PREFACE

The Papers and Information contained in this Proceedings Volume were presented at the 35th Annual Highway Geology Symposium and Field Trip held on the campus of San Jose State University in San Jose, California, August 15-17, 1984. The theme of the conference was "Geotechnical Problems Associated with Transportation Routes on a Major Plate Boundary". This theme was selected to emphasize the difficulties that engineers and geologists have in dealing with the problems of planning, designing and constructing in areas characterized by active faults, evolving (often unstable) topography, and active shorelines. The conference brought together more than 100 scientists and engineers from various parts of the United States to share new information and ideas.

During the conference, 17 papers were presented. Eleven of these papers are in this volume. The abstracts of six presentations are included because a complete paper was not available.

In order to make the Proceedings available as quickly as possible, authors were requested to submit camera-ready manuscripts. The compilers of this volume have not edited the papers and the papers appear as submitted. All of the papers contain new ideas or perspectives, and it is hoped that they will stimulate new thinking and new approaches to solving the problems associated with the complex interaction of geology and transportation routing.

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Department of Geology, San Jose State University

Timothy J. Beck

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October 1984

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Papers

site of the 1979 meeting, Austin, Texas in 1980, and Gatlinburg, Tennessee in 1981. The 1982 meeting was held in Vail, Colorado, and in Stone Mountain, Georgia in 1983. The 35th meeting was held in San Jose, California.

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee composed of approximately 20 engineering geologists and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contributions to the symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. They may succeed themselves for one additional term.

A number of three-member standing committees conduct the affairs of the organization. Some of these committees are: By-Laws, Public Relations, Awards Selection, and Publications. The lack of rigid requirements, routine, and the relatively relaxed overall functioning of the organization is what attracts many of the participants.

HIGHWAY GEOLOGY SYMPOSIUM

History, Organization, and Function

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium was organized and held its first meeting on February 16, 1950, in Richmond, Virginia. Since then, 35 consecutive annual meetings have been held in 23 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Symposium moved to Lexington, Kentucky in 1965, Ames, Iowa in 1966, Lafayette, Indiana in 1967, back to West Virginia at Morgantown in 1968, and then to Urbana, Illinois in 1969. Lawrence, Kansas was the site of the 1970 meeting, Norman, Oklahoma in 1971, and Old Point Comfort, Virginia the site in 1972.

The Wyoming Highway Department hosted the 1973 meeting in Sheridan. From there it moved to Raleigh, North Carolina in 1974, back to the west to Coeur d'Alene, Idaho in 1975, Orlando, Florida in 1976, Rapid City, South Dakota in 1977, and then back to Maryland in 1978; this time in Annapolis. Portland, Oregon was the

Meeting sites are chosen two to four years in advance and are selected by the Steering Committee following presentations made by representatives of potential host states. These presentations are usually made at the steering committee meeting which is held during the Annual Symposium. Upon selection, the state representative becomes the state chairman and a member pro tem of the Steering Committee. Depending on interest and degree of participation, the temporary member may gain full membership to the Steering Committee.

The symposia are generally for two and one-half days, with a day-and-a half for technical papers and a full-day for the field trip. The symposium usually begins on Wednesday morning. The field trip is usually Thursday, followed by the annual banquet that evening. The final technical session generally ends by noon of Friday.

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally cultural stops are scheduled around geological and geotechnical points of interest. In Wyoming, the group viewed landslides in the Big Horn Mountains; Florida's trip included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and Glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area which illustrated the interaction of fault activity, urban landslides, and coastal erosion with the planning, constructing, and maintaining of transportation systems.

At the technical sessions, case histories and state-of-the-art papers are the most common. Highly theoretical papers are the exception. The papers presented at the technical sessions are published in the annual proceedings. Some of the proceedings are out of print, but copies of most of the last fifteen proceedings may be obtained from the Treasurer of the Symposium, David Bingham, of the North Carolina Department of Transportation in Raleigh 27611. Costs generally range from $5.00 to $12.00, plus postage.

*HIGHWAY GEOLOGY SYMPOSIUM

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Paul Price	-	1970
K. B. Woods	-	1971
R. J. Edmonson	-	1972
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Burrell Whitlow	-	1978
Bill Sherman	-	1980
Virgil Burgat	-	1981
Henry Mathis	-	1982
David Royster	-	1982
Terry West	-	1983
David Bingham	-	1984

*In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis, Indiana designed a plaque to be presented to individuals who have made significant contributions to the Highway Geology Symposium over a period of years. The award, a 3.5" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

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The Geotechnical Setting of the San Jose Area, California

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Abstract

The greater San Jose area, in the southern portion of the San Francisco Bay region, is typical of many parts of California in terms of the geologic hazards and difficulties that must be overcome to satisfactorily plan, construct, and maintain a transportation system. Resistant geologic materials in combination with recent tectonic events have created northwest-southeast topographic barriers that must be breached. The many active faults of the area have the potential to cause damage because of ground shaking, ground displacement, and ground failure. Landslides, partially the result of the tectonically weakened rocks and steep slopes, are threats to the transportation corridors. Ground subsidence because of groundwater withdrawal, erosion by wave action, flooding because of combined rainfall runoff and high tides, and tsunamis are water related problems in the region. The mapping in detail of geologically hazardous areas, combined with a better understanding of the mechanics of geologic processes should

permit better planning, construction, and maintenance of modern transportation systems.

This paper is designed to be an introduction to the general geotechnical setting for the greater San Jose area and it will serve as background for the other papers in this volume. This area is outlined on the location map (Figure 1) and is in Santa Clara County at the southern end of the San Francisco Bay region.

The San Francisco Bay area is in the California Coast Ranges Physiographic Provence bordered on the east by the Great Valley Provence, on the west by the Pacific Ocean, and to the south by the Transverse Ranges (Figure 2). The Coast Ranges Provence is characterized by northwest-southeast trending subparallel mountain ranges rising in elevation to approximately three thousand feet. These ranges are separated by short discontinuous valleys, the Santa Clara Valley being a somewhat larger than average example. The province's southern border, the Transverse Ranges, is characterized by an east-west strike of the geologic structures. Figure 3 illustrates the rugged topography of the Santa Cruz Mountains and the San Francisco Peninsula which is crossed by the San Andreas fault (the plate boundary separating the Pacific plate on the west from the North American plate on the east). On the Peninsula, the topography east of the San Andreas fault consists

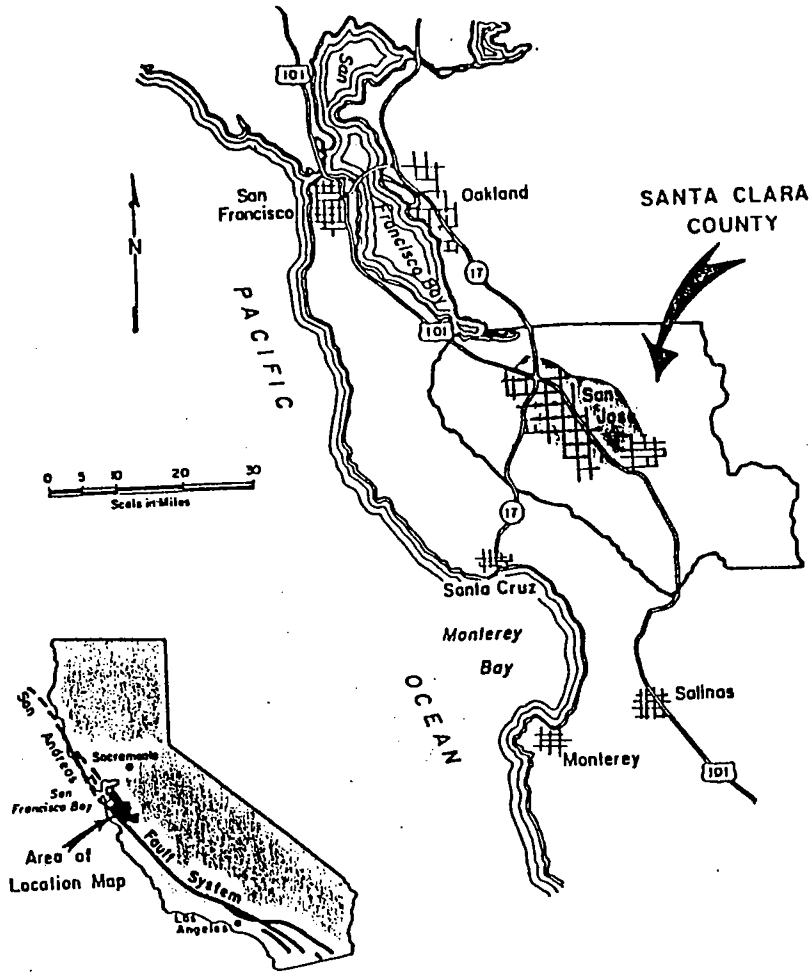

Figure 1. Location map for the San Jose area within the greater San Francisco Bay Region of central California. Note the lengthy shoreline (San Francisco Bay and Pacific Ocean) in this region.

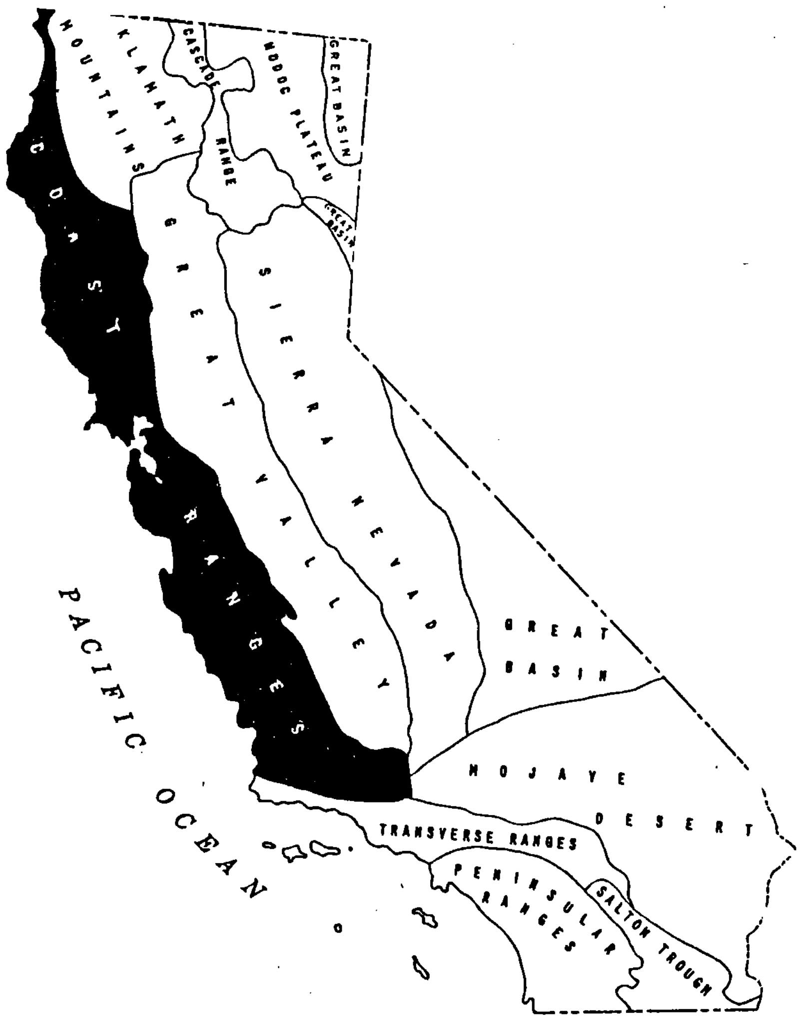

Figure 2. Physiographic provinces of California. The Coast Ranges Province in which San Jose is located is shaded. Geologic structures in this province are generally oriented northwest-southeast in contrast to the east-west oriented structures of the Transverse Ranges (modified from Bailey, 1966).

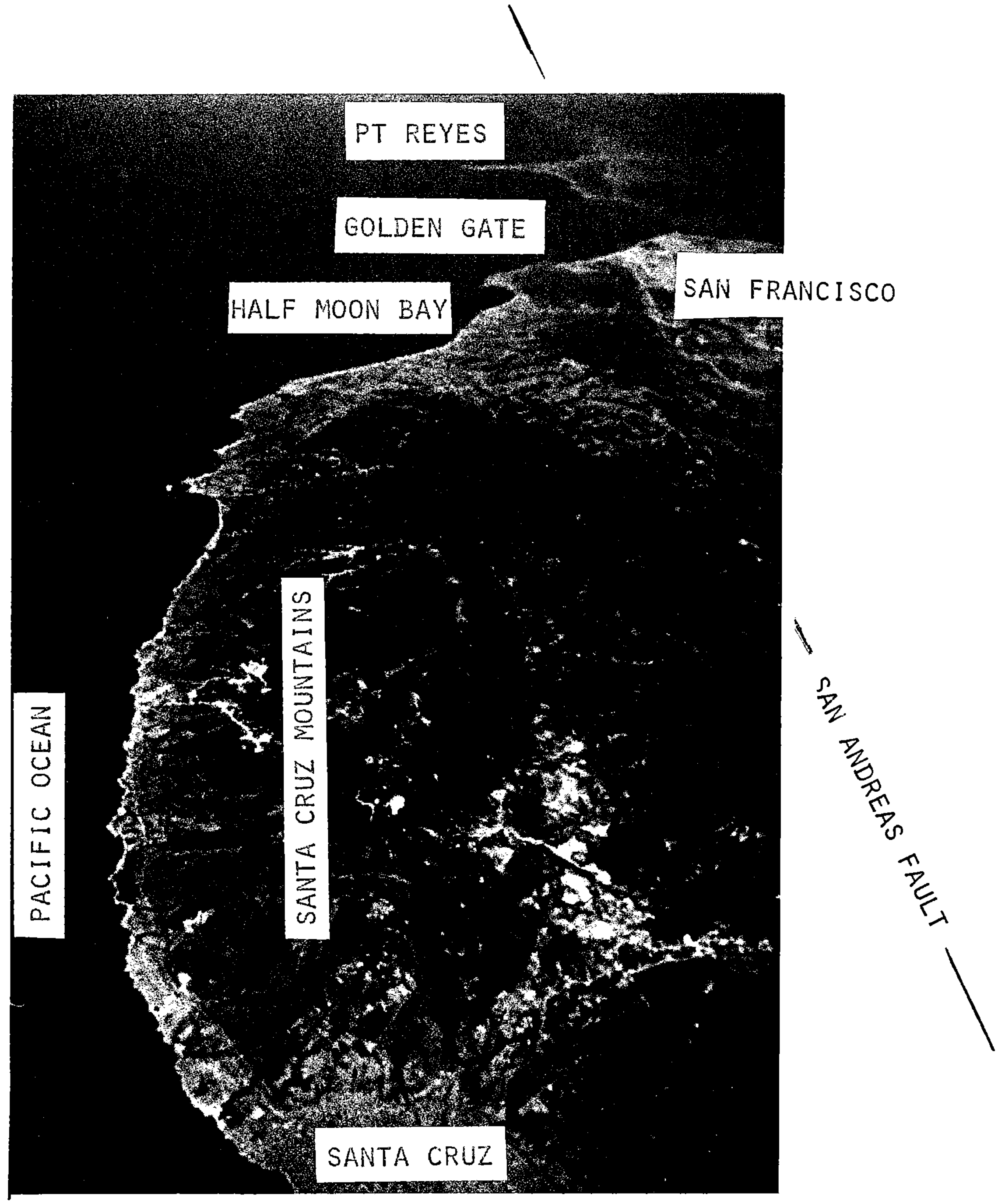

Figure 3. Photograph of central California coastal areas from Santa Cruz on the south to Point Reyes to the north. Note the lineament, the San Andreas fault, northwest- southeast across the San Francisco Pensinsula. Contrast the topography on the peninsula east and west of the San Andreas fault. (Modified from transparency, Pilot Rock, 1975)

of rolling surfaces cascading to the San Francisco Bay. This contrasts with the more rugged terrane west of the fault. Marine terraces fringe the Pacific Ocean. A different perspective, (Figure 4) toward the southeast, shows the extensive urbanization surrounding San Francisco Bay. Many of these developments are on land reclaimed from the bay. The low ranges east of San Francisco Bay, include the mountains of the Diablo and Mt. Hamilton ranges.

Geologically, the region is dominated by three "basement" materials: granitic-metamorphic, Franciscan, and Mesozoic marine (Figure 5). The granitic metamorphic core complexes are exposed in Montera Mountain, near the city of Santa Cruz, in the Gavilan range, and on the Monterey Peninsula. Among these core complexes is the Mt. Diablo diapere in the northeastern part of the San Francisco Bay region. The Franciscan core assemblage occupies much of southern Marin County, the northern portion of the San Francisco peninsula, and the ranges east of the City of San Jose. This Franciscan material is a very complicated and heterogeneous geologic unit and is a geotechnical challenge. Because of the extreme heterogeneoty, the geologist is unable to extrapolate geologic data with confidence from one point to another. Late Mesozoic marine materials crop out along the Pacific coast and in the foothills east of the City of Oakland. Most of the valleys are filled with unconsolidated and frequently water-saturated alluvium.

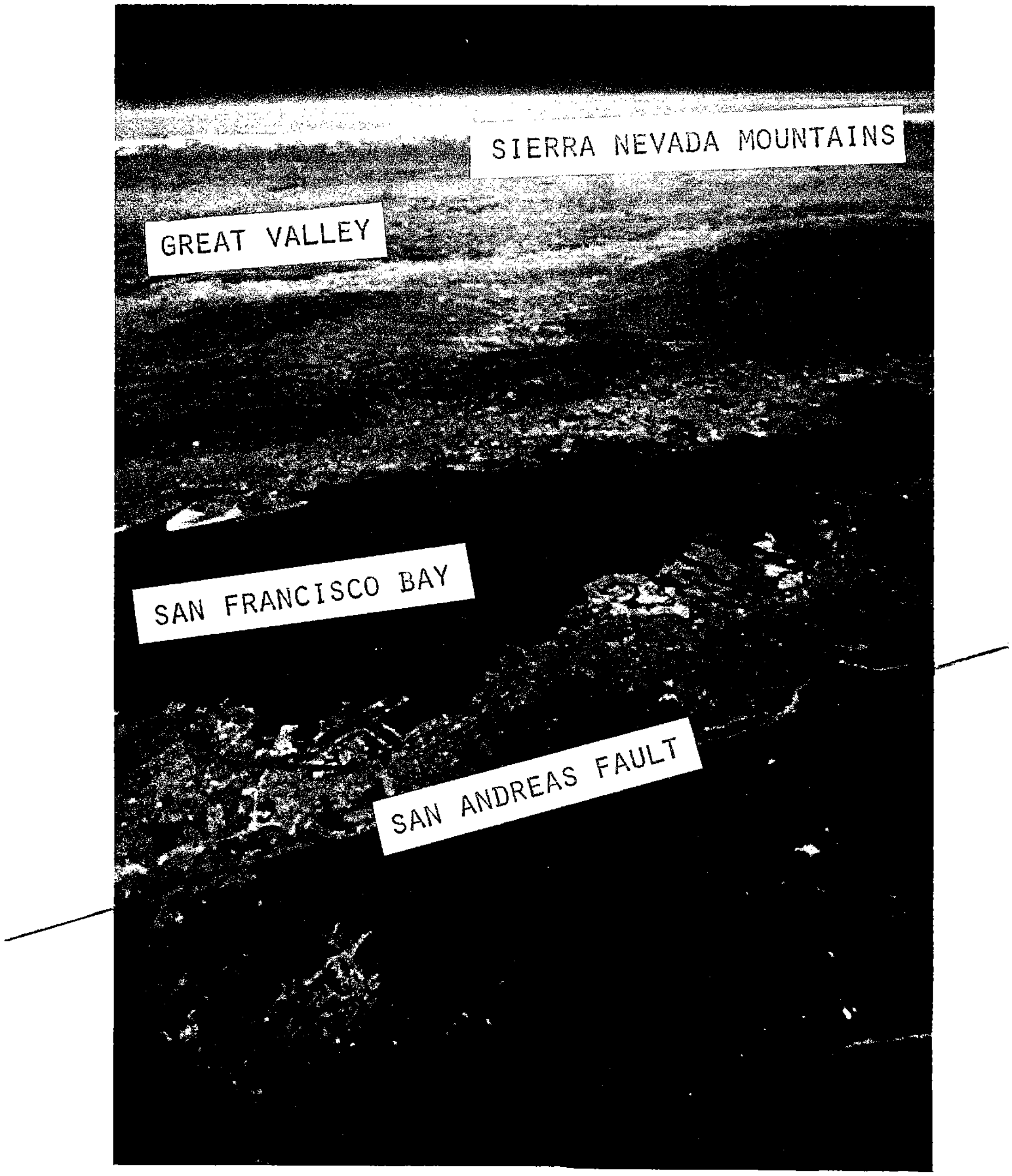

Figure 4. Photograph looking toward the southeast shows the extensive urbanization surrounding the southern end of San Francisco Bay. The Central Valley and the Sierra Mountains are located to the east toward the horizon. (Modified from transparency, Pilot Rock, 1975)

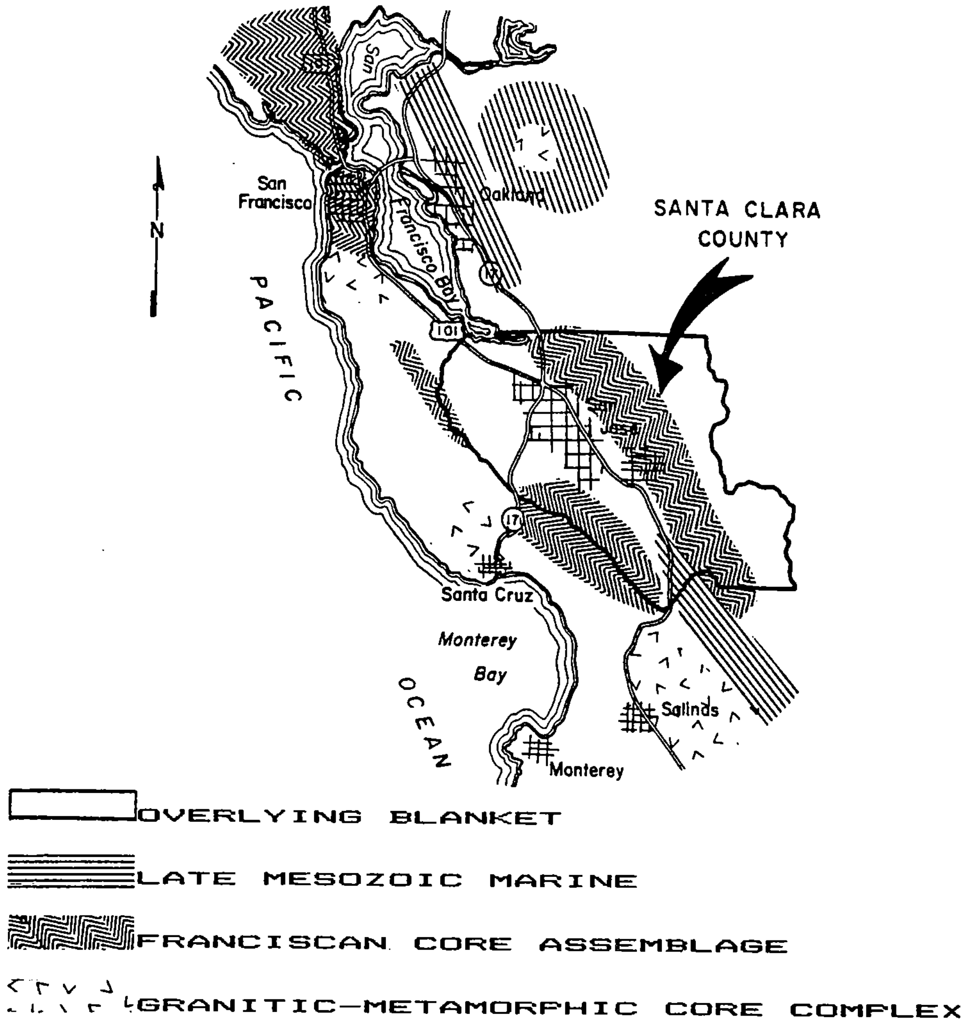

Figure 5. Simplified geologic map indicating the locations of the "basement" rocks of the San Francisco region.

