with one rather than the other. No reliable criteria have been determined for deciding a preference between window mapping and line mapping. It remains a matter of site conditions and personal preference. A third site for testing the system was mentioned at the beginning of this paper. The site is located near Clifton Forge, Virginia, in the Iron Gate anticline. A large road cut is planned through the anticline, but currently rock exposures are poor along most of the planned construction. At a site such as this, detailed line mapping is not possible and window mapping appears to be the best alternative.

It was determined during the study that a combination of the window mapping and line mapping techniques is sometimes useful. At the Cedar Bluff sites, much of the two slopes was mapped by setting up 50-ft-long windows, 30-40 ft apart, but mapping within them as if doing detailed lines. This proved to be a satisfactory compromise between the two techniques for this particular location. As expected, the computer-based data collection system worked equally well for either technique, using the same data collection format for both. Some of the data were simply not collected depending on which procedure was being used.

The rapid data collection and evaluation system has been shown to provide a number of time-saving services. It decreases data collection time by about 20-50 percent and provides data plots for preliminary analyses in hours instead of days. It is also capable of transferring the collected data to a larger computer system for additional analysis by use of sophisticated FORTRAN programs.

A few changes could be made to the system to improve its ease of operation. The pocket computer data-collection program could be rewritten to allow a short remark to be recorded after each joint if desired. By using new knowledge, the prototype hardware interface can be rebuilt to provide a less complex form, which makes it simpler to use. In addition, a newer model of microcomputer could be used for the preliminary analyses and stereonet plotting. Microcomputers that are smaller, will connect to any television set, and are less complicated to disassemble and transport are now available in the same price range. Finally, a number of programs are currently being written for the microcomputer system that will handle involved calculations of factors of safety for potential failures once they have been identified. The proposed changes will make this computer-based data collection system simpler to use while making it an even more powerful analytic tool.

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The Malibu Landslide

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A case history of one of the most highly publicized landslides in the history of the California Department of Transportation is presented. In the spring of 1979, the recognition of an incipient and potentially dangerous massive slide above steep sandstone bluffs adjoining the Pacific Coast Highway (CA-1) between Malibu and Santa Monica necessitated closure of this vital transportation link. The installation of an extensive and unique warning system plus a rock fall catchment area permitted partial reopening one month later while the slide investigation proceeded. The correction ultimately selected involved a partial unloading and slope reinforcement. The four-way warning system proved to be extremely useful in permitting the partial reopening of the highway while providing for the general safety of the traveling public and adjacent home owners. The use of rock dowels for slope reinforcement reduced the required excavation to achieve stability by more than 1 million yd^3 in extremely difficult terrain.

What has become known in Southern California as "the Malibu landslide" is located adjacent to the Pacific Coast Highway (CA-1) 1.5 miles west of Topanga Canyon in the central part of the Santa Monica Mountains between Santa Monica and Malibu. This area is characterized by steep natural slopes and rocky escarpments. Landsliding, a common occurrence along this part of the Pacific Coast Highway, is a natural process of bluff recession due to past oversteepening by wave erosion. The bluffs are composed primarily of thick-bedded sandstone with interbeds of siltstone and conglomerate. These sedimentary rocks have been intruded by volcanic rocks that are now highly altered. They have been folded and faulted to create a complex geological structure. Some of the joint patterns of the rock are oriented adversely and thus contribute to the instability of the slope.

Figure 1 is an oblique aerial photograph of the

Figure 1. Oblique aerial view of Malibu slide.

Figure 2. Rock slide on Pacific Coast Highway, April 13, 1980.

bluff that shows the active slide area, the Pacific Coast Highway, and homes on its seaward side.

In January and February 1979, California Department of Transportation (Caltrans) maintenance personnel observed a significant increase in the number of rock falls along this stretch of the highway. One extremely large rock (>100 tons) appeared to be precariously balanced on the bluff face and was judged to be a menace, not only to the heavy traffic on the highway (average daily traffic = 30 000 vehicles) but also to the homes on its seaward side. Removal of the rock was accomplished on February 16, 1979, by emergency contract. On April 13, 1979, a rock fall occurred that was of sufficient magnitude to warrant closure of the Pacific Coast Highway (see Figure 2).

