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Commemorate the Legacy of Ralph B. Peck
Seventh International Conference on Case Histories in Geotechnical Engineering
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and Symposium in Honor of Clyde Baker

MISSISSIPPI RIVER ROAD GABION WALL/SLOPE STABILIZATION

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ABSTRACT

Lee County widened Mississippi River Road north of the Keokuk, Iowa in the early 1990s, removing material from the toe of slopes along the alignment. Gabion walls were constructed to provide grade separation. Significant precipitation occurred in the spring of 2010, and two (2) wall sections (about 100 feet each in length) failed. Based on our site exploration and instrumentation monitoring data, the gabion wall sections appeared to fail due to additional lateral load from a soil mass sliding on top of the shale bedrock and a buildup of high ground water levels. To support the additional load of the soil mass, reestablish the gabion wall/slope, and to keep the road open to traffic, a tied back, closely-spaced drilled shaft wall was designed to remediate the slide and augment the original gabion wall. This paper describes the investigation, analyses and the design and construction of the remedial measures adopted.

INTRODUCTION

The project area is located near the intersection of 274th Avenue and Mississippi River Road/County Road X28 about 0.7 miles north of the Keokuk, Iowa city limits (Fig. 1). The typical slope stratigraphy in the Keokuk area consists of colluvium (material that moves down the slope) over loess soils. Glacial deposits can be encountered beneath the loess and overlying rock. The rock deposits consist of the Lower Pennsylvanian sandstone and shale, Mississippian limestone and shale and St. Louis limestone. Over geologic time, the rock surface in Keokuk became extremely irregular with the Pennsylvanian present in some areas and both the Pennsylvanian and Mississippian eroded in others.

Slope movement along the west banks of the Mississippi River has historically been a problem in Keokuk, Iowa. Although slope movement can occur in any of the soils and/or shale layers, failures in relatively steep slopes usually occur within the soil layers above the rock; however, slope movement can also occur in the shale.

BACKGROUND

Lee County (County) widened Mississippi River Road north of the Keokuk city limits in the early 1990s, removing material from the toe of slopes along Mississippi River Road/County Road X28. Gabion walls were constructed to provide grade separation between the slope and road. Where the wall sections failed, the gabion wall had 5 levels and a height in the range of 12 to 15 feet. Each level of the wall is about 3 feet high. The top of the slope at the failed sections is about 80 feet above the road with a 2 horizontal to 1 vertical grade above the gabion wall. The new road consisted of a Portland cement concrete pavement with curb and gutter.

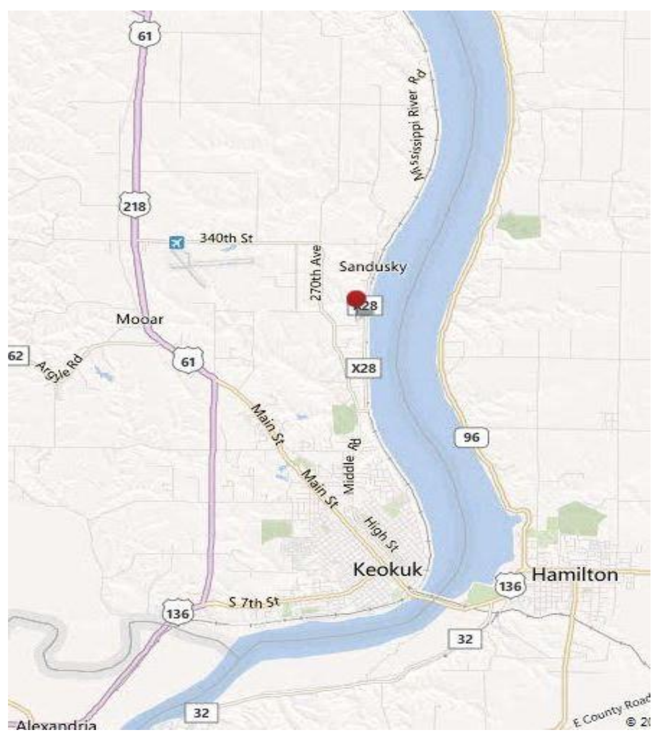


Fig. 1 – Site Location Plan

Movement in two sections of the gabion wall, each about 50 to 100 feet in length, was first noticed about June 1, 2010 (Fig 2.). At that time, the north wall section (about Station 56+50) showed some outward movement (bulging) while the south wall section (about Station 55+50) showed noticeable rotation about the toe with the wall leaning towards the road. The south wall section subsequently failed on July 24 or 25, 2010. The ground surface in front of the failed wall section at Station 55+50 was noticeably raised above the curb elevation and extended laterally over the curb. In addition, the top of the bottom level gabion cages were barely above the ground surface. The condition of the ground surface and lower gabion cages was similar at the north wall section (Station 56+50) that was still intact.

The owner of the property above the failed wall sections reported observing tension cracks in the slope above the wall. According to the property owner, the tension cracks developed the spring of 2010. During a site visit of the area above the referenced wall sections, several tension cracks and slumped areas were observed. One crack extended up the slope from the area where the power pole (Fig. 2) is leaning noticeably towards the road. A bench in the slope was also observed up on the slope and appeared to be a previous road or trail.