Within the San Francisco Bay region, there has been much Quaternary activity (Figure 6). Not surprisingly much of the recorded tectonic displacement is characterized by right-lateral motions typical of those associated with the San Andreas fault. The San Andreas, Hayward, and Calavaras faults create secondary stress regimes. These regimes are characterized by thrust components. When considering the fault rupture potential in the region, not only are strike slip faults present but thrust faults as well.

The combination of the resistant core materials and the Quaternary structural activity, creates many topographic barriers (Figure 7). These topographic barriers are important in the construction of transportation corridors as east-west routes must go over or through these elevated terranes.

In general, the geologic hazards in the San Francisco Bay region are similar to those which should be anticipated anywhere within the State of California (Figure 8). A study by the California Division of Mines and Geology evaluating the projected dollar losses from geologic hazards within California for the period 1970-2000, suggests that more than 90 per cent of the losses can be attributed to four basic hazards. Earthquake shaking is the dominant one. The second greatest hazard is the loss of mineral resources. This loss ("hazard") occurs most frequently when expanding urban centers render unavailable many geologic

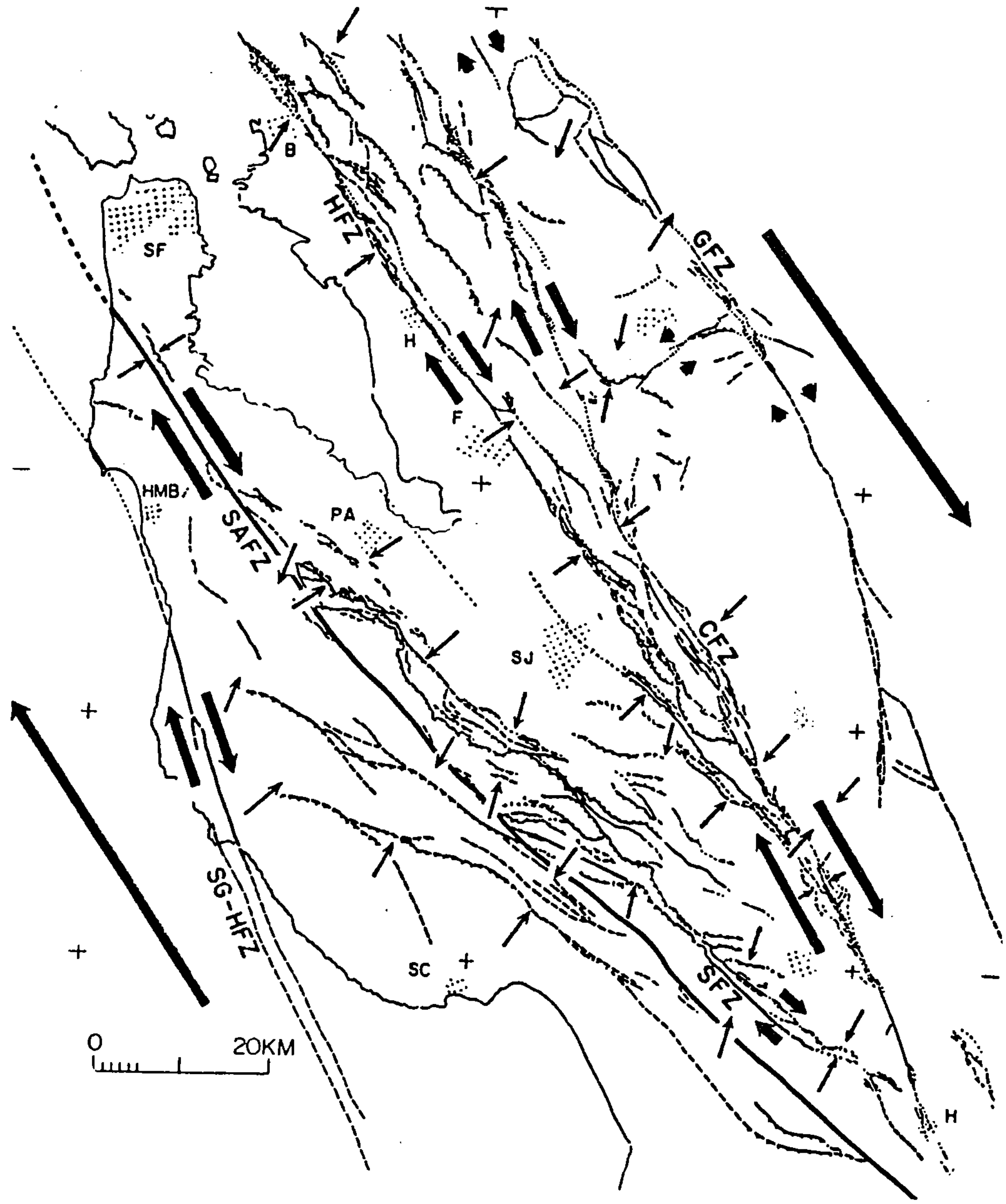

Figure 6. Folds and faults that have developed or that show evidence for movement during Quaternary time (from Page, 1982, p. 8). SAFZ - San Andreas Fault Zone, HFZ - Haywood Fault Zone, CFZ - Calaveras Fault Zone, GFZ - Greenville Fault Zone, SG-HFG - San Gregario - Hosgri Fault Zone

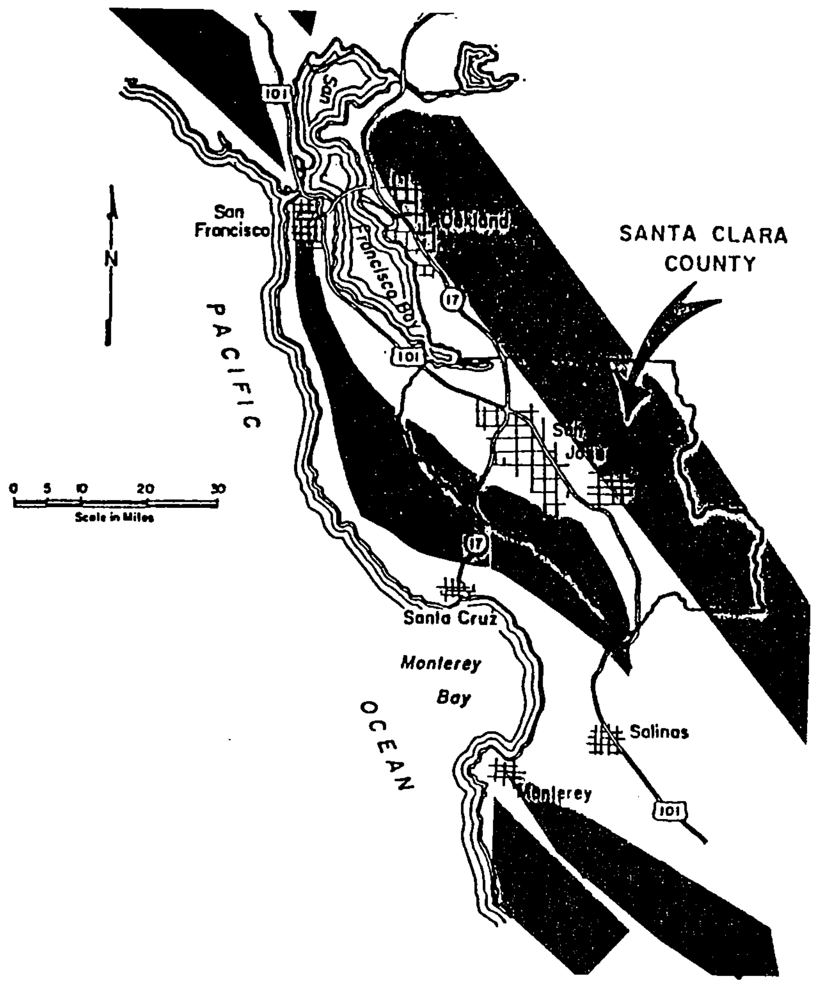

Figure 7. Topographic barriers (mountains and ridges) that have resulted from a combination of resistant geologic materials and Quaternary structural activity.

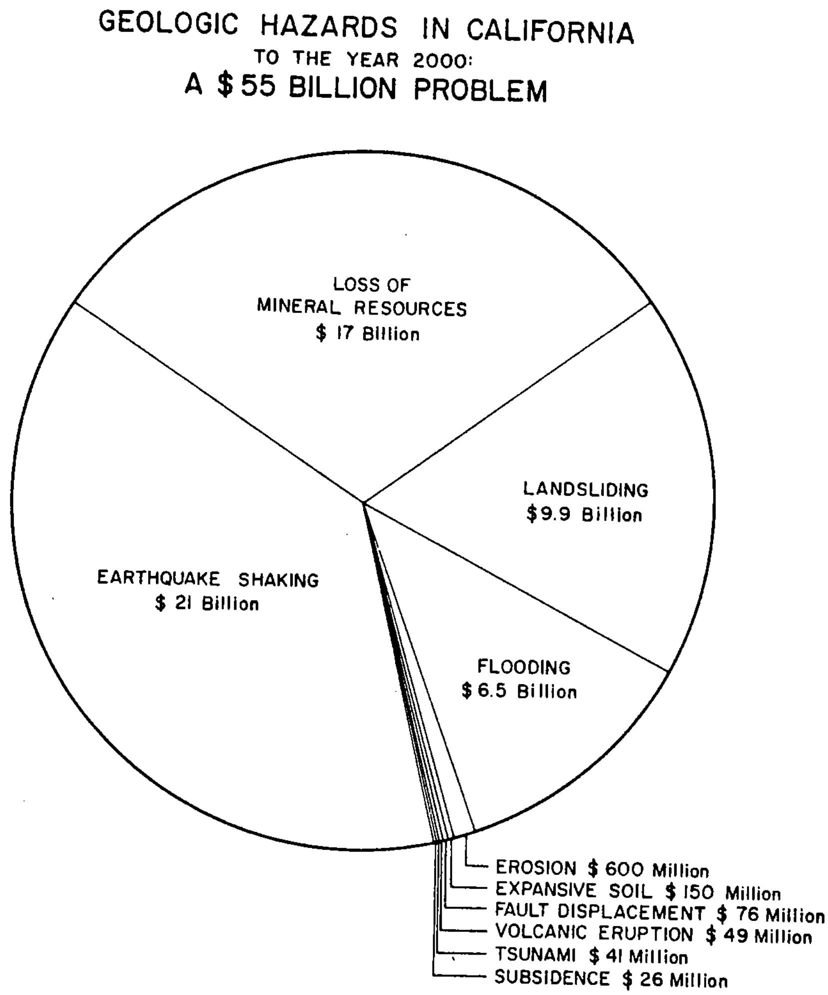

Figure 8. Pie-diagram indicating the types of geologic hazards and their relative importance in the State of California. These are the costs to the public that could be anticipated between 1970 and 2000 if current planning and construction practices were to be followed (Alfors, et al, 1973, p.5).

resources. Principal among these lost resources are the sand and gravel deposits. Landsliding is the third major problem in California. The fourth, flooding is expected to produce more than 10 billion dollars in losses within the next 30 years. Other geologic hazards factors are important locally, but their combined losses do not produce a large percentage of the overall damage potential for the period 1970 - 2000.

Earthquakes are one of the major problems in the San Francisco Bay area. The disruption of transportation corridors by ground displacement and/or seismic shaking has and will occur (Figure 9). The concern and debate over the stability of engineered transportation structures in active seismic areas has been going on for years. An example is the great debate between Andrew Lawson from University of California, Berkley and Bailey Willis from Stanford University over whether the San Francisco Golden Gate Bridge could survive in its present location (Figure 10). Happily, it can be reported that the Bridge has stood the test of time, at least for the past 50 years. Unfortunately, many newer structures including some of the overpasses under construction during the 1971, San Fernando earthquake did not survive. Some of these overpasses collapsed, prompting the California Department of Transportation to retrofit and upgrade the overpasses within the state, a major project, which is in the process of being completed.

The San Francisco Bay region is laced with active faults. The

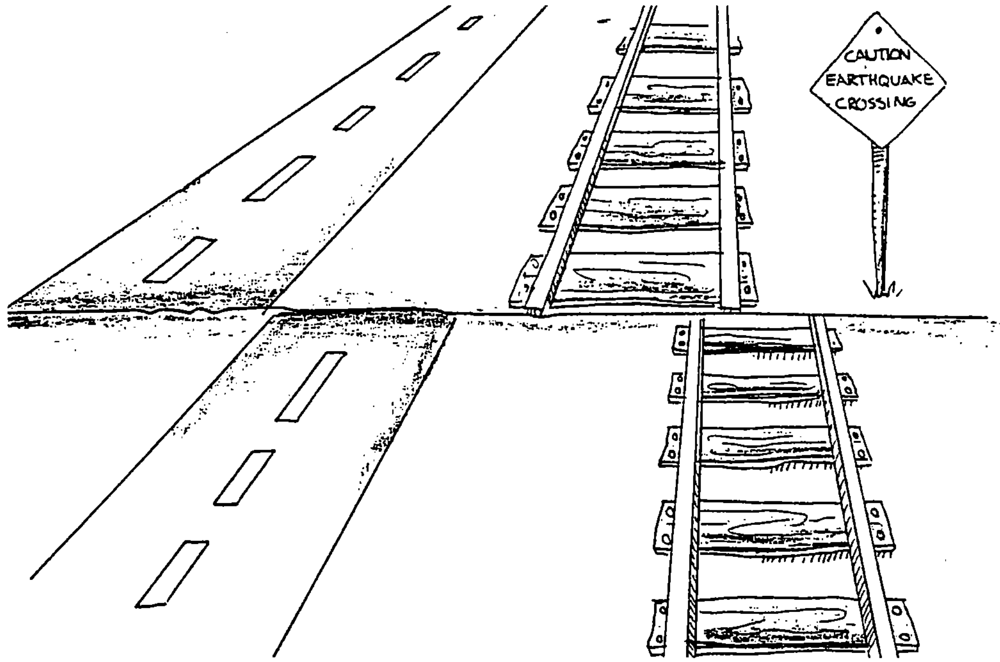

Figure 9. Ground displacement as the result of fault movement - a major disruption to transportation systems.

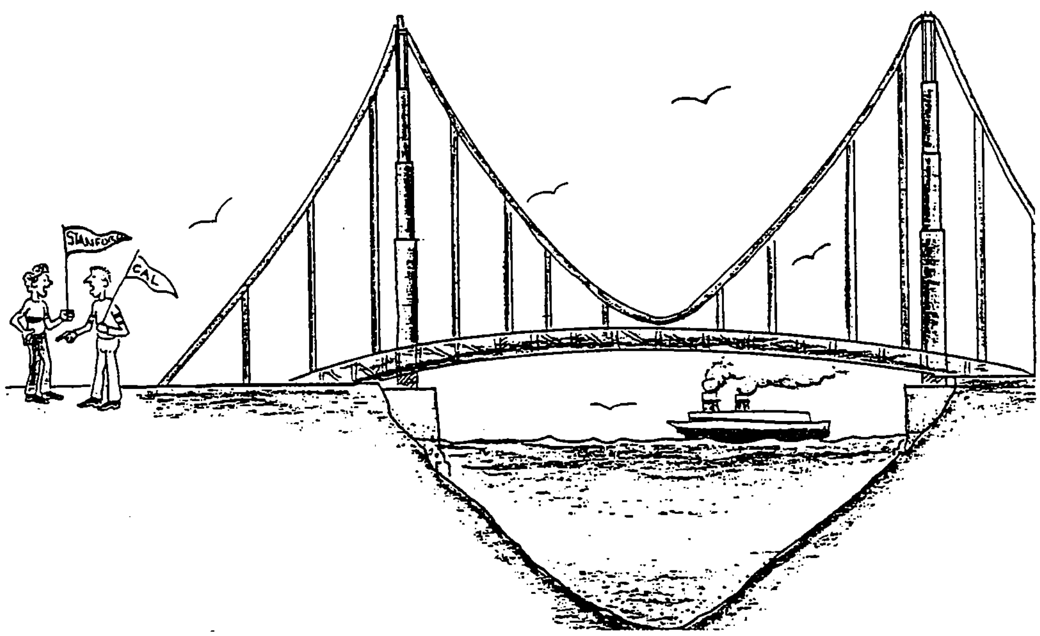

Figure 10. Bailey Willis (Stanford University) and Andrew Lawson (University of California, Berkeley) consider the seismic stability of the Golden Gate Bridge.

enhanced lines of Figure 11 identify those faults along which historical movement has been recorded. In addition to the 1906 event about which a great deal is known, both the Hayward and Calavaras faults which dominate the East Bay area have moved within historic time. Near Hollister just south of San Jose, aseismic creep (tectonic creep) was discovered in 1956. This is a slow, progressive, and cummulative displacement along a fault. Structures built across these creeping faults are deformed and eventually significantly damaged. Although creep is not a violent event, it contributes to the long term damage and increased maintenance costs. The active faults of the area have a history of repeated displacements. Consider the plot of earthquake epicenters for events greater than RM 5.8 within approximately 100 km of the community of Hayward just to the north of San Jose (Figure 12). RM 5.8 or larger magnitude events were selected because this is the earthquake size at which ground failure, ground rupture, and significant damage would be anticipated. The length of historical record within the San Francisco region is short, approximately 150 years, but for that period, it is clear that repeated events are the "norm". A closer evaluation of one event other than the well known 1906 event is appropriate. The epicenter for 21 October, 1868 earthquake was on the Hayward fault. The earthquake had a typically large "felt" area (Figure 13). Within the community of San Jose and on the San Francisco peninsula, Modified Mercalli intensities

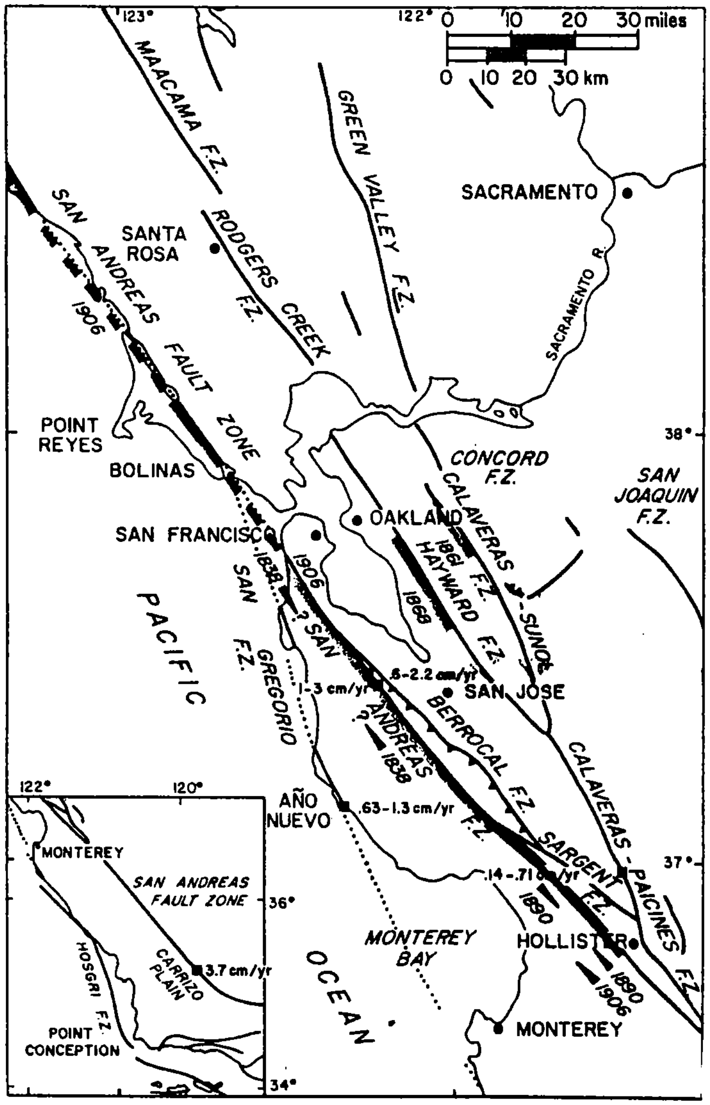

Figure 11. Faults within the San Francisco Bay area for which there is evidence for movement during historic time, approximately the past 150 years (from Slemmons and Chung, 1982, p. 116)

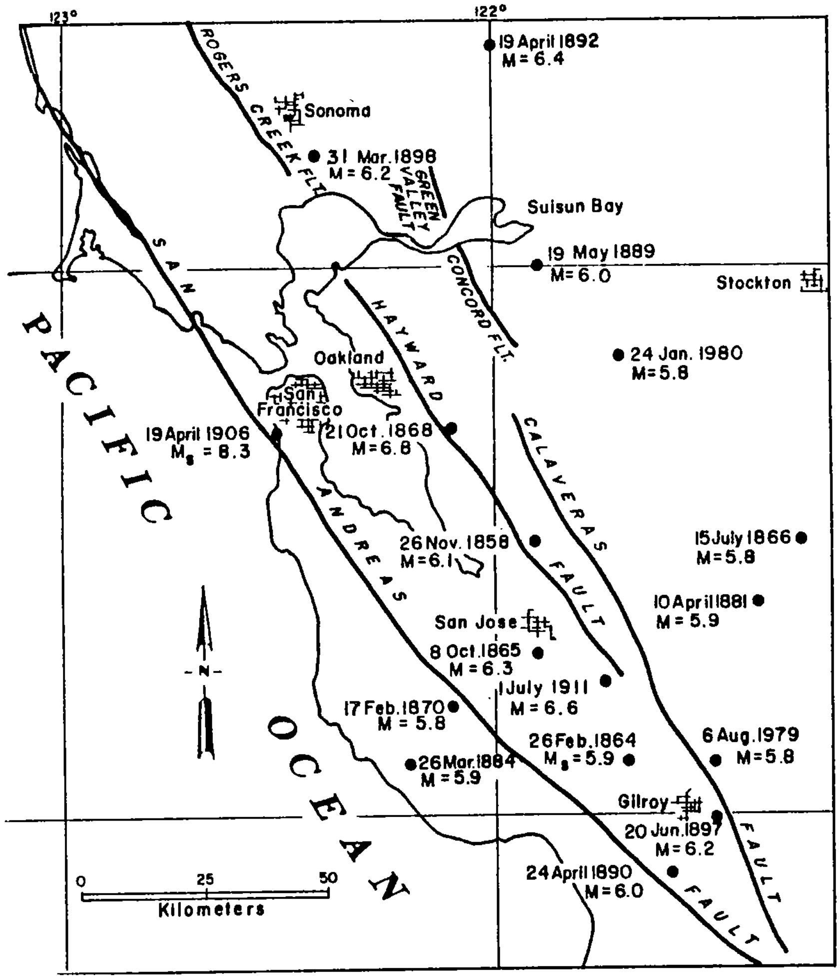

Figure 12. Epicenters for earthquake events with Richter Magnitudes greater than 5.8 which have occured in the San Francisco Bay area during approximately the past 150 years (from Toppozada and Parke, 1982, p. 327).