A subsequent field review of the area by Caltrans geologists and engineers immediately above the bluff where the closure had been made revealed an extensive system of cracks and fissures extending as far as 350 ft up slope from the bluff face. The generally geometric pattern of the cracks and the 1.5-ft-wide, 3-ft-deep scarp at the head of the slide, shown in Figure 3, indicated the existence of an unstable mass of up to 60 000 yd^3 of soil and rock

Figure 3. Fissure at head of slide.

above the bluff face. The bluff face, except for one area later identified as "the chute", appeared to be relatively stable. An earthquake of Richter magnitude 5, which occurred January 1, 1979, and had an epicenter about 8 miles southwest of the slide, may have been a factor in the initial cracking, although this could not be established definitely.

MONITORING

The first problem posed by the slide was how to open the Pacific Coast Highway, at least partially, while ensuring the safety of those traveling the road and the residents of the homes on the seaward side. The plan that ultimately evolved was the development of a barrier suitable for the catchment of relatively small earth and rock falls and a monitoring system that would provide advance warning of a large mass movement. At any indication of mass movement, the road would be closed and the houses evacuated. The feasibility of such an approach was based on past experience by Caltrans geotechnical personnel, which showed that certain types of incipient slides that are properly instrumented and monitored provide hours to days of warning of sudden mass movement. It was reasoned that development of an adequate catchment zone and monitoring system would permit partial reopening of the highway while exploration leading to the development of a permanent correction could be undertaken.

Three "early warning systems" were installed in the badly fissured area above the bluff between mid-April and early May. A plan view of the monitoring system is shown in Figure 4. Steel pins were driven into the slope at various locations, and measurements were taken between them on a daily basis so that the rate of movement across various segments of the hillside could be determined. Beginning April 18, four acoustic emission [subaudible rock noise (SARN)] stations were established for the purpose of obtaining a relative measure of slide activity. These microacoustic devices consist of highly sensitive geophones capable of sensing rock noise well below human acoustic range. Individual clicks or events per minute at a given site are counted by trained personnel (see Figure 5). Having established a background level of activity for a particular site, an increase in events or counts per minute provides warning of mass movement. Eventually, six acoustic emission stations were established and monitored on a near-continuous basis. On the evening of April 25, three of the six stations then in operation indicated a sudden acceleration in count rate, which prompted closure of the pedestrian walkway along the highway. A plot of counts per minute versus time for one such station (PATH) is shown in Figure 6. Three days later, noise levels dropped to the original background level so that reopening of the walkway was permitted.

Figure 4. Malibu slide instrumentation.

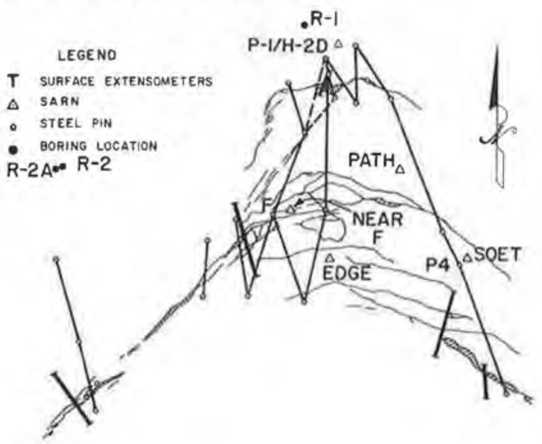
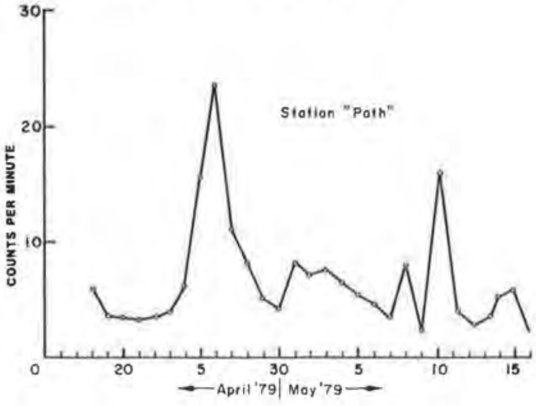


Figure 5. Monitoring SARN.