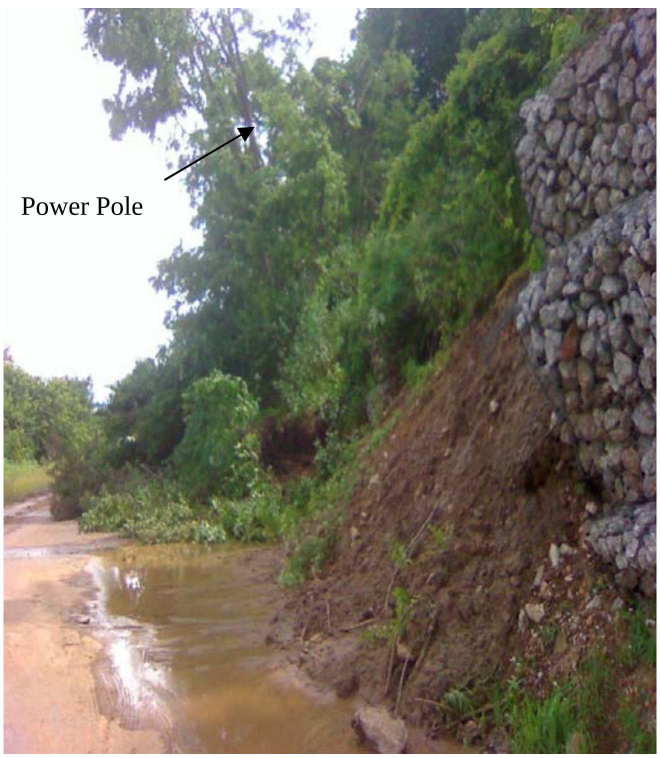


Fig. 2 - 2010 Slide

Additional slope movement occurred in June 2011. The primary area of additional movement was located on the south end of the 2010 slide mass, which includes the power pole on

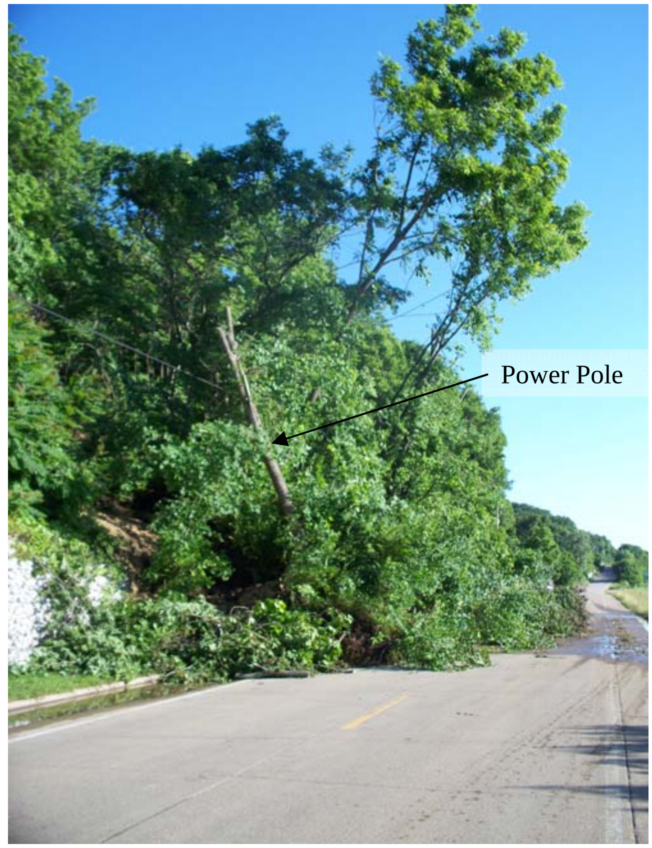


Fig. 3 - 2011 Slide

the south end of the 2010 south slide area (Fig. 3). The power pole moved down the slope as a result of the 2011 slide. As opposed to the 2010 slide, which caused lateral movement of the gabion wall, the 2011 slide mass moved over the wall and into the road. The 2010 and 2011 areas represent progressive movement within a larger slide mass.

Four (4) borings extending into the shale bedrock were performed in October 2010 at the approximate locations shown on the Boring Location Plan (Fig. 4). Slope inclinometer guide casing was installed in Boring 1; PVC well casing was placed in Borings 2 through 4. Subsurface conditions at the boring locations generally consisted of fill overlying native soils, underlain by residual soil above shale bedrock. The thickness of the fill and residual soils overlying the shale bedrock beneath the slope surface varied from about 11 to 11½ feet. Figure 5 indicates the stratigraphy of the slope.

A slope inclinometer was used to measure progressive change in the angle of inclination of the guide casing placed in Boring 1. The inclinometer readings provide an indication of small lateral movements and in the case of evaluating a slide, provide an insight into the location of the failure plane at the inclinometer casing location.

