The Loma Prieta, California, Earthquake of October 17, 1989—Landslides

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STRONG GROUND MOTION AND GROUND FAILURE
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CONTENTS

LANDSLIDES

ANALYSIS OF EARTHQUAKE-REACTIVATED LANDSLIDES IN THE EPICENTRAL REGION, CENTRAL SANTA CRUZ MOUNTAINS, CALIFORNIA

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CONTENTS

ABSTRACT

The reactivation of large landslides during the 1989 Loma Prieta earthquake provided a unique opportunity to evaluate the seismic stability of preexisting landslides and to test the dynamic slope-stability methods currently used in geotechnical practice. For this study, we characterize two reactivated landslides, using the investigative techniques of geologic mapping, subsurface logging and sampling, and laboratory direct shear testing developed over decades of landslide investigation. We then compare actual landslide behavior during the earthquake with the results of several analytical methods, and test the sensitivity of key parameters in the calculations.

Results of both pseudostatic and cumulative-displacement analyses are less conservative than what actually occurred at the two landslide sites during the earthquake. We obtain a relatively high pseudostatic factor of safety of 1.1, using minimum shear strengths from direct shear tests and a seismic coefficient of 0.20. Cumulative-displacement calculations yield displacements within an order of magnitude of the 33 to 61 cm of displacement measured in the two landslides, generally lower than the actual field displacements.

Comparisons with actual measured landslide displacements show that the stability analyses are highly sensitive to the angle of internal friction, a parameter that is difficult to determine accurately in the laboratory. Backanalysis of the shear strength of the basal rupture surface for static stability can help define a realistic range of friction angles. The cumulative-displacement analyses are also sensitive to the acceleration-time history of ground motion at the two landslide sites. Models of earthquake-triggered landslide failure could be refined by using more rigorous physical models, but the resulting displacement calculations will not improve until earthquake ground motions at landslide sites can be better characterized.

INTRODUCTION

The combination of existing slope failures and a history of seismic ground shaking has led to the current practice of incorporating both static and seismic loading conditions into slope-stability studies in California. Although the analytical techniques and models used for static-loading conditions have been standardized, uncertainty exists among practicing engineers and geologists regarding the accuracy of the seismic analytical methods. For typical residential- and commercial-development investigations, earthquake loading is simulated by applying a seismic coefficient to a static-limit-equilibrium calculation. The value of this coefficient is selected on the basis of experience and judgment; therefore, the values chosen to be appropriate for particular sites by various workers commonly differ. State-of-the-art slope-stability-analytical methods have been modified and used by scientists at the U.S. Geological Survey to predict hillslope behavior during earthquakes (Wieczorek and others, 1985; Wilson and Keefer, 1985); however, the ability of available analytical methods to predict the occurrence and amount of displacement of earthquake-triggered landslides is not well established. Very few case studies of actual earthquake-triggered landslides have been published as of this writing (1993) (for example, Clark and others, 1979; Wilson and Keefer, 1983; Keefer, 1991; Jibson and Keefer, 1993).

The 1989 M=7.0 Loma Prieta earthquake caused discrete ground cracking in the preexisting-landslide terrane of the central Santa Cruz Mountains. Much of this ground cracking occurred along preearthquake landslide boundaries, suggesting reactivation of old bedrock landslides in the region and providing an opportunity to compare the actual seismic stability of old landslides with the results of analyses for dynamic slope stability as currently conducted in geotechnical practice.

TOPOGRAPHIC AND GEOLOGIC SETTING

CENTRAL SANTA CRUZ MOUNTAINS

The Santa Cruz Mountains are part of the northern Coast Ranges of California. This rugged mountain range forms the spine of the San Francisco peninsula, extending approximately 135 km northwestward from the Pajaro River in the south to near the community of South San Francisco in the north (fig. 1). Elevations of ridgecrests range from about 450 to 1,150 m above mean sea level.

The study area lies between about 8 and 19 km south of the community of Los Gatos at elevations of 280 to 340 m (fig. 1). The landslides chosen for study occurred in an area that underwent particularly intense ground cracking during the earthquake (fig. 2). The two sites selected for detailed study are situated 10 to 13 km northwest of the 1989 main-shock epicenter.

The study area, which is from 1.5 to 3.0 km southwest of the mapped trace (that is, probable 1906 surface rupture) of the San Andreas fault zone, is underlain by tightly folded Eocene, Oligocene, and Miocene sedimentary rocks (fig. 2). This Tertiary bedrock section consists of relatively soft, poorly indurated, marine clastic sedimentary rocks, about 7,600 m thick (Clark, 1981). In the study area, the rocks have been strongly folded and locally overturned into a series of northwest-southeast-trending anticlines and synclines. Specific Tertiary bedrock formations underlying the study areas are the Butano Sandstone, Vaqueros Sandstone, and San Lorenzo Formation.

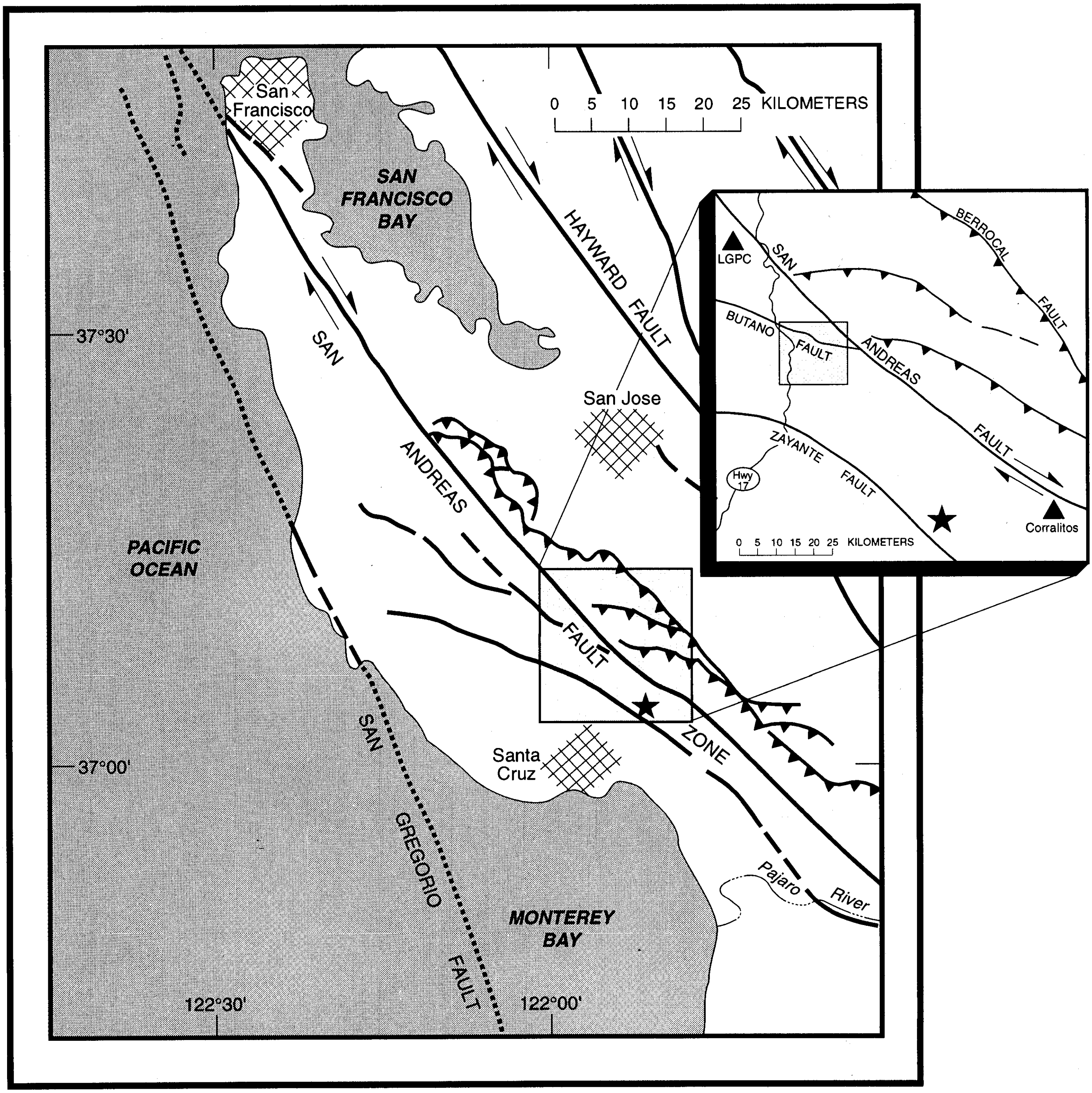
PREEXISTING "ANCIENT" LANDSLIDES

The study area encompasses a widespread landslide terrane. In general, the geomorphology of this terrane is characterized by "stepped" topography, including very steep, curved scarps, flat-lying benches, and steep-walled, deeply incised creek canyons. Typically, several sequences of scarps with intervening flat-lying benches form the slope face between the ridgetops and creek canyons. Geomorphic features range in appearance from relatively fresh to subdued, strongly suggesting periodic movement of individual landslides within a large landslide complex. The large areal extent of these landslides and multiple scarps suggest large cumulative displacements of these features.

"Ancient landslides" are defined here to be landslides that originated thousands to tens of thousands of years ago, although the most recent movement may have been only years or decades ago. Indeed, geomorphic and subsurface geologic data suggest periodic and episodic reactivation of these landslides. Although the exact date of landslide initiation in the study area is unknown, other landslide complexes in California are associated with Pleistocene glacial and climatic changes during the past 100,000 years (Stout, 1969).

The landslide masses are composed of weak earth materials derived from fractured, folded, and faulted Tertiary sedimentary rocks that have been subjected to periods of intense rainfall, ground-water fluctuations, and strong earthquake shaking. The sedimentary rocks consist primarily of fine-grained, thin-bedded, highly fractured shale, claystone, and siltstone that contain numerous shears, fractures, and other discontinuities. The resulting permeability of the near-surface materials enables them to collect, transmit, and store water from various sources (for example, rainfall, runoff, and lateral ground-water flow).

Figure 1.—San Francisco Bay region, Calif., showing location of study area (small shaded box), major faults (dashed where approximately located, dotted where concealed; half-arrows indicate direction of relative movement) and thrust faults (sawteeth on upper plate), and epicenter of 1989 Loma Prieta earthquake (star). Triangles, strong-motion-recording stations used in this study; LGPC, Los Gatos Presentation Center station.