Figure 6. SARN measurements for Malibu slide.



The two monitoring systems described required the presence of surveyors or geophysicists on a continuing basis. It was deemed extremely hazardous for personnel to work on the unstable and very steep bluff at night for monitoring purposes. With this in mind, a third warning system was installed that would automatically trigger an alarm at roadway level in the event of significant slope movement. These devices, designated as "surface extensometers", were installed across selected zones of high activity based on previous survey measure- ments. They consisted of wire several feet long connected to two steel pins 3-4 ft above the ground surface (see Figure 7). A constant tension was maintained by a spring mounted in a metal housing attached to the system. Movements exceeding 0.15 ft closed an electrical contact, which in turn would trigger a battery-driven alarm system at roadway level. The alarm itself consisted of a flashing yellow light and an air horn (see Figure 8).

A fourth safeguard was the presence, on a 24-h basis, of a maintenance worker who continuously observed the slope, provided warning of rock falls, and could immediately initiate highway closure if so warranted by activity on the unstable slope. The

Figure 7. Surface extensometer installation.

Figure 8. Surface extensometer alarm system.

maintenance worker on duty was instructed to inform geological personnel assigned to the project immediately in the event of an alarm in order that monitoring of the acoustic emission stations could be initiated. In the event that two extensometer alarms were triggered, he was instructed to close the roadway immediately and alert the County for possible evacuation purposes.

During the second week in May, survey points were established on the face of the bluff below the slide mass to verify that the bluff was not moving.

Concurrent with installation of the monitoring systems, a soil-and-rock berm approximately 8-10 ft high was constructed near the toe of the slope. This berm consisted primarily of slide debris that covered the northbound outer lane (see Figure 9). A 10-ft-high chain link fence was installed on top of the berm. The purpose of the berm-fence system was catchment of small earth and rock falls, which would safely permit the opening of the remaining three lanes. Partial opening of the highway occurred on May 6, 1980, after a period of relatively little slide activity. Shortly after the highway was reopened, however, rock and soil falls increased in frequency and magnitude and quickly filled the catchment area. Slide activity precluded the safe removal of debris that collected behind the berm. Consequently, the Pacific Coast Highway was again closed to vehicle traffic on May 8. Construction of an improved catchment system was immediately begun. This consisted of H-beam uprights with heavy timber lagging. Individual H-piles were set in 10-ft-deep predrilled holes and backfilled with lean concrete. The H-pile wall was 12 ft in height from highway profile grade. Secured to the wall was an additional 12 ft of chain link fence for rock catchment (see Figure 10). This system substantially increased the storage capacity for rock and soil fall. On completion of the H-pile timber wall on May 20, three lanes of the highway were again opened. The new system functioned effectively throughout the remainder of the preconstruction phase of the slide.

Figure 9. Soil berm catchment system on Pacific Coast Highway.

Figure 10. H-pile catchment wall and fence on Pacific Coast Highway.

ELEMENTS OF FAILURE

Information obtained from the exploration and monitoring indicated that geological conditions for the slope on which the material was failing differed from conditions exposed in the bluff area. This relation is consistent with geologic mapping by Yerkes and others of the U.S. Geological Survey in 1973. A generalized geologic cross section of the slide is shown in Figure 11.

The beds exposed in the bluff below the failing material have been mapped as Vaqueros Formation and are composed mostly of thick-bedded sandstone with interbeds of siltstone and conglomerate. These beds have been intruded by irregular masses of diabase. Analysis of discontinuities in the rock indicates a stable condition except for the aforementioned "chute area".

