figure 1. Photograph showing the slope failure.

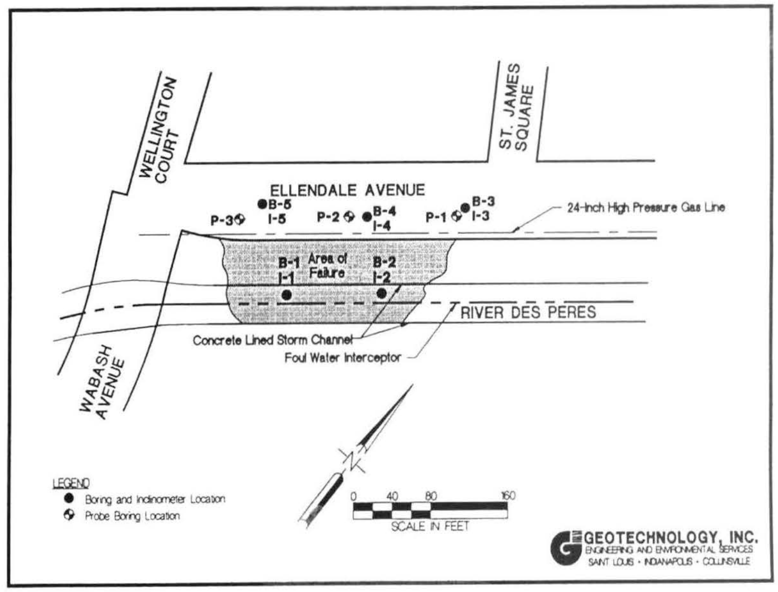

Figure 2. Site plan and boring locations.

Emergency repairs commenced on June 10, 1995 and consisted of driving a wall of PZ27 and PLZ23 steel sheet piles to the top of bedrock approximately 12 feet south of the Ellendale Avenue pavement in the area affected by the slope failure (Figure 3). The steel sheet pile location was chosen by MSD primarily to arrest the advancement of the failed slope, thereby providing some immediate protection to the east bound lane of Ellendale Avenue and the underlying high pressure gas line. Ellendale Avenue was re-opened to traffic on June 27, 1995, following completion of the sheet pile installation. The gas line has remained in service throughout. The long term remediation system, described below, incorporates the steel sheet piles as driven.

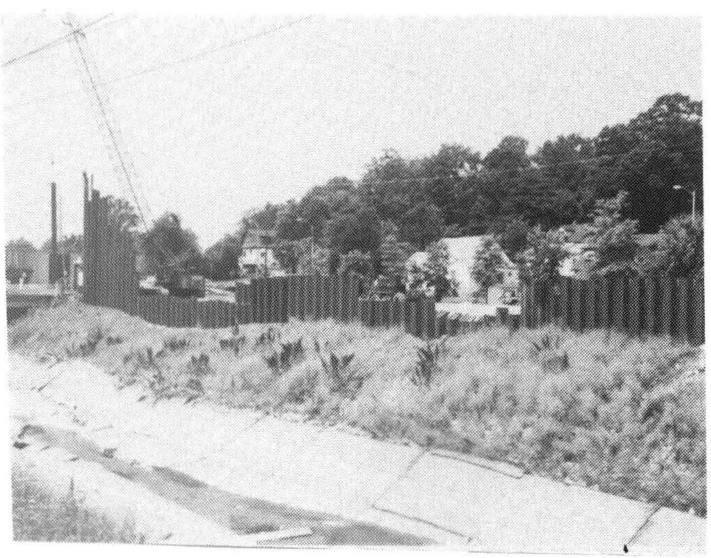

Figure 3. Photograph showing the sheet piles being driven.

BRIEF HISTORY OF RIVER DES PERES

The River Des Peres watershed consists of approximately a 111-square mile area that contains the western one-third of the City of St. Louis and a large portion of south, east central and north central St. Louis County. Included in the St. Louis County area are all or part of 42 municipalities.

