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1974, pp. 359-363. https://doi.org/10.56577/FFC-25.359

in:

Ghost Ranch, Siemers, C. T.; Woodward, L. A.; Callender, J. F.; [eds.], New Mexico Geological Society 25 th Annual Fall Field Conference Guidebook, 404 p. https://doi.org/10.56577/FFC-25

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LANDSLIDES ON "BRAZOS PASS"

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"Brazos Pass" is a part of the northern scenic route of U.S. 64 which crosses the Tusas Mountains in north-central New Mexico. The Tusas Mountains are a southern extension of the San Juan Mountains of southern Colorado. Not being one of New Mexico's more impressive ranges, they have long been one of the most inaccessible areas in New Mexico because of the lack of low passes through difficult terrain, a rather sparse population and a general economically depressed area. Until U.S. 64 from Tierra Amarilla to Tres Piedras was built, the traveler going from Tres Piedras and the eastern parts of Rio Arriba County to the county seat at Tierra Amarilla, had to travel south almost to Espanola or north through Antonito, Colorado. Alternatively, a sparse network of circuitous, unmaintained dirt trails could, with patience and luck, provide access through the range. Therefore, this road has been a boon to the local residents as well as to the developing tourist industry.

Planning for the new road began in the early 1960's and the first project was let to contract in August, 1964. The last part to be paved was completed late in 1971. Portions of the road follow pre-existing unsurfaced county and forest roads; other parts, including the part discussed here, penetrated virgin territory. Some of the planning, design and construction was done on a "crash" basis with soils data largely gathered from snow-covered terrain. Because of limited funding, geological studies were of a preliminary nature and were not supported by hydrological studies or by slope stability tests.

The mountains are geologically complex, with a Precambrian core of granite, quartzite and other metamorphic rocks, overlain by scant sediments near the margins and extensive Tertiary volcanics elsewhere. The area has been modified by Pleistocene glaciers and glacial outwash deposits and by widespread landsliding.

The problem area, primarily on landslide terrain, extends from the floodplain of the Rito de Tierra Amarilla, about ten miles east of the village of Tierra Amarilla, and climbs along the south flank of Penasco Amarilla (yellow boulder) to "Brazos Pass," about 2,000 feet higher. The major bedrock formation of the area is the Cretaceous Mancos Shale, a gray, marine, clay shale. During Pleistocene glacial periods, it was covered by a thick outwash deposit of gravel, remnants of which cap the highlands of the Brazos divide. Apparently the parent glacier for these gravels, the Ritito Conglomerate, occupied the south flank of the San Juan Mountains in Colorado. Quaternary downcutting and erosion have left a very large area of semi-stabilized landslides of glacial debris and gravel mixed with the underlying shale. This old landslide complex extends southward from the Brazos River at the Brazos Box to the Canjilon area, a distance of some twenty miles, and nearly covers the entire western slope from crest to valley through this section. Lakes, swamps and peat bogs dot the area and abundant precipitation contributes to a high water table.