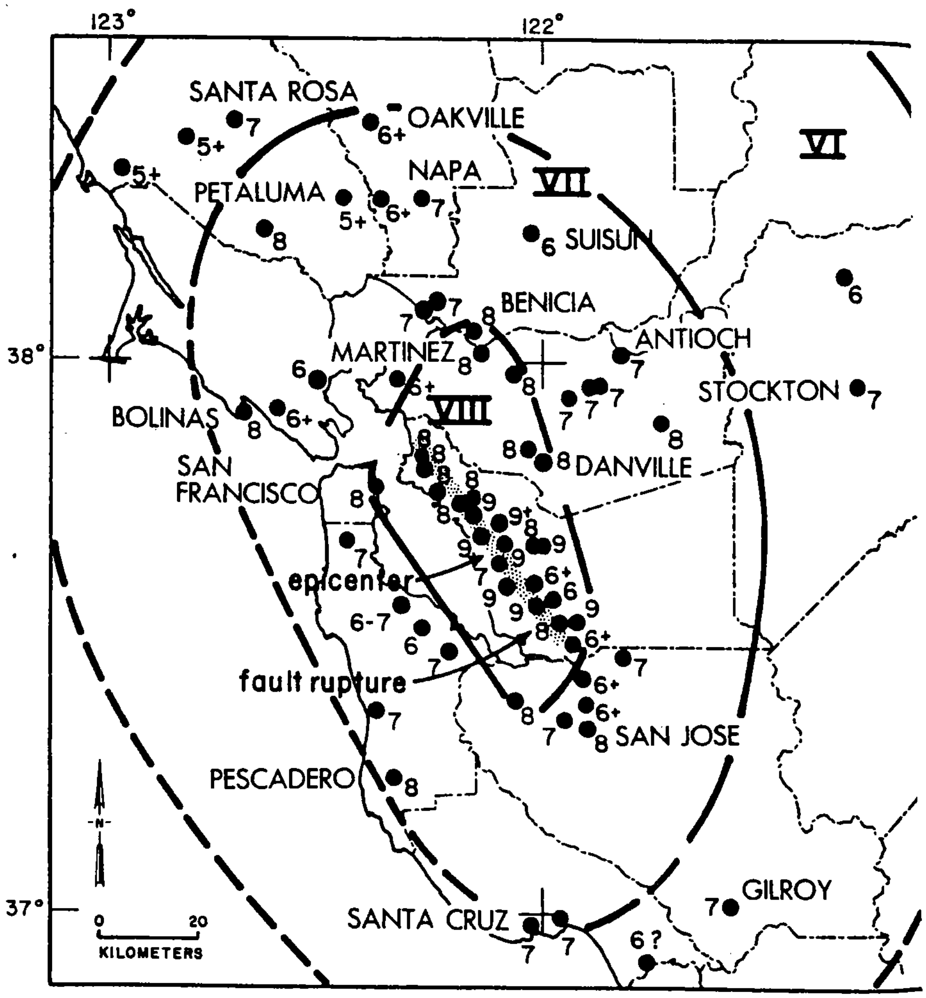

Figure 13. Isoseismal map for the 21 October 1868, earthquake located on the Hayward fault (from Toppozada and Parke, 1982, p. 322).

greater than VIII were reported. The area bounded by the VII Modified Mercalli isoseismal line encompasses most of the currently urbanized region of the San Francisco Bay area.

Our confidence and engineering abilities have led to the belief that construction adjacent to major faults can be safely achieved. For example, in Figure 14, the location of the San Andreas fault, is indicated by the two arrows. The Stanford University linear accelerator, a 2 mile long sensitive engineering structure, was built near the fault and operates satisfactorily. The sinuous line across the photograph is Highway 280, one of the principal north-south freeways in the region.

In addition to the problem of ground shaking and ground rupture, ground failure is associated with seismic events. Drawing upon the work of Youd and others, 1982, the liquefaction susceptabilities of some of the land in the south part of the bay region have been delinated (Figure 15). The land bordering the open water of the San Francisco Bay, has a locally high liquefaction susceptibility because of the presence of shallow, water-saturated, granular materials. The open circles on the figure locate regions in which liquefaction has occured during past earthquakes such as along Coyote Creek in 1906. The unshaded areas are those regions in which liquefiable materials are present but because of the low water table, the region only warrants a moderate rating. The low rated areas are underlain by older, cemented

Figure 14. Vertical photograph showing the location of the San Andreas fault with respect to the Stanford Linear Accelerator. For scale the accelerator is two miles in length. Highway 280 is oriented approximately north- south in the photograph. (Modified from a transparency, Pilot Rock, 1973)

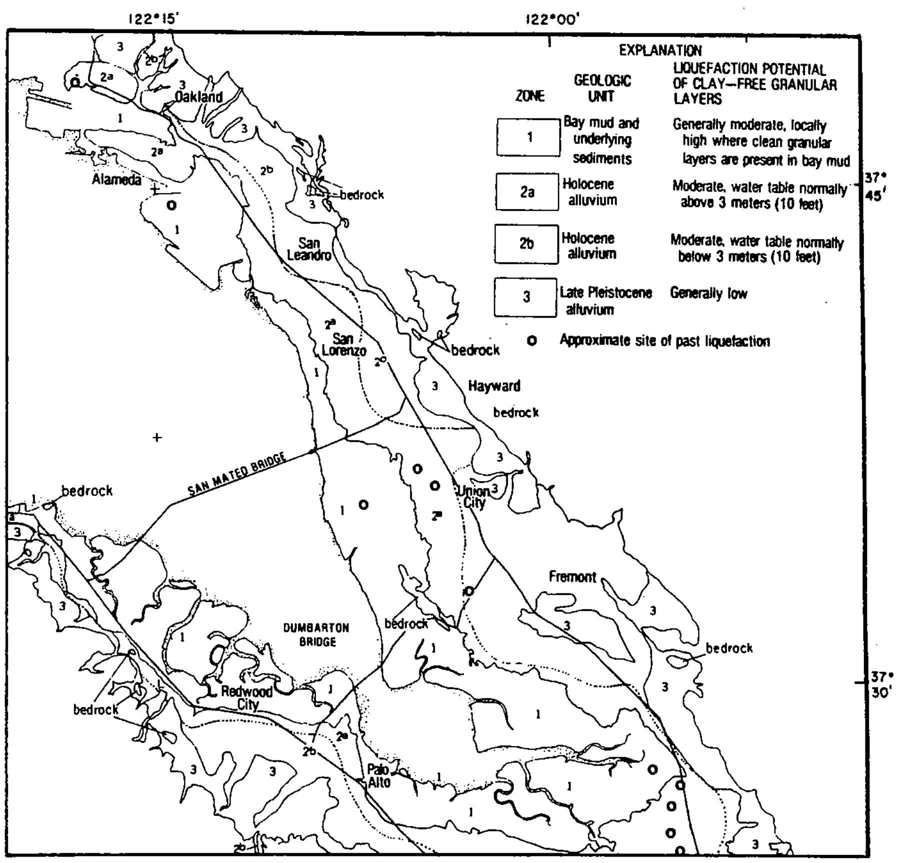

Figure 15. Liquefaction susceptibility for the area bordering the southern portion of San Francisco Bay. Susceptibility is based on the presence or absence of clay-free, well-sorted, water-saturated, fine-grained materials, primarily sands (modified from Youd, 1982, p. 350).

materials, and low water tables. Outboard of the shaded zones are bedrock materials in which liquefaction is not a problem.

Slope instabilities occur because of steep topography, weak rock, (rock which often has been distorted and broken by tectonic activity) and a topography that creates orographic effects leading to the concentration of rainfall and associated runoff. These factors create environments in which weak slope materials become saturated and unstable. Not only are there numerous situations in which slopes fail onto highways and other transportation routes, situations also exist in which the engineered materials within the road fills become saturated and fail (Figure 16). These failures lead to many difficulties such as reducing the ability of residents to move about an area and limiting the access ways that are necessary for the timely response of emergency vehicles in the event of a major disaster.

Flooding is a major problem in portions of the San Francisco Bay area. In the vicinity of the San Francisco airport, floods occur during periods of heavy rainfall and high tide. The same senario occurs in the southern portion of Marin county. A combination of short response times and rapid runoffs from the mountain valleys combined with high tides leads to the flooding of many of the area roadways.

Groundwater withdrawal is a problem for many portions of the Santa Clara valley. As a result of the increasing population

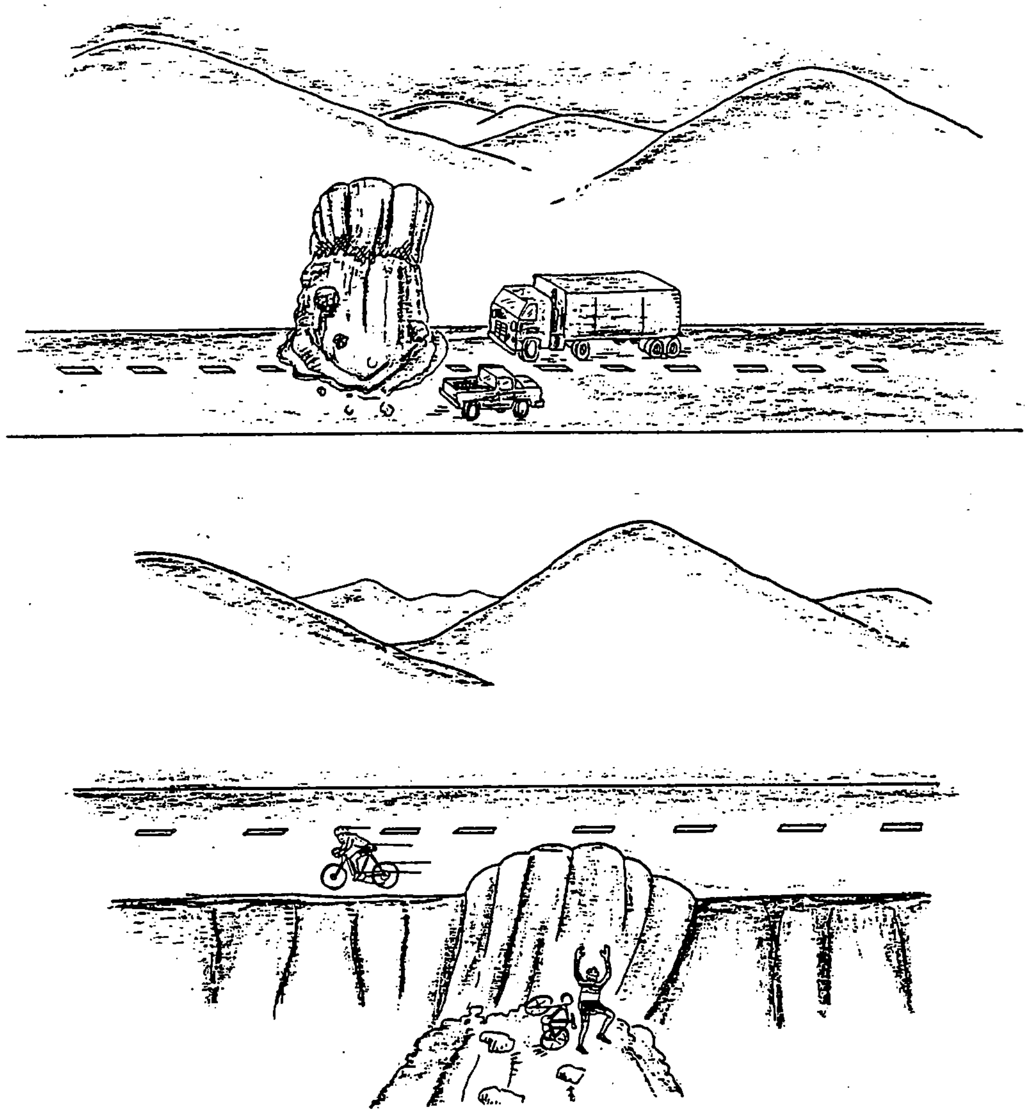

Figure 16. Two principal modes of failure of transportation routes from landslide activity -- failure of slope materials onto the roadways and failure of the road fills.

during the 1900's, the demands upon the groundwater resources greatly increased. The associated pumping lead to the reduction in the pressure head and the compaction of the aquicludes causing ground subsidence (Figure 17). In communities, such as Alviso on the southern shore of San Francisco Bay, this subsidence permitted flooding by marine waters from the bay. Now many homes are protected by dikes which is at best only a short term solution. Within the San Jose area, this subsidence created a large shallow bowl shaped depression with a maximum ground elevation change of -13 feet (Figure 18). This regional subsidence alters the gradients of streams, engineered drainage ways and roadways, and increases the probability of flooding both freshwater and marine. Because of an extensive groundwater recharge program and a carefull monitoring of pumping rates, subsidence was halted by 1970, but could be restarted if care is not exercised in the utilization of groundwater.

The seismically active region bordering the Pacific Ocean has generated tsunamis (seismic sea waves) which have impacted the California coast (Figure 19). These tsunamis can be generated from distant sources such as the Aleutian and the Chiliean trenchs. In addition, waves have been generated from faults near the California coast. An important difference between these events is that when a wave is generated at a great distance a significant amount of warning time exists, permitting the evacuation of exposed coastal

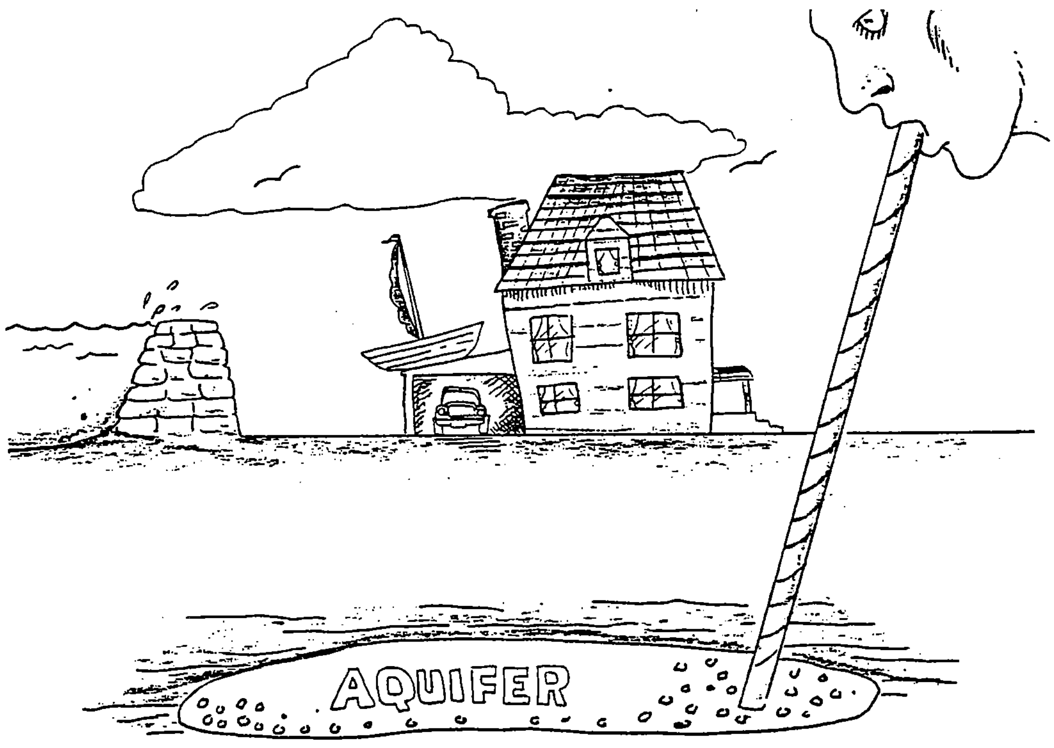

Figure 17. Depletion of the groundwater reserves contribute to the lowering of the ground surface and the increase in the potential for marine water flooding.

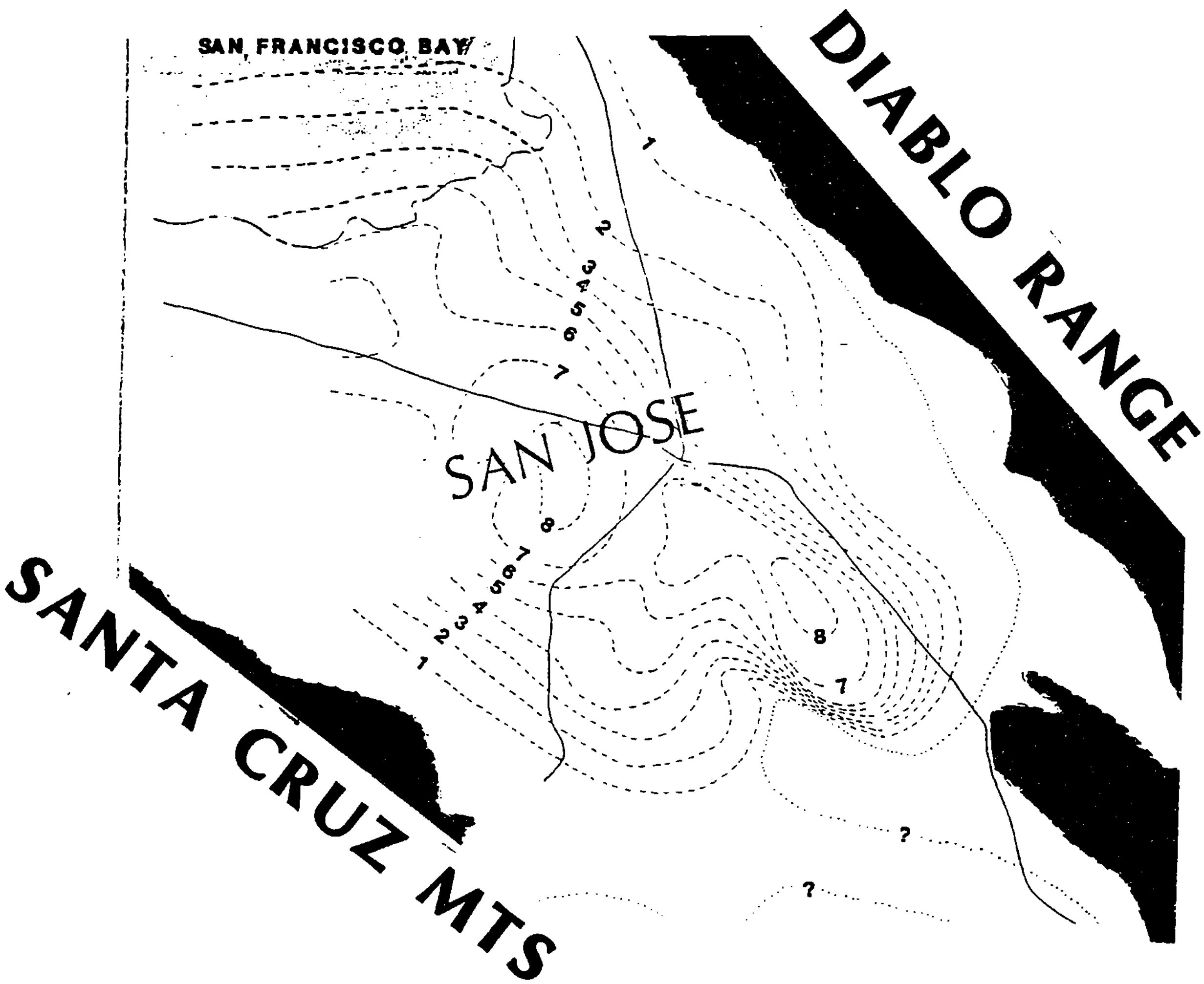

Figure 18. Ground subsidence as the result of groundwater withdrawal for the period approximately 1900-1970. Contours are in feet. Maximum subsidence recorded in area was at one station of -13 feet.

areas. In the case of a locally generated event, the wave arrives with little time for emergency response. The San Francisco Bay is not in itself form a complete barrier to the potential damage of tsunamis. Although the shallowness of the bay does attenuate the waves, the combination of residual tsunami height, high tide, and low lying regions adjacent to the bay creates the strong possibility of significant flooding.

One of the growing problems is the conflict between individuals utilizing the sand and gravel resources to supply our modern industrial society and the residents of our expanding urban environment. Many sand and gravel operations which for years have operated in the "country", now find themselves being surrounded by expanding surburbia. Many residents of these communities consider the sand and gravel operations offensive (Figure 20). These individuals are concerned about the noise, the truck traffic, the dust, etc. and want the pits closed or operations severly curtailed. Once the pits have been paved over and houses constructed, the chances of utilizing these resources are very small. These conflicts will become more and more frequent. They must be resolved taking into consideration the viewpoints of all parties involved.

Because of the lengthy and exposed California coastline, one of the difficulties in developing a transportation route adjacent to the ocean is simply having adequate protection to prevent the

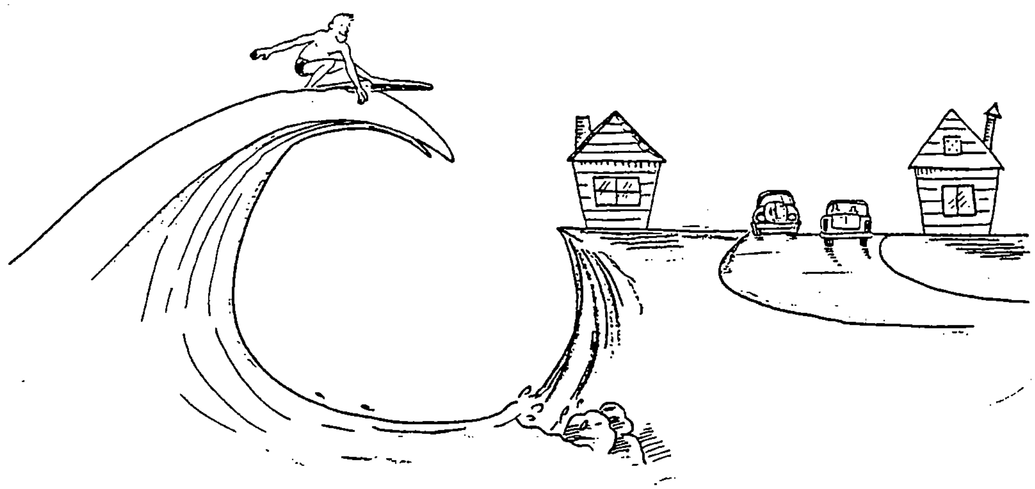

Figure 19. The coastal areas of California have been impacted numerous times by tsunamis (seismic sea waves), some of which have developed locally, others have traveled great distances originating in ocean trenches near Alaska, Japan, Chile, etc..

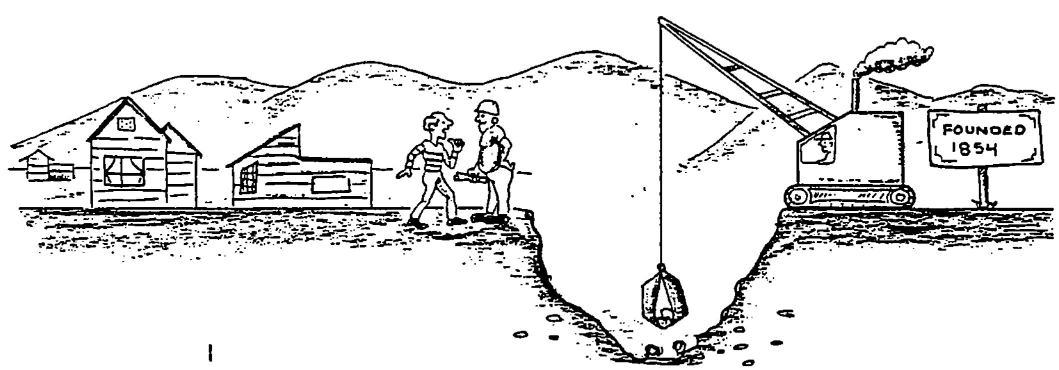

Figure 20. The continuing and growing conflict between gravel pit operators and the residents of expanding suburbia.

ocean from eroding it (Figure 21). Erosion is a natural process. In many places along the coast because of the weak rock, the erosion rate is high. Development of some harbor facilities, groins, jetties, etc., have contributed to the acceleration of the normal rate and the subsequent loss of portions of the coastal highways. An example of this is at the southern end of the Half Moon Bay Harbor. The development of the harbor breakwater intercepted longshore drift and accelerated shoreline erosion. This ultimately led to the loss of a portion of Highway 1 south of the harbor.