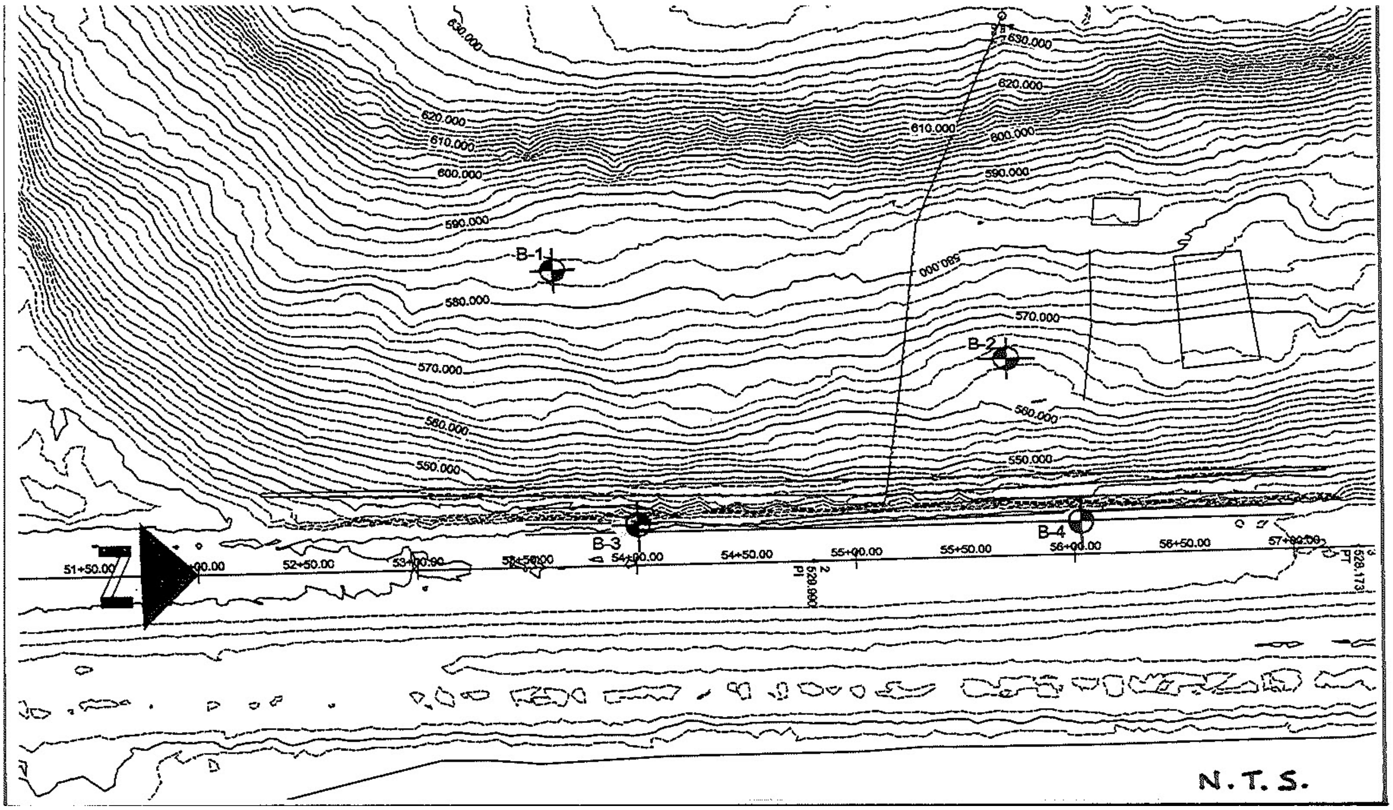
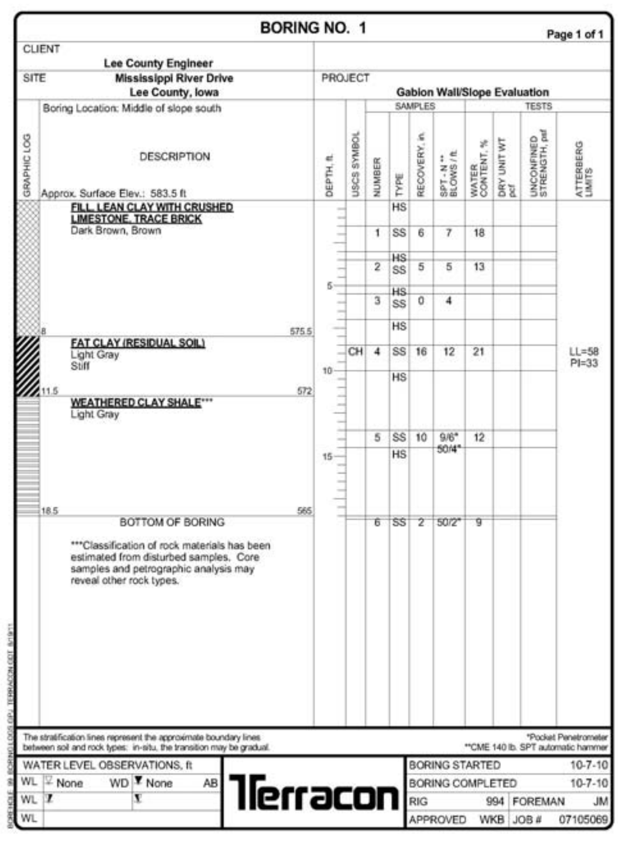


Fig. 4 – Boring Location Plan

The slope inclinometer provided valuable information pertaining to the movement of the slide at this site. Representative plots showing cumulative and incremental movements along the 2-foot intervals of the measuring probe are shown in the Inclinometer Plots (Fig. 6). As shown in the plots, movements at the location of Boring 1 was noted at about the 11 to 12 foot depth. Unfortunately, the slope inclinometer casing moved during the 2011 slide to the extent that further readings are not possible. The primary movement appears to be near the interface between the residual soil and the shale; however there are likely additional shallow failures within the slide area as the ground break-ups and excess rains saturated the undulating ground on the slope.

