03 Jun 1988, 10:00 am - 5:30 pm

Geotechnical Investigation into Causes of Failure of a Gabion Retaining Wall

Edward A. Nowatzki
University of Arizona, Tucson, Arizona

Brian P. Wrench
Steffen Robertson & Kirsten, Johannesburg, South Africa

Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 6.96

Geotechnical Investigation into Causes of Failure of a Gabion Retaining Wall

Edward A. Nowatzki
Associate Professor, Civil Engineering Dept., University of Arizona, Tucson, Arizona

Brian P. Wrench
Principal, Steffen Robertson & Kirsten, Johannesburg, South Africa

SYNOPSIS: This paper describes the post-failure analysis of a 26m long x 4m high gabion retaining wall located in a suburb of Johannesburg, South Africa. The wall had been built just beyond the toe of a natural slope with most of the gabion units resting on the bed of a small river. The river bed soils consisted of approximately 2.5m of soft, dark-grey, silty clay underlain by massive granite bedrock. The water table at the toe of the wall was within 0.1m of the river bed surface. Failure of the wall occurred over the weekend after backfilling to grade behind the wall had been completed.

Stability analyses were conducted using both total (undrained) and effective (drained) shear strength parameters for the clay. The results of the analyses showed that the wall should be stable with FS = 1.2 for effective stress parameters and that the wall should be unstable with FS = 1.0 for undrained strength parameters. The details of the testing program and the selection of strength parameters is described in the paper.

INTRODUCTION

This paper describes the post-failure analysis of a 26m long x 4m high gabion retaining wall located in a suburb of Johannesburg, South Africa. The wall was built as part of the development of an industrial park along the east bank of the Klein Jukskei River in Strydom Park.

The original topography of the site sloped steeply downwards to the river. The wall was constructed just beyond the toe of the natural slope with most of the gabion units resting on the river bed, except for a length of approximately 10m near the middle of the wall where the units were founded about 1m below the river bed. Following completion of the wall, an imported fill was placed and compacted between the wall and the natural slope. Final grade was relatively flat with a gentle slope from east to west.

The retaining wall itself consisted of 2m x 1m x 1m gabion cages containing angular rock fill. The front face of the wall was constructed of two rows of gabion units for a length of approximately 26m. The height of the wall along this length was 4m above the river bed. The north and south wing walls of the structure were 9m and 6.5m long, respectively. Figure 1 shows a sketch of the structure prepared by the contractor.

Construction records indicate that backfilling behind the wall was completed on or about 8 July 1983. The wall failed over the weekend of 9-10 July 1983 following a period of heavy rain. The consulting firm of Steffen Robertson and Kirsten (SRK) was retained by the contractor to perform a geotechnical investigation to determine the cause of the failure and to make recommendations for the redesign and/or repair of the wall. A preliminary field investigation revealed that a classic rotational type of failure had occurred along the central portion of the wall. A maximum downward displacement of approximately 1.4m was evident along the intersection of the semi-circular failure surface with the backfill surface. Bulging of the river bed at the toe of the wall was clearly visible. These characteristic features of the failure are shown in Figures 2a and 2b. Reconnaissance of the site revealed the presence of a concrete wall, approximately 3m in height, located just downstream of the retaining structure as shown in Figure 2c. This concrete wall acted as a dam before development in the area took place. Although it was breached at the time of the failure, the wall could back up water under flood conditions. The size of the spans between piers of the bridge located just upstream of the gabion wall, as shown in Figure 2d, suggested that flow in Klein Jukskei River could be substantial. The reconnaissance also revealed the presence of rock outcrops on the opposite river bank and to the north and south of the site. These outcrops formed a natural channel that directed the flow of the river toward the wall. The site conditions described above had a significant impact on the recommendations made by SRK.