What efforts are under way that would allow the increasing population to live with the geologic hazards that are in the San Francisco Bay area? One approach is to use effective planning, to focus the investigative efforts. Because of the limited geotechnical expertise and limited resources (time and money), there is little chance that the entire route of a proposed transportation corridor can be evaluated in infinite detail. The use of maps such as those provided by the government of Santa Clara County, on which areas have been designated in which geologic hazards may exist permits the better utilization of our investigative resources (Figure 22). On this map, areas which are shown as red zones (the darkest grey tones) are those in which geologic hazards are believed to exist and additional geological investigations are warrented. Green zones are areas in which no

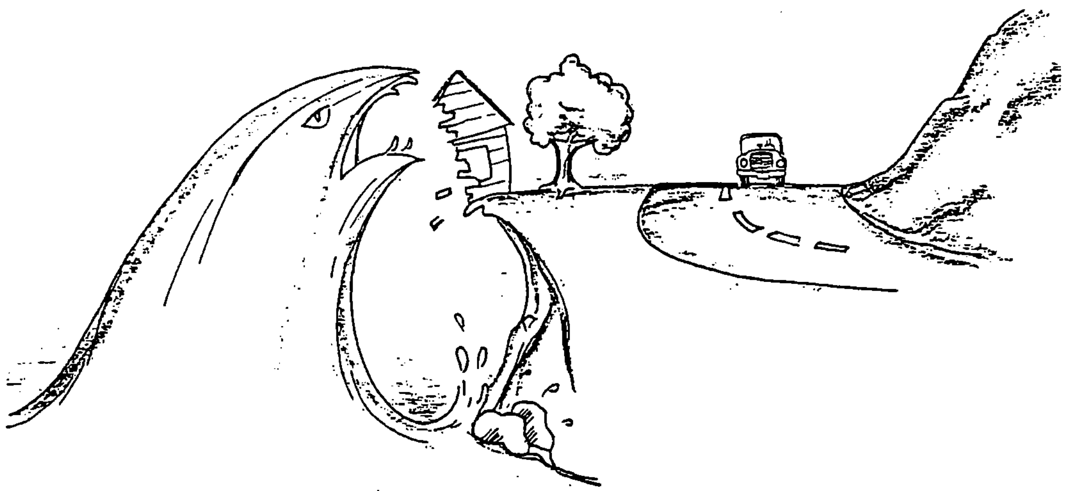

Figure 21. The dynamic Pacific Ocean and its tremendous potential for erosion is a constant threat to coastal projects including transportation networks. Normal erosion rates can be accelerated by man's inappropriate activities or failure to consider the long term consequences of the action.

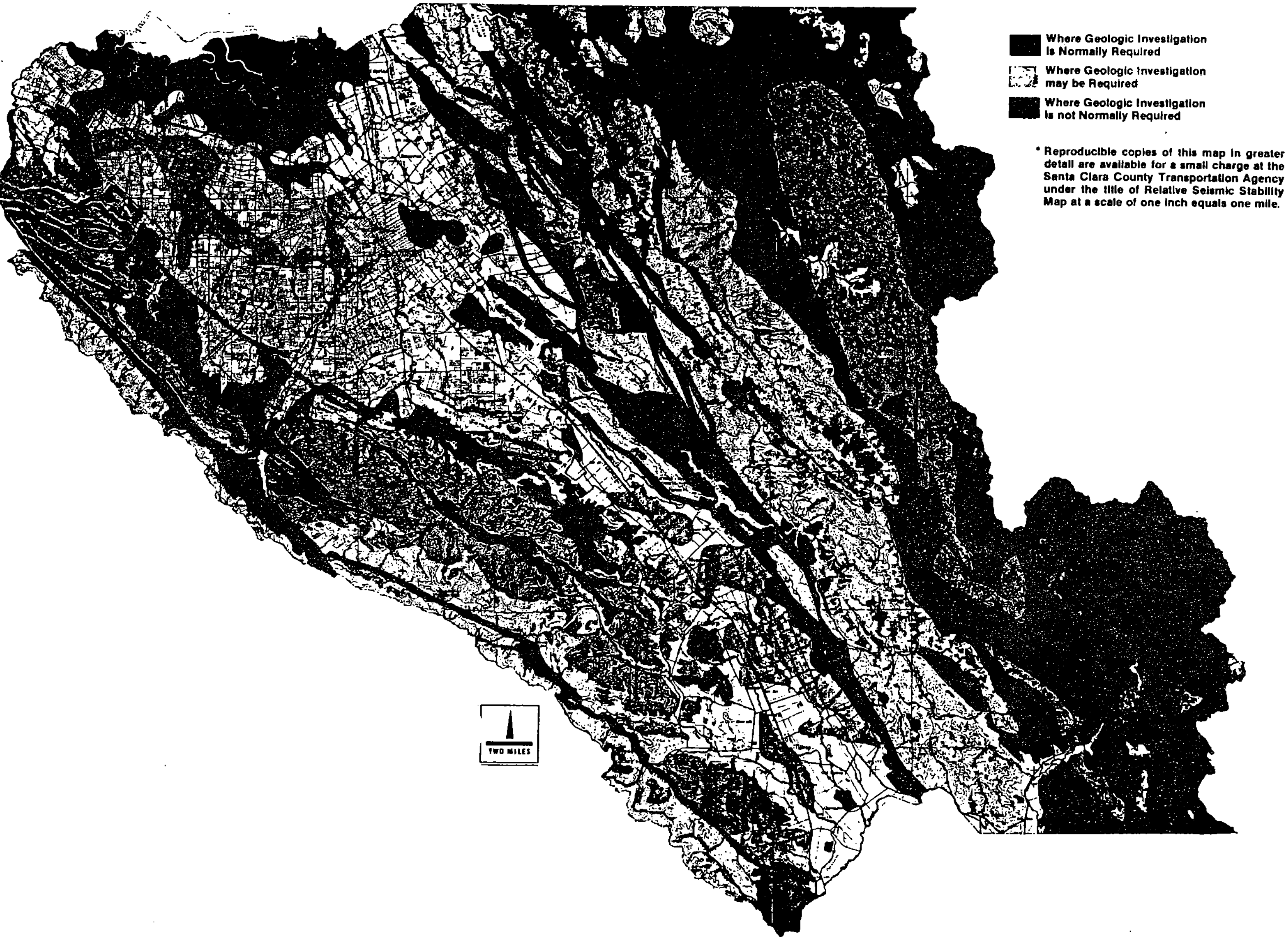

Figure 22. "Investigative Needs" map, available from the Santa Clara County government, indicates those areas for which geologic reports would be appropriate. The requirement for a geologic report is based on generalized knowledge about the presence or absence of particular geological hazards within a selected area (Santa Clara County, 1976)

hazards are known to exist and additional geologic work probably is unnecessary. The yellow zone, a cautionary zone, is one in which specific information is lacking; caution should be exercised and additional geologic investigation may be prudent. Much about the activity and the mechanics of fault rupture has been learned during the Alquist-Priolo special studies which were required by the state legislature following the 1971 San Fernando earthquake. As a result of this legislation, the State Geologist was required to establish zones approximately 0.25 miles in width, along faults which are believed to be potentially active. Within these zones, special geological studies are required to insure that no structures for human occupancy will be built on ground capable of undergoing fault rupture.

The California Division of Mines and Geology completed a study of the possible impacts of a Richter Magnitude 8.3+ earthquake on the San Francisco Bay region. Most of the major transportation routes within the area would have at least local blockages (Figure 23). Some of the more significant routes such as Highway 17 in the east bay and Highway 101 on the Peninsula could sustain damage that would require long term repair. The trans-bay bridges are believed to be capable of withstanding a major earthquake. The problem will be the bridge approaches, many of which are located on weak, water-saturated material and are likely to fail making the bridges useless. In addition to the blockages that will occur along the

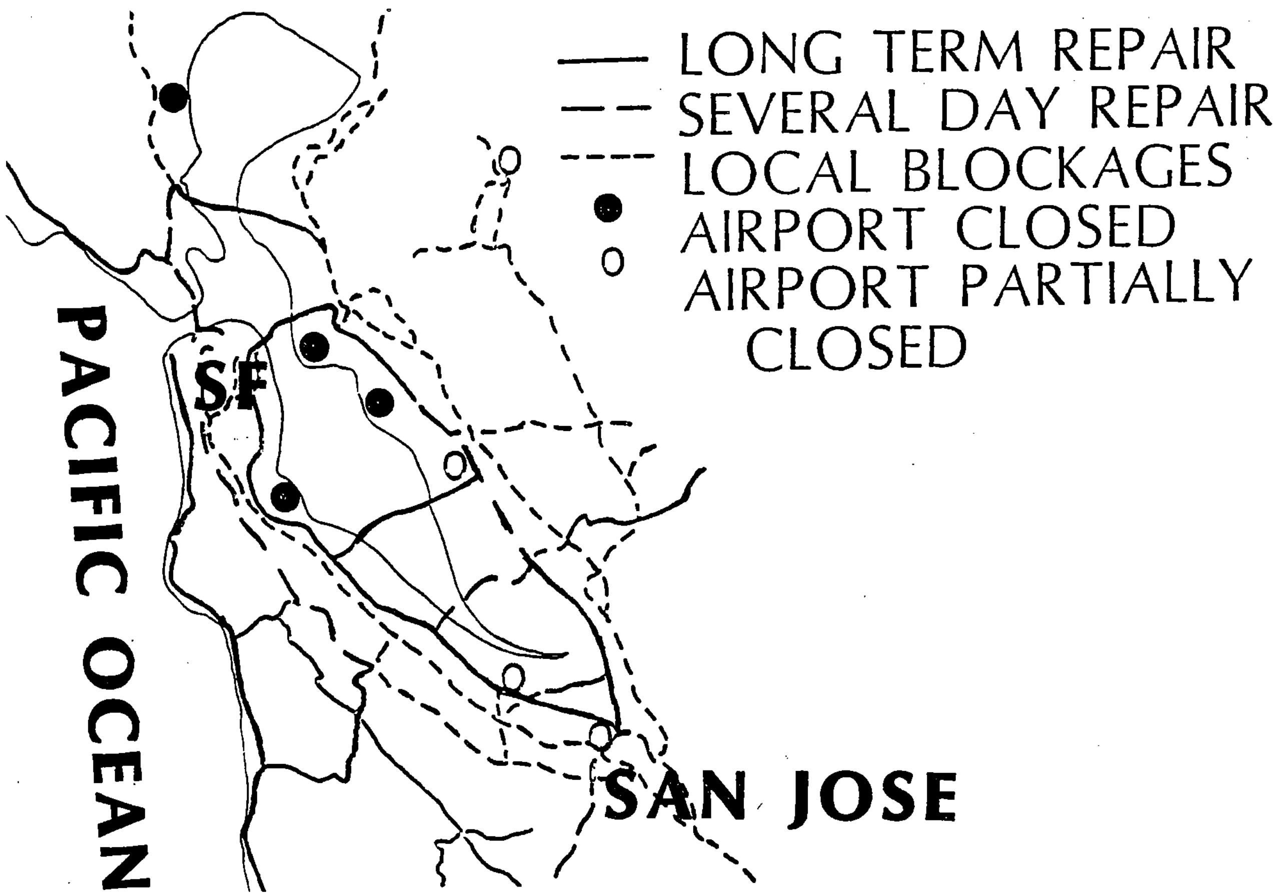

Figure 23. The potential impact of a Richter Magnitude 8.3+ earthquake on transportation routes within the San Francisco Bay region (modified from Davis, 1982, p. 337).

freeways, the major airports, primarily San Francisco, Alameda, and Oakland probably will be closed because of damage to the runways resulting from the failure of the weak, water-saturated materials underlying them. Moffett Field and San Jose airports probably will operate at reduced levels because of similar failures.

One of the factors that must be considered in designing transportation corridors and routes in California is the conflict that frequently arises between the geotechnical values and "other" considerations. Much of California's population is well-financed, well-educated, and interested in a variety of environmental concerns. Frequently geotechnical engineers are chagrined when a final route is selected based on factors other than geotechnical considerations. An example is the controversy that exists between a road location which is unsatisfactory because of slope instabilities and the only reasonable alternate road location which threatens some endangered species (Figure 24).

The building of transportation systems within the San Francisco Bay region faces a number of challenges. These challenges to transportation development, range from ground shaking, ground rupture, and ground failure, to problems involving erosion, flooding, landsliding, and the important and frequently overlooked problem of having an adequate supply of sand and gravel. The approach that is suggested for overcoming these difficulties lies in part in better geologic mapping. There is the need to

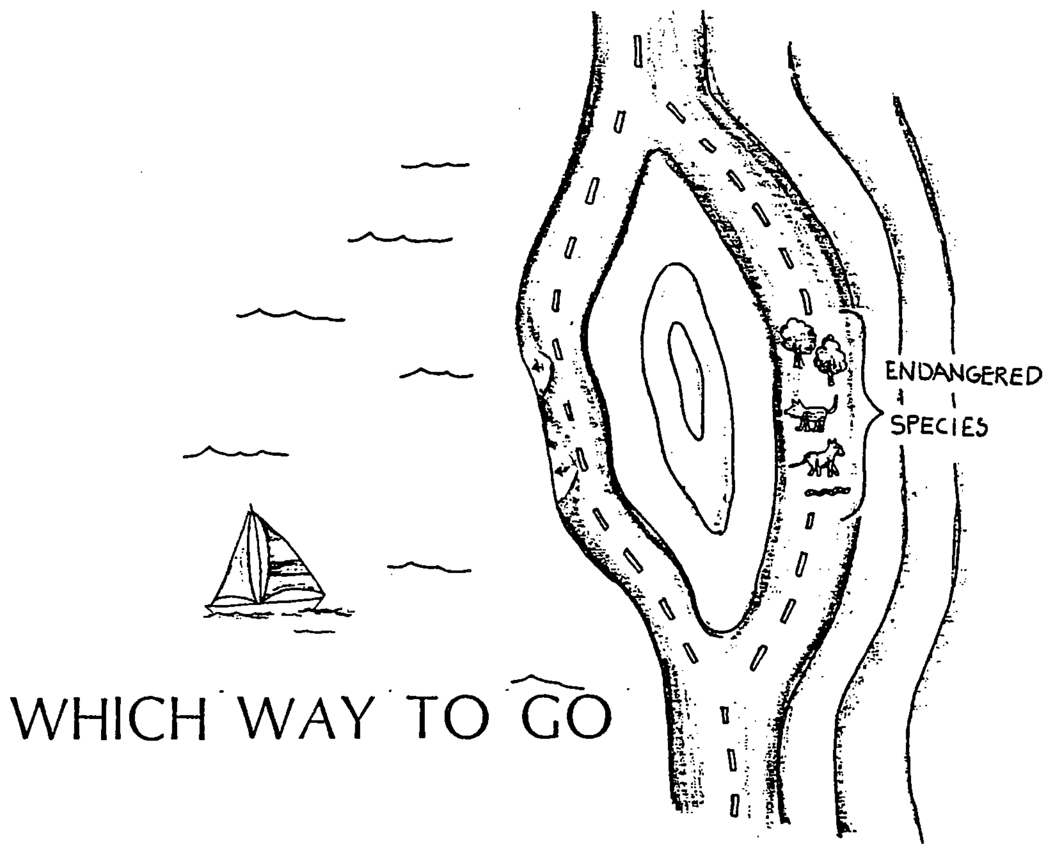

Figure 24. Map suggesting the type of conflict that often develops over the selection of a highway route when different significant factors are involved. Which factor should be considered the more important? Who should make the final decision? How should the final decision be made?

know where the hazards are located. Not only is there the need for better mapping, but also the need for better understanding of the mechanics of each of these of hazards. Hopefully better mapping and better understanding will produce better planning such that the initial costs as well as long term expenses can be kept to a minimum.

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1984 INVENTORY OF FOOTHILL LANDSLIDES

SANTA CLARA COUNTY, CA

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ABSTRACT

Recently active landslides in Santa Clara County were inventoried as part of a joint study with the Santa Clara County Geologist and the Office of Planning and Land Development. Interest in this project evolved from the County's need to document slope behavior following the heavy rains of the 1982-83 seasons. The study area consists of the foothills bordering the Santa Clara Valley. Numerous faults transect the hills, producing extensive zones of sheared and weakened bedrock. The study area was divided into 3 segments; East, West and South. Slides within each segment were mapped by Young, Vassil, and Sparrowe (respectively) at a scale of 1" = 500'. Data compiled for each slide include dimensions, type of failure, lithology, and structures involved.

The project provides a basic data base for future development, zoning and seismic hazards planning. Numerous types of failures, including rotational slumps, mud and debris

flows, and block falls were observed. Clusterings of slope failures occur near faults and under certain lithologic and environmental conditions. Data generated by this study should be interpreted with care due to limitations which include a) the broad, regional scope of the project, b) limited access to large private tracts, c) the high density of vegetation and residential structures in some areas, and d) owner-effected repairs which often masked evidence of slope failures.

INTRODUCTION

Santa Clara County is located at the south end of the San Francisco Bay. It encompasses the floor of the Santa Clara Valley, the bay marshlands, and the foothills of the Diablo and Santa Cruz Mountain Ranges (Figure 1). Higher than average precipitation during the 1982 and 1983 rainy seasons triggered numerous landslides within the bordering mountain and foothill regions. This paper reports on a landslide mapping and indexing project undertaken by the authors at the request of the Santa Clara County Geologist and Office of Planning and Land Development. The study is intended to serve as a planning tool, and as a rudimentary data base correlating locations and types of slope failures with geologic, geographic, environmental conditions.

REGIONAL GEOLOGY

Tectonics

The dominant feature, governing structure and tectonic activity in Santa Clara County, is the San Andreas fault system. This northwest

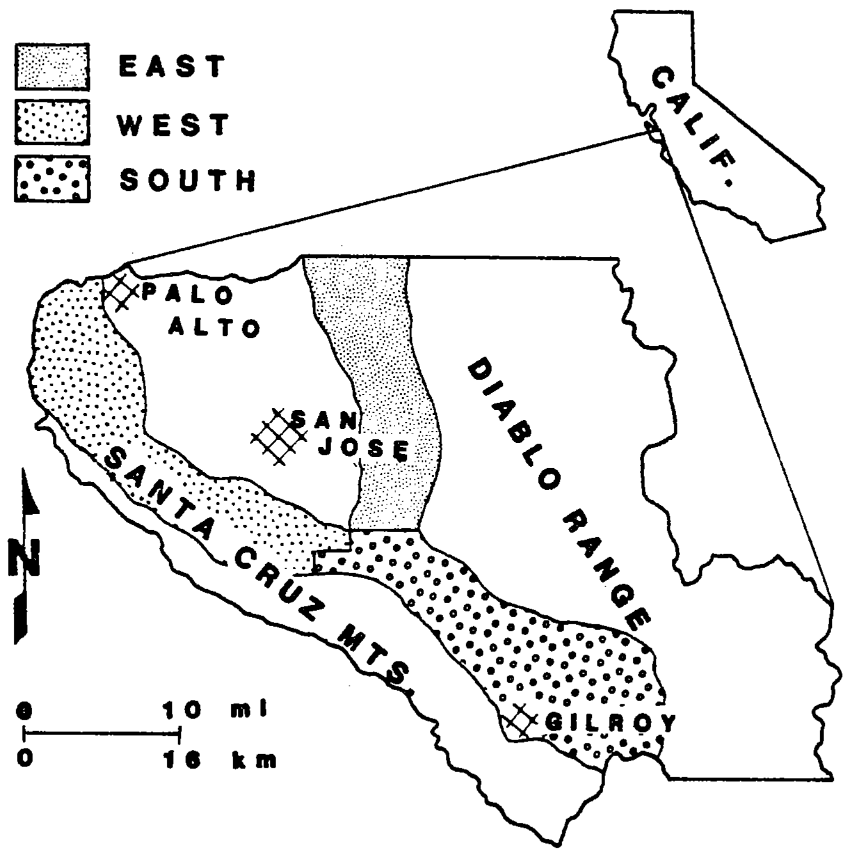

Figure 1. Landslide inventory study area.

trending fault system delineates the boundary between the North American and Pacific plates. It exhibits a right lateral sense of motion as the Pacific plate slides northwest past the North American plate.

The San Andreas fault runs from near the southernmost portion of Santa Clara County, through the Santa Cruz Mountains and foothill regions of Los Gatos, Saratoga, and Los Altos Hills. It is closely associated with numerous smaller, stress-related, strike slip and thrust faults. Towards the extreme south end of the County, the Calaveras fault zone splays off the San Andreas fault, near Hollister, CA. The currently active Calaveras and its subparallel related faults cut across the foothills of the Diablo Range along the east side of Santa Clara County, from Gilroy and Morgan Hill, to East San Jose and Milpitas. These faults and the Hayward fault, continue beyond Santa Clara County and up along the eastern side of the San Francisco Bay Area.

The uplift of the Coast Range to the west and the Diablo Range to the east of the valley is considered a result of subduction along the western boundary of the Pacific plate. The compression which accompanied the collision between plate boundries highly folded and faulted the materials which make up the Diablo and Coast Ranges. In the early Miocene, the modern San Andreas fault came into existance. Major strike-slip faulting further completed the compressional folding and faulting which had occurred earlier.

Lithology

The most widely occurring formations in the county are the Santa Clara, Berryessa, and Franciscan. The Santa Clara Formation consists of weakly consolidated sandstones, siltstones, and conglomerates. Landslides are common in these soft, easily eroded sediments. The Berryessa Formation is predominatly shale with some sandstone. It occurs between traces within the Calaveras fault zone. Further to the south and along the western margin of the study area, the Franciscan Formation is enountered. The Franciscan Formation is a highly folded, sheared and faulted complex which contains shales, sandstones, cherts, schists, serpentines, and altered volcanics. Other geologic formations in the area include carbonates, basalt, alluvium and landslide deposits.

METHODS OF DATA ACQUISITION

Due to the size of the foothills region, the County was divided into three subregions; East, West, and South. The East County, mapped by Young, included the Diablo Range foothills from Milpitas to South San Jose. The West County, mapped by Vassil, extended along the foothills of the Santa Cruz Mountains from Palo Alto and Los Altos Hills in the north, through Saratoga and Los Gatos, to San Jose in the south. The South County region, mapped by Sparrowe, included the area from South San Jose to Morgan Hill and Gilroy, where the Santa Cruz and Diablo Coastal Ranges come close to coalescing (as do the main traces of the San Andreas and Calaveras faults).

Topographic roadmaps at a scale of 1" = 500', with 20' contour intervals were provided for each region. Field work involved driving through the study areas, searching for indications of recent sliding. In areas of rangeland and open space, recent slides were easily distinguished through fresh scraps, toes, and disrupted vegetation. In the more developed areas, fresh scarps were visible primarily in only the largest and most active landslides. It became necessary to "see through" the effected repairs on developed slopes. Slides were often mapped on the basis of a combination of new retaining walls, fresh arcuate tension cracks in roads and driveways, new asphalt, displaced curbs and sidewalks, disrupted vegetation, fences and signposts, distressed houses, and drainage diversion projects.

Slides were mapped using red ink, on the topographic road maps. Arrows, indicating the direction of movement, and identification numbers for each slide were included on the maps. Numbers cross-indexed the observed slope failures with data sheets that were compiled for each slide (Figure 2). Data sheets, devised by the Santa Clara County Geologist, were utilized to catalogue each slope failure with respect to location, nature of failure (mudflow, rotational slump, debris avalanche, etc.), dimensions, slope gradient, lithology, drainage structures involved, age of slide, and potential risk to life and property. All roads actually traversed in each area were indicated on the maps in green ink. Potential slope failures, where suggested by topography, drainage and vegetation, were mapped in blue ink. Figures 3 and 4 illustrate a well defined rotational slump east of Morgan Hill and the corresponding map segment.

The three subregions were selected on the basis of geologic, geographic, and environmental characteristics that identified them as distinct units. These Santa Clara County subregions will be examined more closely in the next three sections.