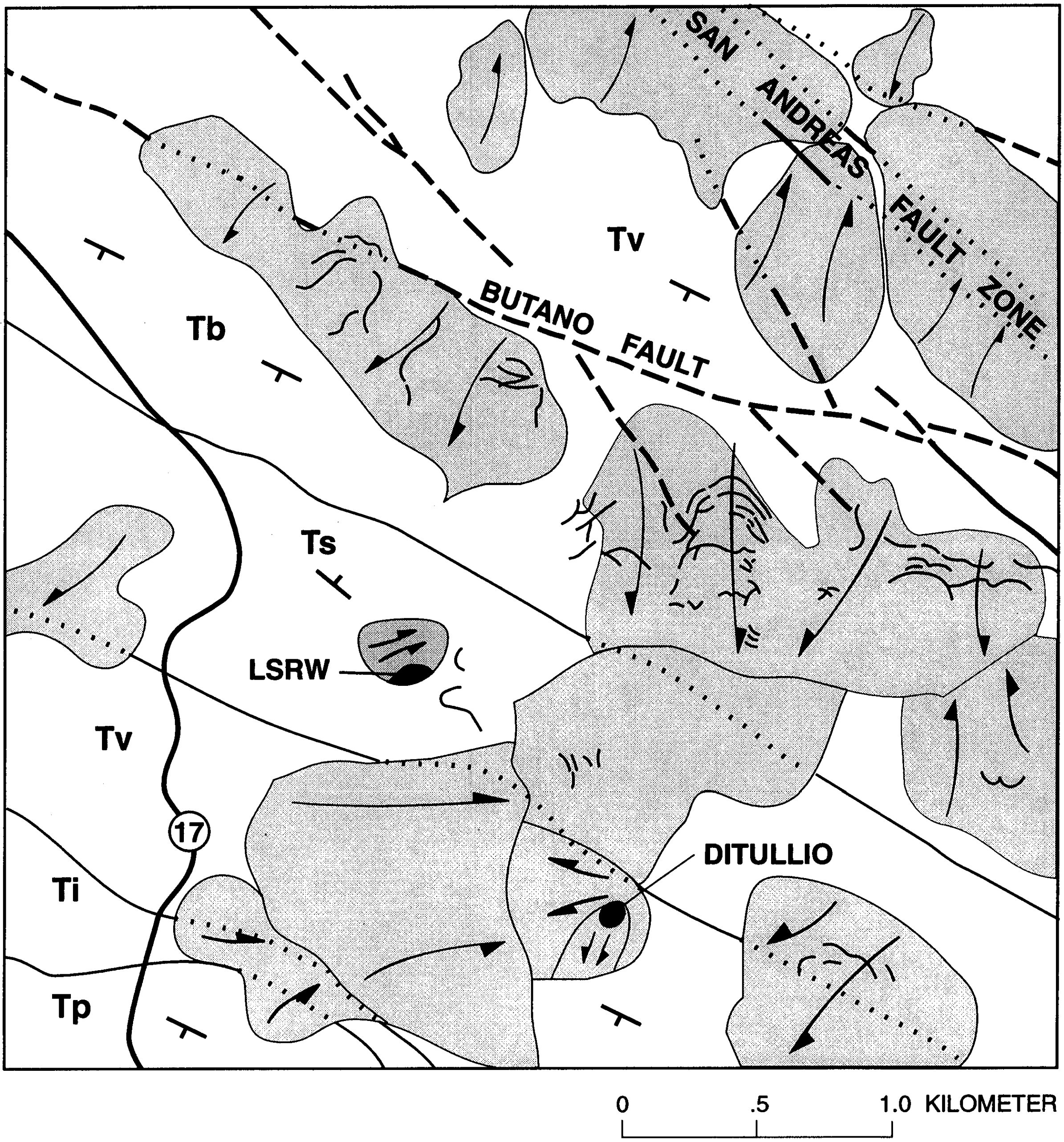
EXPLANATION

Fault- Dashed where approximately located; dotted where concealed. Half arrows indicate direction of relative movement.

Thrust fault- Sawteeth on upper plate

▲ **Strong motion recording station**- **LGPC**, Los Gatos Presentation Center

★ **1989 Loma Prieta earthquake epicenter**



EXPLANATION

TERTIARY BEDROCK UNITS

- **Tp** Purisima Formation
- **Tl** Lambert Shale
- **Tv** Vaqueros Sandstone
- **Ts** San Lorenzo Formation
- **Tb** Butano Sandstone

Preexisting landslide- Half arrows indicate direction of downslope movement.

Reactivated-landslide study site

1989 coseismic ground fissures

Contact- Dotted where concealed

Strike and dip of bedding

Fault- Approximately located; dotted where concealed.

LANDSLIDE CHARACTERIZATION

SITE SELECTION

A general reconnaissance of more than 50 large earthquake triggered or reactivated landslides was performed after the 1989 Loma Prieta earthquake. Of the landslide sites, approximately 20 were selected for more rigorous inspection. Early inspections focused on identifying landslides that met the following criteria: (1) deep-seated failures that involved bedrock materials, (2) a well-defined pattern of ground cracks and surface displacements indicating probable reactivation of the basal rupture surface, (3) location within preexisting landslides or landslide complexes, and (4) terrain amenable to detailed surface and subsurface investigation and not so steep or heavily vegetated as to prevent drill-rig access. We also preferred sites where ground water was anticipated to be deeper than several meters, so that a geologist could safely enter and log large-diameter boreholes without the need for dewatering. Shallow ground water prevented full characterization of landslide parameters at 4 of the 20 selected sites.

Many of the earthquake-reactivated landslides appear to have been only partially mobilized, without shear failure along the entire basal rupture surface. The pattern of surface deformation at several sites, however, indicated that some landslides had been fully mobilized by the earthquake. Two of these landslides, here referred to as the Lower Schultheis Road West and Ditullio landslides, were characterized by a combination of field mapping and subsurface exploration. Large-diameter boreholes were excavated and logged by a geologist descending into the borehole to directly observe and sample landslide rupture-surface materials. Geologic and geotechnical conditions, deformational features, and the investigative procedures used at each site were described in detail by Cole and others (1991).

SUMMARY OF SUBSURFACE CONDITIONS

The preexisting-landslide masses are composed of three distinguishable rock and soil materials: (1) regolith, (2) sparsely to highly fractured bedrock units, and (3) clay shear and gouge material, including landslide rupture surfaces.

Figure 2.—Geologic map of study area (see fig. 1 for location), showing locations of Lower Schultheis Road West (LSRW) and Ditullio landslides discussed in text. Geology from Clark and others (1989) and McLaughlin and others (1991); areas of preexisting landslides (shaded) from Cooper-Clark and Associates (1975) and this study; coseismic ground fissures from Spittler and Harp (1990).

1. The regolith consists of oxidized and fractured rock and soil derived from the underlying bedrock that have been subjected to seismic shaking, cyclic changes in ground-water levels, and other weathering processes. Regolith at the landslide sites contains angular, weathered, gravel- to boulder-size sandstone and siltstone fragments in a matrix of clayey silt to silty sand. The upper approximately 4 m of regolith typically is a plastic silty clay.
2. Displaced sedimentary bedrock sequences of the San Lorenzo Formation are present at depth in the Lower Schultheis Road West and Ditullio landslides; the bedrock is highly fractured and oxidized above major shear surfaces at 7.3- and 18-m depth, respectively (figs. 3, 4). Below these shear surfaces, the bedrock is less fractured, denser, and relatively unoxidized. Interbedded sandstone and siltstone are present at the Lower Schultheis Road West landslide, and massive sandstone at the Ditullio landslide.
3. Thin seams of clayey gouge occur in the regolith and fractured bedrock at several landslide sites explored in the study area. The gouge is generally a wet silty clay with a soft to firm consistency and medium to high plasticity. The gouge seams are commonly from about 1 to 10 cm thick. Polished surfaces were observed in many of these seams. At the Lower Schultheis Road West landslide, several continuous and discontinuous gouge zones are present at 4- and 7-m depth. Thicker gouge zones are present within, and at the base of, the regolith. A relatively thin seam of gouge forms the contact between highly fractured rock and underlying unfractured rock. At the Ditullio landslide, a 2- to 5-cm-thick gouge seam separates oxidized, fractured rock from underlying unfractured rock at about 18-m depth; this gouge seam is associated with a zone of crushed siltstone and sandstone, as much as about 80 cm thick (fig. 3).

The results of subsurface exploration indicate that the reactivated landslides occurred within preexisting landslide masses that are characterized by relatively coherent, rather than disrupted, landslide materials. Complex, shallow perched ground water and both shallow and deep preexisting rupture surfaces were penetrated. The shallow shear surfaces typically were observed at the base of the regolith at 5- to 15-m depth, and deeper shears were penetrated within the underlying fractured sedimentary bedrock. The primary geologic controls on these shear surfaces appear to be the regolith-bedrock contact and adversely oriented bedding planes and fractures in the bedrock.

REGOLITH DETACHMENT

On the basis of our downhole observations, the contact between overlying, oxidized regolith and underlying, unoxidized bedrock was commonly a well-developed shear contact. The base of the overlying regolith is generally at 5- to 15-m depth. The underlying bedrock, though part of a larger, ancient landslide mass, typically is unoxidized (or only locally oxidized) and closely fractured. The observed shear contact separating the two units is a saturated, clay-rich, soft, plastic gouge ranging from approximately 2 to 5 cm in thickness. At one of the landslides we explored but do not analyze here, a sheared water well

