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Use of gabions for localized slope stabilization in difficult terrain

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ABSTRACT: Two case studies are used to illustrate the application of gabion rock baskets for localized slope stabilization and erosion protection in difficult terrain. The gabion concept has been used for more than a century in numerous erosion control and bank protection projects. These flexible, environmentally sound and economical elements can also be used in small-scale rock slope stabilization projects where more invasive methods are either impractical or cost prohibitive.

1 INTRODUCTION

Gabions have been used throughout the world as wall elements for erosion control projects, soil reclamation work, retaining structures and stream channel linings. Though they vary in shape and size, gabion units are generally rectangular containers (baskets) made of hexagonal woven galvanized steel wire mesh and filled with cobble-size rock (Fig. 1). Units can be used individually or can be laced together and stacked atop one another to form a flexible gravity-type wall.

Employed in lieu of rigid structural materials such as concrete, wood and steel, gabion use is driven by benefits such as:

- simplicity and ease of installation
- adaptability to difficult sites
- use of local fill material
- construction by relatively unskilled labor
- flexible/forgiving elements
- ease of maintenance
- porosity
- economy
- natural appearance; can easily be vegetated

Gabions demonstrate adaptability as earth retaining structures in that they may be built in multiple tiers and set at nearly any inclination to accommodate a wide range of slope geometries. Many different stacking arrangements are possible including battered and stepped-back fronts. Higher tiered walls require greater basal widths and/or use of counterforts to brace against overturning moments induced by backfill (Gray & Sotir 1996). Various standard designs for different wall heights and backfill conditions are supplied by gabion manufacturers.

2 ANALYSIS METHODOLOGY

Although yielding by their nature, gabion retaining structures are designed in the same manner as rigid gravity walls. Gabions depend mainly on the shear strength of the rock fill for internal stability and their mass or weight to resist external lateral earth forces (Gray & Sotir 1996).

Traditional limit equilibrium methods are used to perform stability analyses. External loads on gabion walls are typically assumed to act as a Coulomb or Rankine active earth wedge. As with nearly all

Figure 1. Gabions Containers (rock baskets)

retaining structures, gabion walls must be stable against overturning, basal sliding, bearing capacity failure and deep-seated rotational movement (slope failure). Internal stresses should also be evaluated to verify that stresses induced to the units do not exceed allowable values.

General gravity retaining wall analysis and design procedures found in most geotechnical engineering texts discuss lateral earth pressure and slope stability theories (e.g. Bowles 1996 and Spangler & Handy 1982). The assumptions, limitations and parameters for each analysis method should be understood prior to undertaking design, and should be related to the specific characteristics of the gabion retaining wall structure in question.

Vendors have also developed specialized gabion design manuals for use by practitioners. Maccaferri Gabions, Inc. (1995) provides both a detailed guide for the analysis and design of gabion retaining walls and computer design software. All of the previously described failure modes are analyzed with the software. Slope stability of the wall system can also be assessed by numerous computer software programs commercially available.

3 DAM ACCESS ROAD EDGE STABILIZATION

A 9.7 km unimproved dam access road in Central Arizona required the use of gabions to stabilize numerous road-edge wash outs from local storm water inflow. The Horse Mesa Dam road was constructed during the 1920's for access to the dam site. The dam is operated and the road is maintained by The Salt River Project. Road alignment follows a highly irregular hillside contour and makes several switchbacks and steep grade changes along its route, with numerous cuts and fills across a steep rock face hillside. At the bottom of the canyon, the road parallels the south bank of Canyon Lake. The local terrain consists of steep ½:1 to ¾:1 (horizontal to vertical) road cut sections. Above and below the road cuts, the natural slope ranges from 45-75 degrees inclination from horizontal.

The site is at an average elevation of 550 meters above mean sea level and is considered to be part of the Sonoran Desert. Summer temperatures range between of 40° to 47° C and the annual rainfall is on the order of 19 cm. The 50-year design rainstorm for a 1-hour period is 50 mm, with an infiltration of only 25%. Vegetation is sparse on the rocky slopes, primarily consisting of cactus, desert grasses and scrub brush. Erosion is extreme in this environment.