The formation within which the failure occurred has been mapped as Topanga sandstone with inter-

Figure 11. Generalized geologic cross section.

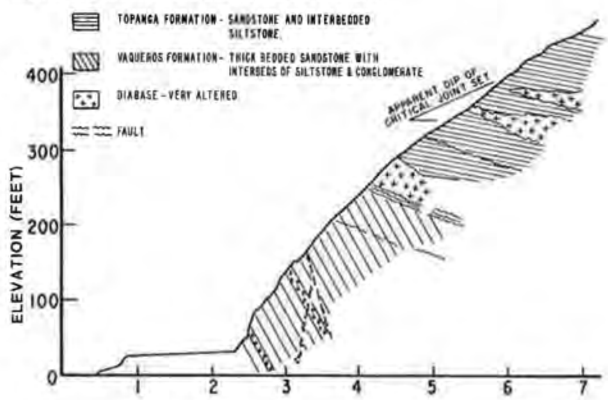
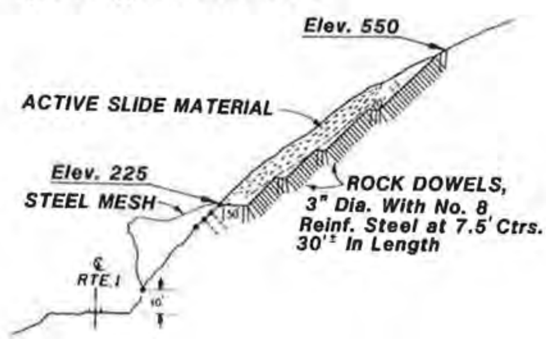


Figure 12. Typical rock dowel section.



bedded siltstone. This formation has also been intruded by diabase that is now intensely altered. Some of the joint patterns exposed in the upper slope are oriented adversely and contribute to the instability. Joint-controlled failure is also suggested by the geometric pattern of the cracks that developed.

Initial cracking of the hillside may have occurred during the January earthquake, but that cannot be proved. The rains of January through March certainly reduced the stability of the hillside.

The pattern of cracking and the types of failure that were occurring suggested that the depth of failed material was shallow. Drilling and seismic refraction indicated a thickness of about 30 ft of disturbed material. The active slide material is shown in section in Figure 12.

LABORATORY TESTING PROGRAM

To better establish the strength characteristics of the materials involved and to provide input for design of a correction, selected samples of weathered sandstone and volcanic rocks common to the site were subjected to a series of multistage triaxial compression and direct shear tests. The purpose of these tests was to determine fracture strength at varying confining pressures and the residual shear strength (strength available after excessive strain and fracture have occurred). Wetting of the weathered sandstone resulted in complete degradation to sand in a relatively short period of time. This demonstrated the need for effective surface drainage for final correction of the slide.

Triaxial test specimens of the sandstone were prepared by diamond coring 2-in-diameter samples from selected boulders. Triaxial tests (unconsolidated undrained) were conducted under an initial confining pressure of 5 psi to determine rupture and thus bond strength of the cementing agent. Samples were sheared in a dry state at a strain rate of approximately 1 percent/min. Shearing continued after rupture until a residual strength value was obtained. Subsequently, confining pressure was increased and shearing continued. This procedure permitted the development of an effective stress Mohr envelope for each specimen. Results of the triaxial test series are summarized in Table 1.

Results of triaxial testing on specimens of the sandstone fragments indicate that the cohesive or binding agents provide a highly competent material prior to fracturing if water is not present. On exposure to water, very slight finger pressure was necessary to crumble the various sample fragments that remained intact after the tests whereas extreme finger pressure was required to break or crumble fragments before the test. The tests revealed high resistance to sliding friction while the specimens were in a dry state. Shear planes that developed after the initial rupture during triaxial testing were irregular or rough compared with the cut or smooth surface for the direct shear test, and thus exhibited ϕ angles in excess of 50°.