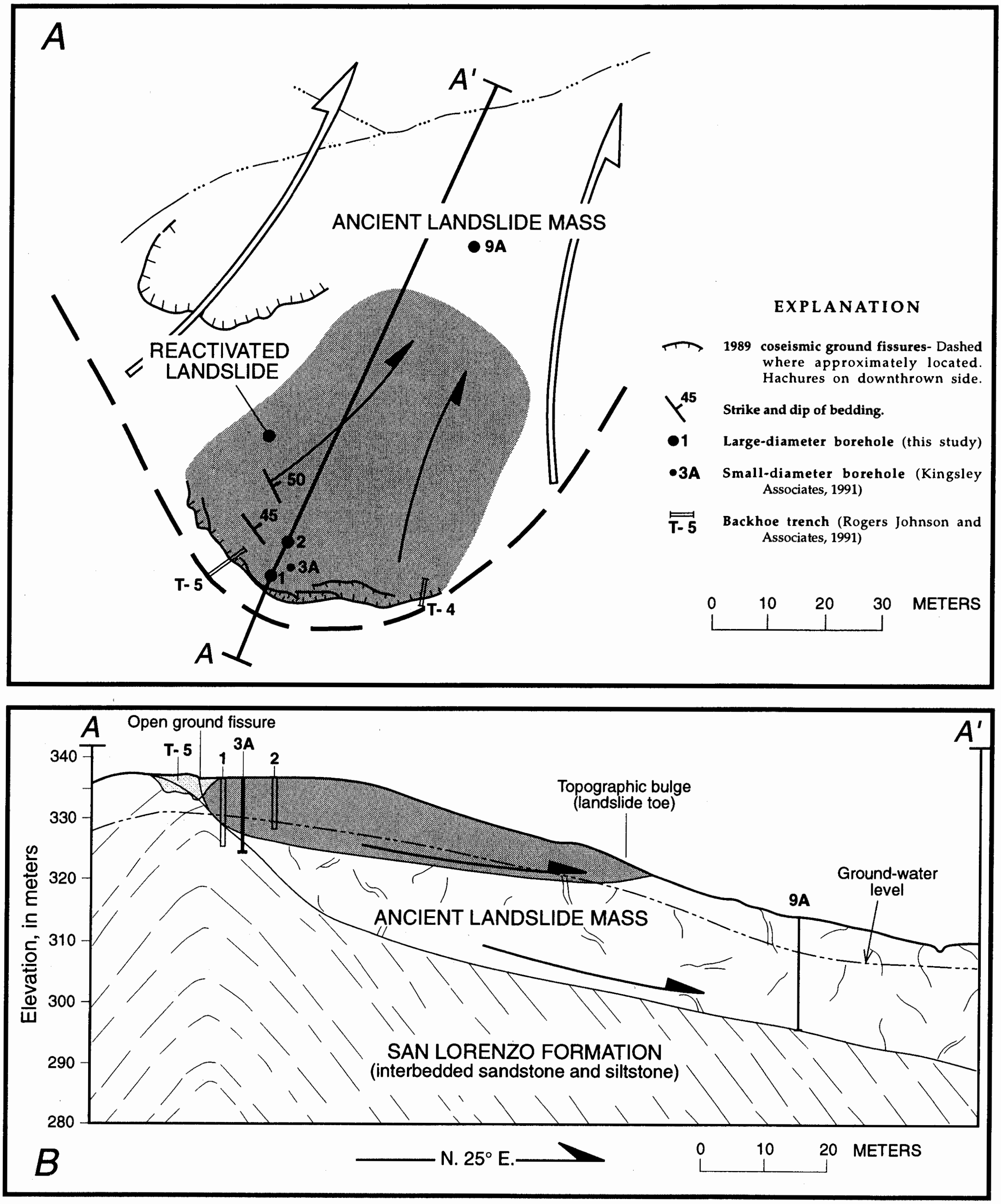


Figure 3.—Simplified geologic map (*A*) and cross section (*B*) of Lower Schultheis Road West landslide (see fig. 2 for location). Half-arrows indicate direction of downslope movement.

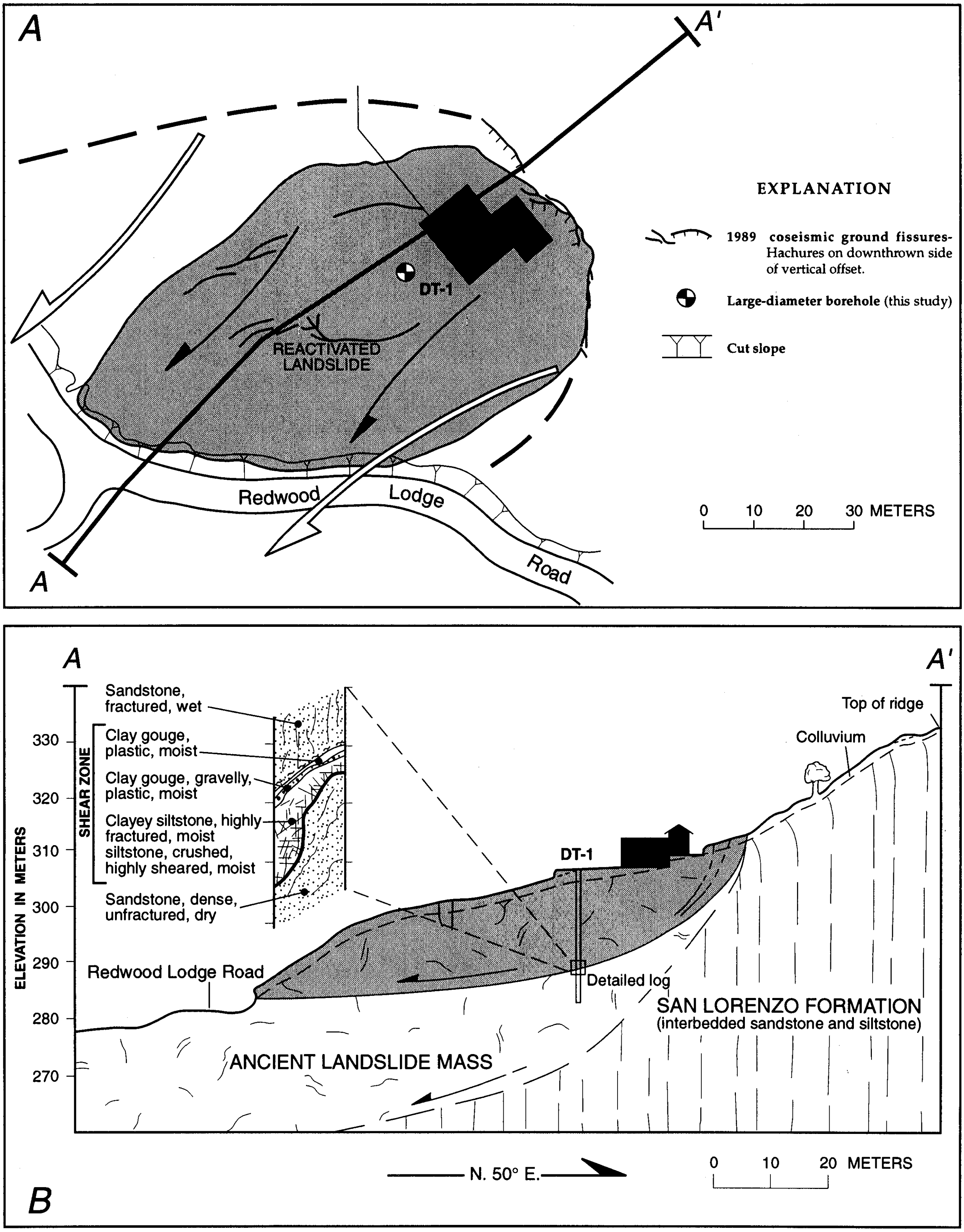


Figure 4.—Simplified geologic map (*A*) and cross section (*B*) of Ditullio landslide (see fig. 2 for location). Half-arrows indicate direction of downslope movement.

provided a piercing point on the reactivated rupture surface. A large-diameter borehole adjacent to the well revealed that this rupture surface coincided with the regolith-bedrock contact.

Although the geomorphology of the area is best explained by the presence of deep-seated landsliding, logging of large-diameter borings revealed that shallow landslide debris was much more disrupted than the deeper landslide material. The degree of development and fresh appearance of the shears separating the regolith from the underlying rock suggests that the Loma Prieta landsliding event mobilized these shears and that they had been mobilized frequently in the past as well.

STRUCTURAL CONTROL ON DEEPER FAILURES

Bedding-plane shears were observed at the Lower Schultheis Road West landslide and in trenches excavated in the headscarps of other landslides in the study area (Cotton and others, 1991; Keefer, 1991). An interpretation that old landslides failed repeatedly along a preexisting bedding-plane shear is supported by the difference in physical characteristics between the overlying landslide debris and underlying bedrock at the Lower Schultheis Road West landslide (fig. 4). Regional geologic data suggest that the upslope parts of several other landslides may be underlain by bedding planes dipping steeply downslope.

For the landslides to have fully failed in the past, the basal surfaces must eventually break out to the slope surface, whereas the bedding planes likely remain steeply dipping. Therefore, rupture surfaces that coincide with bedding planes in the upslope (headward) parts of a landslide mass dip less steeply than these bedding planes beneath much of the landslide mass. The presence of steeply dipping bedding planes could help to explain how many slope failures from the 1989 Loma Prieta earthquake were restricted to only the steeply dipping, upslope components of the landslides. Many of these landslides, however, do not appear to have been influenced by dip-slope conditions (D.K. Keefer, oral commun., 1992); thus, the limited movement of the landslides must be explained by other mechanisms.

Subsurface geologic evidence does not demonstrate conclusively whether the landslides were caused originally by seismic activity or by excessive water during wetter climatic conditions, or by some combination of these two mechanisms. Subsurface exposures, however, confirm the presence of multiple rupture surfaces of quite different geometry and stratigraphic position, arguing for the existence for multiple rupturing events since the landslides first formed. The deep-seated rupture surfaces that incorporated relatively fresh bedrock into the landslide mass also involved a much greater volume of the slope than the shallow ruptures observed at the regolith-bedrock contact. Such massive failures must have been generated by an event quite different in magnitude from the 1989 Loma Prieta earthquake. The largest measured displacements in this earthquake were less than 3 m (Keefer, 1991). Thus, it apparently reactivated only shallow segments of relatively small areal extent within deep-seated, large preexisting landslide masses.

KEY PARAMETERS IN SLOPE-STABILITY ANALYSIS

The key parameters in slope-stability analysis include landslide geometry, generally analyzed as a representative cross section; ground-water levels or pore-water pressures acting on the landslide, generally considered as a phreatic surface; the intensity of earthquake ground motion at the site, typically inputted in the form of a single seismic coefficient or an appropriate acceleration-time history; the unit weights of the landslide materials; and an average shear strength along the basal rupture surface.

LANDSLIDE GEOMETRY AND GROUND-WATER CONDITIONS

The geometry and ground-water parameters were determined directly from field measurements. Geologic cross sections were carefully constructed through the landslides on the basis of field measurements. Establishing that a landslide had actually moved downslope, in contrast to simply forming a scarp by settlement of loose graben debris, required careful examination but could be verified by the presence of sheared flanks or compressional-toe areas. Determining landslide depth is complicated by the fact that multiple shear surfaces were observed in boreholes penetrating the preexisting-landslide masses. In the absence of well-defined subsurface piercing points, the selection of which of the multiple shear surfaces was reactivated involves careful correlation between borehole logs and surface deformational features, as well as reasonable assumptions regarding landslide behavior. Only one well-developed shear surface was penetrated in the Ditullio landslide, thus making the determination of landslide depth straightforward. The selection of the activated Lower Schultheis Road West landslide rupture surface was based on the fresh appearance and continuity of the regolith-bedrock contact, correlation with the headscarp fissures and downslope zone of compression, and observations from other similar types of landslides in the study area.