EAST COUNTY FOOTHILLS

Background

The East County is characterized by broad, low, undulating foothills and alluvial fans. The hills form the base of the Diablo Range, whose highest peak, Mount Hamilton, reaches an elevation of 4213 feet. Near its crest, the Diablo Range receives an average annual precipitation of 25 inches. The valley floor and lower foothills receive an average of 14 inches

1983/84 Landslide Inventory Data Sheet: Santa Clara County

O 1. Slide Code (map quad — slide no.): ________ — ____ Inventory Date: ________

O 2. Location: ____________________ Road ____________________ Map Coordinates

O 3. Slide Type: Rotational Slump Skin Slip
(Circle) Mud Flow Debris Avalanche
Composite Block Glide Rock Fall

O 4. Approximately dimensions: ______ length ______ breadth ______ thickness yards^3

O 5. Repair work evident (Check): yes ______ no ______

O 6. Approximate slope adjacent hills (Circle):
0-5%; 6-10%; 11-15%; 16-20%; 21-25%; 26-30%; 31-45%; above 45%.

O 7. Vegetative Cover (Circle):
Bare Rock, Bare Soil, Grass, Chaparral, Trees-Brush, Wooded, Dense Forest, Landscaped

O 8. Water Barriers (Circle):
springs, ponds, "blue-line" streams infringed upon, landslide dam

O 9. Construction Involved (Circle):
Grading, Building Pads, Homes, Appurtenant Structures, Fences, Power Poles, Sewer Lines, Septic Systems, Water Lines, Natural Gas Lines, Bridges, Driveways, Diverted Drainage, Publicly Maintained Roads, Other: ____________________.

O 10. Age of Slide:

Pleistocene,	Ancient,	Prehistoric,	"Old"	Recent,	Active
10,000 yrs.	(1000-10,000 Yrs.)	200-1000 yrs.	(19th Century)	(1900-1980)	1981-84

O 11. Risk: To Property: 0, 1, 2, 3, 4, 5. (5 Max.); To Life: 0, 1, 2, 3, 4, 5. (5 Max.)

O 12. Estimated Damage (to date):
(Circle) 0 - $500; $500-$5,000; $5,000-$50,000; $50,000-$500,000; $500,000+

X 13. Date of First Historic Movement (if known): ____________________ Date and Reference

X 14. Previously Mapped: ____ No ____ Yes ____ Unknown Source of Information ________ (circle & specify) USGS

CDMG, SCWD, SCCO., Thesis Other

X 15. Soil(s) Unit(s) Involved: ____________________ From Soil Conservation Service 1,000-Scale Maps

X 16. Relative Seismic Stability Rating: DS, ES, FS, Other: ____________________ (From Seismic Safety Element Maps)

X 17. Bedrock Formation Involved: ____________________
(Circle & Specify) From Maps by U.S.G.S.; CDMG; Thesis

Site Specific Reports Other

X 18. Annual Rainfall Contour: ____________________ inches
From Isohyetal Maps

X 19. Comments re: cause, recommendation (if any):

X 20. Name of Observer: ____________________

O = Field Observation
X = Data from office sources

Figure 2. Landslide data sheet.

Figure 3. Rotational slump east of Morgan Hill.

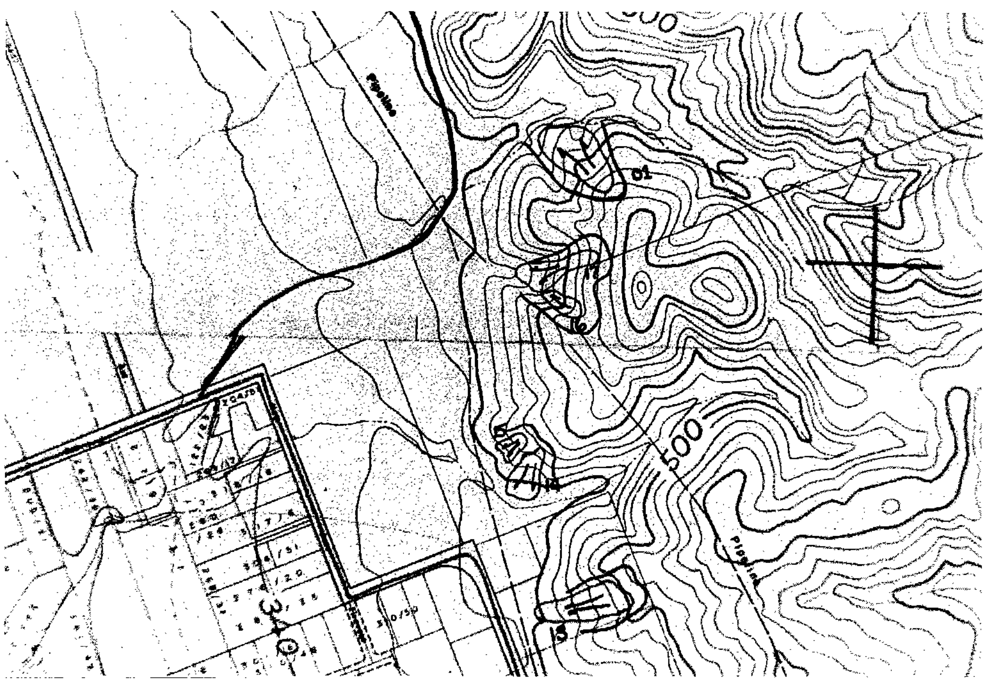

Figure 4. Rotational slump (01) illustrated on contour map.

of rainfall annually. Most of the rain falls in a few moderate to high intensity storms during the rainy winter season. The season ranges from October to April, and is followed by a relatively warm (80 - 100 degrees), dry summer. Oaks and shrubs line the stream channels and the hills are covered by grasslands which are used primarily for grazing purposes. During the dry summer months, the parched hills are especially susceptible to grass fires. When these occur, vegetative cover is destroyed. Removal of vegetative cover probably contributes to slope instability problems during the wet months.

Drainage from the Diablo Range flows into the San Francisco Bay. Small channels coalesce to form streams crossing broad alluvial fans at the valley floor. Response times for these small drainages tends to be rapid. During the high intensity rains of January 1982 and March 1983, channels overflowed. In Berryessa and Milpitas, runoff was seen pouring off the hills as sheetwash during peak discharge periods. In both events, high antecedent rainfall had saturated the slope prior to the damaging storms. Numerous old landslides were reactivated, and new ones formed as a result of the high intensity rains.

Slope failures in the East County

The east foothills are pocked with the scars of numerous old landslides. Rotational slumps into drainage channels are common. Large scale, shallow seated slumps spill onto the valley floor. This imparts: a hummocky, irregular appearance to the terrain along the base of the foothills. Many of these slides occur in the Santa Clara Formation as well as within the material of the Calaveras fault zone.

In San Jose's Berryessa section, Piedmont Road follows the base of the foothills, parallel to the Calaveras fault. Following the rains of early March 1983, a scarp appeared along the shoulder of Piedmont Road, in the toe area of what appears to be a large, shallow old slidemass. The new slide, a rotational slump, necessitated removal of a single family residence whose backyard abutted against Piedmont Road. The scarp propagated upslope, onto Piedmont Road, affecting both the road and utility lines. The shallow slump developed a toe which accumulated beneath the house and extended into the cul-de-sac on which the property fronts. Although the house has been removed, and a relatively dry winter followed the unusually wet 1983 winter, the toe continues to accumulate. It has since disrupted the road repairs as well as the previously uninvolved homes on either side of the now abandoned property.

The most destructive active landslide in the east foothills occurs along Boulder Drive, in the San Jose Highlands, near Alum Rock Park. The area was developed in the early 1960s, as a quality residential neighborhood. Slope stability problems plagued the development from its earliest phases. Distinct "problem areas" appeared as slow downslope movement, during the 1970s, damaged residences and disrupted roadways, limiting access into and out of the community. Repair crews persisted in maintaining the chronically distressed road segments. Following the 1982 and 1983 rains, the rate of sliding accelerated substantially. A number of residences, as well as the lower portion of Boulder Drive, have since been abandoned.

The San Jose Highlands slide mass dramatically illustrates processes observed in many of Santa Clara County's foothill regions. A large, pre-existing slide mass has become signficantly more active, developing new scarps following the record setting rains of 1982 and 1983. The Boulder Drive slide was known to be active prior to the heavy rainfall years. In other areas, especially along the western margins of the County, new slides occurred within slides previously mapped as "old", and "dormant".

The rolling topography and semi-arid climate have combined to make the East County prime agricultural land. Generally, the steeper slopes support livestock grazing and the base of the foothills supported large orchard tracts. During the last two decades, the burgeoning Santa Clara Valley electronics industry has attracted a major population influx. Demographic needs have switched land use along the base of the foothills from agricultural to residential and light industrial-commercial usage. Continued growth is pushing the residential sector higher into the hills. Although the east foothills have experienced chronic slope stability problems throughout their known history, landslides were not considered a problem until development placed lives and property in their paths.

WEST COUNTY FOOTHILLS

Background

The Santa Cruz Mountains are fault block mountains whose steeper, upturned slopes border the Santa Clara Valley. Alluvial fans cover much of

Saratoga, Los Altos, Los Gatos, and parts of Cupertino. The main trace of the San Andreas fault passes through Los Gatos, Saratoga, Los Altos Hills and Palo Alto. The foothills exhibit high relief, are heavily vegetated, and densely populated.

Moisture laden oceanic air masses drop approximately 50 inches of rain per year near the summit of the Santa Cruz Mountains. High intensity winter storms account for most of the precipitation. Numerous steep canyoned streams facilitate drainage. Much of the flow is directed into flood control and water storage catchments. These include Uvas, Almaden, Guadalupe, Vasona, Lexington and Stevens Creek reservoirs. Water in the reservoirs is used for recreation and for groundwater recharge. Much of it eventually drains into the San Francisco Bay.

Increased demands for housing during the last 15 to 20 years has led to extensive development in the West County foothills. "View" lots in this part of the County comprise the area's most expensive real estate.

Landsliding in the West County foothills

As a result of the high precipitation of 1982 and 1983, the western portion of Santa Clara County supports a greater number of landslides per unit area than either of the other regions studied. Most of the western slides occur within large, old slide masses closely associated with the highly sheared, folded and faulted units bordering traces of the San Andreas, the Berrocal thrust, and the Sargent faults. Large colluvial deposits associated with the older slide features developed numerous failure planes following the saturated conditions of the 1982 and 1983 winters.

Sliding occurred frequently in road fill, especially where gullies or creeks concentrated runoff. Foundation and driveway fill failures were also triggered by the high amounts of rainfall. Slumping into creek banks distressed many of the roads, driveways and structures in the West County foothills. Outcrops of the notoriously slide prone Santa Clara Formation behaved characteristically poorly during the wet winters. A number of landslides involving roads, property and residences in the Mt. Eden - Sarahills - Michaels Drive sections of Saratoga appear to be colluvial failures associated with older slides in the Santa Clara Formation. Cut slopes along Interstate 280 sustained a number of failures, especially in the vicinity of Stanford University. The nearby chronically slipping Elena Landslide flowed onto I 280, necessitating major repairs in an attempt to stabilize the failure. Slope failures damaged roads, structures and property sites in the hilly, residential neighborhoods near Los Altos City Hall. Destructive landslides occurred on the ridges and in the ravines of Los Gatos and in the Almaden Valley Area.

The largest active landslide in the West County is the Congress Springs landslide, in Saratoga. The slide mass involves a block glide failure whose glide plane is at a depth of approximately 200 feet. The 50 million cubic yard slide occurs within a larger, ancient landslide. Due to dense vegetation, development, steepness of terrain and the massiveness of the feature, it is difficult to percieve the entire Congress Springs slide in the field. It appears to consist of many differentially moving segments. Some residences and roads have suffered distress since the 1982 and 1983 seasons, while other structures within the slide mass are being rafted, intact, downslope.

Intense development, thick vegetation, and slide repairs masked many of the recent landslide features in the West County. Slides were often mapped on the basis of "suggestive evidence" rather than actual slide features. When a combination of freshly disrupted vegetation, distressed structures, curbs, sign posts and fences coincided with new repairs on roads and driveways, arcuate tension cracks, new drainage culverts and conduits, new retaining walls, and major landscaping projects, a site was interpreted as having had recent landsliding activity.

Where slides are clearly visible, it is apparant that they usually manifested as debris slides and flows, or as composite features. The composite slides were most common. They tended to include rotational slumps in their upper portions, with oblate mudflows extending downslope.

SOUTH COUNTY FOOTHILLS

Background

The southernmost portion of Santa Clara County extends from Coyote, south of San Jose to Gilroy.The Santa Clara Valley ends south of Gilroy, where a topographic high creates a divide between water draining northward, into the San Francisco Bay, and water draining southwestward, into the Monterey Bay. Coyote Creek, which discharges into the San Francisco Bay, provides the major drainage system. Two foothill reservoirs, Anderson and Coyote, store water for recreation and flood control in the South County foothills. Both reservoirs are situated close to active faults.

Traces of the Calaveras fault pass beneath the right abutment and spillway of Coyote Dam, as well as beneath Cochrane Bridge at Anderson Lake. Most of the County's recent earthquake activity has taken place along the Calaveras fault, in the vicinity of Mount Hamilton, and southward, towards the Hollister area. The April 1984 shock of magnitude 6.3 was focused in Halls Valley, along the Calaveras fault. Halls Valley constitutes a linear feature, trending northwestward through the Diablo Range, between Morgan Hill and Mount Hamilton.

Major land use patterns in the South County continue to remain agricultural. The foothills are used as grazinglands, while the valley floor is largely cultivated. Current population trends are for residential development and population growth. Projections for the next 10 to 15 years anticipate that growth and development will accelerate rapidly in the South County.

The South County combines many of the characteristics of the east and west foothill areas to the north. The southeast hills support rangelands, with oaks and shrubs mostly in the drainage courses. This is the side that hosts the Calaveras and its related faults. The southwestern hills are more heavily vegetated, drain the Santa Cruz Mountains, and are closely associated with the San Andreas fault.

Landsildes in the South County

South County landslides include rotational slumps, mudflows, composite slump-flows, surficial skin slips, block falls and debris avalanches. Failures have been observed in the fill prisms of roadways, and along natural gas pipelines. Reservoir induced sliding occurred along the margins of Anderson Lake during the drought years of the mid-1970s, when water levels were low and continue today at a diminished rate. Old and active landslides are common in the foothills near both Anderson and Coyote reservoirs.

The rotational slump illustrated in Figure 4 occurred in the foothills of Morgan Hill. The fresh head scarp represents the latest failure episode on a slide mass that has been moving for over twenty years. The slump occurs in the Santa Clara Formation. It is associated with a spring from which water is piped into a storage tank on the valley floor. The Santa Clara Formation, composed of very recent (Lower Pleistocene) loosely consolidated nonmarine gravel, sand, clays, siltstones, and basaltic volcanic rocks is prone to failure, and hosts many rotational slumps and debris flows where it occurs. The slides drawn in this figure occur in a northwest trending outcrop of the Santa Clara Formation.

Since the South County is closely bounded by both the San Andreas and Calaveras faults, it is highly susceptible to landsliding induced by seismic shaking along any of the fault traces transecting the area. Several slope failures were observed near Anderson Reservoir in Morgan Hill following the April 1984 quake. Fill failures damaged Dunne Avenue near Cochrane Bridge. Rock avalanches fell onto Cochrane Bridge from distinct avalanche chutes in the Berryessa Formation. The north trace of the Calaveras fault passes beneath Cochrane Bridge. Fault creep, estimated at approximately 2 cm per year for the Calaveras fault, has kept the bridge in a state of chronic distress since it was first constructed. Structural damage to the bridge following the 1984 earthquake necessitated closure and abandonment of the structure.

Landslides in the South Country are closely associated with changes in pore pressure due to heavy rainfall, seepage from springs, water lines and reservoirs, weak lithologic units (particularly the Santa Clara Formation), poorly compacted fill, seismic shaking events, and stresses accumulating from continual creep along the Calaveras fault. Since the impact of development is just being felt in the South County, slope stability problems can be minimized if recognized and taken into account during the planning phases.

CONCLUSION

The 1984 inventory of foothill landslides in Santa Clara County provides a general overview of the foothill slope conditions. It is hoped that as the data is analyzed, correlations may be observed which will aid in predicting which slopes will be more prone to landsliding than others. Data from this study will be available to the public in the near future. All data should be interpreted with care due to the broad, regional nature of the study.

As of this date, results from the landslide study point to the need for site specific investigations whenever development is being planned in the foothill regions.

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ENGINEERING GEOLOGY OF THE CARMEL VALLEY ROAD ROCKSLIDE, MONTEREY COUNTY, CALIFORNIA

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On April 5, 1983, a major rockslide located about eight miles east of Carmel Valley, California, closed the Carmel Valley Road. The closure lasted for four months and resulted in significant economic and social impact on the several thousand people who live in the Carmel Valley area.

The rockslide occurred along a ridgecrest located above a steep, 200-foot-high cut slope. The underlying bedrock consists of fractured granitic/metamorphic Salinian basement. Detailed engineering geologic mapping and borehole information indicated that the rockslide was 300 feet long, 240 feet wide, 70 feet deep, and had a volume of approximately 150,000 yards.

Large-scale mapping of bedrock fractures in the cut slope defined a set of planar features that controlled the failure. Fracture analysis indicated that the rockslide moved toward the free face of the cut slope as a wedge failure and generated rockfall debris that closed the road. The failure of the rockslide was induced by high pore pressures generated during the last two abnormally wet winters. Piezometric data indicated that the near-surface ground-water regime and the rockfall activity were very sensitive to short-term storms of low to moderate intensity.

Landslide control works selected to mitigate the rockslide and rockfall hazards included: 1) mass grading, 2) mid-slope rockfall interceptor benches, 3) horizontal underdrains, 4) surface drainage control measures, and 5) anchored wire mesh.

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Characteristics of Mudflows: Some Examples from the 1980 Mount St. Helens Eruptions

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ABSTRACT

The large mudflows generated during the May 1980 eruptions of Mount St. Helens brought the scientific community face to face with the actual and potential disastrous consequences of large mudflows. These mudflows caused extensive flooding, erosion, and burial of transportation routes and bridges, as well as interruption of major navigation channels in the Columbia River system.

The mudflow hazard is real; however, little is understood of the mechanics governing the flow. Realistic approaches to land-use planning and engineering design and location of transportation routes should be based on a thorough understanding of the behavior of mudflows. In order for theoretical models to be developed, detailed observations of actual mudflows must be made to establish flow characteristics.

This paper examines the Mount St. Helens mudflows as to their flow properties (velocity, depth of flow, viscosity), material characteristics, and water content. Observations of the flows and resulting deposits indicate flow varied from laminar to very turbulent and behaved as a highly viscous non-Newtonian flow; also, material characteristics were highly variable.

INTRODUCTION

The term "mudflow" or "debris flow" is applied to a whole spectrum of flow processes involving mass movements of earth materials. Definitions of these terms vary considerably in the technical literature. Mudflows and debris flows are members of a gradational series of processes intermediate between flash floods and mud slides characterized by different proportions of water, fine solid particles, and rock debris. Mudflows or debris flows originating from volcanic activity are defined as lahars. Although some authors (Sharp and Nobles, 1953; Varnes, 1978) make a distinction between the terms "mudflow" and "debris flow," depending on whether fine-grained or coarse clastic material predominates, the two terms will be used synonymously in this paper.

Mudflows have been observed in many areas of the United States and the world. They are generated as a result of extreme climatic conditions such as heavy rainfall or rapid snow melts due to warm weather. Also, large-scale mudflows often are generated by the eruption of volcanoes.

Primarily, mudflows are important geologic agents with regard to large volumes of sediment transport, and they contribute significantly to the geomorphologic evolution of the earth's surface. However, mudflows are of more concern to mankind because of their potential to inflict damage to life and property. On

May 18, 1980, numerous mudflows occurred as a result of the eruption of Mount St. Helens in the State of Washington. The most massive of these mudflows originated in the form of a lateral blast and avalanche in the North Fork Toutle River and traveled for a distance of more than 120 km downstream. This mudflow destroyed or damaged many homes, bridges, roads, logging facilities, and equipment. The sediment volume, estimated to be more than 30.8 million m^3 clogged the channels of the Cowlitz and Columbia Rivers closing them to shipping and increasing the flood hazard of the Cowlitz. The estimated cost of damage to the roads and bridges was $112 million, and the estimated dredging cost was $45 million (Rainier Bank, 1980).

Assessment of mudflow hazard potential is essential for the prevention of loss of life and property in mudflow-prone areas. This requires routing of mudflows down stream valleys which involves hydraulic computations of flow depths and velocities. Although methods of routing water floods in streams are available, no such method exists for mudflows. This is due to the fact that the theories of fluid mechanics for mudflows are not fully developed.

The U.S. Department of Interior and the Washington Water Research Center are supporting research at Washington State University to develop the theories of fluid mechanics for mudflows. An essential first step in this research is to examine the rheological parameters and flow characteristics of actual mudflows.

The purpose of this paper is to summarize the numerous observations of the mudflows around Mount St. Helens, Washington, and analyze the rheological parameters of the muds and their flow characteristics.

The author spent considerable time in the field during the summers of 1980-81 and fall of 1981 examining and sampling mudflow deposits. Also, much time was spent discussing data with Mr. Tom Pierson of the U.S. Geological Survey Cascades Volcanic Observatory. A great deal of data on mudflows has been collected by numerous investigators and this paper is an attempt to pool this information and evaluate typical mudflow behavior.

LITERATURE REVIEW

Some of the best information available on mudflows is from eyewitness reports during the 1980 eruptions of Mount St. Helens and subsequent examinations of the remaining deposits. Personnel from the United States Geological Survey (USGS) have resurveyed a number of stream channel cross-sections, measurements of which were made before the eruptions, to document changes in channel morphology by the mudflows and by continued stream erosion. In addition, several investigators have estimated mudflow velocities at many locations in the Toutle, Pine, and Muddy River drainages

(Cassidy et al., 1980; Fink et al., 1981; Janda et al., 1981; Pierson, 1981). These estimates were based on run-up around channel bends and differences in heights of mud marks between the upstream and downstream sides of standing trees. Cummans (1981) reported average velocities based on eye-witness reports of time of arrival of peak flows.

Janda and others (1981), Pierson (1981), and Higgins and others (1983) report observations of the textural characteristics of mudflow deposits, as well as some interpretation of field observations and aerial photography concerning mudflow formation and flow behavior. Several others have published or collected field data (Fink et al., 1981; Fritz et al., 1982), which have been drawn from for this paper.

ORIGIN OF MUDFLOWS

Mount St. Helens, located in southwestern Washington, erupted violently at 8:32 a.m. Pacific Daylight Time (PDT) on May 18, 1980. The beginning of the eruption was marked by a massive debris avalanche and horizontal blast that moved down the northern side of the cone (Figure 1). Much of the debris was deposited in the upper 27 km of the North Fork Toutle River Valley. During the eruption, dense clouds of hot gases and volcanic debris spilled over the sides of the crater or fell from the vertical plume and flowed down the snow- and ice-covered slopes of the volcano.

Mudflows affected the North Fork and South Fork Toutle River and their tributaries that drain the western side of Mount St. Helens, as well as the Muddy River and Pine Creek drainages on the eastern and southeastern slopes (Figure 1). The largest mudflow developed in the North Fork Toutle River and modified over 120 km of stream channel.

Janda and others (1981) suggested that there were at least three major modes of mudflow formation:

1. The North Fork Toutle River mudflow developed primarily by slumping and flow of saturated debris avalanche material.
2. High velocity mudflows originated on the cone from the catastrophic ejection of volcanic debris, ash, water, and entrapped gases.
3. Small mudflows were formed on the cone by rapid melting of snow and ice by heat of pyroclastic flows and deposited tephra.