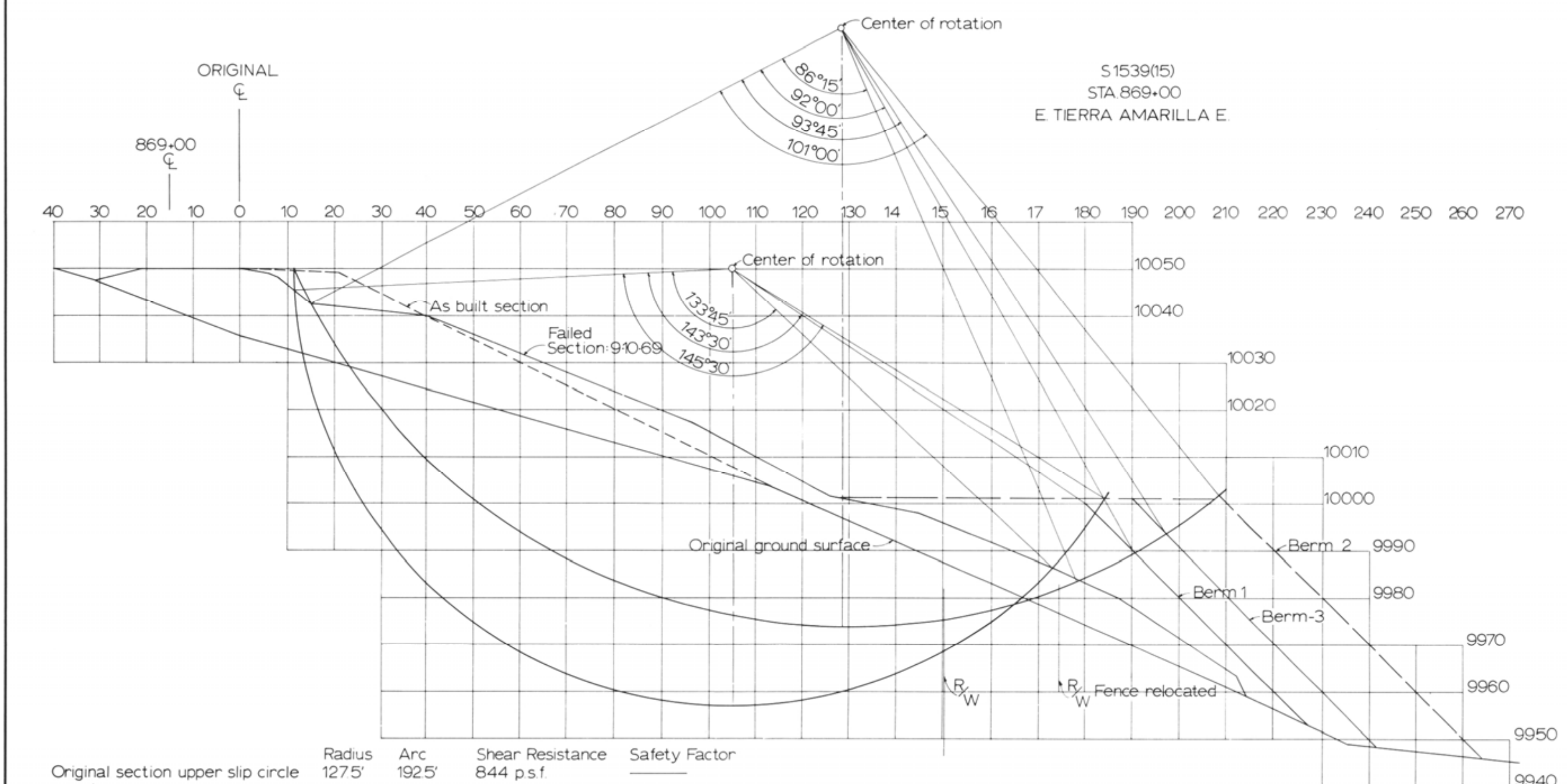
Cut-slope failures (Fig. 1) began soon after completion of project S-1539(8) in 1966 and have continued to the present time. Only a few cuts have remained unaffected and many show massive instability. The failures range from classical rotational shear failures to earth and mud flows. Some mud flows are acting on incredibly flat slopes, commonly 6:1 or flatter.

Figure 1. Cut-slope failure near Station 700+00. Note the leaning trees and the scarp where toe material has been removed from the roadway.

Because of limited maintenance funds, no corrective measures have been taken to control the failing cut-slopes. Highway maintenance forces will be able to correct a great number of the cut-slope failures utilizing the cosmetic approach described by McGuffey (1973), whereby the slope is cleaned and reshaped and a two foot minimum thickness of open graded materials is placed over the zone of failure. It is not expected that this approach will be effective on some of the larger failures. Rock buttresses and horizontal drains may be required to stabilize the slopes in some areas.