The River Des Peres channel improvements were needed to solve the sewage and stormwater drainage problems of the River Des Peres area. Construction was begun in 1923 and was completed in 1933. Some additional work to improve the lower channel, placing riprap, and constructing access ramps, was accomplished from 1935-1940 as part of WPA Projects. The channel includes a lined open channel with two foulwater interceptors, one a masonry structure completed in 1912, and a relief sewer of reinforced concrete completed in 1970. Both interceptor sewers range in size from 6 to 9-feet in diameter and for the most part, run beneath the channel bottom. A comprehensive overflow regulation system improvement project is underway, that includes replacement of the foulwater interceptors and installation of pump stations and gate structures.

The open portions of the RDP channel consist of a mixture of reinforced concrete and natural limestone bedrock bottoms with varied side wall and slope construction. In this project area, the sidewalls consist of sloped reinforced concrete walls or grouted riprap with a section of dry-laid riprap above. The bottom of the channel is reinforced concrete.

SUBSURCE CONDITIONS

The subsurface exploration program (Geotechnology, Inc, 1995) included drilling 3 probes to rock to assist in the sheet pile length selection and drilling 6 borings with soil sampling and rock coring at the top and bottom of the slope at the location shown on Figure 2. Typical boring logs for the upper and lower profile are shown on Figure 4.

Lower subsurface profile (below channel bottom) - Below the reinforced concrete channel bottom, interlayered soft to very soft gray silt, clayey silt, silty clay and sandy silt with a trace of organics and wood, typical of alluvial deposits, was encountered to depths of 7.5 and 11.5 feet in borings B-1 and B-2 respectively. These fine grained soils are underlain by loose to medium dense gray fine sand and silty sand and gray fine to medium gravel until limestone bedrock was encountered at 16 feet beneath the channel bottom.

Upper subsurface profile (bellow Ellendale Avenue) - Below the pavement section of Ellendale Avenue, consisting of asphalt, concrete and crushed rock subbase to a depth of five feet, or topsoil in the median between the east and westbound lanes, the soil consists of an upper stratum of soft to stiff, gray and brown, clayey silt intermixed with silty clay to a depth of 27 to 36 feet beneath the ground surface. These alluvial terrace deposits are underlain by dark brown silt, sandy silt, loose to medium dense, silty sand, clay or gravel alluvial deposits until limestone bedrock was encountered at 43 feet in borings B-3 and B-4 and at 35 feet in boring B-5. The limestone bedrock, based on the recovered cores, is moderately hard to very hard, finely crystalline, with chert imbedded, and no visible bedding planes. The percent core recovery and rock quality designation (RQD) were relatively high, with recovery ranging from 88 to 100 percent and RQD ranging from the upper fifty's to the upper nineties.

Groundwater - Groundwater was observed generally from 1 to 3 feet beneath the channel bottom, and between 20 and 21 feet beneath Ellendale avenue.