METHODS OF INVESTIGATION AND RESULTS

Field Investigation:

Two NX size boreholes were advanced, one (BH-1) on top of the fill 5m behind the retaining wall, the other (BH-2) at the toe of

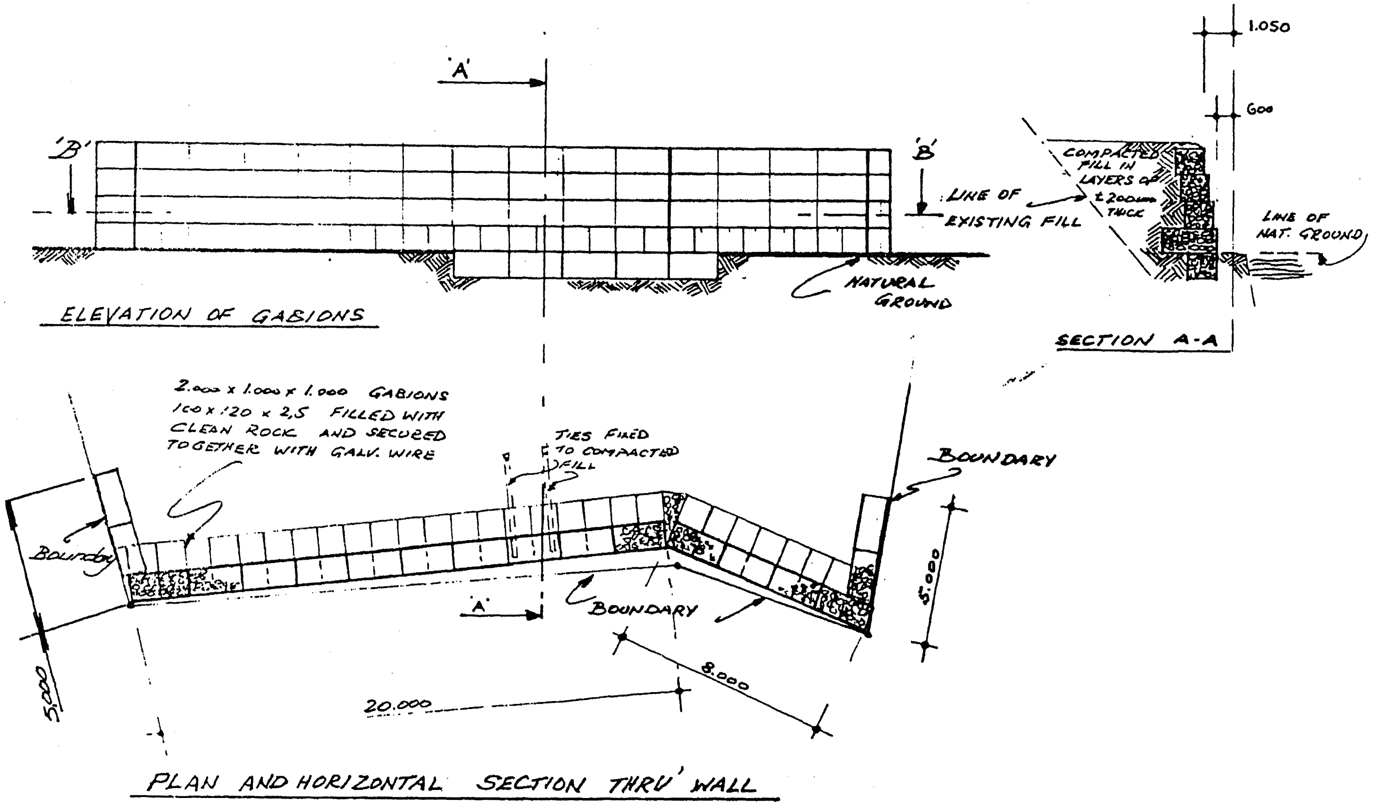


FIG. 1

Sketch of Gabion Wall Showing Configuration and Dimensions

the wall in the river bed. Both boreholes extended through the fill materials and underlying soils and penetrated 2m into the granite bedrock. Standard Penetration Tests (SPT) were carried out to estimate the insitu densities of the fill and subsoils. Disturbed and undisturbed soil samples were recovered from the boreholes for laboratory testing. The results of the field investigation are summarized in Table 1.

TABLE 1
SUMMARY OF RESULTS OF FIELD INVESTIGATION

Borehole	Depth	Average Description	SPT
BH-1	0-4.4 m	Dry to slightly moist medium-dense clayey sand and gravel, fill.	9
	4.4-4.8 m	Moist dark-grey soft clay.	3
	4.8-7.4 m	Unweathered, coarse-grained, widely fractured granite.	-
BH-2	0-2.8 m	Very moist dark-grey clay.	3
	2.8-4.8 m	Unweathered, coarse-grained widely fractured granite.	-

Field vane shear tests to determine the undrained shear strength (s_u) of the clay were also performed. Peak and residual values of s_u were measured.