PINE CREEK MUDFLOW

During the initial eruption, a large, dense cloud spilled over the rim of the crater and flowed down the eastern and southeastern flanks of the cone. It is assumed that the cloud was a pyroclastic density flow (Pierson, 1981). Typically, pyroclastic flows are capable of melting a considerable volume of snow and ice as they travel down slope. The estimated minimum average velocity of the flow was 38 m/s (Pierson, 1981). It is believed that this was the triggering mechanism of a large mudflow that moved down the lower slopes of the cone into Pine Creek and the Muddy River and eventually to Swift Reservoir (Figure 2).

On the lower slopes of the cone, a distance of 3.8 km from the crater, the mudflow was approximately 10 m deep, moving at approximately 28 m/s (1 in Figure 1). Peak discharge is estimated to be about 280,000 m^3/s. At this location, evidence of large-scale eddying was reported. This suggests that the pyroclastic flow was still inflated and flowed along with the mudflow it had initiated. Downstream 1 km, the depth and velocity had changed little; however, there was no evidence of eddying (Pierson, 1981). Fink and others (1981) report a minimum velocity of 31 m/s near site 2 in Figure 1. Where the main portion of the mudflow entered the East Fork Pine Creek, the minimum velocity was estimated at 24 m/s with flow depths of 10 to 15 m and a peak discharge of approximately 29,000 m^3/s (3 in Figure 1).

The lower reaches of Pine Creek have a much wider channel than the upstream sections. Pierson (1981) reports the average flow depth decreased to approximately 8 m and the average velocity varied from 9 to 12 m/s in these reaches. Estimates of peak discharge at the mouth of Pine Creek are in the range of 7300 m^3/s.

Initial peak velocity of the mudflow in Pine Creek ranged from 25 to 30 m/s near the cone but decreased rapidly over the entire distance to Swift Reservoir. Froude numbers calculated for surveyed cross-sections suggest flow remained at or above critical velocity along essentially the entire flow path (Pierson, 1981).

Pierson (1981) describes the texture of the mudflow deposits on Pine Creek, according to Folk's classification, as muddy, sandy gravels or gravelly, muddy sands. The deposits are very poorly sorted and show little change in grain-size distribution in the downstream direction.

MUDDY RIVER MUDFLOW

The pyroclastic density flow that moved down the eastern slopes of the cone and generated mudflows into Pine Creek entered Smith Creek and Ape Canyon, tributaries to the Muddy River (Figure 1). Large mudflows were generated that flowed the length of Muddy River to the upper end of Swift Reservoir (Figure 3).

The mudflow that moved through Ape Canyon was greater than 20 m in depth with a peak discharge estimated to be about 67,000 m^3/s (Pierson, 1981). Janda and others (1981) report an estimated velocity of 32 m/s at the head of Ape Canyon. Immediately upstream of the canyon mouth the velocity was about 28 m/s, and at the confluence with Smith Creek, run-up on the opposite canyon wall suggests a velocity of about 27 m/s. The mudflow moved down Smith Creek above the confluence with Ape Canyon; however, no data are available for that area. The Smith Creek valley below the confluence with Ape Canyon widens (250 to 300 m) and has a low gradient (0.013). The mudflow slowed in this stretch of the channel and deposited a considerable quantity of debris (Pierson, 1981).

Another mudflow split from the Pine Creek mudflow and flowed down Muddy River canyon to join the Ape Creek mudflow (4 in Figure 1). Cassidy and others (1980) estimate an average velocity half-way down the Muddy River canyon of 23 m/s. Peak discharge was about 20,000 m^3/s.

At the confluence of Clearwater Creek and Muddy River (5 in Figure 1), the combined mudflows traveled at an estimated velocity of 14 m/s (Pierson, 1981). The velocity continued to decrease downstream. Cassidy and others (1980) estimate a velocity of 6 m/s for the Muddy River mudflow at Swift Reservoir. Estimated average peak velocities vary little in the lower 10 km of the Muddy River.

Pierson (1981) describes the textural characteristics of the Muddy River mudflow deposits the same as for the Pine Creek mudflow, i.e., muddy, sandy gravels or gravelly, muddy sands. The deposits are very poorly sorted and exhibit a fine-skewed distribution. However, the Muddy River deposits are generally coarser than the Pine Creek deposits. In the downstream direction, the deposit shows distinct fining and some sorting, probably due to considerably lower velocities in Muddy River. Froude numbers calculated for surveyed cross-sections suggest that the Muddy River mudflow flowed subcritically through the lower channel reaches (Pierson, 1981).

Cassidy and others (1980) generated a combined hydrograph of the Pine Creek and Muddy River mudflows entering Swift Reservoir (Figure 4) on the basis of the water surface elevation record at Swift Dam. The hydrograph exhibits steep limbs and a peak discharge of approximately 2700 m^3/s. Time to peak was about 1.5 hours after the eruption with a volume of 13.8 x 10^6 m^3.

Janda and others (1981) report several mudflows occurred after the large Muddy River mudflow. These flows are thought to have occurred between the waning phases of the May 18 eruption and the end of the June 12, 1980 eruption. The deposits consist of silt- and clay-rich matrices with dispersed cobbles, boulders, and varying amounts of pumice and other tephra (Figure 5).

SOUTH FORK TOUTLE RIVER

The lithologic character and sequence of mudflows on the South Fork Toutle River are very similar to those in the headwaters of Muddy River and Pine Creek. Janda and others (1981) report that during the initial phase of the May 18 eruption, large volumes of water-saturated pyroclastic material flowed very rapidly down the western flank of the volcano. This material melted snow and ice and developed into a single, sharp-crested mudflow peak in the South Fork Toutle River. From stratigraphic evidence, they suggest the peak flow occurred within minutes after the 8:32 a.m. blast. A number of witnesses observed this peak many kilometers downstream (Cummans, 1981).

It is thought that in the upper reaches of South Fork Toutle River the mudflow was highly turbulent. Janda and others (1981) report mudflow-scoured surfaces and thin mud deposits as much as 54 m above the general depositional level of the mudflow (6 in Figure 1). These deposits were not formed by cross-valley banking due to radial acceleration around bends, but instead are thought to be from cross-valley sloshing. The mudflow deposits and eyewitness accounts indicate the flow became less turbulent and slowed considerably within a short distance downstream. The overall average velocity between the area of high turbulence and the confluence of the North Fork and South Fork Toutle River is estimated to be approximately 7 m/s (Cummans, 1981). Estimates of velocity from cross-valley banking of mudlines in this area indicate a range of 7 to 29 m/s (Wigmosta et al., 1981).

Depth of the mudflow decreased in the downstream direction. In the area of highly turbulent flow near the cone, flow depth was reportedly at least 15 m; however, at a stream gage immediately below the confluence of the North Fork and South Fork Toutle River

(7 in Figure 1), the mudflow peak caused a 6.4 m rise in stage. On the Cowlitz River at Castle Rock, peak discharge of the mudflow caused a 0.98 m increase in stage, and at Longview there was no increase in stage, only some increase in turbidity (Cummans, 1981).

Mudflow deposits changed in texture and composition in the downstream direction, certainly a reflection of the change in velocity and discharge. Deposits near the volcano exhibit a granular matrix. A short distance downstream the deposits of the peak mudflow contain abundant sand, silt, and clay. Figure 6 shows typical grain-size distributions of deposits near Toutle. Investigators have identified little deposition from the South Fork mudflow below its confluence with the North Fork Toutle River.

NORTH FORK TOUTLE RIVER

The beginning of the May 18, 1980 eruption was marked by a tremendous rock avalanche on the northern side of the cone. Sliding of the rock mass released pressure within the cone, which caused steam and magmatic explosions behind the slide scarp. The sliding debris became engulfed in the resulting blast cloud and evolved into a large hot avalanche. One lobe of the avalanche traveled westward down the North Fork Toutle River for 22 km (Figure 1), filling the valley an average of 45 m deep with

2.8 km^3 of debris. Another lobe moved into Spirit Lake. The deposit has a very hummocky, irregular surface and consists of rock, ash, pumice, snow, and glacial ice. Much of the avalanche deposit was at least partially saturated by water and steam. Additional moisture probably was provided by rapid melting of snow and the crushed portion of glacial ice incorporated in the deposit.

The North Fork Toutle River mudflows were largely generated on the surface of the avalanche deposits (Janda et al., 1981; Voight et al., 1981; Cummans, 1981) rather than on the cone. Eyewitnesses report the mudflows originated from slumping and flowing of water-saturated parts of the avalanche deposit. Mudflows reportedly moved over the irregular surface, ponded in depressions and broke out of the depressions as larger flows. Flow between ponded areas was estimated to be only a few meters deep and at velocities from 12 to 15 m/s (Janda et al., 1981).

The mode of mudflow generation, irregular flow course, and ponding delayed the peak mudflow from leaving the end of the avalanche deposit until about 1:30 p.m. Cummans (1981) reports that flows into Swift Reservoir on the southern side of the volcano had nearly subsided and the peak mudflow from the South Fork Toutle River had moved into the Cowlitz River by 1:30 p.m. The magnitude of the North Fork Toutle River mudflow was most evident downstream of the debris avalanche deposit where channel shape and drainage patterns were modified considerably.

Mean flow velocities of peak flow between Camp Baker (8 in Figure 1) and the town of St. Helens were estimated at about 4 to 5 m/s. Immediately below the confluence of the Green River, mean flow velocity was about 12 m/s. The rest of the estimates in the North Fork Toutle River ranged from 6 to 8 m/s (Janda et al., 1981). Timed visual observations reported by Cummans (1981) were lower, due probably to local ponding at channel constrictions and log jams.

Many investigators have noted that near the main channel of flow, trees were sheared or toppled and highly abraided. Laterally from the main flow trees remained standing, suggesting velocity had decreased. Mudlines on standing trees indicated sharp velocity drops laterally from the main flow channel. Splash marks were evident on trees immediately adjacent to the main flow but disappeared laterally. Near the lateral margins of mudflow flooded areas, there are examples of mobile homes, houses, and logs that floated passively on the mud and were let down in nearly their original location.

Janda and others (1981) report recessional surge lines in the mud veneers on steep channel walls. Too, abrupt flow fronts at least 3 cm high were preserved where the margin of peak flow was deposited on smooth surfaces.

The mudflow deposits on the avalanche and in the stream valley above the town of St. Helens consist of subangular pebble and cobble gravel with abundant silt- and clay-size material in the matrix. Large angular clasts of materials with low cohesion, such as avalanche deposits, colluvium, older weathered mudflow deposits, and weathered tephra, occur commonly on the surface of the mudflow deposit and sporadically within the deposit. These clasts are often more than 1 km from their probable source (Janda et al., 1981).

TOUTLE AND COWLITZ RIVERS

Below the confluence of the South Fork and North Fork Toutle River, much of the river flows through narrow bedrock canyons and then widens near Tower (9 in Figure 1). The peak mudflow eroded considerable alluvium and colluvium along the narrow canyons; however, considerable deposition occurred between Tower and the Cowlitz River. Velocity in the narrow canyons was estimated at 3.2 m/s (Janda et al., 1981). Visual examination of the lateral channel deposits in the Cowlitz River suggest they are similar in texture to those in the Toutle River. Therefore, the peak mudflow probably changed little as it flowed through the Toutle and Cowlitz Rivers.

As the mudflow moved through the Cowlitz River at Castle Rock, USGS personnel made periodic measurements of water levels

(Figure 7). Levels rose over a 6-hour period to a crest of 8.92 m at midnight or nearly 6 m above pre-eruption levels. Several temperature measurements have been reported for the lower Toutle and Cowlitz Rivers (Cummans, 1981). The highest temperature at the Highway 99 bridge over the Toutle River (10 in Figure 1) was 33°C at 9:45 a.m. on May 19. Considering the distance from the source, temperatures may have been markedly higher upstream.

OLDER MUDFLOWS

Mullineaux and Crandell (1962) reported valley fills of 2,000-year-old mudflow deposits stretching up to 64 km down the North Fork Toutle River and into the Cowlitz River. The character of these deposits are similar to those from the 1980 mudflows. They are poorly sorted, unstratified, and are composed typically of subangular rock fragments. Typically, the measured sections describe pebbles and cobbles or pebbles and granules in sand matrix. Large boulders are common, especially closer to the mountain, although boulders of several feet in diameter are contained in the deposits near Castle Rock.

Another mudflow about 330 years old was described by Mullineaux and Crandell (1962). This deposit formed a short section of the southern bank of the North Fork Toutle River immediately west of Spirit Lake. The deposit consists of unstratified pebbles, cobbles, and boulders in a sand matrix. The boulders are typically scoria, some up to 1.5 to 1.8 m diameter. Twigs and branches with bark were found in the mudflow, some completely converted to charcoal; others are charred only on the exterior. An in-place stump protruding into the mudflow reportedly still had bark and was charred only where large volcanic bombs were in contact with it.

The condition of the wood suggests that the mudflow mass was originally very hot. The mass cooled as it flowed and by the time it reached the North Fork Toutle River it was not hot enough to char wood. Mullineaux and Crandell (1962) suggest that the transport of the charred wood intact indicates a fairly gentle transport.

Hyde (1975) described older mudflow deposits that extend at least 34 km from the slopes of Mount St. Helens down Swift Creek and into the Lewis River valley. The deposits consist chiefly of sand- and granule-size material with a few pebbles and cobbles. Evidence suggests some of these mudflows were deposited hot.

FLOW CHARACTERISTICS

The above descriptions suggest the mudflows at Mount St. Helens maintained neither steady nor uniform flow. The author viewed videotape records of the North Fork Toutle River mudflow on reserve at the Washington Department of Natural Resources, Division of Geology and Earth Resources library. Some of the best footage recorded mudflow movement over the debris avalanche deposit. The mud slurry appeared viscous, and flow depth, direction, and velocity varied considerably. Some of the flow over the avalanche had a smooth surface; however, there were many turbulent zones exhibiting a great deal of splashing as the mudflow flowed over irregularities. Pierson (1981) reports indications of laminar flow in a few locations by streamlines of light-colored material on the flow surface that were not distorted for long distances. He too noted the very turbulent zones at hydraulic jumps.

Evidence of highly viscous non-Newtonian flow behavior was noted by Cassidy (1980), Janda and others (1981) as well as by the author. As noted above, some deposits showed abrupt flow fronts of at least 3 cm in height. At many locations evidence shows that flows supported boulders at or near their surfaces (Figure 8). It is common to find isolated boulders surrounded by finer grained material in deposits at a great distance from the volcano. In many areas the bark was removed on trees nearly the total height of the mudflow. This suggests coarse material was supported throughout the flow (Figure 9).

Consideration of the sources of the mudflows and temperature measurements mentioned above, suggests they initially were above ambient temperature, although they may have cooled as they traveled down snow- and ice-covered slopes and entered large streams. Too, other investigators have reported evidence in some of the older mudflow deposits that indicate elevated temperatures.

Crandell and Mullineaux (1978) note that earlier mudflow deposits from Mount St. Helens tend to lack abundant clay materials which is typical of deposits around Mount Rainier. Evidently, large areas of hydrothermally-altered rock (the source of clay materials on Mount Rainier) did not exist on Mount St. Helens in the past and do not presently. Therefore, the modes of initiation, as well as the character of the material and flow behavior may vary for different volcanoes depending on weathering and alteration of surficial materials.

CONCLUSIONS

In volcanic settings, evidence suggests that some flows were hot, i.e., thermally altered rocks and charcoal fragments. Temperature probably is influential in mechanics of the flow. Thus, this parameter must be considered in developing flow theories. Different flow regimes were observed by investigators at Mount St. Helens also; there, subcritical to supercritical flow existed alternately at and below about 10 km from the crater. These two instances and films of Mount St. Helens flows, suggest (a) both flow regimes can occur, and (b) the fluidity of the flows sometimes has those characteristics of water flow and at other times apparently is non-Newtonian in nature.

Origin of mudflows can vary as widely as the flow characteristics, i.e., as long as both solids and fluids are sufficiently abundant, mudflows can occur. Thus, the mechanics of motion can be addressed independently from the initiation of motion. Using this approach, an initiation scenario can be defined and motion mechanics, once developed, can be used to describe the flow as it moves downstream.

ACKNOWLEDGMENTS

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The author wishes to thank Mr. Tom Pierson of the U.S. Geological Survey for sharing of information and ideas, as well as the many cooperative technical staff members at the Cascades Volcano Observatory. Also, special thanks go to Dr. John J. Cassidy, former director of the Washington Water Research Center, who spent a great deal of time discussing mudflow mechanics on Mount St. Helens with the author.

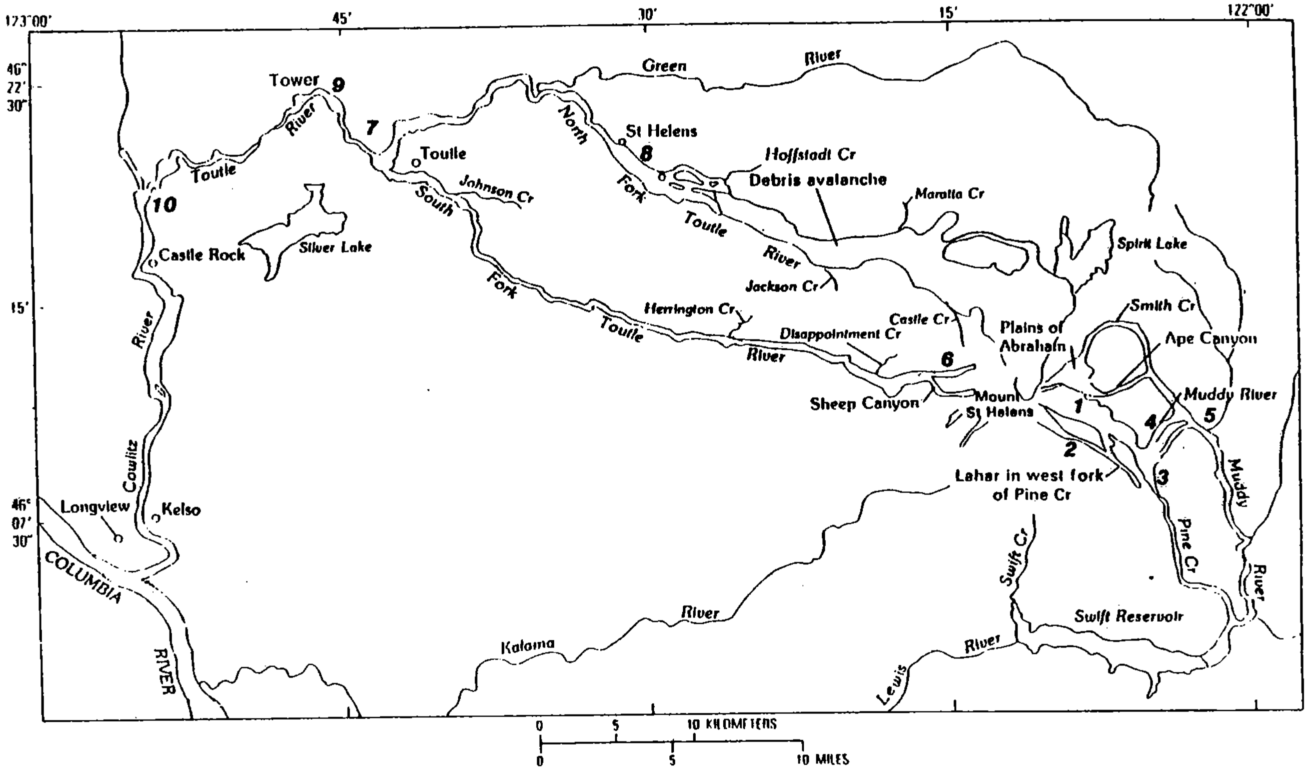

Figure 1. Location map of Mount St. Helens and streams affected by mudflows (after Janda et al., 1981).

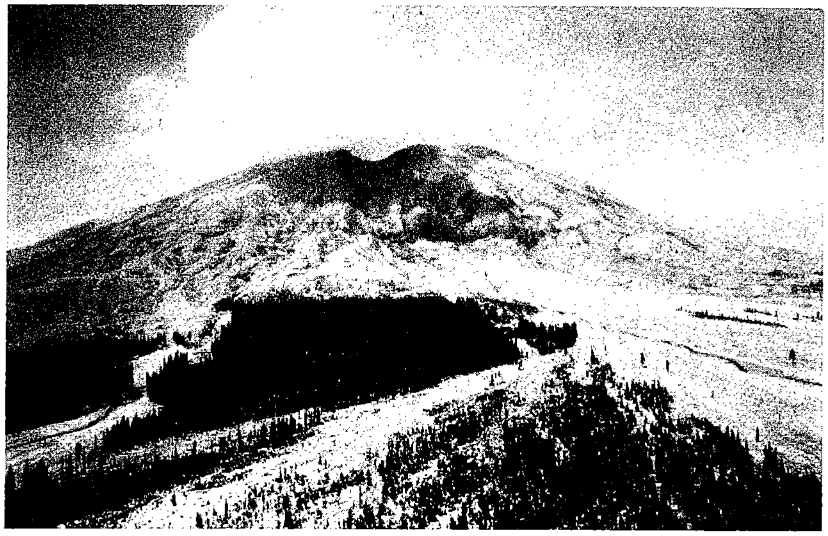

Figure 2. Mudflow deposits on the east side of Mount St. Helens.

Figure 3. Mudflow deposit in the Muddy River arm of Swift Reservoir (photo courtesy of John J. Cassidy).

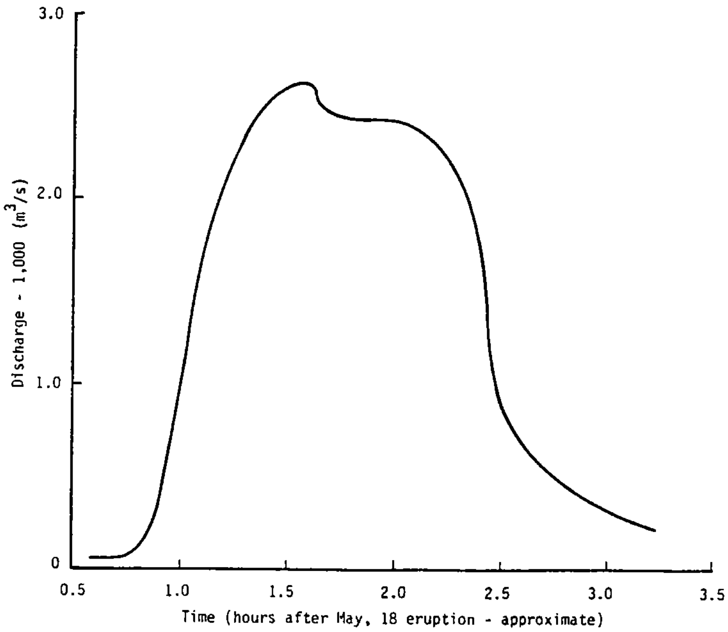

Figure 4. Combined hydrograph of Pine Creek and Muddy River mudflows (after Cassidy et al., 1980).

Figure 5. Eroded mudflow deposit in Muddy River drainage.