3.1 *Geology & Soil Conditions*

The turnoff to the access road from SR-188 is about 7.3 km east from Tortilla Flat, Arizona. The site is situated in rugged terrain within a small tributary canyon to Fish Creek Canyon that has down-cut into a sequence of massive Tertiary age volcanic and volcanoclastic rocks referred to as the Superstition Volcanic Field. The geologic unit exposed in cuts is a volcanic tuff breccia facies of the Geronimo Head Formation (Euge 1994). The tuff was formed during the mid-Tertiary as a result of a major caldera collapse within the Superstition Volcanic Field about 16M years ago (Sheridan 1978).

Native residual and cut materials were used as road fill that can be best described as a silty sand, gravel and cobble mix. Rock fragments are composed of basalt, rhyolite and pumice. Decomposed granite has been imported over many years as road surfacing to create a more even driving surface. The blend of native soil, import soil and rock creates a backfill with a maximum density of 2000 kg/m^3 at 12% moisture content.

3.2 *Historic Repairs*

Local runoff from storms stream down steep hillsides and flow along the road's inside bar ditch. Waters continue along the ditch to locations where road cross sections dip toward the downhill edge. Flows then stream across and down the road, breaking over the downhill face of the road embankment at intervals of every 15 to 300 meters and causing edge washouts ranging from a few feet in all dimensions to as much as 6m across and 3m deep.

In the 60-year history of the access road, numerous attempts have been made to control roadside erosion and washout. Most of these efforts were made by dam maintenance personnel and operations engineers whose primary duty was to the upstream dam and generation facility. Typical repair areas still intact incorporate the use of reinforcing bars and welded wire fabric. Bars ranging in diameter from 6 to 25mm were grouted into the rock face near the bottom of road fills and along any rock outcrops on the sides at about 0.6 to 1.0m spacings, then tied together across the breadth of the washout area. Welded wire fabric (100mm x 200 mm rectangle size) was then be placed on the inside face of the criss-crossed bars to retain fill rock and soil. A typical example is shown in Figure 2.

These short-lived repairs would survive only for a few season until fill washed out from under or around the mesh, or until vehicle tire loads would result in lateral failure of the system. On occasion, concrete would be mass-poured into these roadside voids in an attempt to fill the space. Eventually, fills would erode around the concrete, causing them to wash down the steep hillside.

Over time and many attempts at repairs, these minor problems became a constant drain on the facility operating budget and were unable to provide reliable site access during major storms.

Figure 2. Rebar-Mesh Washout Repair

3.3 *Gabion Repairs*

In the latter part of 1993, a series of major rain storms caused significant damage to the access road. During 1994, the Salt River Project contracted the repair of 38 localized washouts along a 3.2 km stretch of the access road. Because of the irregular nature of the repair sections and the urgency of repair work, small gabion retaining structures were selected as the method of repair. Three typical cross sections were designed for use under various subsurface conditions (Fig. 3).

Work incorporated placing on the order of 450 linear meters of 91mm x 91mm gabion baskets. Individual gabion units varied in length from 0.9 to 3.7 meters. 76mm to 200mm nominal diameter fill stones were imported in from the Metropolitan Phoenix area since little native material met the size and specific gravity requirements. Where feasible, gabions beared on level concrete pads anchored into near-surface rock (Fig. 4). In areas where cobble and boulders were present, the bottom row of gabions was placed from 15cm to 45cm below grade.

To prevent loss of fine soils into gabions, filter fabric was placed between gabions and the backfill. Gabions were installed so that runoff water principally flowed over the top of the baskets, preventing edge erosion from water draining off of the road. This was done by either setting a gabion section lower that the adjacent units and the roadway, or by dipping the top row to allow sheetflow and create a spillway effect. Examples of both types are shown in Figures 5-6.