CONTRIBUTING FACTORS TO MOVEMENT

Although slope movement can occur in any of the soil layers above the bedrock, the limited slope monitoring data indicated that the slope movement was occurring near the interface of the residual soil (fat clay) and weathered shale. During widening of the road, removal of the soil from the toe of the slope perhaps contributed the most toward slope instability. The gabion wall did not have the weight or internal strength to replace the large volume of soil removed from the toe. In addition, the fill present in the upper parts of the slope was likely placed as dumped fill with low in situ density and strength. As visible from field observations, there was excessive water seeping out of the slope and near mud flows were visible at the toe of the slope.



BORING NO. 1 Page 1 of 1

CLIENT Lee County Engineer

SITE Mississippi River Drive, Lee County, Iowa

PROJECT Gabion Wall/Slope Evaluation

Boring Location: Middle of slope south

Approx. Surface Elev.: 583.5 ft

DESCRIPTION	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	SPT - N ** BLOWS / ft.	WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED STRENGTH, psf	ATTERBERG LIMITS
FILL, LEAN CLAY WITH CRUSHED LIMESTONE, TRACE BRICK, Dark Brown, Brown				HS						
			1	SS	6	7	18			
				HS						
			2	SS	5	5	13			
	5			HS						
			3	SS	0	4				
8 — 575.5				HS						
FAT CLAY (RESIDUAL SOIL), Light Gray, Stiff		CH	4	SS	16	12	21			LL=58 PI=33
11.5 — 572	10			HS						
WEATHERED CLAY SHALE***, Light Gray			5	SS	10	9/6" 50/4"	12			
	15			HS						
18.5 — 565 BOTTOM OF BORING			6	SS	2	50/2"	9			

***Classification of rock materials has been estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

*Pocket Penetrometer
**CME 140 lb. SPT automatic hammer

WATER LEVEL OBSERVATIONS, ft: WL None WD None AB

Terracon

BORING STARTED 10-7-10
BORING COMPLETED 10-7-10
RIG 994 FOREMAN JM
APPROVED WKB JOB # 07105069

EXHIBIT A-2

Fig. 5 – Stratigraphy

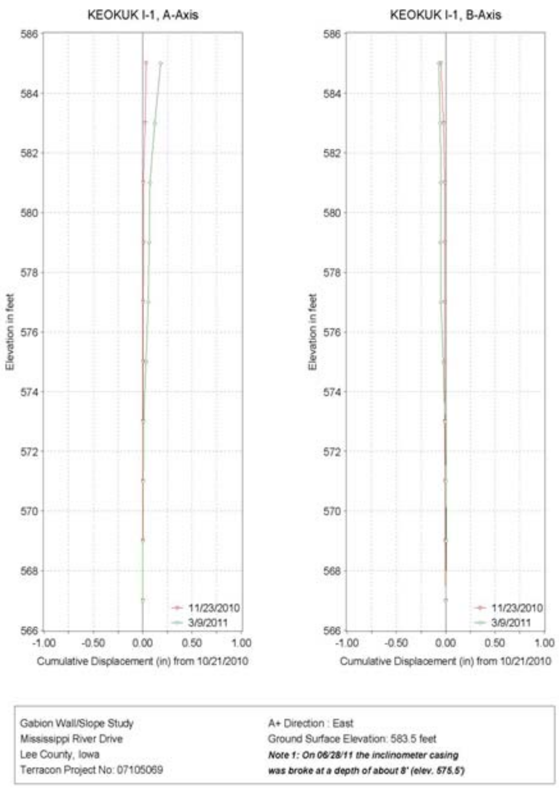


Fig. 6 - Inclinometer Plots

Considering the steepness of the slope, prior movement could have occurred prior to construction of the gabion walls resulting in lowering of the in place strength of the native soils beneath the fill. The presence of the multiple tension cracks and slumps above the wall sections were all indications of that the slope had likely experienced previous progressive movements. With the amount of rain that occurred in the years prior to failure and in the year the failure took place, a previous slide may have been activated. The toe of the failure appeared to emerge at the road level.

STABILITY EVALUATION

Although the original wall failure was located closer to Station 55+50, the cross-section selected for our analysis (Station 53+78.8) (Fig. 7), was based on the location of the inclinometer. Considering the results of the borings and the 2011 slope movement, the cross-section at the failed gabion wall section may differ from the cross-section selected, and in particular, the thickness of the fill/residual soil; however, for the purposes of our analysis, this cross-section was used. The location of the failure surface was approximated from the visible toe of the slide, surface cracks, and the inclinometer readings.

The post-failure movements and the geometry of the slide presented an opportunity to conduct back analyses of the slide areas so that estimates of appropriate average shear strength parameters along the failure plane could be obtained. The back calculation of the slide can be considered a large scale in-situ shear test providing better estimates of the soils' shear strength parameters than a small scale laboratory specimen. The calculated values represent the average shear strength parameters along the estimated failure plane. Appropriate pre-failure piezometric pressures (water pressures) were also estimated from the monitoring wells. The technique of back analysis is widely used in connection with landslide remediation studies. The method has limitations; in cases of progressive failures the position of failure surface may be controlled by strong or weak layers within the slope and use of an estimated failure surface in performing the back analysis may be too simplistic. Since remediating a failed slope involves a relative improvement of the marginal stability, the back analysis was considered appropriate.