Three fill slope failures developed during the summer and fall of 1969, about a year after construction of the subgrade. The surfacing had not yet been placed, since this was a stage construction project. One of these failures was a steep cut and fill section at Station 641+100, one in a moderate fill section placed upon a gentle slope below a small lake at Station 840+00 and another on a relatively large side-hill fill section at Station 869+00.

Subsurface exploration and on-site conditions indicated that each slide had pronounced rotational movement with minor translatory or flow characteristics. A shallow ground-water source of adequate volume to have caused the problem was



	Radius	Arc	Shear Resistance	Safety Factor
Original section upper slip circle	127.5'	192.5'	844 p.s.f.	———
Original section lower slip circle	93.5'	217.9'	927 "	———
Berm no. 1 Upper slip circle	127.5'	205.0'	721 "	1.17
Berm no. 1 Lower slip circle	93.5'	233.8'	767 "	1.21
Berm no. 2 Upper slip circle	127.5'	224.4'	620 "	1.36
Berm no. 2 Lower slip circle	93.5'	237.0'	746 "	1.24
Berm no. 3 Upper slip circle	127.5'	209.1'	674 "	1.25

SCALE: 1"=10'
Wm Gonzales

Figure 2. Cross-section at Station 869+00, showing tested configurations.

located in each slide. It was decided that the best solution for correcting these slides would be to construct a supportive berm along the flanks of the fill which would restrain the driving forces moving through the maximum anticipated failure arc of each slide. A variety of trial berms were examined to determine the necessary berm size. The center of rotation of the maximum failure was estimated to be at the intersection of a horizontal line projected from the crown of the slide and the perpendicular bisector of an arc passing through the crown and foot of the slide. The center of rotation for a more shallow seated failure was placed somewhat higher (Fig. 2). Moments were taken about each center of rotation. The berm size needed to increase the factor of safety of each slide 25 to 50 percent was calculated utilizing Baker and Yoder's equation (1958), $S = (W_1 D_1 - W_2 D_2)/rl$, where the driving forces are designated W_1 times their lever arm D_1, and the resisting forces are designated W_2 times their lever arm D_2. The radius of the slip surface is r and the included length of the arc is l. For purposes of calculation, the slides, having slumped to a point of relative stability, were assumed to have a factor of safety (f.s.) of 1.0. The f.s. for the condition was defined as the ratio of the failed condition to that of the improved condition.

Using this method, it was found that original highway embankment through the failed areas had factors of safety near 0.64. At station 643+00, the berm selected brought the f.s. to 1.29 using a maximum depth failure arc. At station 840+60, the selected berm gave a f.s. of 1.26 with a shallow slip circle and 1.24 with a full depth arc. At Station 869+00, the berm selected gave factors of safety of 1.25 and 1.24 for maximum depth and shallow seated slides respectively.

In addition to the berm, the aquifers were drained with perforated pipe drains located from 4.5 to 17 feet below grade with average depth of about 14 feet. All perforated drains were located in the borrow ditch above the road and were backfilled with free-draining granular material. Access manholes were placed at most cross-drains for maximum serviceability.

The project, S-1539(15), was implemented during the summer of 1970 and was immediately followed in April, 1971 by project S-1539(14) for surfacing of the highway. Other than continuing cut-slope failures, no problems occurred until the very wet winter and spring of 1972-73.