Lateral earth stability was the only analysis required for these structures since they were founded on either bedrock or highly stable rock fill (q_a > 0.24 MPa) and were close to bedrock. When considering the largest repair structure (H = 4.6m) at the steepest configuration (23cm step-back for 91cm x 91cm gabions), factors of safety for sliding and overturning were 2.52 and 1.63, respectively for

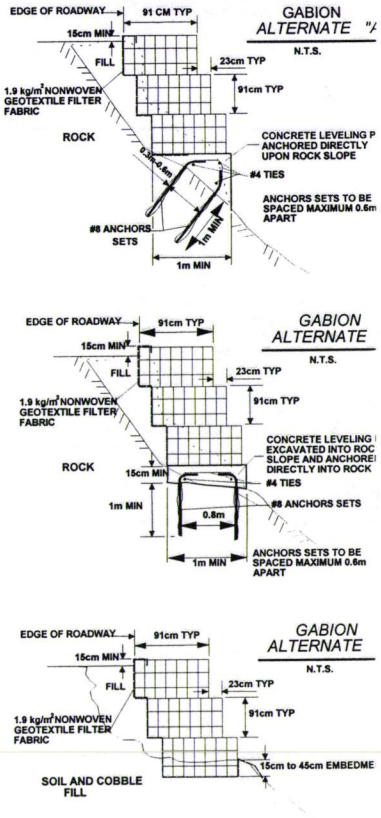


Figure 3. Gabion Repair Cross Sections

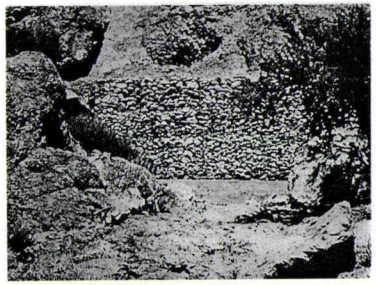


Figure 4. Gabion Repair on Concrete Pad

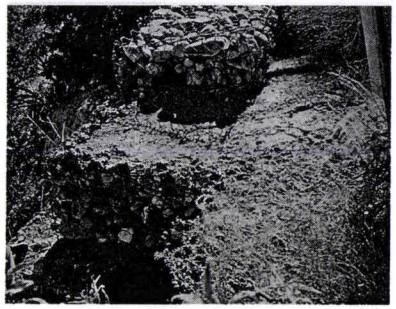


Figure 5. Single Gabion Lowered for Spillway

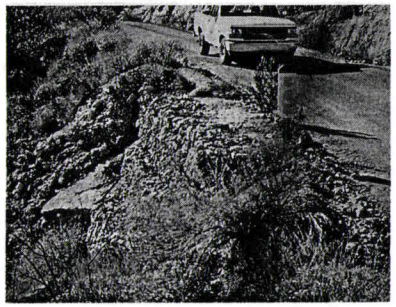


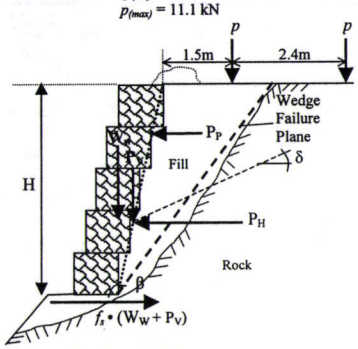
Figure 6. Gabion Structure Lowered for Spillway

standard load conditions (project required use of standard loads for design). For the maximum possible load case, factors of safety for sliding and overturning were 2.00 and 1.07, respectively. This was deemed acceptable for this site by the project designers. The analysis is shown in Figure 7.

Gabion repairs were performed over a 10 week period at a total installed cost of about $242 per cubic meter.

In the fall of 1994, the site encountered a rainstorm that resulted in 50mm of rain fall in a single hour. Gabions showed only minor erosion damage to the upper baskets edges and the road remained in service. Minor repairs were undertaken within one month of this storm, using earthen and concrete roadside berms to better direct future flows onto gabion structures. Since the 1994 repair, no other repair work has taken place on these gabion structures. A comparison of repair locations over this time period is shown in Figures 8-9.

Wall Properties: height of wall, H = 4.6m
91x91 cm baskets staggered @ 23cm
Unit weight of gabion stones = 2640 kg/m^3
Gabion basket porosity = 35%
Soil Properties: unit weight = 2000 kg/m^3
friction angle, φ = 40°
cohesion = 0 kPa
Wheel loads: $p_{(std)}$ = 3.6 kN
$p_{(max)}$ = 11.1 kN



Wall weight, W_w = 21.4 kN
Wall friction angle, $\delta = 2/3\varphi$ = 26.7°
Wall angle to active plane, β = 101.3°
Base friction coefficient, f_s = 0.84
Earth pressure coefficient, K_a = 0.132
Horizontal component of thrust at wall, P_H = 7.4 kN
Vertical component of thrust at wall, P_V = 3.7 kN
Horizontal thrust cause by point load, $P_{P(std)}$ = 1.0 kN
$P_{P(max)}$ = 3.2 kN