The slope stability analyses were performed using computer programs SLOPE/W and STABL. Both of these programs use limit equilibrium methods (LEM) of analysis. The failure surface estimated from the limited data obtained from the geometry provided by the County and the inclinometer readings is indicated in the Subsurface Cross-Section (Fig. 7).

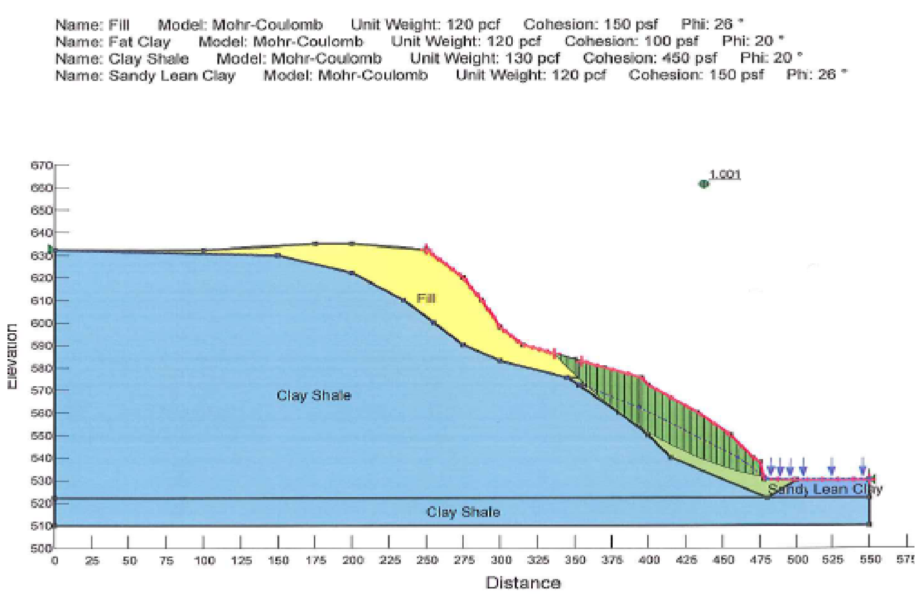


Fig. 7 - Subsurface Cross Section Station 53+78.8/Back Calculation

The cross-sectional model shown in Fig. 7 was used as the basis for our analyses to evaluate shear strengths along the known failure surface that would result in a FOS close to 1.0. Fig. 7 shows the results of back analysis indicating an incipient failure in the slope with the current geometry. The water level (piezometric pressure) was estimated from the water levels observed at Borings 2 through 4 as shown in Fig. 7. Based on the back analysis, the average shear strength parameters for the various stratigraphic units were estimated as shown in Table No. 1. below:

Table No. 1. Effective Stress Strength Parameters

Material	Cohesion (c´) psf	Frictional Angle (ϕ´) degrees
fill, lean clay	150	26
fat clay (residual soil)	100	20
clay shale	450	20
sandy lean clay	150	26

Our analysis of remedial measures utilized the shear strength parameters obtained from the back analysis.

Every slope will have a finite failure probability associated with its particular geometry. The appropriate factor of safety for a slope should reflect the degree of confidence the engineer has in the selection of soil and piezometric conditions, as well as the consequence of failure. Suggested factors of safety (summation of resisting forces divided by the summation of driving forces) have been compiled in the literature. For remediation of this slope, we recommended using a FOS of about 1.5. A FOS 1.5 represents an approximately 50% improvement in stability over the existing condition.

REMEDIAL MEASURES

The results of our analysis were discussed with the County. If no action was taken to remediate the slope, continued movement and further deterioration of the gabion wall would likely occur. As the water pressure builds up in the slope, sudden movement of large masses are expected as has occurred in the past. Consequence of no action could be a safety hazard for the traffic on the county road as well as continuous maintenance in debris clearing. This alternative was not acceptable to the County.

To rebuild the gabion wall without additional structural support, a deep subsurface drainage system would be required to reduce the pore water pressures in the slope and increase the shear strength of the soils. The deep drainage system, however, will be very slow due to the low permeability of the soils. Due to the terrain, installing of a deep subsurface drainage system would also be very difficult. The usual methods of geometric change with drainage and/or in-situ treatment were not considered feasible or too slow in achieving timely stabilization; a more immediate improvement in the stability was needed. For this reason, mechanical methods to structurally augment the gabion wall were considered.

Mechanical means to reconstruct the wall/slope and decrease the risk of future movement were discussed with the County and are summarized below. The costs provided below were for use in comparing the relative costs of each alternative.

- A system of concentrated tieback anchors on the slope with horizontal subdrains, filling in of tension cracks, and restoration of the gabion wall were considered for further design and implementation. This alternative was expected to result in a FOS of about 1.3, an approximate 30% improvement in stability over the existing condition, but still below a FOS of 1.5. For comparison purposes only, the cost for a tieback system and subdrains was estimated to be at least $300,000.
- Removal and replacement of the slide mass and installation of subsurface drainage was considered a possible means to achieve the preferred FOS of 1.5. Due to the expected cost, this alternative was not evaluated.
- Mechanical stabilization comprised of a new retaining structure consisting of a series of closely-spaced straight-sided drilled shafts with tiebacks was considered to provide a positive means to mitigate slope movement and achieving a FOS of 1.5; however, the cost of the drilled shaft wall was expected to be more expensive than the tieback anchors (without a wall) alternative. For comparison purposes only, the cost of the drilled shaft alternative was estimated to be at least $600,000.