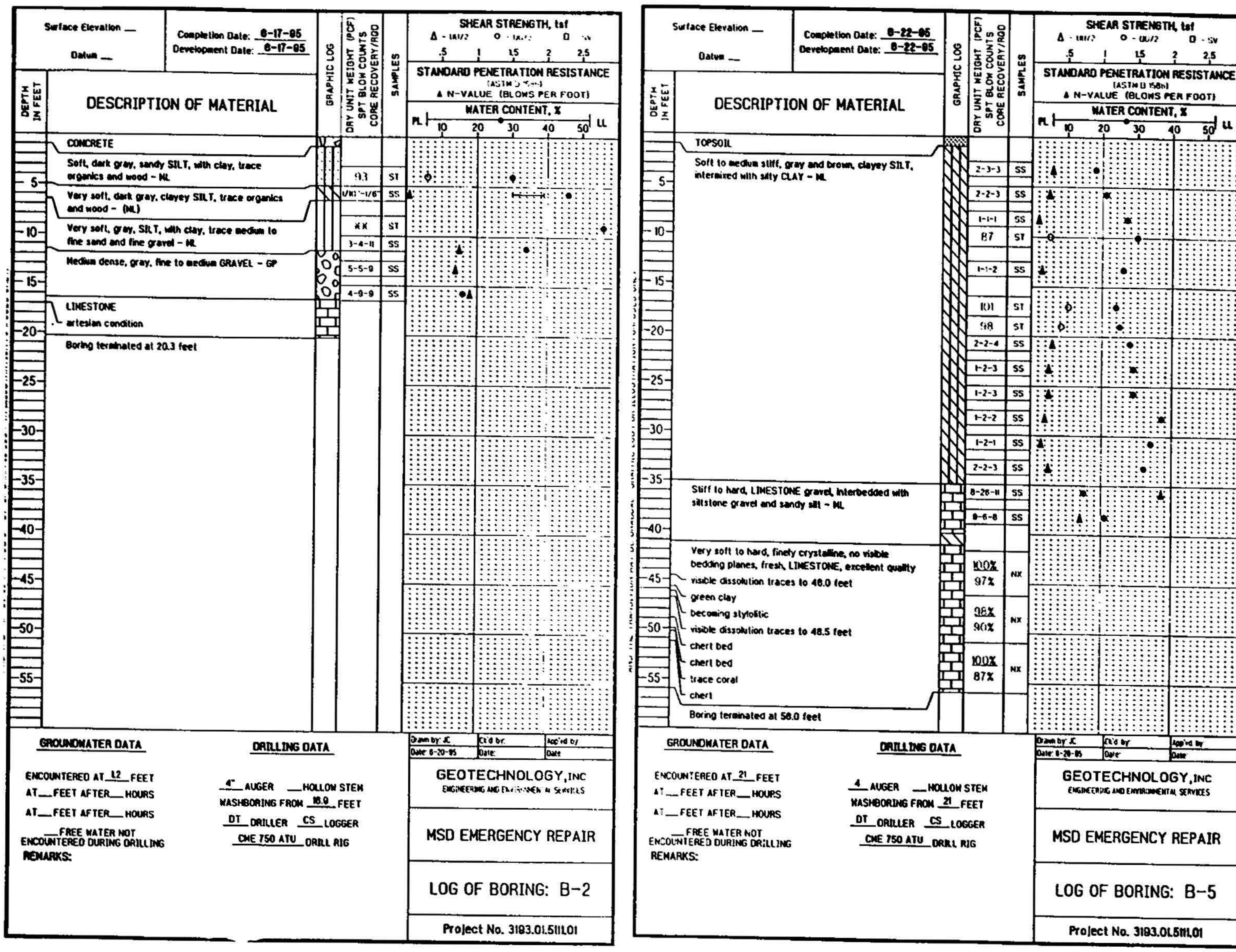

Figure 4. Typical lower and upper boring logs.

INCLINOMETER DATA ANALYSIS

Five inclinometers were installed at the site. Two, I-1 and I-2, were installed at the toe of the slope in the bottom of the channel, and the rest were installed at the top of the slope and were numbered I-3 through -5 as shown in Figure 2. All inclonometer casings were embedded approximately 10 feet into rock and were completely grouted from the outside. Inclonometer data were collected on a regular basis until the failure surface was established with sufficient certainty. Data were collected and analyzed utilizing equipment and computer program manufactured and developed by Slope Indicator company (Slope Indicator Co., 1996). Typical plot of upper and lower inclinometers are shown in Figure 5. The top inclinometers indicated small displacements on the order of 0.1 inches due probably to the disturbance from the sheet pile wall installation. Further, these displacements appeared to be dissipating with time, as indicated by comparing the last readings to the previous. Data from toe inclinometers indicated relatively large displacement continued to occur, particularly at I-1, which is located at the center of the slide mass. Total displacements were approximately 0.5 inches and 0.17 inches for inclinometers I-1 and I-2, respectively. Data from I-1 and I-2 indicated the slip surface to be located approximately 6 feet below the bottom of the channel. Comparisons of the latest two readings indicated 0.08 inches and 0.09 inches displacement for I-1 and I-2, respectively, indicating that displacements was continuing to occur at both locations prior to remediation.