The ground-water levels shown on the cross sections (figs. 3, 4) were those penetrated during subsurface exploration. Because the 1989 Loma Prieta earthquake occurred near the beginning of the 1989–90 rainy season during a period of prolonged drought, the conditions in boreholes are believed to represent ground-water conditions immediately before the earthquake. We realize that changes in water-well levels and surface streamflow after the earthquake suggest that trapped ground water may have been freed by fracturing during the earthquake and that water levels may have dropped significantly (Rojstaczer and Wolf, 1992). However, perched water in the analyzed landslides is unlikely to exit through new fractures extending into the landslide debris. Furthermore, we know of no reports or evidence of ground water flowing to the surface from within these small landslide masses after the earthquake. Nonetheless, we performed sensitivity analyses to test the effects of changing ground-water levels on the stability analyses (see subsection below entitled "Sensitivity Analysis").

EARTHQUAKE GROUND MOTIONS

Earthquake ground motions can be entered into the displacement calculations from published maps (K.L. Lee, unpub. data, 1977; Makdisi and Seed, 1977; Hynes-Griffin and Franklin, 1984; Lin and Whitman, 1986), as well as directly from acceleration-time histories recorded on strong-motion instruments located near the study area. We used data from the Corralitos station, part of the California Division of Mines and Geology's California Strong Motion Instrumentation Program (CSMIP), and the Los Gatos Presentation Center station (LGPC, fig. 1), operated by the Charles F. Richter Seismological Laboratory of the University of California, Santa Cruz. The Corralitos and LGPC stations are approximately 7 and 19 km, respectively, from the epicenter (fig. 1).

The intensity of ground motion at a particular site is influenced by several factors, including earthquake magnitude, epicentral distance, local geologic structure, source and rupture mechanism of the earthquake, wave-interference effects, geotechnical site conditions, and topographic amplification. The selection of appropriate acceleration-time histories for seismic stability is generally problematic because most landslide sites are at great distances from actual recording stations and the conditions at these sites generally differ from those at the recording stations. The large magnitude and complex rupture mechanism of the 1989 Loma Prieta earthquake, in combination with extremely irregular topographic conditions, indicate that ground motion in the study area may have varied considerably. Thus, selection of appropriate ground-motion parameters at the landslide sites involved an assessment of the site conditions at the two closest recording stations (fig. 1). Several studies indicate that the earthquake had a strong azimuthal effect on ground motion (Beroza, 1991; Steidl and others, 1991; Wald and others, 1991). The earthquake ruptured bilaterally, and so the region northwest of the epicenter (including the LGPC station and both landslide sites) was influenced by northwest-propagating rupture, whereas the region southeast of the epicenter (including the Corralitos station) was more affected by southeast-propagating rupture (table 1). Furthermore, rupture was predominately reverse slip to the northwest of the hypocenter and predominately strike slip to the southeast. According to Wald and others, the bilateral rupture resulted in a larger overall stress drop and correspondingly higher ground motion to the northwest of the epicenter, and lower ground motion directly updip of the hypocenter near the Corralitos station. Campbell (1991) suggested that the northwest-trending geologic structure may explain the higher ground motion and lower attenuation rate northwest of the epicenter.

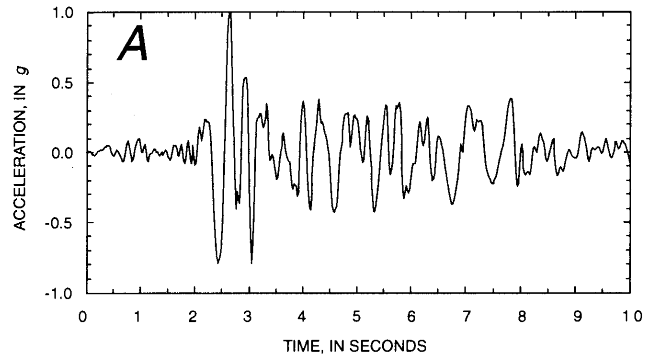
The Corralitos station (fig. 1) recorded a peak horizontal ground acceleration, k_{max}, of 0.63 g (California Division of Mines and Geology, 1990), whereas the LGPC station appears to have recorded a k_{max} value of greater than 1.00 g (off scale). Because the recorder was not firmly anchored to the floor and an apparent baseline offset occurred during the strong ground shaking, seismologists have cautioned that the data from the LGPC station may not be a reliable indicator of the actual ground motion at this site (Karen McNally, oral commun., 1991). For the purposes of our analysis, however, the baseline correction for this offset (approx 0.03 g north-south), was considered insignificant relative to the k_{max} value. The amplitude of long-period waves on the corrected acceleration-time history for the LGPC station is similar to the modified record from the Corralitos station scaled to 1.00 g (fig. 5).

The site conditions at the LGPC station (fig. 1) are thought to more closely resemble those at the landslide sites than do those at the Corralitos station because (1) the LGPC station and the landslide site are at similar distances from the San Andreas fault and the Loma Prieta epicenter, (2) topographic conditions at the LGPC station and the landslide sites are similar in terms of slope position and elevation, and (3) the LGPC station is located along the same azimuth from the epicenter (320°) as the two landslide sites. In the absence of recorded data in the immediate vicinity of the landslide sites and because of very high ground motions in the surrounding region, the acceleration-time history of the LGPC station was selected initially to represent "bedrock" ground motions at the landslide sites during the earthquake. We also used the Corralitos station record, both unscaled (k_{max}=0.63 g) and scaled (k_{max}=1.00 g), to test the sensitivity of the calculations to ground motion (see subsection below entitled "Sensitivity Analysis").

Table 1.—*Distance of two strong-motion stations used in this study from earthquake and landslide features*

[The Lower Schultheis Road West and Ditullio landslides are about 12 and 11 km, respectively, northwest (az 320°) of the epicenter of the 1989 Loma Prieta earthquake. Both landslides are about 2 km southwest of the Loma Prieta rupture zone, as measured from the San Andreas fault zone]

	Distance (km) to:				
Strong-Motion station (fig. 1)	Loma Prieta epicenter (azimuth)	San Andreas fault	Loma Prieta rupture zone	Lower Schultheis Road West landslide (elev., 335 m)	Ditullio landslide (elev., 351 m)
Corralitos (elev., 320 m).	7.0 (080°)	0.3	0.3	16.4	15.9
Los Gatos Presentation Center (elev., 366 m).	19.0 (320°)	1.4	1.5	7.0	8.2



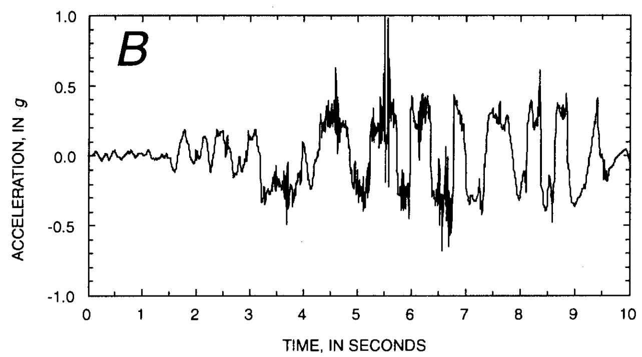


Figure 5.—Corrected north-south component of motion on strong-motion records of October 17, 1989, main shock from California Division of Mines and Geology station at Corralitos (*A*) and University of California, Santa Cruz, station at Los Gatos Presentation Center (*B*) (see fig. 1 for locations). Record in figure 5*A* is scaled to 1.0 *g*.

UNIT WEIGHT AND SHEAR STRENGTH

Moisture content and density were determined in the laboratory on selected undisturbed samples of the landslide mass by measuring the weight and volume of material collected and sealed in brass tubes. For the Lower Schultheis Road West landslide, we identified two geotechnical units in the landslide mass. The soil (upper unit) has a moist unit weight of 2.06 g/cm^3 and a saturated unit weight of 2.16 g/cm^3; the regolith (lower unit) has a moist unit weight of 1.96 g/cm^3 and a saturated unit weight of 2.08 g/cm^3. For the Ditullio landslide, only one geotechnical unit was identified, with a moist unit weight of 1.92 g/cm^3.

Shear strengths were determined initially by laboratory measurements on samples collected from subsurface rupture zones. Backcalculations were subsequently performed under static-loading conditions for comparison with the laboratory results. Collecting representative and oriented samples of landslide rupture surfaces is a difficult task. To obtain the best possible samples, we collected tube samples of the landslide rupture surfaces from large-diameter boreholes. At each sample location, a geologist hand-excavated a horizontal shelf into the landslide material several centimeters above and parallel to the shear surface. Tube samples were pushed and driven through the landslide debris, through the shear surface, and into the underlying materials. The samples were sealed and transported to our laboratories for various direct shear tests (consolidated drained and consolidated undrained, peak, and residual) on "undisturbed" and remolded samples.

Because of the rapid failure of the landslides, we chose consolidated-undrained ("quick") direct shear tests, rather than triaxial strength testing, to represent the loading conditions imposed by the earthquake. Although some drainage can occur during direct shear testing, the permeability of the gouge material from these landslides is very low, and the sample does not have sufficient time to drain during the relatively rapid test. Triaxial tests can measure pore pressures, but shear surfaces must be oriented at an angle to the sample axis to avoid stresses perpendicular to the shear surface. In addition, direct shear tests can be performed on relatively short samples, whereas triaxial samples must be about twice as long as the diameter. Thus, the triaxial procedure requires a longer sample, which involves more disturbance during sampling and is difficult where thin, soft gouge material lies between hard, brittle rock. We favor direct shear tests over triaxial tests for these reasons and because they offer the best chance for shearing the actual rupture surface in a similar mode of failure to that in the landslide.

The residual shear strengths measured in direct shear tests are approximately 70 to 75 percent of the peak values. This result approximates the reduction to 80 percent of peak shear strength estimated by Makdisi and Seed (1977) and recommended by Hynes-Griffin and Franklin (1984), owing to cyclic-loading effects. Therefore, we used the residual shear strengths to characterize the cyclic strength of the basal rupture surfaces. Two measures of residual shear strength, lower-bound and average or "best fit" curves, were drawn by linear regression through the data points (fig. 6). The shear-strength parameters, in terms of angle of internal friction (ø) and cohesion (*c*), that describe the lower-bound and best-fit values are ø=25°, *c*=0 kg/cm^2 and ø=24°, *c*=0.22 kg/cm^2, respectively.