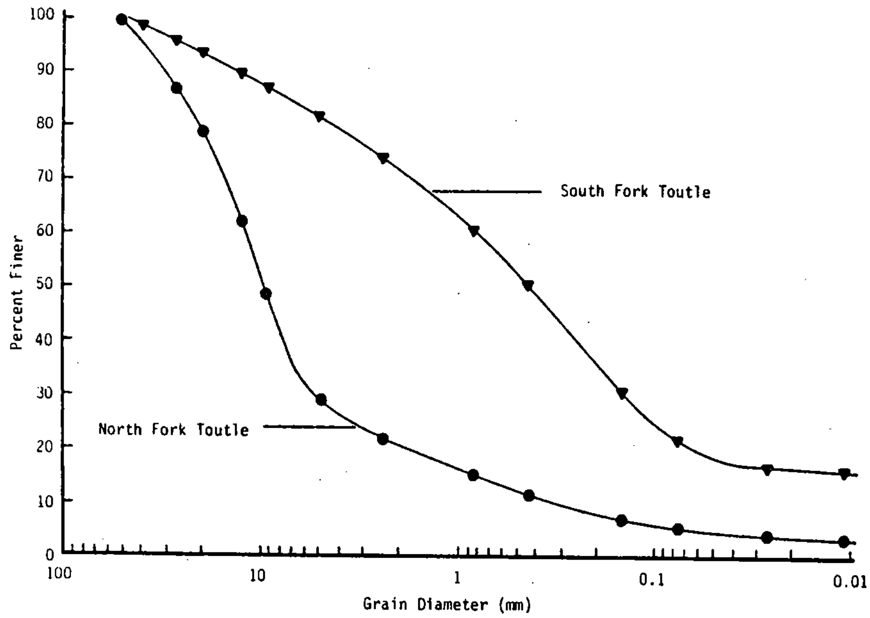

Figure 6. Grain-size distribution of the May 18, 1980 peak mudflow deposits near Toutle, WA.

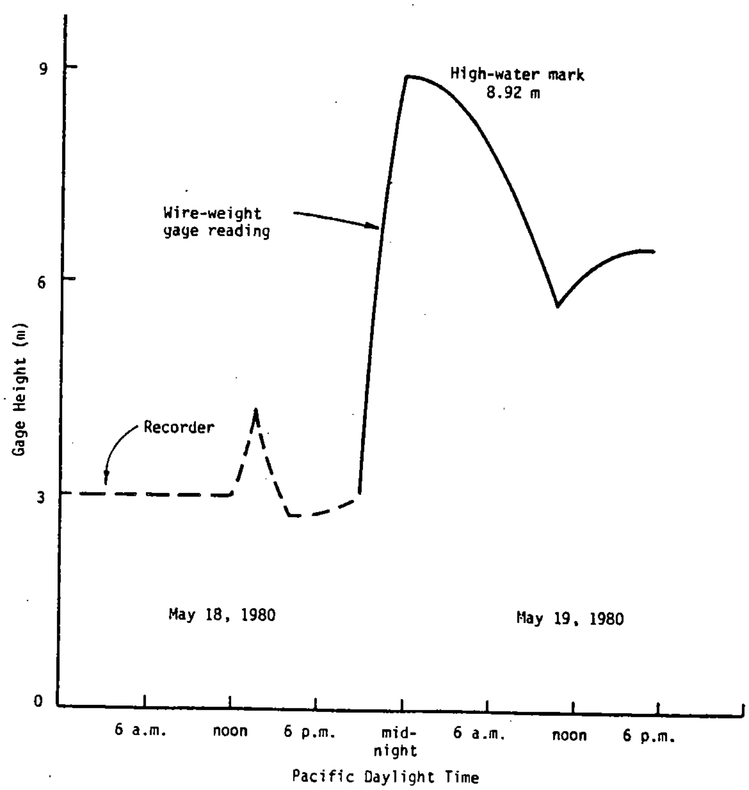

Figure 7. Stage from recorder and wire-weight gage readings. Cowlitz River at Castle Rock, WA, May 18-19, 1980 (after Cummans, 1981).

Figure 8. Large boulders deposited on a bridge abutment by mudflows southeast of Mount St. Helens.

Figure 9. Bark removed from trees by the coarse material supported in a mudflow.

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THE CONGRESS SPRINGS LANDSLIDE UPDATED

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State Highway 9, west of Saratoga, California, parallels Saratoga Creek and crosses the toe of a large moving mass of rock and soil, which has been named the Congress Springs Landslide (William Cotton & Associates, 1977). The block-glide-type slide is funnel-shaped, exceeds 200 feet thick, and has maximum lateral dimensions of about 3,000 x 5,000 feet. The total volume is about 50 million cubic yards. The vertical relief from the headscarp to the highway at the base is nearly 1200 feet. The active slide lies within a larger ancient slide much of which appears to have stabilized.

Symptoms of distress have been noticed in the area of the slide for at least 20 years by many affected parties, including Caltrans, San Jose Water Works, Saratoga Heights Mutual Water Company, City of Saratoga, County of Santa Clara, and many private property owners. For years the problems were considered isolated phenomena, but since 1976 the overall hazard has been identified, mapped and monitored. Special geologic hazard zoning, with controls on development, has been established in their respective jurisdictions by the City of Saratoga and the County of Santa Clara.

Continued movement of about 1 1/2 inches per year has been observed in the core of the slide, and total compressional uplift of the highway at the toe has exceeded 13 feet. Most rapid movement followed greatest rainfall with a lag of a few months. After the record two-year drought of 1976-77, the San Jose weather station measured the six wettest consecutive calendar years on record. The total for 1983 alone was 32.57", exceeding by 27.5 percent previous record rainfall for a single year, set in 1888. The effect of extreme rainfall on slope stability has been dramatic in Santa Clara County.

COMPILERS' NOTE: This abstract was originally prepared for a paper presented at the Highway Geology Symposium. As no proceedings contribution was received, the abstract is reprinted here.

ROAD FAILURE BY SUBSURFACE STORMFLOW IN MELANGE TERRANE

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A low volume road on the Six Rivers National Forest of northwestern California has several portions that undergo intermittent slumping. The road was built on ultramafics and metasediments. The area has earthflows which are characterized by Galice formation melange, a mixture of materials commonly having a high clay content.

Three unstable sites were selected on this road to monitor subsurface drainge with piezometers. One site had 26 piezometers up to 20 feet deep; the other 2 sites each had 5 piezometers. Detailed topographic maps were made to locate the piezometers and determine their elevation. Piezometric levels were measured during storms and dry periods of the winter of 1983-84. The specific conductance was determined for water from all piezometers. The specific capacity of each piezometer was also measured.

The results suggest that specific capacity is inversely related to specific conductance and these values can help explain piezometric levels. For example, clayey melange had piezometers with low specific capacities. Melange waters had high conductivities because vertical drainage was retarded, forcing shallow water to flow laterally as subsurface stormflow. The results also indicate that road cuts intersected the piezometric surface and produced exfiltration. Inboard ditches concentrated water that drained through the fill and promoted instability of the fill slope.

COMPILERS' NOTE: This abstract was originally prepared for a paper presented at the Highway Geology Symposium. As no proceedings contribution was received, the abstract is reprinted here.

CORRECTION OF SYCAMORE DRAW LANDSLIDE, SOUTH OF BIG SUR, MONTEREY COUNTY, CALIFORNIA

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ABSTRACT

In early April 1983, the Sycamore Draw landslide, activated by a record succession of two wet winters, buried a section of the Pacific Coast Highway, seven miles south of Big Sur, California. Measuring 250 by 400 feet, the landslide consisted of weathered and fractured granitic-metamorphic rock of the Salinian Block. The landslide severed the major scenic route between San Francisco and Los Angeles and in tandem with a later slide to the south isolated the small community of Partington Ridge for nearly three months. These two factors, along with the safety problems associated with working on the slide from the road level, escalated the cleanup effort from routine maintenance to a major construction operation.

Landslide mitigation measures implemented include, 1) mass grading to remove the head and side scarps of the existing slide and the majority of material from a larger, adjacent, developing slide, 2) constructing midslope and road level debris benches, 3) installing horizontal drains, and 4) constructing a collector system to divert drain water away from the cut slope. Upslope cracking discovered soon after the Coalinga Earthquake was monitored with a stake array throughout construction as were a minor slipout and cracking within the excavation.

The highway was reopened to through local traffic by the end of July 1983 and the remedial work was completed by the end of January 1984 with a total cost of over $400,000.

PHOTO 1 SYCAMORE DRAW LANDSLIDE FOLLOWING THE SECOND 1983 FAILURE

INTRODUCTION

Sycamore Draw landslide is located on State Highway 1, 7 miles south of Big Sur and 35 miles south of Monterey, California. The second massive failure during the winter of 1982-83 buried a 250-foot stretch of the highway, disrupting local traffic and severing the major coastal scenic route between San Francisco and Los Angeles.

A field review was made of the site by Engineering Geologists from the California State Department of Transportation (Caltrans) to recommend remedial measures to insure an expedient reopening of the road and provide safety for construction crews and the traveling public.

AREAL GEOLOGY

Sycamore Draw is located near the western edge of the Salinian Block (see Figure 1) in the Coast Ranges Geomorphic Province of California. The Sur fault, the northern portion of the Sur-Nacimiento fault zone (Page, 1970), runs just offshore to the west of the site, separating granitic and Sur series rocks on the northeast from rocks of the Franciscan Complex to the southwest.

The site is located within a small pluton of charnockitic tonalite (see Figure 2) that is exposed for about seven miles along the Pacific Ocean (Compton, 1960). This body maintains the northwest regional trend of the faults and the Sur series rocks. The landslide is nestled between good exposures of the charnockitic tonalite.

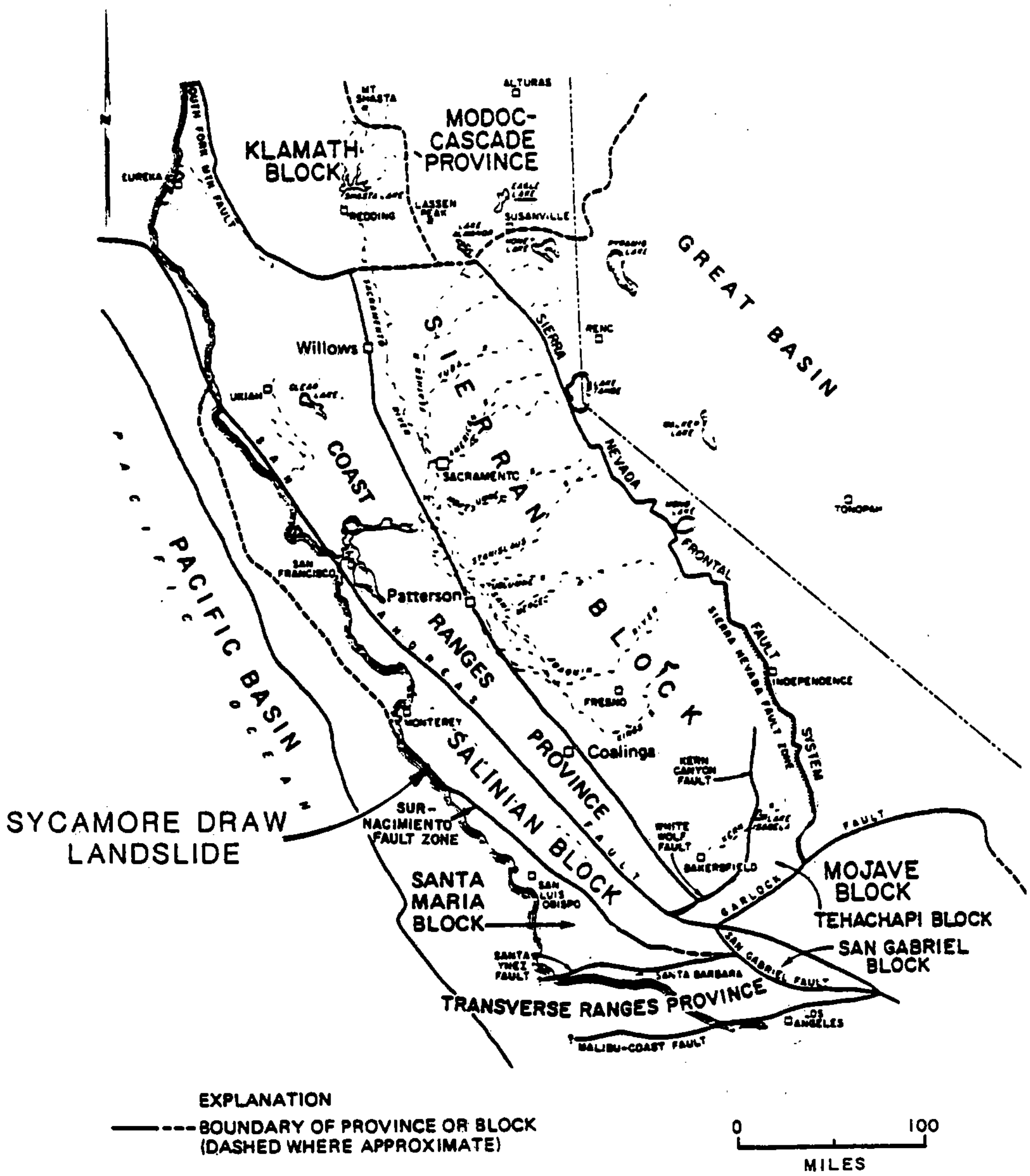

Fig. 1. TECTONIC PROVINCES AND CRUSTAL BLOCKS OF CALIFORNIA (After Wong and Ely, 1983)

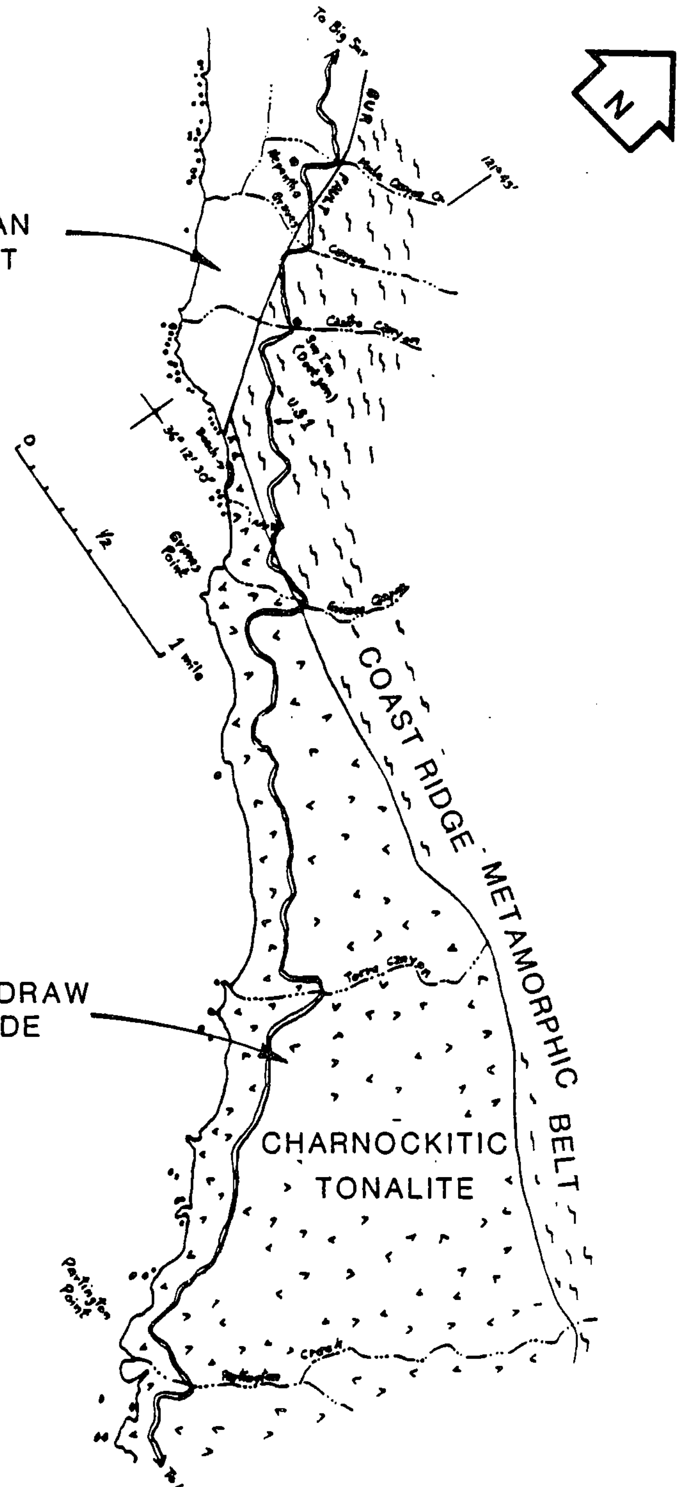

Fig. 2. GENERALIZED GEOLOGIC MAP OF VICINITY SOUTH OF BIG SUR TO PARTINGTON POINT, CALIFORNIA (After Ross, 1979)

SLIDE DESCRIPTION

Surface Features

The Sycamore Draw landslide occurred on an oversteepened ridge between two draws that merge to form a large amphitheater below the road. Preceding the slide, the lower slopes were covered by dense brush and the upper slopes above the active slide area were covered by grass with groves of sycamores, laurels, redwoods and oaks growing in the draws and around spring areas. The natural slopes vary from nearly horizontal on a natural bench, that may represent the base of a previous head scarp above the head scarp, to 45 degrees and average 35 degrees. A narrow private road ascends the hillside to the north of Sycamore Draw crossing over the draw above inactive slide area.

In addition to the existing slide, a larger adjacent slide was identified on the field review. Both are part of a larger ancient slide mass that extends an additional 800 feet to the skyline.

The developing slide faces the bridge spanning Sycamore Draw. The head of the developing slide is delineated by a series of arcuate cracks extending from the crown of the existing slide to the inside of the natural bench. The cracks show a maximum vertical displacement of about a foot before becoming indiscernible as they turn toward an outcrop on the south flank of the draw. The slide toes out above the draw and is expressed by smaller auxiliary slides working in the draw.

Subsurface Features

The slide mass consists of soil and colluvium mixed with loose, fractured and broken weathered granitic rock. Boundaries between the soil, colluvium, weathered rock and fresh rock are generally indistinct, garbled by the periodic movements of the ancient slide mass.

On either side of the ridge that contains the slide are exposed hard ribs of rock. However, in the slide area, the only possible outcrop of hard rock is located at the northern perimeter of the slide.

Previous Activity

Landsliding has been occurring along this coast since well before the first track was blazed; especially at the oversteepened interface between the land and the sea which is the route of Highway 1. Inspection on the ground or of aerial photographs reveals many scars remaining from previous slides. A portion of the pre-highway county trail crosses the upper reaches of Sycamore Draw incorporated into a private road. Sections of this route have been reclaimed by slides. A black and white oblique aerial photograph, circa 1955, shows a scarp developed in the approximate position of the first 1983 slide.

Recent interest by Caltrans in Sycamore Draw began with a review of storm damage in April 1978. In addition to cataloging storm damage to the sidehill embankment, to the road, and making recommendations for repair of the washout section, the inspection (Smith, 1978) noted a small landslide developed above the road. In a foundation report of the following year, Caltrans, (Fox, 1979) determined that the granitic rock is mantled by 5 to 22 feet of fill and landslide debris. An active spring was also noted just above the site. The final correction adopted and constructed was a bridge across Sycamore Draw.

1983 Activity

Activity in 1983 began with a slide on March 23rd consisting of about 33,000 yds^3. Cleanup of the slide was begun at road level, supervised by Maintenance personnel. On April 1st, a second slightly larger slide of 35,000 yds^3 occurred while repair work was in progress killing one equipment operator. This second slide upgraded the status of the slide from a maintenance cleanup effort to a large-scale landslide repair supervised by construction personnel.

Surface and Ground Water

The rainfall totals for the two years preceding the slide were exceptionally high, exceeding 62 inches and depart by over 22 inches from normal (NOAA, 1981-1983). The cumulative effect on the ground water was sufficient to reactivate portions of the large slide mass. The fact that the slides did not occur until the end of the rainy season indicates that they were caused by the progressive buildup of the hydraulic driving forces. Heavy rains during the latter portion of the 1982-83 season (see Figure 3) may have triggered failure of the destabilized mass.

At the time of the field review, water was issuing forth from springs at several locations including the base of the head scarp, on the north side of Sycamore Draw, from a talus pile below the large laurel tree and below the road near the south end of the debris pile.

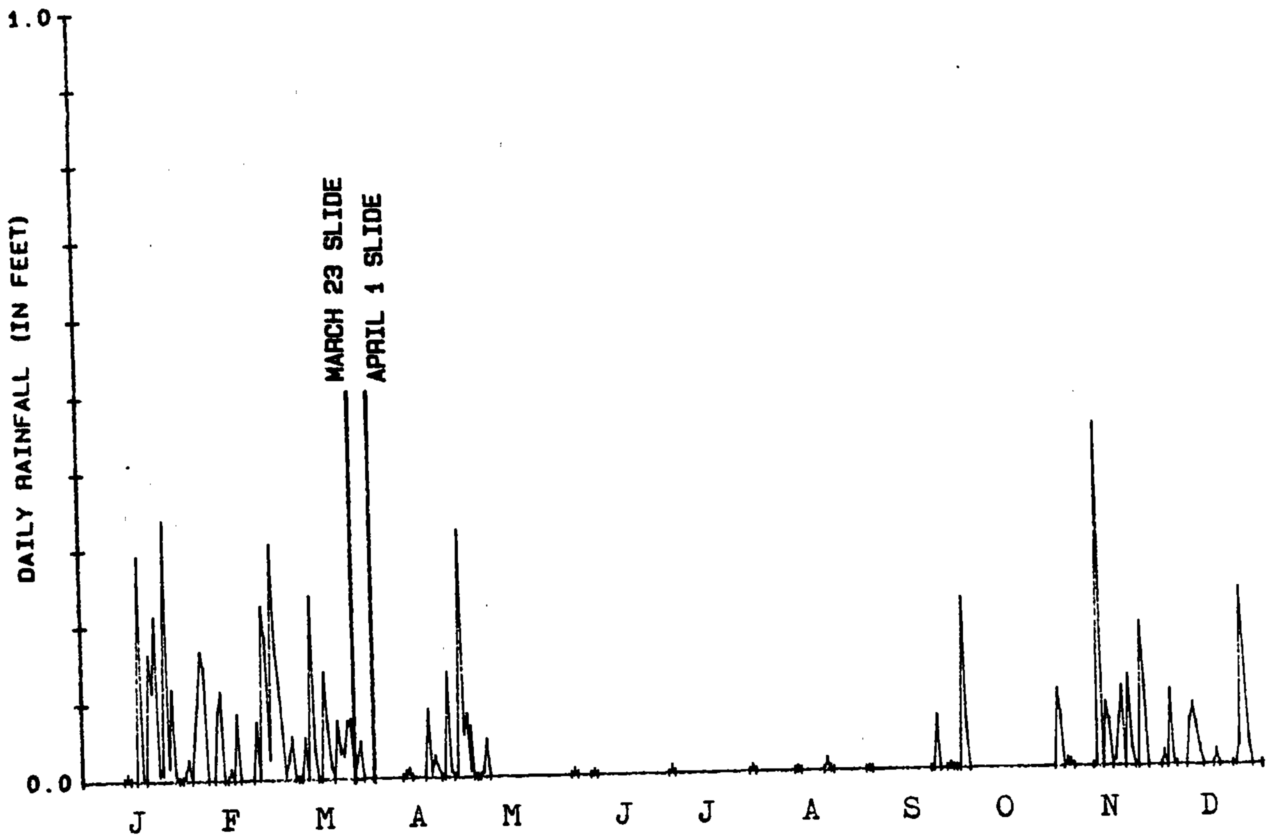

FIG. 3 1983 DAILY RAINFALL WITH THE DATES OF THE TWO SLIDES

MITIGATION MEASURES

During the field review, it was noted that the majority of the material had already failed in the existing slide but that the adjacent slide posed a similar safety hazard in removal from the toe. It was therefore deemed necessary that construction should start unloading the slide from the top and work down.

The recommended stabilizing effort (Works, 1983) would include mass grading to remove the majority of material from the developing slide and the head and side scarps from the existing slide, draining and diverting water from the slope, constructing benches to catch loose material from the large denuded cut, provide access to install and service the drains, and monitoring for the development of additional cracking.

Mass Grading

The maximum recommended slope was 1.25:1 for the cut to maintain the natural slope above the active slide area and act as a buttress of the larger slide above.