Based on the alternatives discussed with the county, the tied back, closely spaced drilled shaft wall was selected as the most positive alternative for design.

TIED BACK, CLOSELY SPACED DRILLED SHAFT WALL DESIGN

A rigid restraining structure was considered to be more positive due to the restrictive site, safety and certainty. The design methodology used for the design of a tied back pile wall was essentially first to perform a stability analysis to see what magnitude of thrust was needed to be applied at the face of the wall along the road to provide a stable slope with a factor of safety of 1.0. Figure 8 shows the analysis where the horizontal thrust "P" at the face of the wall is computed. Once the thrust is computed the approach would be to provide the thrust by means of a tied back wall. Appropriate factors of safety would then be applied to the design of the drilled shafts and the tiebacks. A tied back wall would actively resist the movements of the soil mass. The vertical component of the wall works against the thrust of the sliding mass while the tieback load increases the normal stress on the slip surfaces in the soil and also provides restraint at the top of the wall. The tiebacks also help the stability of the wall where the wall is too tall to be cantilevered.

The thrust that was needed to be applied to the face of the wall was transformed into an apparent active pressure diagram for a tied back wall. The passive resistance was provided by appropriately embedding the pile wall into the shale bedrock. Using the normally applied analytical methods the forces in the tiebacks and the bending moments in the shaft were computed and the detail design of each component performed. The drilled shafts were designed to be of 36 inch diameter, spaced at 6 feet centers. Each shaft will be reinforced with a steel wide flange section and tied back at the top with tiebacks at 45 degrees to horizontal, post-tensioned to 85 kips. A schematic of the tied back wall is shown in Fig. 9. A summary of the drilled shaft and tieback design is shown in Table No. 2.

Table No. 2 Summary of Design

Overall Quantity

Number of drilled shaft: 54 (items)

Number of tie-back anchors: 54 (items)

Drilled Shaft

Element	Description	Values
Length	Total	27 ft
Drill-hole	Diameter	36 in
Reinforcement	W18x97, f_y – 50 ksi Steel	26 ¾ ft.
Concrete	Minimum 28-day compressive strength	4000 psi

Tie-back Anchor

Element	Description	Values
Tie-back Inclination	Uniform	45 °
Bonded length	Minimum	25 ft
Nail Length	Uniform Pattern	L = 12 ft
Anchor Bar	Type	Threaded Bar
	Nominal Bar Diameter	1 ¼ in.
	Material	150 ksi Steel
Drill-hole	Minimum Diameter	6 in.
Corrosion Protection	Grout-protected anchor bar	Class II Protection
	Minimum Cover	As specified
	PVC Centralizers	As specified
Grout	Neat Cement	f_c – 4000 psi
Ultimate Bond Strength	Minimum Specified	Q_u – 4.5 kips/ft

A typical section through the tieback drilled shaft arrangement is shown in Fig. 9. The installation of the tied back, drilled shaft will be augmented with trench subdrains. The tieback at each shaft location needed an arrangement for the tieback installation after the shaft has been constructed. To control the installation, a pipe sleeve will be welded to the wide flange at 45 degrees as shown in Figs. 10 and 11. A typical tieback anchor detail is shown in Fig. 12.

The sequence of constructing the tied back shaft system will be important for a successful installation of the remedial measures. The sequence of construction is expected to include installation of the drilled shafts followed by the tiebacks. Stressing of the tiebacks will need to wait until the shaft concrete and the tieback grout has achieved desired strength. Each tieback will be stressed to a design load of 85 kips.

Terracon produced the design calculations, drawings, specifications and the contract documents in accordance with the Iowa Department of Transportation requirements. The project construction is due to start 2012-13 winter. The initial bid results indicated a cost of just over $1 million dollars, a significant increase over the original estimate.