SLOPE STABILITY ANALYSIS AND DESIGN

Analysis of the existing slope, at its conditions after the failure, and proposed remediations were conducted assuming static conditions. The factor of safety was computed by the modified Bishop method utilizing the STABR/G slope stability computer program (Geosoft, 1987). The slip surface geometry was estimated based on the inclinometer data at the bottom of the channel and the existing scarp at the top. The soil properties at failure were then back calculated for a factor of safety equal to one.

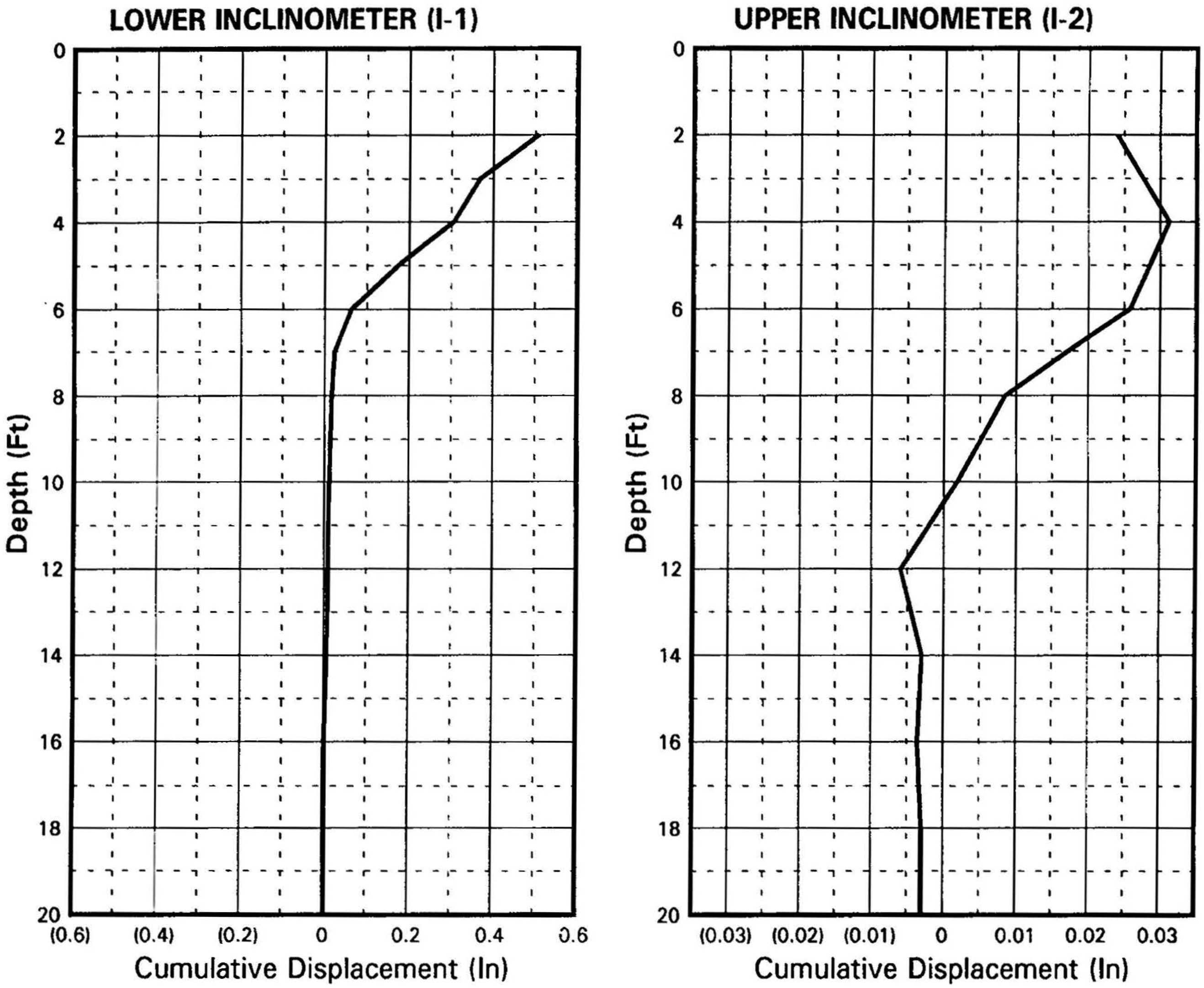

Figure 5. Typical lower and upper inclinometer data.

The analysis was performed for three conditions of water elevation, water at the bottom of channel, water at top of slope, and a rapid draw down scenario, which is believed to have contributed to the conditions that caused the slope failure. Using a moist density of 125 pounds per cubic foot, it was calculated that an internal friction angle of twenty-two degrees would have produced a factor of safety of 1.0, 0.7, and 1.18 for water at the bottom, rapid draw down, and water at the top cases, respectively. Therefore, for all proposed remediations, a friction angle of twenty degrees was used as the residual friction angle. This friction angle compared closely with the residual friction angle obtained from several consolidated-undrained (CU) triaxial tests conducted on representative samples from the site.