Laboratory Testing Program:

The following laboratory tests were carried out on selected undisturbed samples retrieved from the site:

1. Saturated unconsolidated-undrained triaxial tests to determine the undrained shear strength (s_u) of the clay.

2. Consolidated-drained shear box tests to determine the drained (effective) cohesion and friction angle ($\bar{c}$ and $\bar{\phi}$) of the clay.

3. Indicator tests (gradation, Atterberg limits) to classify the clay according to the Unified Soil Classification System (USCS).

A summary of the results of the laboratory and field-strength testing program is contained in Table 2.

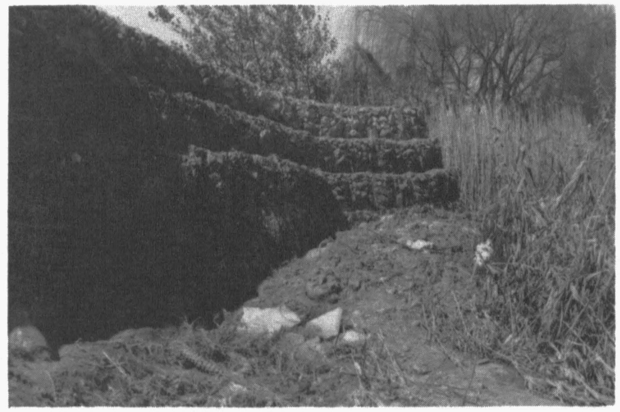
ANALYSES AND RESULTS

Selection of Shear Strength Parameters:

Two types of stability analyses were performed:

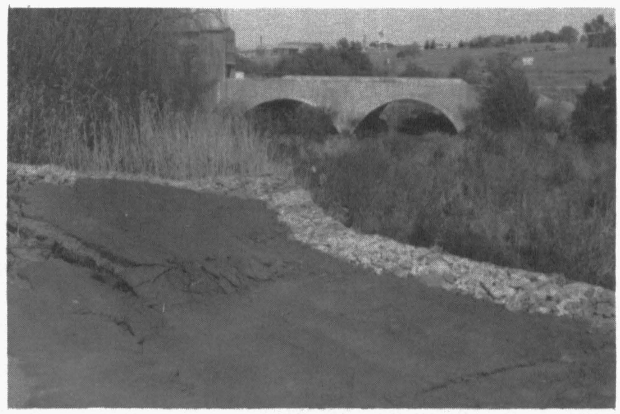
1. Short-term or end-of-construction analysis. This analysis was conducted in

(a)



(b)

(c)



(d)

FIG. 2

Post-Failure Photos of Gabion Wall Showing:

(a) surface displacement of backfill
(b) bulge at toe
(c) view downstream with concrete wall visible
(d) view upstream with bridge span visible

TABLE 2. SUMMARY OF RESULTS OF LABORATORY TESTING PROGRAM

Unit	Test Type	Cohesion (kPa)	Friction Angle (degrees)	Total Unit Weight (kg/m^3)
Fill	SPT	0	35	1880
Clay	Drained Direct Shear	0	30	1550
	Undrained Triaxial	20	0	1550
	Laboratory Vane			
	Peak	6	0	1550
	Residual	3	0	1550
Wall	Assumed[a]	2000	42	2300

(a) Properties of the wall were assumed so as to make the wall rigid relative to the backfill and clay layer in order to model the problem