Figure 7. Gabion Wall Design Analysis for Road Edge Stabilization

Figure 8. Road Edge Gabion Stabilization – 1994

4 RECLAMATION SLOPE STABILIZATION

Reclamation work for a mining development project in a remote high mountain region in northcentral

Figure 9. Road Edge Gabion Stabilization - 1998

Utah required the use of small gabion wall sections to stabilize slopes in rugged terrain. The Uintah Mountain Copper Company needed to install a gravity wall system as part of a pilot project for development of a natural iron oxide (hematite) pigment mine. Limited access required wall materials be both light-weight for ease in transportation and utilize on-site rock for aesthetics. Slope stabilization methods also needed to be inexpensive and compatible with the forest environment, leaving a natural appearance.

Gabion units were selected for placement in a test pit section on a high-energy talus slope/slide zone that undergoes extreme seasonal runoff conditions. This site is at an elevation of 3110 meters above mean seal level and on the edge of a small vegetated ravine. Estimated annual precipitation is on the order of 50cm, with the majority of the moisture from snow that is on the ground from October to June. Primary vegetation includes Douglas Fir trees along steep hillsides. Grassy meadows are intermingled in the Douglas Fir stands, which give way to treeless, barren slopes of the Alpine environment.

4.1 *Geology & Soil Conditions*

The site is approximately 40 km northeast of Duchesne, Utah and due west of Moon Lake in the Slate Creek Canyon Region's north face of Dry Ridge within the Ashely National Forest. The site is located on the down side of an almost vertical displaced normal fault of the Uintah South Flank Fault and elechlon fault system on the north and a series of elechlon faults and a brecciated zone on the south side of the property. This fault displaces Mississippian limestone formations and is underlain by Cambrian quartzite and shales (Wall 1990). The test pit area is within a zone of highly fractured and thinly bedded limestones that are highly weathered (clay-like) within the ore pockets.

4.2 *Test Pit Reclamation Program*

Once ore and overburden rock are removed fron test pit, the area is to be recontoured for reclam: purposes using overburden and native topsoils. S side slopes (30° to 40° from horizontal) prec laying back of the hillside to a stable natural a without a support system. Review of prc worldwide show that similar steep slopes in zones have been successfully stabilized by terr: with gabion structures (Maccaferri 1995). Figu provides a cross section of the proposed ter slope at the Uintah Mountain site.

Maximum overburden at the deepest section the order of 14m. The steep rock face is to be a 5H:1V slope and rock pinned as needed to pr the pit from limestone rock mass wasting. A s of four gabion retaining wall structures ranging 12 to 24 meters in length are to be placed up meters above adjacent terrace grades at step slopes up to 70°, making the reclamation fea Rock backfill is to include native limestone an soils near the top of the baskets to enco vegetation growth on and above the reta structures.

Backfill material was evaluated from road c determine strength an unit weight properties. N fractured and weathered limestone breaks upon excavation and recompaction predominately a clayey gravel and cobble Water content of the clay component can upwards of 20%, with a total mix unit weight compaction) of 2080 kg/m^3. Cohesion varies and a nominal 0.5 kPa capacity was assigned mix. Large-size interlocking angular grave cobble rock were compared to similar backfil internal friction angle of 40° was assumec conservative value for the compacted mix.

Each terrace structure required a separate a including evaluation for both lateral and slope stability (deep-seated failure of Coulomb earth pressure methods were utiliz

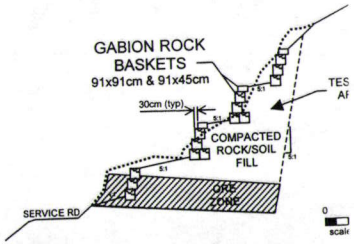
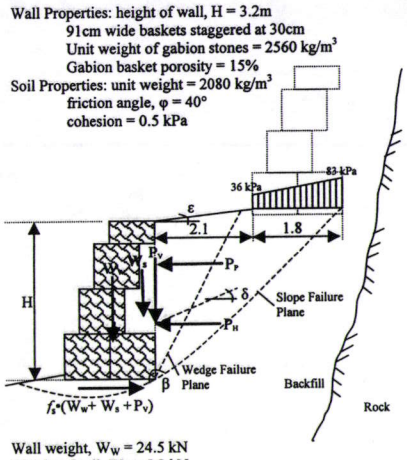