Several slope geometries were investigated in order to determine the optimum solution. However, MSD had specified that the repaired slope should match the geometry of the original slope at the bottom lined portion (1V:2H) to satisfy existing channel hydraulic requirements. This geometry jeopardized the stability of the slope. Similar analysis was conducted to incorporate the concrete lining at 1V:2H and re-grade the remainder of the slope to 1V:5H. The factor of safety for the rapid draw down case was 0.77 which was considerably less than the targeted 1.2 factor of safety for this controlling case.

A rock buttress alternate was analyzed. This alternative was evaluated based on keeping the lower concrete lined channel at 1V:2H, re-grading the remainder of the slope to 1V:3H, and constructing a rock buttress at the toe area. Different layouts of rock buttress were investigated and the factor of safety for each layout was computed until an acceptable buttress geometry, that provided the minimum acceptable factor of safety, was achieved for the rapid draw down case as shown in Figure 6.

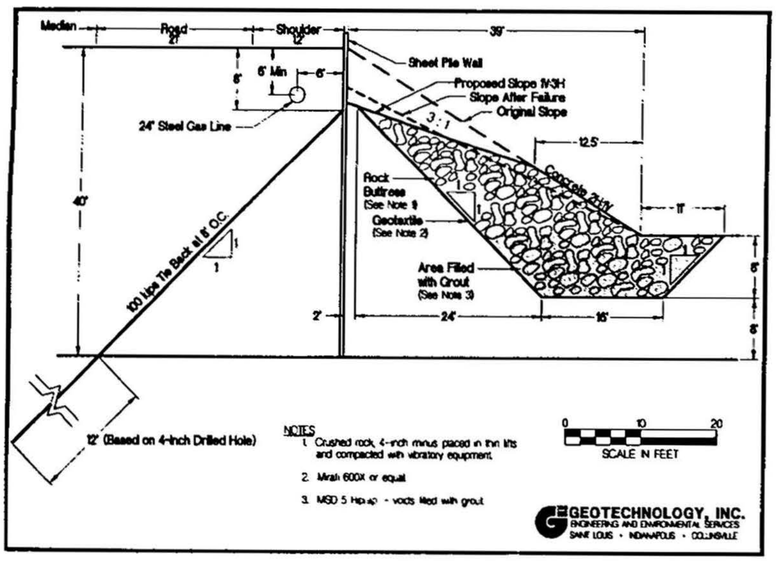

Figure 6. *Slope cross section showing the rock buttress and tiebacks.*

The rock buttress material consisted of 4-inch minus well graded crushed rock placed in relatively thin lifts and compacted with vibratory equipment. Since adequate drainage in the rock buttress cannot be maintained below the channel bottom, MSD 5 rock blanket with predominant rock sizes between 6 and 12 inches (4) was used in this zone, and the voids between the rock pieces were filled with grout to channel bottom surged level. Excavation for the rock buttress was recommended to be accomplished in limited segments perpendicular to the channel alignment to reduce the potential for additional instabilities of the slope. Each segment was required to be bachfilled the same day it was opened.