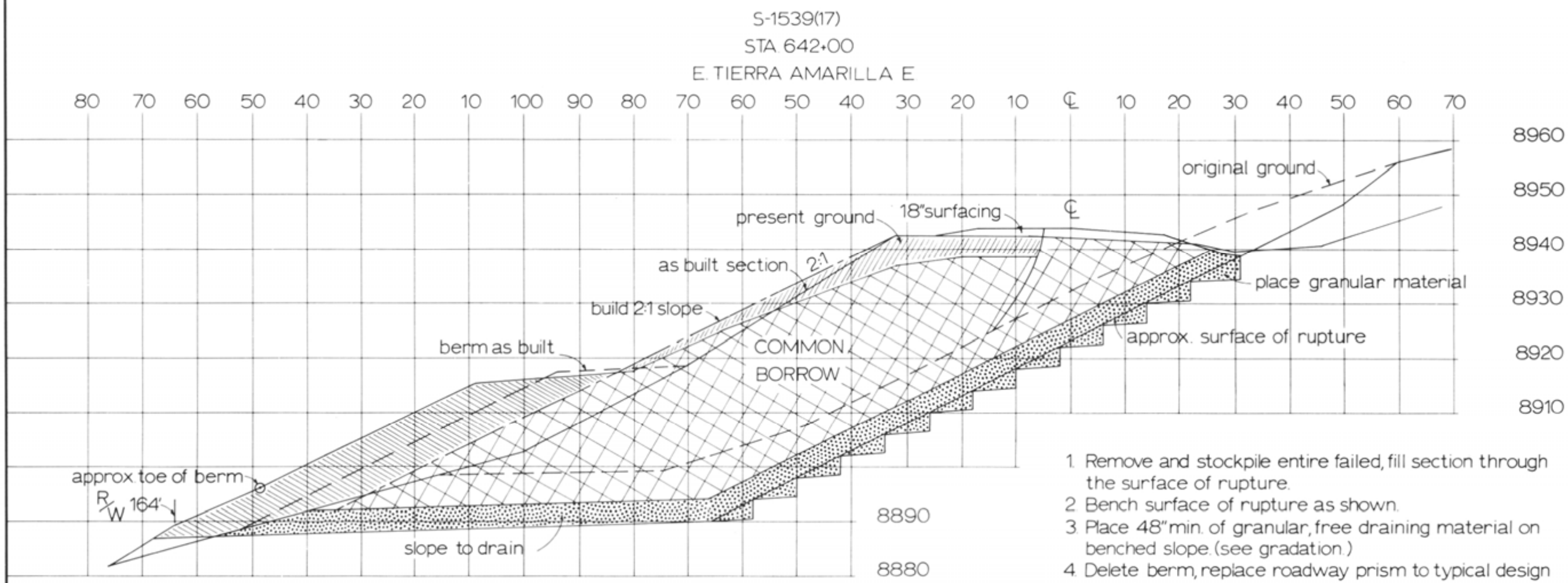
Early in 1973, movement was noted at two places within the roadway fill. One of these areas, at Station 589+50 (Fig. 3), had not shown distress previously. The other area, Station 643+00 (Fig. 4), showed renewed movement at and on either side of the previous slide.

Subsurface exploration of the slide at Station 589+50 revealed that an aquifer in glacial gravel within the old landslide debris had been dammed up by the roadway fill. Hydrostatic pressures had resulted in saturation of the fill, and movement developed along the original ground surface. Test holes established the limits of the aquifer and showed that the till layer beneath the moisture was a gravelly clay, impermeable and competent. A method of correction was selected which involved removal of the slumped material, placement of a granular drainage blanket of open-graded aggregate and replacement of the fill. A 5 x 7 foot reinforced concrete box culvert, which was torn into four pieces by the slide, was also scheduled for replacement.

To compute the stability of the proposed correction at Station 589+50, strength parameters typical of the granular material, $\phi = 30^\circ$ and $c = 0$, were used, since the entire area was to be underlain by granular material. These figures were then inserted into the standard equation for translatory slides to determine the factor of safety. The equation is f.s. $= (\Sigma N \tan\phi + cl)/\Sigma D$ where ΣN is the sum of the tangential forces, ΣD is the sum of the driving forces, ϕ is the angle of

Figure 3. Fill-slope failure at Station 589+50. The hole at the right foreground was caused by erosion through a gap in the severed box culvert.

Figure 4. Upper section of the fill-slope failure near Station 643+00. Another failure lies immediately behind the photographer.