Figure 10. Reclaimed Test Pit Cross Section



Wall weight, W_W = 24.5 kN
Weight of soil, W_S = 5.2 kN
Wall friction angle, $\delta = 2/3\varphi = 26.7°$
Backfill slope, $\varepsilon = 11.3°$
Wall angle to active plane, $\beta = 90°$
Base friction coefficient, $f_s = 0.84$
Earth pressure coefficient, $K_a = 0.225$
Horizontal component of thrust at wall, P_H = 6.4 kN
Vertical component of thrust at wall, P_V = 3.2 kN
Horizontal thrust cause by point load, P_P = 4.8 kN

Figure 11. Gabion Wall Design Analysis For Reclamation Stabilization

determining lateral stability. Slope stability was calculated via a gabion design computer program that uses Bishop's method (Maccaferri Gabions GAWAC R2.0). Figure 11 presents the analysis for the most critical case.

The factors of safety for this section in terms of sliding, overturning and slope stability were 2.48, 2.00 and 1.50, respectively. This was deemed as adequate for the project.

4.3 *Construction of Test Section*

The first phase of this exploratory test pit reclamation development program was approved to determine the impact of such a retaining structure on the hillside and forest environment. Excavation of a 3.6 to 4.6 meter wide, 14 meter long and 0 to 3.6 meter deep phase 1 test pit was completed in early September 1997. Reclamation commenced a few weeks later once all survey and ore body measurements were complete. The gabion retaining wall for this section (the lowest elevation structure shown in Fig. 10) was built in 2 days at a total installed cost of $105 per cubic meter. The wall consisted of a simple 9m long, stair-stepped, three-tiered structure. Three feet of overburden fill was placed above the top of the upper row to improve access and allow observation of seeding tests. Rock fill for gabions utilized a combination of hand-selected 10cm to 30cm nominal diameter rock from the excavation area (matching the color and texture of the intact slope rock) and nearby talus slope residual limestone sands, gravels and cobbles (Fig. 12). A 1.1 m^3 loader and D7 Caterpillar bulldozer were used to place and compact backfill. The area above the wall was terraced and seeded upon completion of the structure.

Weather prevented site access after October 1997. This is typical for the region. Figure 13 shows the completed gabion wall at that time. A long snowmelt and runoff season did not allow return to the site until August 1998. At that time, a detailed evaluation was performed on the structure and other reclamation activities. A slide had deposited up to 3m of material on the terrace above the wall, but did not result in any significant movement. The wall had rotated outward, due partially to the fact that the rows were set steeper than the planned 30cm step-backs. Some fines had washed out of the baskets and deformations were observed. Adequate material was still present so that vegetation was actively growing on the terraced top and along the face of the gabion baskets. The wall appearance (color and texture) blended in well with the surrounding rock faces. Figure 14 shows the wall after one year in place.

5 CONCLUSIONS

1. Gabion retaining structures are simple and economical for use in small-scale slope stabilization projects.

Figure 12. Gabion Fill in Baskets at Reclamation Test Site

Figure 13. Reclamation Gabion Stabilization September 1997

Figure 14. Reclamation Gabion Stabilization August 1998

2. Although highly deformable and flexible, gabion retaining walls are designed by the same traditional limit equilibrium methods used for rigid gravity structures. Both lateral and overall slope stability need to be considered in rockfill slopes. Bearing capacity in rock conditions is generally not significant.

3. Gabions were successfully used to stabilize numerous road-edge wash outs from local storm water inflow along the Horse Mesa Dam access road in Arizona. Since their installation in 1994, no significant maintenance has been required on these structures. The total installed cost for gabion walls at this site was about $242 per cubic meter.

4. Reclamation work for a mining development project in a remote high mountain region in northcentral Utah required the use of small gabion wall sections to stabilize slopes in rugged terrain. After one year in place, the wall has functioned well and has met project objectives. The total installed cost for gabion walls at this site was about $105 per cubic meter.

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