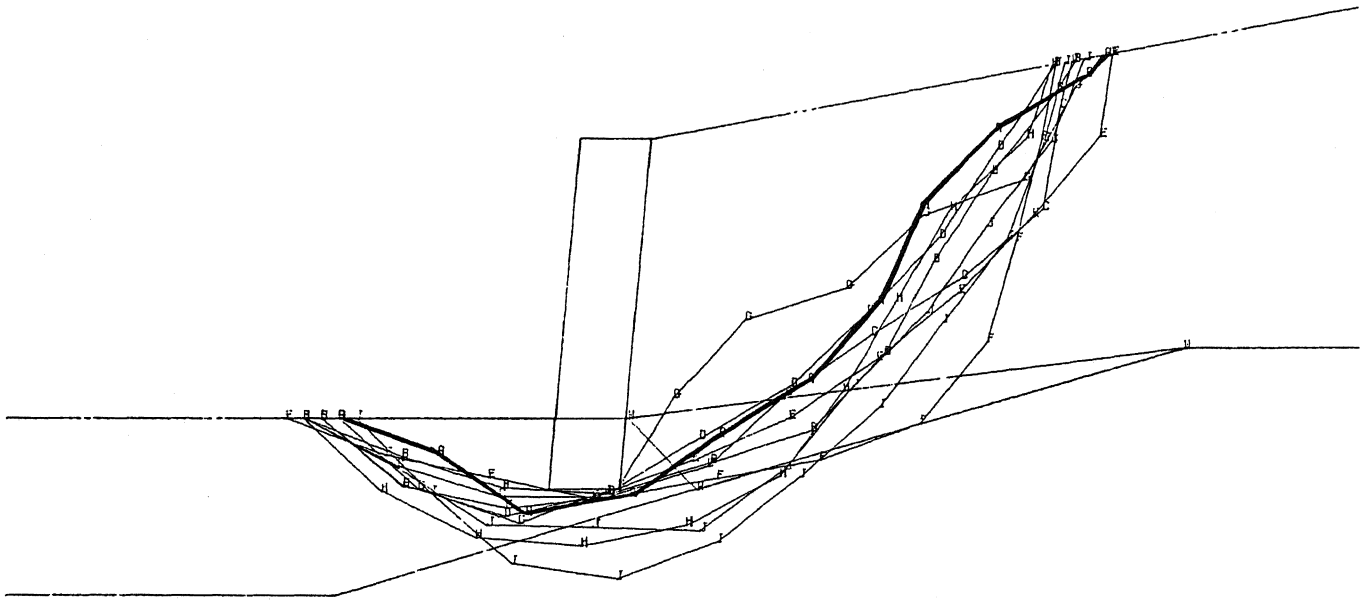


FIG. 3

Failure Surfaces for Short-Term Stability
(s_u = 12 kPa; FS = 1.0)

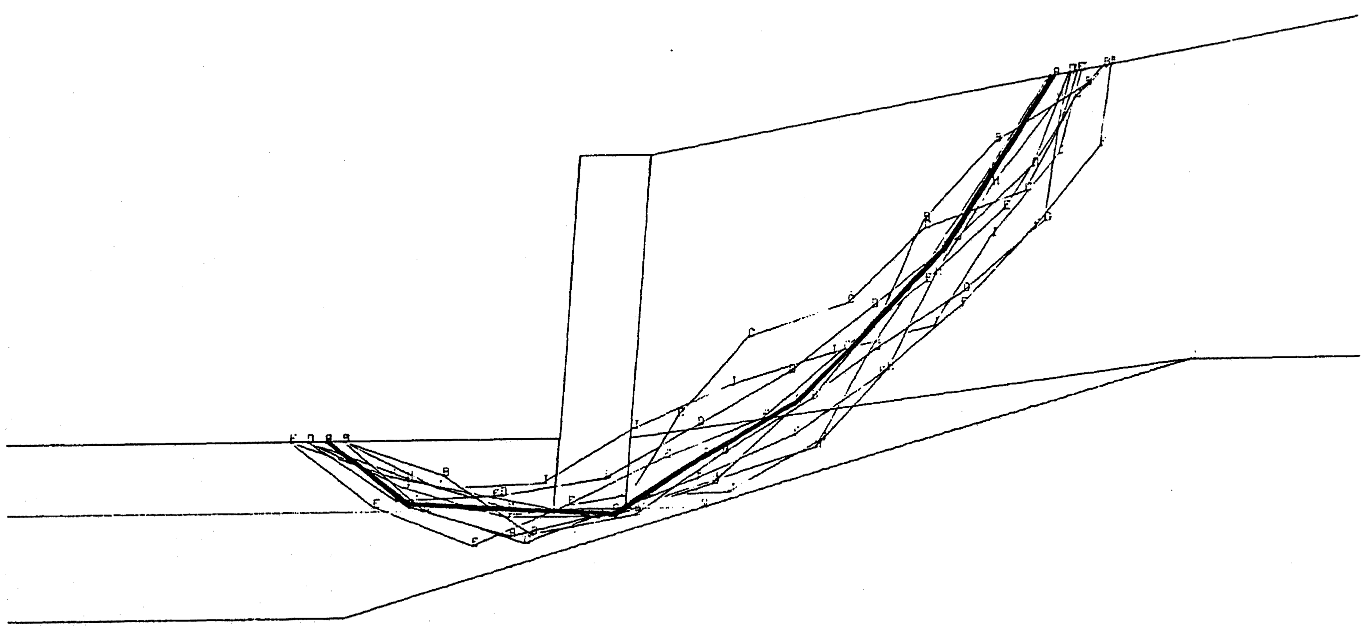


FIG. 4

Failure Surfaces for Long-Term Stability
($\bar{c}$ = 0; $\bar{\phi}$ = 30^o; FS=1.2)