Direct shear samples were prepared from the cores by hand cutting two 0.5-in-long samples and assembling the two halves together in a direct shear box for eventual shearing on the cut faces. Tests were conducted on specimens in a dry state under normal loads that varied from 0.125 to 2 tons/ft². Shearing continued until a residual value was reached.

The direct shear tests of specimens in a dry state, which are summarized in Table 2, revealed peak ϕ angles of approximately 40°. The residual ϕ angles for the direct shear tests varied between 24° and 29°. The latter results are not considered representative of normal conditions, however, due to the smoothness of the shearing surface.

Strength characteristics of the specimens in a weathered but dense dry state are presumed to be typical of subrounded to subangular sand and can be expected to have ϕ angles between 40° and 45° and, in a loose state, between 30° and 35°. No cohesion was evident. Based on the results of the above tests, representative angles of internal friction (residual) of from 30° to 33° were assumed to repre-

Table 1. Results of triaxial compression tests on sandstone specimens.

		Compressive Strength (psi)									Residual Shear-Strength Parameters	
			Residual @ σ_3									
Specimen Group	Test	Rupture @ σ_3, 5 psi	5 psi	10 psi	15 psi	20 psi	30 psi	40 psi	80 psi	ϕ (°)	C (psf)
A-1	0	1040	140	180	--	80	--	--	--	--	--
B-1	1	2920	92	128	--	200	--	324	--	51[a]	--
C-1	2	2700	180	240	--	--	--	440	606	54[a]	--
C-1	3	2600	206	226	290	366[b]	--	--	--	62[a]	--
Weathered volcanics			37	45	--	56	72	--	--	34	2100

[a]Multistage triaxial compression test; high ϕ angle due to cementation.
[b]Measures 324 with the addition of water.

Table 2. Summary of direct shear tests on sandstone specimens.

Specimen Group	Sliding Shear Stress (ton/ft²) Peak	Sliding Shear Stress (ton/ft²) Residual	Normal Stress on Horizontal Plane (ton/ft²)	ϕ Angle (°) Peak	ϕ Angle (°) Residual	Cohesion
A	1.45	1.02	2.0			
	0.73	0.53	1.0			
	0.55	0.30	0.5	39	29	0
	0.24	0.21	0.25			
	0.29	0.10	0.13			
B	1.72	1.23	2.0			
	1.65	0.92	2.0			
	0.93	0.40	1.0	42	24	0
	0.52	0.20	0.5			
	0.27	0.11	0.25			
	0.25	0.05	0.13			

Figure 13. Dowel placement.

sent the controlling shear strength of the design for permanent correction.

CORRECTION DESIGN

Based on the inclination of the apparent critical joint set (Figure 11) and the residual shear strength of the dominant material, it was estimated that the flattening necessary to achieve stability without reinforcement would involve removal of from 1.5 million to 3 million yd^3 of material. Because of the extreme expense of the magnitude of unloading under very difficult conditions, plus obvious aesthetic considerations and lack of nearby disposal sites, the basic concept of a combined unloading reinforcement scheme evolved early in the investigation. It was postulated that an unloading of the active slide material, coupled with the use of rock dowels to "knit" or pin remaining material together, could reduce necessary excavation to approximately 150 000 yd^3, or approximately 10 percent of that necessary without slope reinforcement. Improved stability with dowels was to be achieved in four ways, three of which cannot be quantified but would certainly have a significant effect. These are

1. Development of a shear key;
2. Posttensioning--As the dowels were installed immediately following excavation, it was assumed that adjustment would result in some degree of posttensioning and would, in effect, increase normal load on the assumed potential slide surface and thus increase shear strength;
3. Forcing a deeper failure plane--It was assumed that, after excavation and installation of the dowels, the potential failure surface would be forced through at least partially intact rock blocks with the resulting increase in shear strength; and
4. Improved local stability--It was assumed that the rock dowels would pin or knit individual blocks into position, thus eliminating progressive movement from local failure.