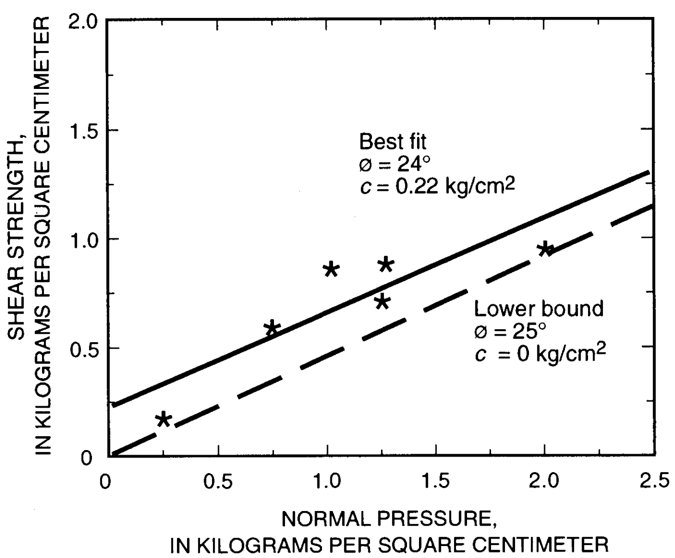


Figure 6.—Residual shear strength versus normal pressure in laboratory consolidated-undrained tests on landslide rupture surface material.

METHODS OF ANALYSIS

PSEUDOSTATIC METHOD

Pseudostatic analysis is a standard method of assessing seismic slope stability in the geotechnical industry. This method involves analyzing landslides by using a traditional method of limit-equilibrium-stability analysis, such as Spencer's or Bishop's method (Chowdhury, 1978; Huang, 1983). In addition to the static forces acting on the landslide body, a static force proportional to the weight of the landslide mass is then added as a permanent horizontal force to simulate the dynamic force of the earthquake. The magnitude of this force is the product of the weight of the landslide body and a seismic yield coefficient, *k*.

The *k* value significantly affects the results of the pseudostatic analysis, yet traditionally it has been arbitrarily chosen on the basis of experience and judgment. Lambe and Whitman (1969) stated that *k* values of 0.1 to 0.2 are commonly assumed. K.L. Lee (unpub. data, 1977) indicated that the *k* value may fall in a somewhat broader range of 0.05–0.25. Chowdhury (1978) suggested that *k* values of 0.10 to 0.15 are typically used in the United States, whereas Huang (1983) suggested a *k* value of 0.27 for design in western California. A *k* value of 0.15 is currently required by the California Division of Safety of Dams for analysis of earthfill dams. In California, *k* values of more than 0.20 are rarely used in practical design for residential and commercial sites. The diversity in chosen *k* values reflects the uncertainty that engineers have with regard to the pseudostatic approach and the selection of design-level earthquakes.

CUMULATIVE-DISPLACEMENT METHOD

As recognized by many previous workers, the pseudostatic method of analysis has some significant limitations. Newmark (1965) noted that the transitory vibrations of earthquake ground motions could cause the factor of safety (FS) of a slope to temporarily drop below 1 several times during a seismic event. An accurate pseudostatic analysis of this situation would indicate failure. However, if the drop in the FS value is so transient that the induced deformations are small and the slope is essentially unchanged, then the term "failure" may not accurately describe the actual condition of the slope.

A method of measuring incremental displacements of the slope during an earthquake was developed by Newmark (1965) as an alternative to the pseudostatic approach for earth embankments. Commonly referred to as "Newmark analysis," "cumulative-displacement analysis," "permanent-deformation analysis," and "sliding-block analysis," this method sums the downslope movements of the landslide during each cycle of high acceleration to compute a cumulative displacement. Thus, this method evaluates performance rather than stability. The magnitude of the computed displacement can be used to assess the degree of damage. For example, if the displacements are several meters, deformation of the slope probably will be heavy; conversely, if the computed displacements are less than a few centimeters, deformation of the slope will be slight. This method requires a complete acceleration-time history as input, as well as a value for the yield acceleration (k_y) of the landslide, defined as the ground acceleration required to bring the FS value to 1 in a pseudostatic analysis.

A basic assumption of the cumulative-displacement method is that the landslide mass behaves as a rigid body and can be modeled as a friction block on an inclined plane. Under static conditions, the block rests on the inclined plane without sliding. As the inclined plane is shaken, the block moves with the plane (without slipping) until the acceleration of the plane exceeds the k_y value of the friction surface, at which point the block begins to slide. When the acceleration of the plane decreases below the k_y value, the block continues to move because of momentum but decelerates because of friction until movement stops. The amount of displacement of the block is a function of the elapsed time while the k_y value was exceeded and the magnitude by which the k_y value was exceeded by the shaking. The displacement of the block during an acceleration pulse is then calculated by integrating the area under the velocity curve. Additional displacements caused by subsequent pulses of shaking above the k_y value are summed to give the cumulative displacement of the block.

For design problems, investigators wishing to estimate landslide displacements commonly use published curves or graphs of the integrated displacements determined for historical earthquakes of different magnitudes by previous workers, such as Makdisi and Seed (1977) or Hynes-Griffin and Franklin (1984). These curves were constructed by double-integrating acceleration-time histories as a function of the k_y value. Makdisi and Seed integrated the average acceleration-time histories calculated for embankments of varying heights that had been subjected to a range of earthquake-induced base accelerations. Hynes-Griffin and Franklin integrated the horizontal components of 348 actual and 6 synthetic earthquake acceleration-time histories. The investigator typically enters a curve with the k_y value of the landslide and the expected peak ground acceleration, and arrives at an estimate of the landslide displacement. Although these curves are relatively simple to use, they are by no means consistent with each other. A comparison of several widely used curves by Jibson (1993) demonstrated that the displacements estimated from different curves can vary by a factor of as much as 100.

Another method of using cumulative displacement during design to predict landslide movement is to select an actual earthquake acceleration-time history that could affect the site. The part of the acceleration record above the k_y value is then double-integrated to arrive at a displacement estimate. For this study, we chose to use the largest-amplitude seismic record in the study area and a convenient computer program, DISPLMT (Houston and others, 1987), to carry out the computations. The effects of using various seismic records are discussed in the subsection below entitled "Sensitivity Analysis."

RESULTS

In our analysis of the Lower Schultheis Road West and Ditullio landslides, we approached the seismic-stability problem in two different ways. First, we calculated the pseudostatic stability and predicted displacements, using the parameters measured during our site investigation. For a typical geotechnical site investigation, either the best-fit or lowest shear strength determined from laboratory measurements would be used. This approach actually represented a hindcast, because accelerograph records from the 1989 Loma Prieta earthquake (unavailable for a true prediction before the earthquake) were used rather than a record already on file before the earthquake. Second, we tested the sensitivity of the displacement results to reasonable variations in the shear-strength parameters. We analyzed the FS and k_y values by Spencer's method with the computer program PCSTABL developed at Purdue University (Carpenter, 1985).

PSEUDOSTATIC ANALYSIS

The pseudostatic FS and k_y values for the Lower Schultheis Road and Ditullio landslides, calculated from the laboratory-determined residual shear strengths, are listed in table 2.

FS values of about 1.1 are calculated for both sites if the lower-bound residual shear strengths and a relatively high k_y value (that is, 0.20 *g*) are used. In current geotechnical practice, an FS value of 1.1 is generally considered acceptable for the seismic stability of engineered slopes. Because the shear strengths used represent residual, rather than peak, values, some geotechnical practitioners would consider this analysis to be overly conservative. In practice, the best-fit, rather than the lower-bound, strengths might have been selected. These best-fit strengths would have resulted in computed FS values of 1.2 to 1.4, indicating considerable stability of both slopes under dynamic loading. Thus, our results demonstrate that, using the laboratory-determined residual shear strengths, the pseudostatic

Table 2.—*Pseudostatic factors of safety (FS) and yield coefficients* (k_y) *in the Lower Schultheis Road West and Ditullio landslides, calculated from laboratory-determined residual shear strengths*

	FS for k_y=0.20	k_y (g) for FS=1.0
Lower Schultheis Road West landslide		
Lower-bound shear strength	1.1	0.23
Best-fit shear strength	1.4	.35
Ditullio landslide		
Lower-bound shear strength	1.1	0.22
Best-fit shear strength	1.2	.30

analysis would not have predicted the slope failures even if the lower-bound values were used. However, the FS value drops below 1.0 when a k_y value of about 0.25 g is used.

CUMULATIVE-DISPLACEMENT ANALYSIS

To estimate the displacements, we first used the curves of Makdisi and Seed (1977) and Hynes-Griffin and Franklin (1984), and then double-integrated a record from the 1989 Loma Prieta earthquake. The curves were entered with the k_y values calculated by the pseudostatic method, using k_{max}=1.0 g. Makdisi and Seed presented upper- and lower-bound curves for M=6.5, 7.5, and 8.5 earthquakes. Upper- and lower-bound curves were interpolated for a theoretical M=7.0 event. The k_y values listed in table 3 are derived from an interpreted geometric-mean curve drawn between the upper- and lower-bound M=7.0 curves. Hynes-Griffin and Franklin presented mean, mean$\pm 1\sigma$, and upper-bound curves; the results listed in table 3 are from their mean curve.

Table 3 shows that the curves of Makdisi and Seed (1977) predict displacements in reasonable agreement with those measured after the earthquake, using the lower-bound residual shear strengths to characterize the landslide rupture surface. If the best-fit residual shear strengths are used, the predicted displacements are too low but are still within the same order of magnitude as those observed. The displacements calculated using the mean curve of Hynes-Griffin and Franklin (1984) underpredict the displacements by almost an order of magnitude, using the lower-bound residual shear strengths.

The results of the double-integrated acceleration-time history for the LGPC station, using the computer program DISPLMT of Houston and others (1987), are also listed in table 3. The k_y value can be inputted as a constant, as a function of time, or as a function of displacement. Uphill movements of the landslide block are accounted for by assuming a relation between the upslope and downslope k_y values and the static FS value. Most accelerograms are asymmetric, and the program accounts for this feature by calculating displacements twice, assuming that each side acts downslope, and averaging the results. For simplicity, a constant k_y value was used.

The cumulative-displacement method predicts landslide displacements within an order of magnitude of the measured displacements, using the curves of Makdisi and Seed (1977). The curves of Hynes-Griffin and Franklin (1984) and the integrated record from the LGPC station (fig. 1) both predict displacements within the range of those measured, but only when the lower-bound residual shear strengths are used. When the best-fit residual shear strengths are used (corresponding to k_y=0.30–0.35 g), the predicted displacements are an order of magnitude less than those observed and an order of magnitude less than those calculated using k_y=0.22–0.23 g. This result indicates that the displacements calculated by integrating the record from the LGPC station are highly sensitive to the k_y value and thus highly sensitive to the shear strengths chosen for the landslide rupture surface. The results of cumulative-displacement calculations for the two reactivated landslides, using five different analytical methods, are listed in table 4. The method of K.L. Lee (unpub. data, 1977) produces the most conservative results. However, the median displacements from all methods range from 10 to 100 cm for k_{max}=1.00 g, which is as close as we should expect from the level of uncertainty in the k_y value and shear-strength parameters, and the sensitivity of the predicted displacements to these measurements.