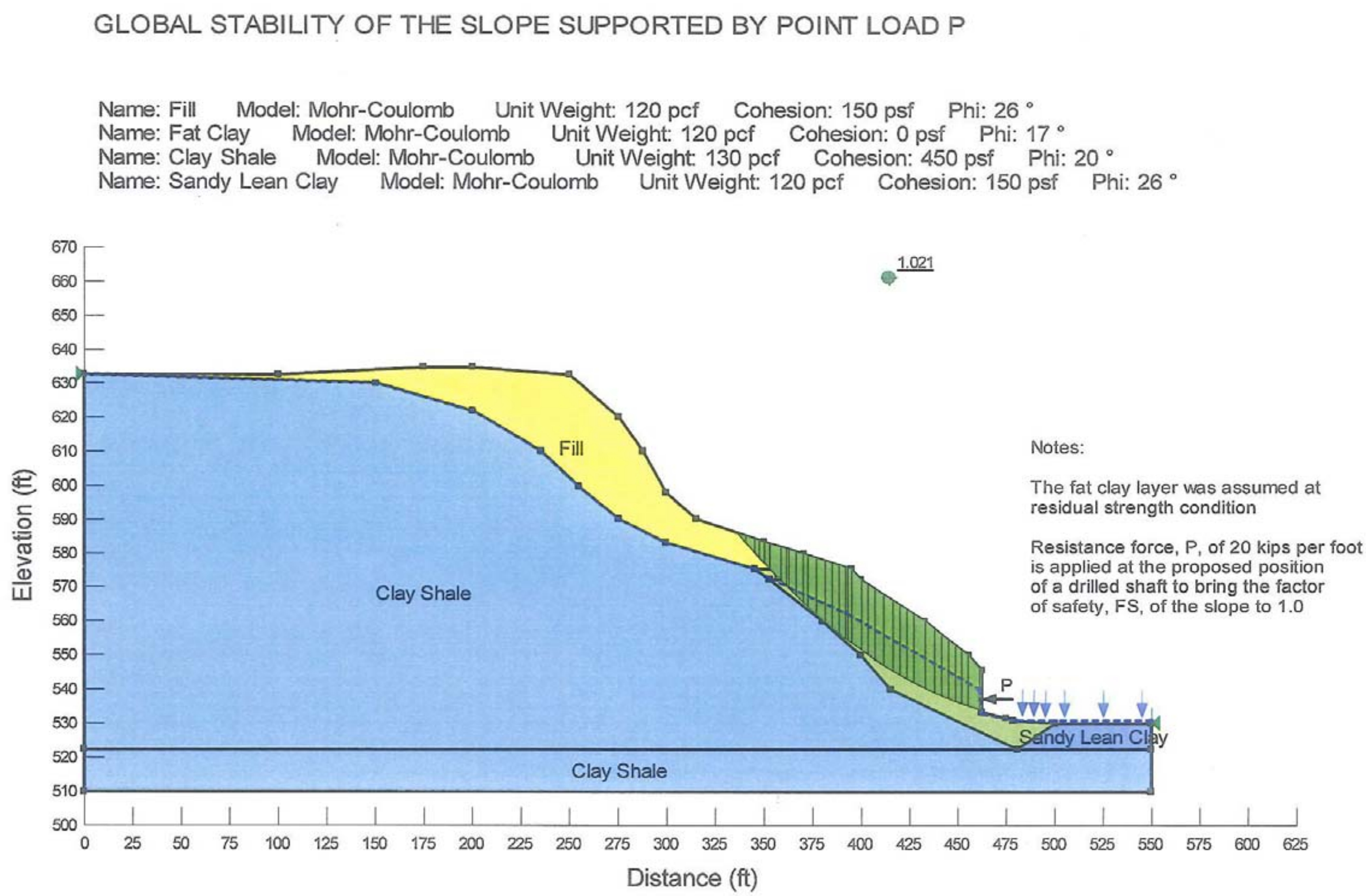


Fig. 8- Analysis For The Horizontal Thrust At The Wall.

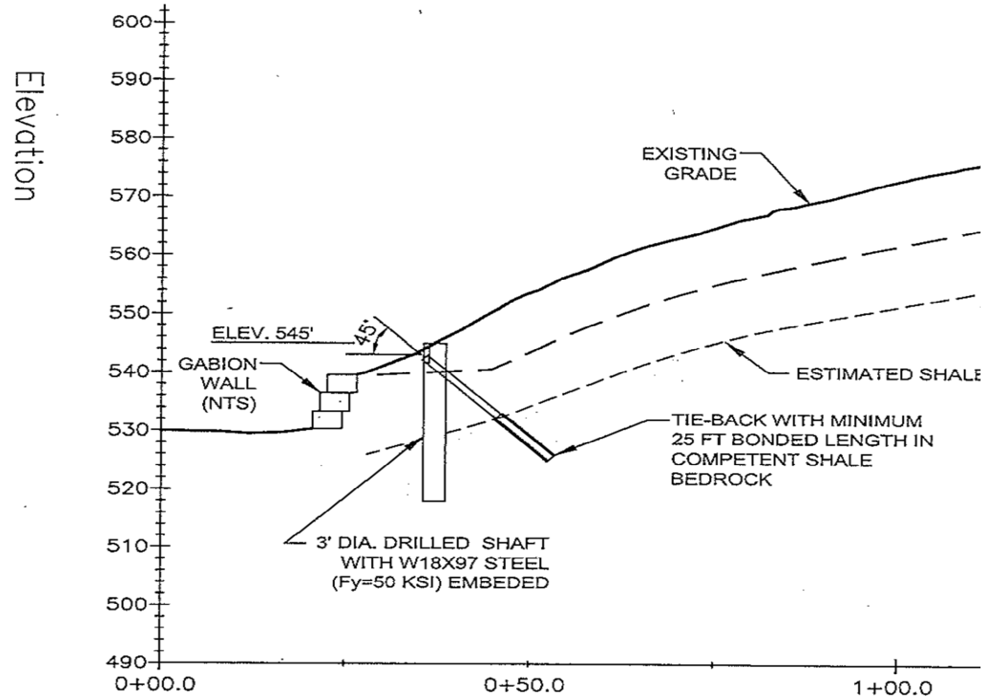


Fig. 9- Tied Back Drilled Shaft Section (typical)