LEGEND

- Excavate / stockpile
- Excavate, stockpile / replace
- Replace

GRADATION for GRANULAR MATERIAL

1"	100
3/4"	90-100
No. 4	25-40
No. 30	5-15
No. 100	0-8
No. 200	0-3

SCALE: 1"=10'
Wm Gonzales

1. Remove and stockpile entire failed, fill section through the surface of rupture.
2. Bench surface of rupture as shown.
3. Place 48" min. of granular, free draining material on benched slope. (see gradation.)
4. Delete berm, replace roadway prism to typical design section with common borrow (stockpiled material.)
5. Resurface repaired area to typical design section.
6. If excessive ground water is encountered during excavation, drain with perforated pipe prior to replacement of fill at the discretion of the Project Supervisor.
7. The benched slope section may be raised or lowered at the discretion of the Project Supervisor in coordination with the Central Materials and Testing Laboratory.

Figure 5. Cross-section at Station 642+00, showing details of the planned reconstruction.

internal friction, c is the cohesion and I is the length of the sliding plane. Computation yielded a factor of safety of 2.8 without the addition of a berm or other stabilizing device. Such f.s. is obviously adequate; however, to guard against plugging of the gravel blanket and subsequent problems, a perforated pipe drain was designed to be placed in the lower part of the aquifer below the uphill borrow ditch. The pipe will divert most of the water before it passes into the drainage blanket and serve to lower the hydrostatic head.

Subsurface inspection in the area around Station 643+00 indicated that the previously installed perforated pipe was working well; no water seepage was found downstream from the pipe. Probing within the roadway prism, above the landslide and below the perforated pipe, showed a number of erratic, small aquifers about 30 to 40 feet below the grade, apparently developed in fractures in the disturbed shale. The depth to the water precluded the use of perforated pipe for drainage and, horizontal drains were judged unreliable because of the improbability of intercepting the water within the generally unfractured shale. The movement of the slide, formerly rotational, had been modified by the berm into translatory displacement, based somewhat below the natural ground surface. The two problems, lack of drainage and translatory movement, suggested the solution used for the slide at Station 589+50, and a granular drainage blanket, set on a stable series of benches below the slide plane, would give a f.s. of 1.74 with the original corrective berm replaced and 1.54 if the berm was not rebuilt. As a compromise, in order to provide additional stability and to utilize waste generated by the repair operation, a correction was designed with a 2:1 slope, rather than the initial 11/2:1 slope. With a f.s. of 1.61, this design was accepted (Fig. 5).

On both slides, the drainage blanket will have a maximum thickness of 48 inches to allow for some plugging by the overlying fill during and after construction. The gradation specifications will require 100 percent smaller than 1 inch, 90 to 100 percent passing the 3/4 inch sieve, 25 to 40 percent below the number 4 sieve and less than 8 percent and 3 percent passing the number 100 and number 200 sieves, respectively. Gravel was obtained from a pit in reworked glacial gravels in terraces along Rito de Tierra Amarilla.

The contract to rebuild these slides will be let in May or June of 1974 and probably will be under construction during the field conference.

In retrospect, using the advantages of hindsight, it is apparent that a more detailed geological and soil survey, including shear tests and hydrological studies, would certainly have been desirable during the preliminary design phase of the roadway. The combination of water, soils characteristics and terrain certainly are unique to this particular area and previous experience in designing and building roads across this type of terrain has been rather limited. As previously mentioned, most of the preliminary engineering work was done on a "crash" basis, and most of the soils investigations were done during inclement weather and adverse conditions. The funding was limited (100% state financed), and detailed geological studies had not yet become a part of the routine preliminary work of the Highway Department. To have insured against the type of failures that occurred would have required an extremely elaborate, and expensive design; however, such a design, which would have provided gravel plating on many of the side hill fill sections, some horizontal drains, more perforated pipe, and rock plating cut sections, would have saved the taxpayer a considerable amount of money over a long term. Neither the soils nor the geology of this area are particularly complex but knowledge of them is critical to the engineering design of roadways and structures.

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