The final design, shown in section in Figure 12, involved unloading to a series of benches 20 ft in width separated by 1:1 slopes. The maximum depth of excavation into the slide mass would be approximately 40 ft, which, based on previous exploration, was judged sufficient to remove the failed material. Rock dowels were to be installed in the configuration shown in Figure 12 on 7.5-ft centers to a depth of 30 ft. Each would consist of a 3-in-diameter hole in which a No. 8 reinforcing bar would be inserted, after which the hole would be backfilled with neat cement grout. In addition, placement of steel mesh over the active portion of the bluff face, also pinned with rock dowels, was planned.

The design involved sufficient rock dowels to provide a minimum safety factor of approximately 1.2 against sliding. The dowels were designed to act as shear pins under the assumption of low active strain, which was believed to be justified as a result of improved local stability; i.e., rock doweling would pin individual blocks into position, thus eliminating progressive movement from local failure.

After careful consideration of a number of alternative corrective schemes, including a shift of the Pacific Coast Highway seaward, the decision was made in June to proceed with the design of the unload-reinforcement scheme. Of primary concern at that time was expeditious commencement of the unloading and reinforcement to ensure removal of the most dangerous portion of the slide mass prior to the onset of the rainy season, which in the Malibu area normally begins in the month of November. In slightly more than a month, Caltrans District 7 project development completed the bidding package, and the project was advertised on July 27, 1979. Bids were opened on August 17. The Novo-Rados Construction Company was low bidder at \$3 013 650 for the bid items. The key items, roadway excavation and rock dowels, were bid at \$8.78/yd^3 and \$10/lineal ft, respectively. The contract was awarded on August 29 and approved on September 12, 1979.

Because the results of laboratory testing indicated that material at the site was extremely susceptible to degradation on application of water, it was considered essential that the four benches developed during the unloading process be given special treatment to prevent degradation resulting from surface runoff.

Based on an evaluation of several possible treatments, the decision was made to use an asphalt membrane to provide a moisture barrier. This material, which has been used for lining reservoirs, pits, and ponds, has been successfully applied to both fabric and compacted native soils. Based on the results of test plots on the native material near the site, the decision was made to apply the membrane without fabric reinforcement in a spray application. Because of the extremely rough surface texture of the benches, an application rate of 0.1-0.2 gal/yd^2 was necessary to provide complete coverage.

On approval of the contract, the existing H-pile catchment system was strengthened and, at key locations, increased in height. Excavation of the slide mass and rock doweling of the underlying material began in early October. Excavated material was pushed down the face of the slope into the catchment area on a work schedule of 12 h/day, 6 days/week. Hauling from the catchment area was performed 10 h/night, 5 days/week. Disposal of most of the debris was accomplished by construction of a buttress fill along another portion of the bluff face adjacent to the Pacific Coast Highway about 5 miles east. Slide removal proceeded at an average rate of 2000 yd^3/day, and maximum production was 5000 yd^3/day. Dowel installation (see Figure 13) was accomplished by using two air-actuated pressure drills on a continuous basis. The construction operation was significantly benefited by the rainfall pattern in the immediate area. In contrast to normal conditions, rainfall was relatively light during the months of November and December, which permitted a period of almost three months of uninterrupted unloading and slope reinforcement. In January and

Figure 14. Completed project.

February, rainfall increased significantly but without adverse effect, probably due to the fact that the most unstable position of the slide mass had been removed by this time.

The project was completed and accepted on July 17, 1980. An oblique aerial view of the completed project is shown in Figure 14.

CONCLUSIONS

The following conclusions can be drawn concerning the effectiveness of techniques used to monitor and correct the Malibu landslide:

1. The slope monitoring system, consisting of survey points, SARN, and surface extensometers placed to provide advance warning of incipient mass movement, proved to be extremely effective. It permitted a partial reopening of the Pacific Coast Highway during the investigation and design phases of the slide correction.
2. The correction method that was ultimately used (slope reinforcement and partial unloading) reduced the excavation quantities required to achieve permanent stability by an estimated million cubic yards or more.

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