order to determine whether or not the failure could have been predicted had a pre-construction stability analysis been performed. In view of the relatively short construction period and the presence of the near-saturated soft clay as a foundation material, it was clear that the end-of-construction stability of the gabion wall and the backfill should be controlled by the undrained strength of the foundation clay. As indicated in Table 2, the clay exhibited s_u values ranging from 20 kPa, as determined from laboratory UU tests, to 3 kPa (residual), as determined from field vane shear tests. Such discrepancies in measured values of s_u may be attributed to differences in test method (Ladd and Lambe, 1963). Methods that cause greater sample disturbance generally yield lower values of s_u. On the basis of the test results presented in Table 2, it was assumed that the insitu undrained strength along the failure surface would be, on average, between the maximum and minimum values measured. Therefore, a value of s_u = 12 kPa was used for the short-term stability analysis.

2. Long-term analysis. This analysis was conducted in order to evaluate the stability of the gabion wall and the backfill if they had been constructed in stages and if the foundation clay had been given time to consolidate under the intermediate loads. The results of this analysis were of more than academic interest. The recommendations for remedial measures depended heavily on whether or not the gabion wall, even in its failed condition, could ever be expected to become stable enough to allow normal activity to take place behind it. If it could, then only cosmetic measures would be needed to remove evidence of the failure. If it could not, complete removal of the wall and either its reconstruction or replacement with some alternate structure would be indicated. The drained shear strength parameters $\bar{c}$ = 0 kPa and $\bar{\phi}$ = 30° shown in Table 2 were used for the long-term stability analysis.

Analysis Procedure and Results:

All stability analyses were carried out by using the computer program STABL 21 which is based on Janbu's simplified method of slices. One of the features of the program is that it allows irregular failure surfaces to be considered between specified locations on the crest and toe. This feature was particularly useful for the post-failure analyses conducted here since the crest and toe locations of the failure surface were known from measurements made the field. Another feature of the program is that it selects the failure surfaces that result in the ten lowest factors of safety and plots each of them for comparison purposes. The surface having the lowest factor of safety is highlighted.

The results of the short-term stability analysis using s_u = 12 kPa are shown in Figure 3. The highlighted failure surface evident in the figure results in a factor of safety of 1.0. The analysis confirms that failure of the wall occurred because of overstressing of the underlying soft clays. As a matter of interest, analyses were also performed for s_u = 3 kPa and 20 kPa. Factors of safety of 0.7 and 1.2 were obtained, respectively. This suggests that even with the maximum measured value of the undrained strength, the wall and backfill would be only marginally stable.

Analyses were also performed using the effective stress parameters. The results of these analyses are shown in Figure 4. The factor of safety of 1.2 suggests that even if the clay were to drain under the loads existing at the time of failure, the long-term stability of the wall would still be questionable, especially if any of the conditions existing at the time of failure should change. Such changes could occur if, for example, the ground water table should rise, or erosion of the foundation materials should take place, or should additional loads be imposed on the backfill from normal activities of the site user.

CONCLUSIONS AND RECOMMENDATIONS

The stability analyses show that the gabion wall and the backfill failed because the foundation soils were overstressed, and that the wall was only marginally stable in its post-failure condition. In addition, the analyses suggest that further failures could occur if any of the present conditions affecting the stability of the wall should change.

Reconnaissance of the site revealed that the wall was located on the outside sweep of the Klein Jukskei River and effectively served to constrict the channel. Visual assessment of the hydraulic conditions at the site suggested that the gabion wall was in danger of being damaged by erosion of the banks and scour of the foundation materials. Such a danger would be especially acute if the small concrete dam downstream of the wall were to be removed.

On the basis of these two threats to the future stability of the wall, SRK concluded that flood conditions at the site must be established before any remedial measures for the foundation stability of the gabion wall could be evaluated. They recommended that, in the absence of such information, the gabion wall should be removed.

REFERENCES

Ladd, C.C. and T.W. Lambe (1963), "The Strength of 'Undisturbed' Clay Determined from Undrained Tests," ASTM, STP No. 361, pp. 359-371.