SENSITIVITY ANALYSIS

The sensitivity of the calculated displacements to variations in both shear strength and acceleration-time history is apparent from various analyses. The dependence on shear strength is of great concern because of the difficulty in obtaining well-constrained values. The shear strength of the rupture-surface materials can be calculated in two somewhat-independent ways: (1) by sampling and testing actual gouge materials in the laboratory and (2) by backanalyzing the landslide to determine the shear strengths required for failure.

We know that the present geometry of the landslide is stable under current ground-water conditions, which provides a minimum shear strength for the gouge. If we assume that the landslide would be unstable when ground water filled it to the surface, then a backanalysis of that saturated condition would provide a maximum shear strength for the gouge. An argument can be made that this

Table 3.—*Seismic yield coefficients* (k_y) *and displacements in the Lower Schultheis Road West and Ditullio landslides, calculated from laboratory-determined residual shear strengths*

[All values assume a peak horizontal ground acceleration of 1.0 g. Shear-strengths: ø=25°, c=0 kg/cm^2 for lower-bound values and ø=24°, c=0.22 kg/cm^2 for best-fit values, where ø is the angle of internal friction and c is the cohesion. Mean 1, geometric mean from Makdisi and Seed (1977); mean 2, mean from Hynes-Griffin and Franklin (1984), with displacements below 10 cm not plotted]

	k_y (g) for FS=1.0	Displacement (cm)			
		Mean 1	Mean 2	Program DISPLMT	Actual
Lower Schultheis Road West landslide					
Lower-bound shear strength	0.23	36	10	23	61
Best-fit shear strength	.35	17	<10	1	61
Ditullio landslide					
Lower-bound shear strength	0.22	38	10	29	33-57 (avg 45)
Best-fit shear strength	.30	24	<10	3	33-57 (avg 45)

assumption is reasonable on the basis of the widespread presence of shallow slope failures within this terrain in the general vicinity during periods of very heavy rainfall. We have no evidence that the Lower Schultheis Road West or Ditullio landslide failed statically during recent years; however, we know that laboratory shear strengths generally are at the high end of the range. Thus, these two approaches can give us some confidence about the range of shear strengths that should be considered for these two reactivated landslides.

Representative samples of the material from rupture zones are generally available only if a geologist retrieves them by hand from within a large excavation, such as a trench or large-diameter borehole. Rupture zones are seldom captured intact in drill-core samples because they are softer and more clay rich than the surrounding rock. During coring, the gouge materials are commonly destroyed by small amounts of relative movement between the core segments above and below the shear surface, and the gouge can be washed away by circulating fluid.

When a sample is tested in the laboratory, precautions are needed to prevent disturbance of the gouge. Direct shear testing is generally favored because the thin rupture zone can be aligned with the shear surface to ensure that the gouge material itself is tested, rather than the harder rock above or below the rupture zone. As a result of the uncertainties associated with laboratory results, it is not uncommon during postfailure landslide studies to discover that laboratory-determined shear strengths overestimate the strengths computed for the landslide surface from backanalyses of landslide stability.

Backanalyses for stability also need to be evaluated carefully. A backanalysis based on a seismic failure is a circular argument, because we test our ability to predict seismic failure from independent measurements of key parameters, including shear strength and seismic ground motion. However, if we backanalyze a static (that is, ground-water driven) failure, we gain some insight into the range of possible shear strengths of the rupture-surface materials.

We backanalyzed the Lower Schultheis Road West and Ditullio landslides under static conditions to determine a reasonable range of shear strengths for examination in our sensitivity analysis. The backanalyses assume that the landslides failed under static conditions at some time in the past and that the rupture surfaces had reached their residual strengths (that is, c=0). The range of ø values that would satisfy the backanalysis for possible ground-water levels is plotted in figure 7. This range was the basis for selecting angles of 17°–25° in our cumulative-displacement analyses.

Cumulative displacements as a function of the angle of internal friction, ø, and the yield acceleration, k_y, of the two reactivated landslides for a peak horizontal ground acceleration of 1.00 g are plotted in figure 8, which shows the results using the curves of K.L. Lee (unpub. data, 1977), Makdisi and Seed (1977), Hynes-Griffin and Franklin (1984), and Lin and Whitman (1986), as well as from the program DISPLMT using the Corralitos and LGPC station (fig. 1) records scaled to k_{max}=1.0 g.

Figure 8 demonstrates that the displacements calculated by all five methods are sensitive to the chosen ø value.

Table 4.—*Summary of cumulative-displacement results using five analytical methods for the Lower Schultheis Road West and Ditullio landslides*

[k_y, seismic yield coefficient; k_{max}, peak horizontal ground acceleration; M, earthquake magnitude; static FS, static factor of safety (3.6 for Lower Schultheis Road West landslide, 2.9 for Ditullio landslide); LSRW, Lower Schultheis Road West. Shear strengths for Lower Schultheis Road West landslide: ϕ=25°, c=0 kg/cm^2, k_y=0.23 g for lower-bound values and ϕ=24°, c=0.22 kg/cm^2, k_y=0.35 g for best-fit values, where ϕ is the angle of internal friction and c is the cohesion. Shear strengths for Ditullio landslide: ϕ=25°, c=0 kg/cm^2, k_y=0.23 g for lower-bound values and ϕ=24°, c=0.22 kg/cm^2, k_y=0.30 g for best-fit values]

Analytical method	Input parameter	Value	Predicted displacements (cm)			
			Ditullio landslide		LSRW landslide	
	k_y for static FS		Lower bound	Best fit	Lower bound	Best fit
Program DISPLMT (Houston and others, 1987): Double integration of accelerations above k_y (average of two runs with accelerations inverted).	Corralitos station record (k_{max}=0.63 g)	Average of two runs	5	3	4	3
	Corralitos station record (k_{max}=1.00 g)	----------------	35	4	49	6
	Station LGPC record (k_{max}=1.00 g)	----------------	29	3	23	1
Makdisi and Seed (1977), with Keefer and others (1991) for M = 7.0.	k_y, k_{max}=1.00 g, M = 7.0	Upper bound ------	82	50	76	40
		Lower bound ------	15	9	14	6
		Geometric mean ----	37	22	35	17
K.L. Lee (unpub. data, 1977) -----	k_y, k_{max}=1.00 g, M = 7.1	Upper bound ------	179	97	160	68
		Lower bound ------	45	24	40	17
		Geometric mean ----	77	42	69	29
Lin and Whitman (1986): Provides expected values, implying not especially conservative values.	k_y, k_{max}=1.00 g, subsurface conditions	Mixed sites -------	84	42	77	29
		Deep-cohesionless-soil and stiff-soil sites.	67	34	62	22
		Rock sites ---------	42	21	38	14
Hynes-Griffin and Franklin (1984)	k_y, k_{max}=1.00 g, confidence level	Upper bound ------	73	47	68	37
		Mean ? -----------	21	12	19	<10
		Mean ------------	11	<10	10	<10
Field measurements ----------	---------------	----------------	33-57 (avg 45)		61	

The curves of K.L. Lee (unpub. data, 1977) and Lin and Whitman (1986) are the most conservative, yielding predicted displacements near or slightly greater than the observed displacements when lower-bound laboratory shear strengths (c=0, ϕ=25°) are used. The other methods predict displacements lower than the observed displacements under the same conditions. However, these results also indicate that the laboratory shear strengths generally predict displacements lower than those actually measured for a seismic event as large as the 1989 Loma Prieta earthquake. The two program DISPLMT analyses for each landslide provide the clearest examples: The best comparison of predicted-to-observed displacements are for ø=18°–22° for the Lower Schultheis Road West landslide and ø=21°–24° for the Ditullio landslide. Use of the lower-bound laboratory shear strengths leads to predicted displacements that are about two-thirds less than those measured on the Lower Schultheis Road West landslide and about one-half less than those measured on the Ditullio landslide. The program DISPLMT results for the Corralitos and LGPC station (fig. 1) records also differ markedly, demonstrating that displacements calculated by using integration methods depend heavily on the selected acceleration-time history.

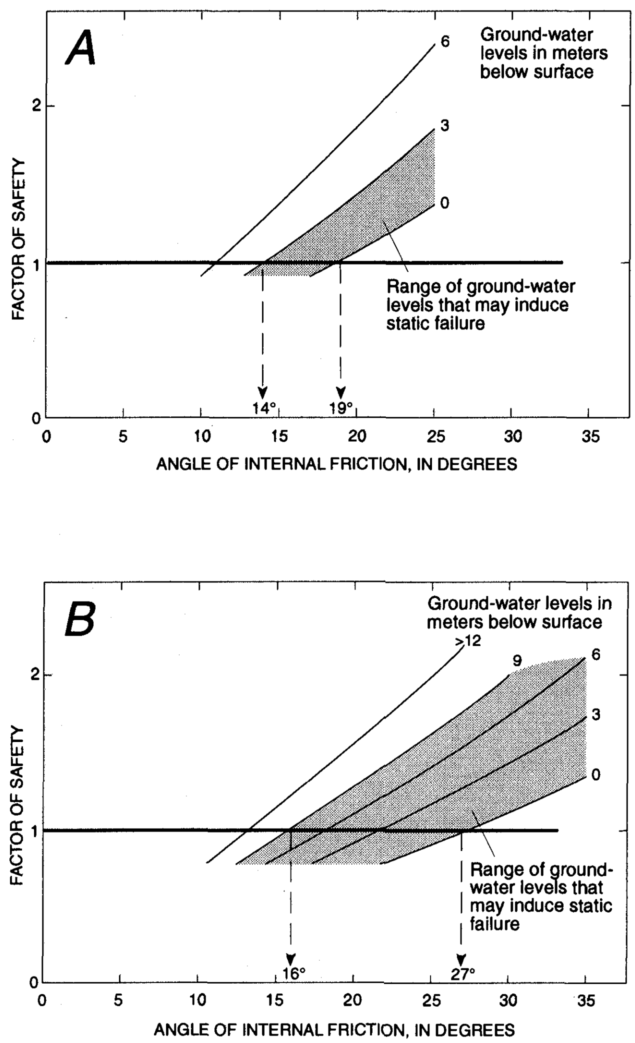


Figure 7.—Factor of safety versus shear strength (expressed as angle of internal friction) at various ground-water levels in Lower Schultheis Road West (*A*) and Ditullio (*B*) landslides (see fig. 2 for locations).