SHEET PILE WALL ANALYSIS AND DESIGN

After a stable slope geometry was defined, the sheet pile wall was analyzed incorporating the new rock buttress at the passive side. The analysis was conducted following the procedure outlined in the Navy design Manual DM 7-2 (NAVFAC, 1982) for an anchored sheet pile wall. In the analysis the following assumptions were made: a differential hydrostatic pressure of 14 feet behind the sheet pile wall to exist since drainage is limited in that area; surcharge load at road level of 400 psf; no sheet pile penetration into rock; and a factor of safety of 2 on the passive pressure. Based on the analysis, one row of tiebacks each carrying a 100 kips design load and spaced at eight-foot centers was required. Layout of this system is shown in Figure 6. A DYWIDAG bar tendon 1-3/8 inches diameter, with double corrosion protection, was selected to carry the designed tie back load Dywidag System International). These bars were designed to be anchored to the underlying rock which necessitated sloping them at 1V:1H. Embedment length in rock, based on 4 inches diameter drilled hole and 70 psi bond stress between rock and grout (Post-Tensioning Institute, 1980), was 12 feet. After testing, tie backs were locked at 60% of their designed loads. The waler was designed as a continuous beam with uniform load supported at tie back locations (US Army Corps of Engineers, 1994)) which resulted in a waler section having two 12x25 channels. Wedge plates were required at the tieback support locations in order to provide the required1V:1H tiebacks inclination. Detail of the waler and the tieback support system are shown in Figure 7. The stresses resulting from the vertical component of the inclined load were checked to ensure they did not exceed the allowable steel stresses of the sheet pile.

The global stability of the entire system including sheet piles, tiebacks and rock buttress was checked and was found to have sufficient factor of safety (NAVFAC, 1982).

PROJECT PERFORMANCE

The project was completed in 1996. Tiebacks were first installed, grouted, tested and locked at 60% of their design load. Rock buttres swork was then started. Since its completion., the slope and the sheet pile wall appear to be performing satisfactorily.

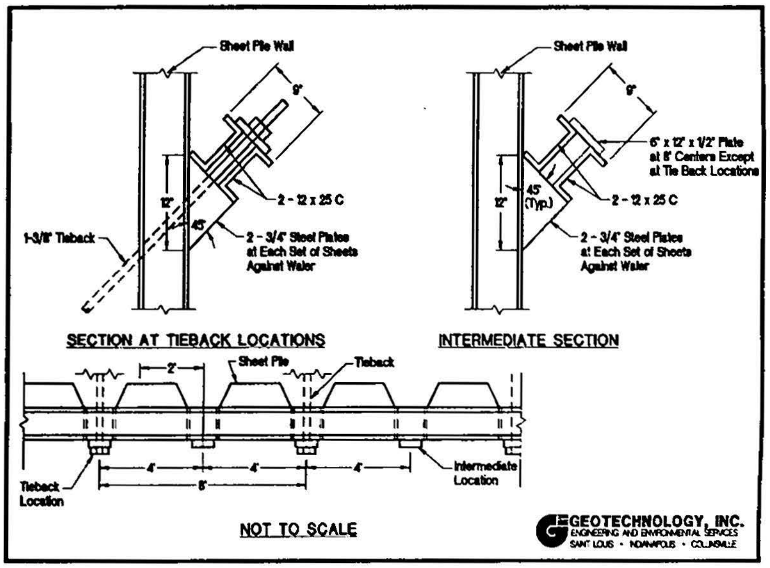

Figure 7. Typical tieback-sheet pile connection detail.

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