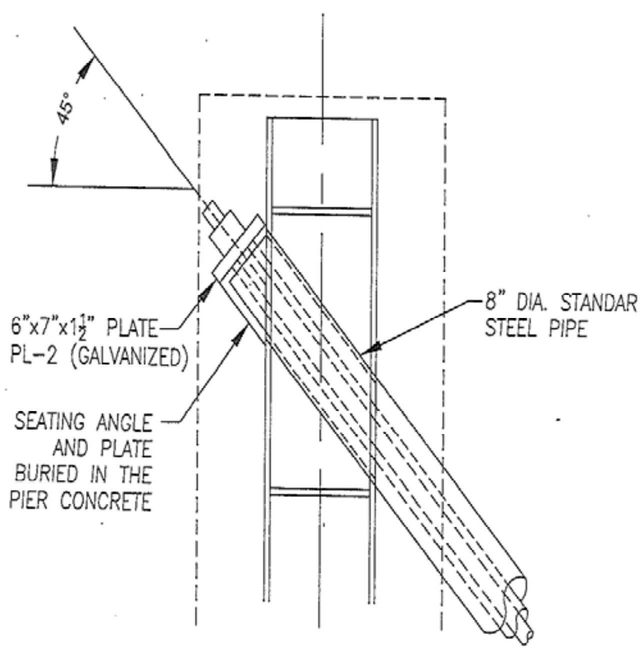


Fig. 10 - Tieback Anchor Detail

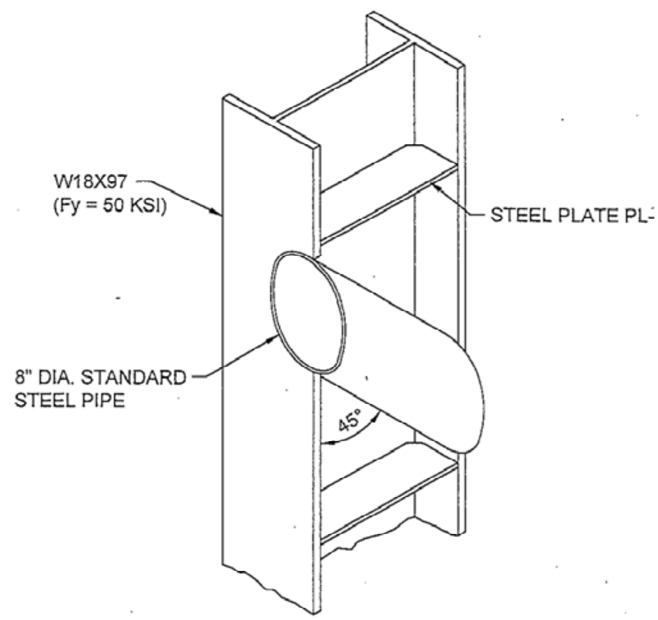


Fig. 11 – View of the pipe sleeve through the steel section

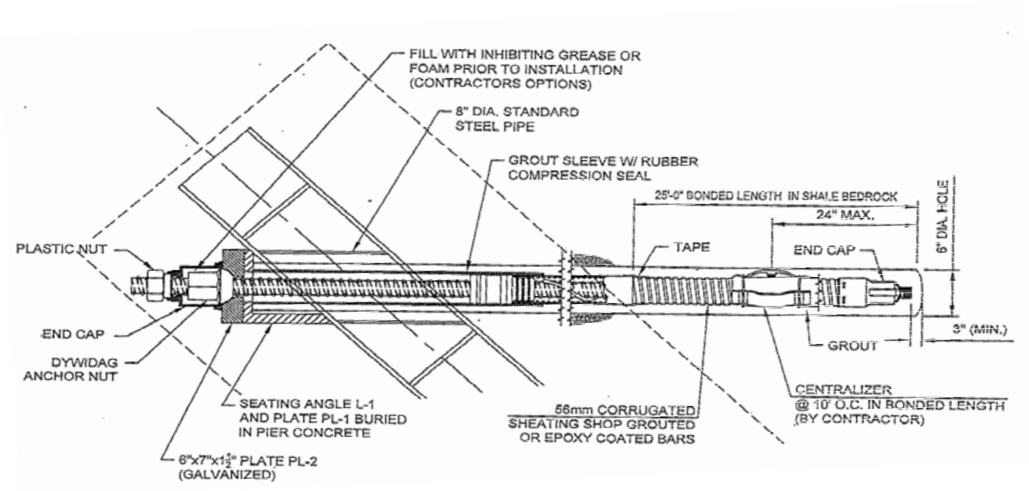


Fig. 12 - Tieback Anchor Typical Detail

CONCLUSIONS

Two sections of a gabion wall installed to retain the cut slope resulting from a roadway widening project collapsed along Mississippi River Road/County Road X28 about 0.7 miles north of Keokuk Iowa. The gabion wall was installed on the cut (west) side of the road, which parallels the west bank of the Mississippi River. The failure caused debris to partially block the county road. Terracon performed a field exploration, installed instrumentation and analyzed the failed conditions. The failed slopes were back analyzed to obtain average shear strength and pore water pressures along the identified slope surface. Based on the back analysis, remedial measures comprising of a closely spaced drilled shaft wall with tiebacks was designed by Terracon.

The remedial measure analysis was based on the premise that the drilled shafts with tiebacks would provide a horizontal thrust at the face of the retaining wall to counter the thrust of the moving mass of the slope above the wall. The thrust needed to be applied by the drilled shafts and tiebacks was equated to an apparent earth pressure on the tied back, closely spaced drilled shaft retaining wall. The paper summarizes the mechanism of the failure observed and the remedial measures adopted.

The wall is due to be constructed the winter of 2012-13. At the time of the conference observations during construction and behavior of the slope would be added to this presentation.

REFERENCES

Sverdrup & Parcel Engineering, Co., September 30, 1960, *"Subsurface Exploration, Union Electric Company, Forebay Crossing Installation, Keokuk, Iowa"*