Although we believe that ground motion at the landslide sites was much greater than that recorded at the Corralitos strong-motion station (fig. 1), we also calculated displacements by using the unscaled acceleration-time history at this station to determine the sensitivity of the results to this seismic record. Cumulative displacements calculated using five analytical methods for k_{max}=0.63 g are plotted as a function of ø value in figure 9. A comparison of figures 8 and 9 demonstrates the importance of the acceleration-time history and peak ground acceleration on the calculations. Using the scaled Corralitos station record (k_{max}=1.00 g), the measured displacements correspond relatively well to those calculated from the program DISPLMT; however, the unscaled Corralitos station record (k_{max}=0.63 g) yields significantly smaller displacements for the same shear strengths.

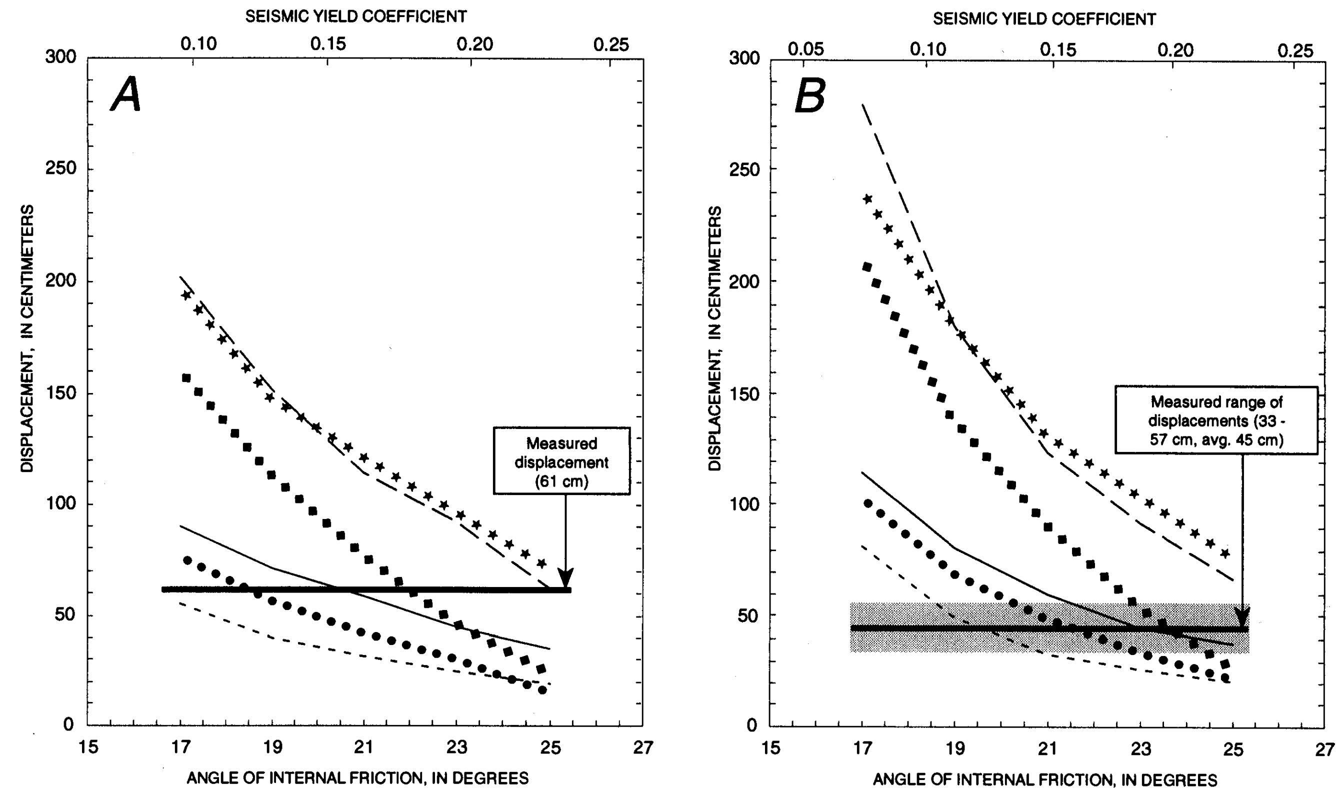
The displacements that can be calculated for an M=7.0 earthquake, using the three curves of Makdisi and Seed (1977) and Hynes-Griffin and Franklin (1984) for the Lower Schultheis Road West landslide, are plotted in figure 10. Because of the bilateral rupture, the 1989 Loma Prieta earthquake was of considerably shorter duration than most other earthquakes of similar magnitude. Therefore, we might expect the measured displacements to fall in the lower part of the range of calculated displacements. Figure 10 shows, however, that they do so only for the smallest ø values. If we assume that the appropriate ø value should be about 21°, then the measured displacements fall in the middle of the range of displacements in the data sets of both Makdisi and Seed (1977) and Hynes-Griffin and Franklin (1984).

In summary, the mean to upper-bound curves for the four chart methods, as well as the DISPLMT integration method, appear to predict displacements that are within an order of magnitude of the displacements measured on the two reactivated landslides when a k_{max} value of 1.00 g is used in conjunction with low shear strengths. It is remarkable, however, how broad the range of calculated displacements can be for an earthquake of a specific size. Adaptation of specific strong-motion records to the actual field conditions at a particular site, to account for the effects of attenuation, distance, topography, or directivity, is subject to uncertainties that can change the predicted displacements by a factor of at least 3. For example, the displacements calculated using the unscaled Corralitos and LGPC station records scaled to the same peak acceleration (k_{max}=1.00 g) vary by a factor of as much as 2 (fig. 8). At this time, it is unclear how the specific record should be chosen or scaled (if at all) for local conditions.

CONCLUSIONS

1. **Preexisting landslides commonly were only partially reactivated.** The pattern of ground cracking within preexisting ancient landslides indicates that only part of many preexisting landslides were mobilized by the ground motion from the 1989 Loma Prieta earthquake. In many places, ground cracks coincided with preexisting-landslide boundaries, even where these boundaries were relatively subdued. The absence of ground cracks displaying shear movement on the flanks of many of these landslides was interpreted to reflect mobilization of only the upslope segments of the basal rupture surfaces of the landslides during earthquake shaking.

2. **Seismic-displacement calculations are highly sensitive to selected yield coefficients and shear strengths.** The results of the cumulative-displacement analyses indicate that calculated displacements are highly sensitive to the seismic yield coefficient, which is a function of the shear strength of the basal rupture surface. The determination of a representative strength across the basal rupture surface, therefore, is crucial to the success of the analysis. Sampling and testing of thin shear surfaces separating hard-rock strata is difficult, and standard "small diameter" exploration methods have a very low probability of allowing identification, adequate sampling, and preservation of thin rupture surfaces. Direct observation and careful hand extraction of oriented samples from large excavations



EXPLANATION

● ● ● ● Program DISPLMT (Houston and others, 1987), scaled Corralitos Station record

◆ ◆ ◆ ◆ Program DISPLMT (Houston and others, 1987), unscaled Los Gatos Presentation Center Station record, k_{max} = 1.00 g

- - - - - Mean ± 1 σ (Hynes-Griffin and Franklin, 1984)

——— Mean for theoretical M =7.0 earthquake (Makdisi and Seed, 1977)

★ ★ ★ ★ Geometric mean (K.L. Lee, unpublished data, 1977)

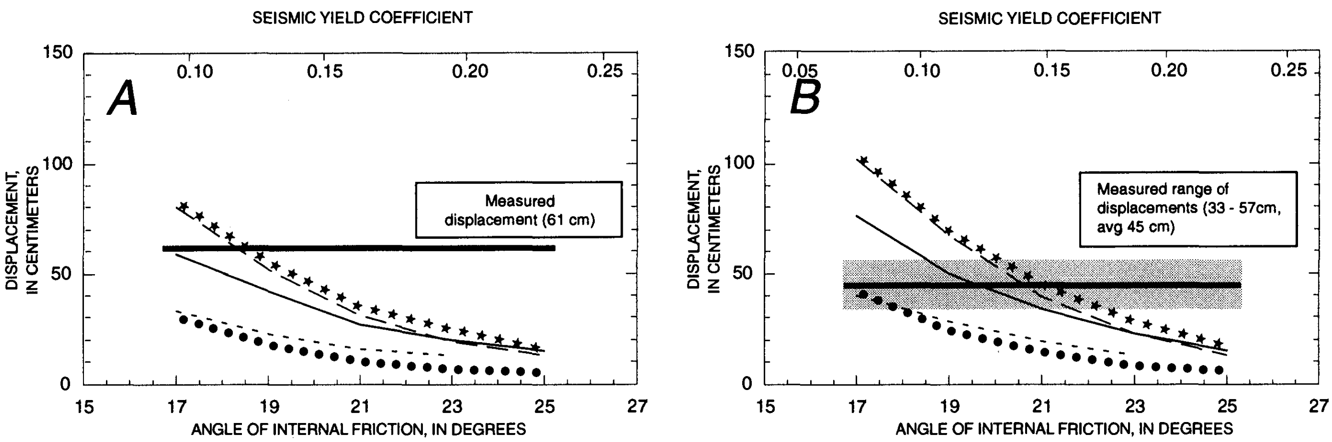
— — Stiff soils or deep cohesionless soils (Lin and Whitman, 1986)

Figure 8.—Predicted displacement versus shear strength (expressed as angle of internal friction) calculated by five cumulative-displacement methods, using records from stations at Corralitos and Los Gatos Presentation Center (see fig. 1 for locations), assuming a peak horizontal ground acceleration of 1.0 g. A, Lower Schultheis Road West landslide. B, Ditullio landslide.

(trenches or large-diameter boreholes) are recommended to observe the landslide geometry and collect the appropriate materials for testing.

3. **Laboratory strengths may be too high to accurately predict stability and displacements.** The pseudostatic method of analysis incorrectly predicted the stability of the Lower Schultheis Road West and Ditullio landslides, using lower-bound laboratory shear strengths and a seismic yield coefficient of 0.20, but would have predicted the failures if a higher seismic yield coefficient had been used. The magnitudes of the deformations on the Lower Schultheis Road West and Ditullio landslides generally were underestimated, using various cumulative-displacement methods and laboratory shear strengths.
4. **Backanalyzed shear strengths can be used to improve the accuracy of calculated cumulative displacements.** Backanalyses give a range of reasonable shear strengths for landslide rupture materials in the form of a range of angles of internal friction of 17°–25°. Backanalyzed shear strengths can be used with laboratory results to better estimate the average shear strength of the basal rupture surface. In this study, use of an angle of internal friction of about 21° produces displacements that generally are consistent with measured displacements, and appears to correctly predict the onset of instability when using the pseudostatic analysis.
5. **Ground-water levels significantly influence slope stability.** Despite a prolonged, 4-year-long drought, shallow ground water was present in many of the landslides investigated. Ground-water zones appear to be perched on impermeable rupture surfaces. Laboratory results indicate high angles of internal friction and relatively low cohesion. The presence of higher ground-water levels in the landslides before the earthquake would have increased the landslide displacements. Measurements of ground-water levels are important to make in the field and to include in calculations.
6. **Displacement calculations are also sensitive to the selected acceleration-time history.** The range of displacements calculated from published compilations of earthquake records is clear testimony to the degree of sensitivity to this parameter. Because the effect of the long-period part of the seismic record is significant, the peak ground acceleration, though important, may not be as critical as other factors, such as duration of shaking, attenuation with distance from the source, and seismic focusing effects (including topographic amplification and directivity). Seismic-stability analyses should be based on more than one seismic record whenever practical.

This study emphasizes that available methods of cumulative-displacement analysis are useful, but investigators should select variables with care so as not to underesti-



EXPLANATION

● ● ● ● Program DISPLMT (Houston and others, 1987), unscaled Corralitos Station record

★ ★ ★ ★ Geometric mean (K.L. Lee, unpublished data, 1977)

- - - - - Mean ± 1σ(Hynes-Griffin and Franklin, 1984)

— — Stiff soils or deep cohesionless soils (Lin and Whitman, 1986)

——— Mean for theoretical M =7.0 earthquake (Makdisi and Seed, 1977)

Figure 9.—Predicted displacement versus shear strength (expressed as angle of internal friction) calculated by five cumulative-displacement methods, using peak horizontal ground acceleration of 0.63 g from station at Corralitos (see fig. 1 for location). *A*, Lower Schultheis Road West landslide. *B*, Ditullio landslide.

mate the stability of preexisting landslides. With a thorough characterization of landslide geometry, local geologic conditions, and engineering properties, and a selection of conservative strength parameters for the basal rupture surfaces, existing analytical techniques are sufficient for order-of-magnitude predictions when the lowest laboratory shear strengths are used.

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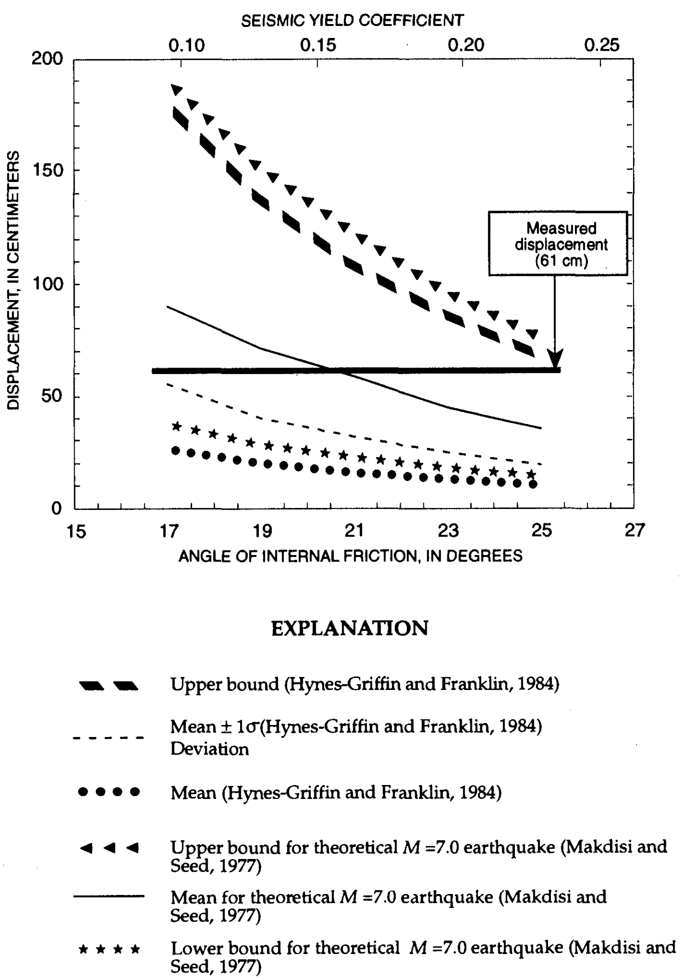

Figure 10.—Predicted displacement versus shear strength (expressed as angle of internal friction) calculated by two cumulative-displacement methods for Lower Schultheis Road West landslide (see fig. 2 for location).

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APPENDIX: DESCRIPTION OF LANDSLIDE SITES

The two reactivated landslides discussed in this paper were chosen from an initial group of more than 50 relatively large landslides in the study area. Detailed surface and subsurface data were collected from five of these landslides. This appendix provides descriptions of the two landslides discussed above. More complete descriptions of these sites, as well as two additional sites, including maps, cross sections, borehole logs, and laboratory procedures, were presented by Cole and others (1991).

LOWER SCHULTHEIS ROAD WEST LANDSLIDE

LOCATION AND SCOPE OF INVESTIGATION

The site of the Lower Schultheis Road West landslide, referred to as "Upper Laurel" by Cole and others (1991), is near Laurel, Calif., along the north-facing flank of an east-west-trending spur ridge (fig. 2) that is underlain at depth by siltstone and sandstone of the San Lorenzo Formation. A surveyed topographic map of the entire 7.7-ha parcel was provided by the property owner. Surface mapping and profiling of the landslide was performed at a scale of 1:240, using a semi-total-station (STS) survey instrument for horizontal and vertical control. Two large-diameter boreholes, LD–1 and LD–2 (fig. 3), were excavated and logged to 11.3- and 8.5-m depth, respectively.

SURFACE CONDITIONS

The geomorphology of the ridge and adjacent valley indicate that a large, ancient landslide underlies most of the north-facing hillside. A system of arcuate ground fissures near the ridge crest indicates that the earthquake reactivated a part of this preexisting landslide. The system of cracks defines a graben that follows the ridge crest for approximately 60 m before curving downslope to form a broad arc (fig. 4). The head of the reactivated landslide mass was downthrown about 25 cm and offset laterally about 61 cm. No ground deformation was observed along the probable lateral margins. The probable toe of the landslide is indicated by a zone of compression coinciding with a slight topographic bulge. The reactivated landslide appears to be about 61 m wide by 76 m long. The total area of the reactivated landslide is approximately 0.4 ha.

SUBSURFACE CONDITIONS

The two large-diameter boreholes, LD–1 and LD–2 (fig. 3), were located 4.6 and 12.2 m, respectively, downslope from the headscarp crack. Both boreholes penetrated a near-horizontal, well-developed shear surface at 4.9-m depth; this shear surface separates overlying, oxidized regolith from underlying, unoxidized fractured rock. A deeper, steeply dipping shear surface was penetrated in borehole LD–1 at 7.3-m depth; this deeper shear surface is a sheared siltstone interbed within massive sandstone that separates the overlying fractured rock from dense, relatively intact rock. The shallower shear surface appeared to be thicker, more moist, and generally better developed than the deeper shear surface within more indurated rock.

Subsurface data indicate that the earthquake did not reactivate the uppermost part of the preexisting landslide mass. Trenches excavated by Rogers E. Johnson and Associates (1991) demonstrate that the deeper (7.3 m deep) rupture surface exposed in borehole LD–1 (fig. 3) extends along a 30° plane to the ridge crest and passes undisturbed beneath the 1989 graben that forms the upslope boundary of the 1989 reactivated landslide (fig. 4). The reactivated headscarp (1989 graben) is 5.9 m downslope from the intersection of the buried slide surface with the present ground surface. The slide surface at this point is 3.2 m deep. Thus, the earthquake apparently failed to reactivate an upper wedge of preexisting landslide material. (The deeper shear surface is a bedding-plane fault that probably formed during folding of an underlying bedrock anticline; the shallower shear surface exposed in boreholes LD–1 and LD–2 was not exposed in the trenches.) Rogers E. Johnson and Associates found a similar relation between a buried rupture surface and an arcuate, 1989 earthquake-triggered landslide headscarp in the same vicinity. The preexisting rupture surface forming the base of the dormant wedge of preexisting-landslide material may have acquired more cohesion over time, owing to root reinforcement and clay development during pedogenesis. The associated increase in shear strength may have helped to maintain stability in the shallow (upper 3 m) part of the landslide. The maximum thickness of the stable wedge coincides with the approximate base of significant roots, suggesting that root reinforcement could have increased resistance to sliding in this wedge. Terwilliger and Waldron (1991) discussed the role of root reinforcement on the stability of shallow landslides.

Ground water was penetrated in the two large-diameter boreholes, as well as in boreholes drilled by Kingsley Associates (1991). Isolated zones of ground water were perched on impermeable shear surfaces. Significant amounts of ground water also were penetrated below the deepest shear surface (8–10 m below the ground surface).

DITULLIO LANDSLIDE

LOCATION AND SCOPE OF INVESTIGATION

The site of the Ditullio landslide is on Redwood Lodge Road, about 1.2 km southeast of the Lower Schultheis Road West landslide. The Ditullio landslide is near the top of a southwest-facing slope within and near the southern margin of a previously mapped, very large landslide complex (fig. 2). The Ditullio landslide is 92 to 122 m long by 76 m wide, covering an area of about 0.6 to 0.8 ha. Surface mapping and profiling of the landslide was performed at a scale of 1:240, using an STS survey instrument for horizontal and vertical control.

SURFACE CONDITIONS

The headscarp is characterized by echelon and parallel fissures that form a 76-m-long, arcuate zone of ground cracks. Individual cracks within this zone indicate 0.3 to 0.6 m of extension and 0.3 m of vertical downdropping. The zone coincides with the apparent headscarp of an ancient landslide. The locations of the toes of both the ancient and 1989 landslides are unknown; however, this ancient landslide probably is part of a landslide complex that extends about 350 m downslope to the creek.

Several linear, echelon cracks that extend 52 m downslope from the scarp along the probable north boundary of the landslide may represent shearing along its right flank; however, these cracks could also be due to settlement of a septic leachfield. No evidence of offset or deformation was observed along the left flank. The earthquake-triggered landslide does not appear to cross Redwood Lodge Road, located 61 to 92 m downslope from the headscarp (fig. 3).

SUBSURFACE CONDITIONS

A large-diameter borehole, DT–1 (fig. 4), was located about 40 m downslope from the headscarp cracks. Borehole DT–1, which was drilled and downhole logged to 22.5-m depth, exposed a well-developed, slickensided shear surface at about 18-m depth. The gradational oxidized-unoxidized contact is at 20.7-m depth; however, oxidized sandstone bedrock immediately below the landslide plane is very dense. Figure 4, which includes a detailed log of the landslide rupture surface, illustrates the relation between the 1989 reactivation, the underlying ancient landslide material, and deeper bedrock structure.

No ground water was penetrated in borehole DT–1, even after remaining open for 12 days.

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