In the absence of a detailed topographic map, a survey party determined that there was enough room between the top of the cut and the centerline of the highway that it would be possible to bring down the excavation at a slope normal to the road varying from 1.49:1 to 1.40:1, allowing for one 20-foot midslope bench, a 30-foot debris bench at road level, an 8-foot shoulder, and a 12-foot lane. Five sets of slope stakes were established above the uppermost cracking. The surveyors returned periodically to check grade.

Approximately 150,000 yds^3 of material were removed during the course of the excavation, however, several times that amount were moved as a result of the rehandling necessary. Wasted material stacked up filling in the slide scar, the roadbed and covering the bridge before the material could run down toward the ocean below the road.

Without a detailed topographic map, it was not possible to determine in advance whether the excavation would daylight at any point above the exposed failure surface of the existing slide. The ends of the excavation which extend outside the established slope stakes were tapered into Sycamore Draw to the north and daylighted to the south.

It was necessary to modify the design of the cut several times to stay behind the slide scar. The surveyors determined that the excavation would daylight in the existing slide scar and the southern end of the cut was resloped at a slope of 1.37:1 beginning above the existing top of the excavation. The slide scar was encountered again at the level of the midslope bench. It was necessary to reduce the bench width to 15 feet in this area. The slope was steepened to 1:1 until the cut was under the slide scar and then flattened to 2:1 to the road.

Ripping was required in two main areas, above the existing slide scar and on the nose of the cut where it rounds into the draw. The material consisted mostly of felsic veins and hard ribs. The fractured nature of the granitic rock allowed easy ripping of the majority of the rock. Only isolated dornackers above the existing slide scar required light blasting. Some of this rock could have been left in place had the width not been needed for the midslope bench. Below the midslope bench, knobs of rock were left in the oversteepened cut in the previous position of the slide scar.

Access Roads: Access was achieved via three roads during the course of repair. All three roads made use of a narrow private road that switch backs up the slope to the north of Sycamore Draw.

The upper access road provided initial access and required the use of the full length of the private road including restoring the unmaintained section crossing Sycamore Draw that had previously slid out.

The middle access road was begun at both ends joining in the draw. The road on the north side of the draw traversed the sidehill. The road from the south passed the base of the outcrop defining the limit of the slide.

The middle road was used for access during the majority of the repair. When first completed, the road descended steeply into a draw and before it was abandoned, the road descended steeply into the cut.

The lower access road utilized an abandoned, earlier route of the private road that had washed out in the draw about 100 feet above the roadway. As the cut was levelled up, a ramp was constructed off the south end allowing caravans of local traffic to cross the slide for the first time since the slide occurred.

Small Failure: During the night of May 23, a slipout totalling about 125 yds^3 developed on the northern portion of the cut in an area bounded by several seeps. This failure was located at the working level immediately adjacent to the normal overnight equipment parking area.

The affected area was limited to about 60 feet horizontal and 20 feet vertical. The slide toed out on the edge of the working area and no cracks were found extending onto the bench.

The development of the failure was relatively rapid as no cracks had been observed previous to that night. The actual rate of failure was probably fast enough to perceive but not fast enough to endanger workers had they been present.

The failed material was removed leaving a small bowl-shaped depression on the slope. No attempt was made to weight the toe because the ballast would pose more of a potential hazard than it would mitigate.

Safety Monitoring

On May 2, 1983, a magnitude 6.7 earthquake struck about 75 miles due east of the site, centered near Coalinga, California (see Figure 4). The earthquake was profoundly felt by nearly everyone, sending construction personnel running down the hillside. The Modified Mercalli Intensity at the site was estimated at V. Although an earthquake of that size is unlikely to produce accelerations high enough at that distance to induce failure, it heightened the sense of safety awareness at the site that lasted throughout the repair.

In order to guard against undetected movement above the excavation or in the slides below, a program of monitoring was instituted. The monitoring included spotters positioned to observe any rapid activity, daily inspection

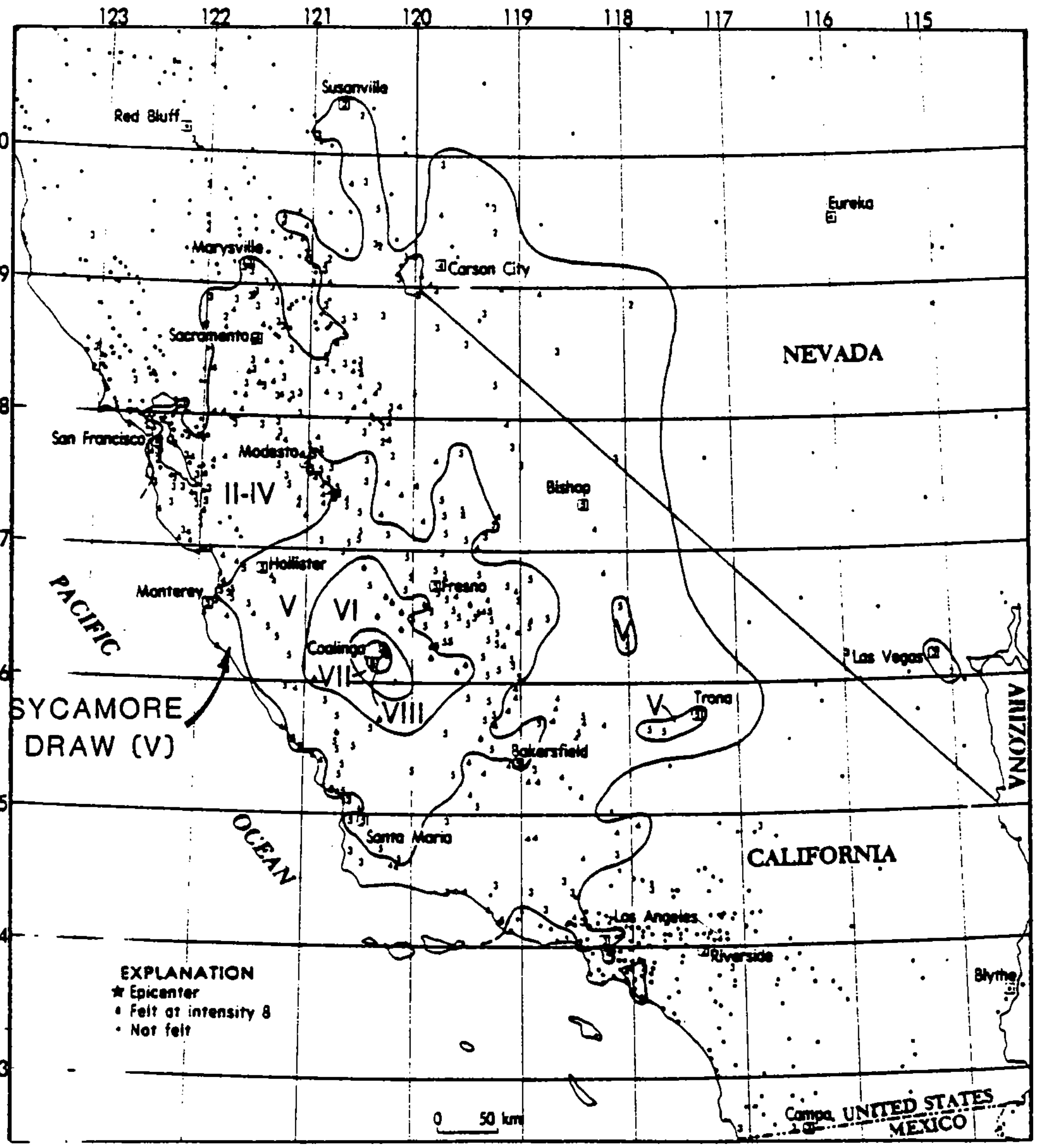

Fig. 4. ISOSEISMAL MAP FOR THE COALINGA, CALIFORNIA EARTHQUAKE OF MAY 2, 1983. (After Stover, 1983)

of the slope above and below the excavation and the establishment of stake arrays to monitor any new developing cracking above or within the excavation.

Spotters: Four to five spotters were utilized throughout the repair. Equipped with FM headsets to communicate with the equipment operators, portable radios to communicate with each other, and remote controlled warning devices, the spotters visually monitored sudden changes on the slide or in the excavation. Three spotters were positioned outside the cut, each with specific responsibilities. The spotter at road level on the south end of the slide observed the head scarp of the existing slide scar and later material being pushed over the head scarp. The spotter at road level on the north end of the slide observed the toe of the developing slide and later material pushed over toward the draw. The spotter positioned across the draw observed for activity above or immediately below the excavation. Additional spotters were positioned within the excavation to check the cut slope immediately above working equipment and the edge of the waste pile.

The spotters at road level were also responsible for dissuading people from walking across the debris pile during work.

Daily Inspection: Daily inspection of the cut slope, the brush-covered slope between the top of the cut and the upper access road and the grassy slope, was performed to detect the development of additional cracking. These inspections were the basis for establishing monitoring stations.

Stake Arrays: The hillside above the upper access road was immediately, following the Coalinga earthquake, inspected for fresh cracking, but none was found. However, two days later as the access road reached the top of the excavation, a fresh crack was detected. This new crack was located above the existing slide crossing the grass-brush line.

Three stake arrays were established to monitor movement. Two of the stations were located along the crack and the third was located to detect extention further to the north. This station, CT-1 (see Figure 5), shows the greatest total movement for any of the monitoring stations of 0.3 foot.

Continued inspection of the grassy slope revealed several other potential cracks. Cracks were most easily observed in bare patches, along trails and in areas of rodent activity. Three additional stations were added on the grassy slope above the cut during the period of excavation. When the events from the Coalinga sequence greater than magnitude 4.0 from May 2 to August 1 (Uhrhammer, et al, 1983) are overlayed on the total movements for Stations CT-1 through 6, there appears to be little relation with movement other than the propagation of the first crack in the days following the main shock (see Figure 5). Five of these stations have continued to be monitored monthly at the end of construction.

Five additional stations were added to monitor potential problems within the area of excavation as determined by the geologist and the spotters observing the cut. Two stations were installed in response to a small slipout on the cut face to insure that there was no upslope migration of failure. Two stations were established on the nose on the north end of the cut in an area of seeps where the middle

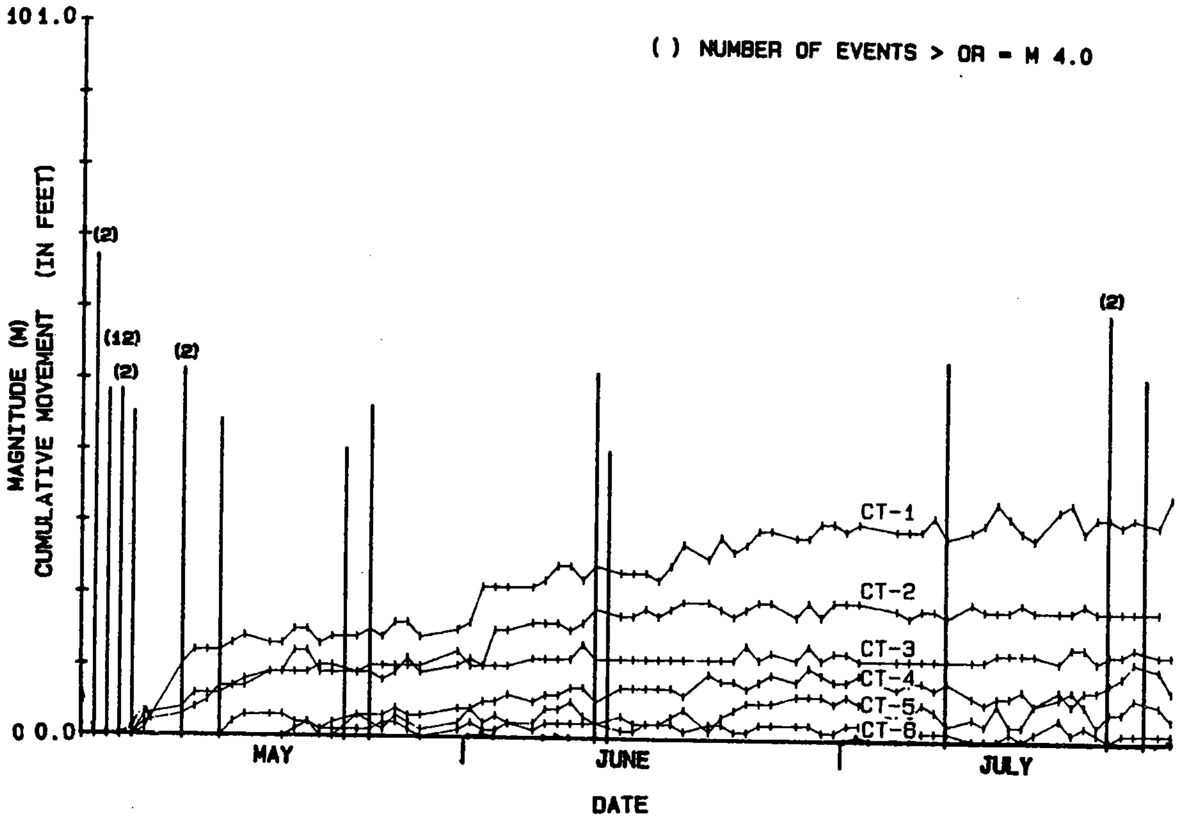

Fig. 5. CUMULATIVE MOVEMENT OF MONITORING STATIONS CT-1 THROUGH CT-6 DURING CONSTRUCTION PERIOD AND LARGER EVENTS OF THE COALINGA EARTHQUAKES 1983.

access road leaves the cut. These stations were installed in response to possible cracking and pulling away of loose rocks on the slope. One station was established in response to a possible crack on the south end of the cut and was abandoned when no measureable movement was detected.

The magnitude of movement was not considered significant with four of the five stations totalling less than 0.05 foot of movement and they were abandoned at the end of excavation.

Debris Benches

Midslope bench: Significant quantities of water were encountered beginning June 15 and preparations were made to leave the midslope bench. Water totalling about 57,000 gpd (gallons per day) was flowing from the cut slope and floor of the excavation. As the cut was lowered, a diagonal trend in the springs became apparent dipping to the north. A temporary bench was formed with the water diverted away to the south. To insure all the springs had been encountered, the bench was lowered an additional 20 feet. The final bench was 15 feet wide and drained the water toward Sycamore Draw.

Initial access to the bench was possible from the middle access road but permanent access for servicing the horizontal drains is via a steep switchback from the lower access road.

Road level catchment area: When the excavation was completed, the additional width between the road and the toe of the slope was used as a combination catchment area and drainage ditch. The long exposed cut consisting of mostly loose, fractured material is sure to erode and this bench provides a buffer against rolling material. Water from the lower springs is directed along the bench toward the bridge.

Horizontal Drains

A total of 21 horizontal drains were installed, the majority of which were drilled along the horizontal bench. Nine horizontal drains were drilled along the midslope bench. The first four holes had strong initial flows but tapered off as the next hole was drilled. Holes 5 and 6 also produced substantial water. The initial flow in Hole 6 was 86,200 gpd which levelled off to a constant 57,000 gpd.

Four horizontal drains were drilled below the midslope bench when additional substantial springs were encountered. A temporary platform was left for the drill rig while the rest of the cut continued to be lowered. One of these holes produced a maximum initial flow of 43,200 gpd.

Two holes were drilled to tap a seep area on the south end of the cut below the midslope bench. Neither one of these holes produced any water.

Five horizontal drains were installed at road level south of the bridge. These drains produced a maximum initial flow of 61,700 gpd and effectively dried up the springs immediately adjacent to the road.

Additional spring areas in the bowl of the excavation were inaccessible by the drill rig at road level.

Drain Water Diversion

Following the completion of excavation, the horizontal drains on the midslope bench were collected in an 8-inch CMP and routed off the north end of the bench into Sycamore Draw. This manifold was only able to collect about 50 percent of the total quantity of water on the bench. The additional water is flowing around the casing in the drain holes or popping out of the slope as springs.

An attempt was made to force more of the water through the drain pipes by sealing off the enlarged holes at the surface. During drilling, bentonite and rags were unsuccessful in sealing off some holes because of the quantity of water and fractured nature of the rock. Following the end of excavation, five minute setting cement was employed but was also unsuccessful as the water continued to find a path around the plug through the fractured rock. Although the collector system is not totally effective, most of the water is running off the bench into the draw and the drains are successfully reducing the pore pressure in the hill.

The flowing drains below the bench were collected in PVC pipe and routed down the slope to road level. No attempt was made to install a collector system for the drains on the south end of the cut because they have not produced any water.

HUMAN IMPACT

In addition to the impact to the tourist industry, in the community of Partington Ridge, up to 60 people were isolated for nearly three months with the closure of the road by a huge landslide on May 1 less than two miles to the south. Many people working in Big Sur were forced to walk around the slide on a trail twice daily. Several people were able to park a car on either side of the slide before the road had closed substantially shortening the walk. Fuel and supplies were flown in weekly by National Guard helicopter.

One construction worker was killed cleaning up the first slide when the second slide occurred.

CONSTRUCTION COSTS

The work was performed under emergency contract for about $400,000. Additional expenses were incurred to pay the spotters and install the horizontal drains.

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Slope Stability Monitoring in the Digital Age

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ABSTRACT

Recent advances in digital data acquisition equipment and in geotechnical instrumentation provide engineering geologists and geotechnical engineers with heretofore unknown capabilities for monitoring ground stability. It is now possible to instrument a slope with tiltmeters and other devices, fully automating the data acquisition process using dispersed data loggers or a central microcomputer-based recording system. Software-controlled display options allow stability conditions throughout the slope to be assessed within minutes. In most cases the same microcomputer that performs field data collection can be used in the office for standard functions such as bookkeeping and word processing.

Modern data collection and analysis methods benefit the geotechnical professional by increasing productivity, reducing turnaround time, enhancing the quality of interpretations, and broadening the range of services available to clients. Continuous monitoring of slope stability using digital systems provides early warning of impending hazards and can reduce liability risks by permitting problems to be detected and addressed before they occur.

1.0 INTRODUCTION

The purpose of this paper is to present an overview of current state-of-the-art capabilities for monitoring ground deformation and slope stability. The paper focuses on the two principal aspects of monitoring: data measurement and data recording. The transmission of data from measurement instruments to recording equipment is also discussed. This presentation is not intended to provide an exhaustive review of the wide range of measurement and recording devices currently on the market. Rather, the emphasis is on capabilities and the choices that have emerged in recent years. Recent advances in sensor design and digital electronics provide geotechnical professionals with an unprecedented opportunity to increase the cost-effectiveness of their work, open new markets for their professional services, and increase the value of the services they render to their clients. It is these advances and their applications that will be addressed.

Development of automated data collection, processing, display, and control systems has been dramatic in a wide spectrum of fields in recent years. Some examples of industries and applications include the following:

- Control systems for the chemical, paper, and refining industries: Automated monitoring and control of batch processes, including adjustment of feedstock streams, control of temperatures and pressures, and automated warning systems.

- Geophysical exploration: Multichannel high-speed digital data acquisition systems, automated processing and playback procedures.
- Production geophysics: Continuous digital well logging and processing; real-time monitoring of hydraulic-fracturing and other well stimulations.
- Petroleum engineering: Digital measurement and analysis of pressure build-up and flow tests for evaluation of permeability and reservoir geometry.
- Air quality: Continuous measurement of industrial emissions and sounding of warnings at threshhold values.
- Hydrology: Continuous monitoring of water quality, digital recording of well tests, correlation of well field data recorded on a common time base.

As has already been the case in many other fields, modern sampling, recording, and data analysis techniques will soon transform engineering geology and geotechnical engineering. How this will take place is reviewed in the following pages.

2.0 THE EVOLUTION OF RECORDING SYSTEMS

The first system for recording scientific and engineering measurements was the human memory. This system is still in widespread use and has the advantages of requiring no investment in capital equipment and nearly universal availability. An example of effective use of this system would be drilling a hole into a landslide and periodically lowering a broom handle into the hole. If the hole shears off as a result of slippage on a slide plane, the broom handle will no longer fall to the bottom of the hole. The mind remembers this fact as an indication that the slide is moving. There are, however, several disadvantages to this system, including the following: slow, labor-intensive data collection; unreliable information retrieval when data quantities begin to exceed several points; and inability to effectively process large quantities of information.

Engineering geologists and geotechnical engineers long ago discovered that the accuracy and quantity of stored data could be increased by writing measurements down in a log book. This approach constitutes the second stage in the evolution of recording systems and is probably the most commonly used system today. It, too, has the disadvantage of being costly because it is labor-intensive and limits the geologist-engineer to slow data collection rates. Furthermore, few forms of processing are available without first manually plotting the data or transferring them to an electronic medium.

With the birth of modern electronics in the twentieth century came the developement of analog recording systems. Analog systems, which were the state of the recording art until the early 1970s, permitted the continuous recording of large quantites of data. Familiar examples of analog systems are strip chart recorders, drum recorders, X-Y recorders, and magnetic tape recorders. One important advantage of analog recording was a bandwidth and amplitude range of stored measurements. Bandwidth is defined as the range of frequencies to which a given device is sensitive. Whereas an engineer can manually record geotechnical data at a rate of perhaps one sample every few seconds (equivalent to a low-pass frequency of about 0.1 Hz), an instrumentation-grade tape recorder is sensitive to signals well above one kilohertz. Such a tape recorder will also resolve slow changes in sensor output (e.g., the output of a strainmeter or tiltmeter) lasting several hours or days. It can record a large number of data channels simultaneously, permitting geotechnical behavior in several parts of a slope or landslide to be simultaneously observed.

Although analog systems represented a great advance over manual recording, they too have certain disadvantages. Among them are:

- The requirement of other specialized analog devices for all processing and playback functions.
- The requirement of sequential rather than random access to stored data leading to long data turnaround times.
- High power consumption, often an important factor in field applications.
- High expense for all but the simplest system.

The great advances in microelectronics and digital logic that have occurred over the last ten years have revolutionized the fields of data acquisition and signal processing. Major innovations continue to be made and applied throughout science and industry and are beginning to appear in engineering geology and geotechnical engineering. Digital systems permit rapid and accurate data measurement and recording, and open up processing and interpretation options not previously available to the practicing engineer. The advantages and capabilities of digital systems are so numerous that they will only be touched upon here. Based on the experience of the author, some of the most important are:

- Ability to monitor a large number of field instruments simultaneously and automatically. A single microprocessor-controlled data logger or computer can monitor from one to several hundred field instruments (piezometers, tiltmeters, extensometers, etc.) at the same time (Figures 1 and 2). Such systems contain an internal, battery-powered clock that provides a common time base for all measurements so that events in different parts of a slope or landslide can be precisely correlated. After the computer program controlling data acquisition has been started, operation is automatic and no personnel are required.
- High data density. Up to 800,000 data points can be stored on one $5\frac{1}{4}$-inch floppy disk. Several million can be stored on a digital tape cassette. Because of these high capacities, a single disk or cassette is sufficient for many geotechnical field projects. Other high-capacity data storage options also exist, such as bubble memory or hard disks.
- Centralized or decentralized field recording. The engineer has the option of recording all data at a central microcomputer station or at several dispersed microprocessor-controlled data loggers (Figure 2). If data loggers are used, the data must be transferred to the

field or office microcomputer for processing at the close of field work.

- Low power requirements. Most microcomputers currently on the market operate on 115 VAC power. Digital data loggers can operate for weeks or months on a single set of batteries.
- Low cost and versatility. Off-the-shelf data acquisition hardware is currently available that interfaces directly with the Apple IIe, the IBM-PC, and other popular microcomputers. Data acquisition parameters (e.g., sample rates, scale factors, and display modes) are controlled by a program in the microcomputer. The microcomputer is also used for all processing and playback functions. When not in use for recording or data manipulation, the microcomputer is free to perform other office functions such as word processing, bookkeeping, etc. Printing and plotting of field data can both be performed on one low-cost dot-matrix printer.
- Compactness. Unlike older analog processing which required a number of specialized devices, all necessary digital processing can normally be done with a single microcomputer system and the appropriate software. Systems are on the market that combine all data acquisition and computing requirements in a single compact package (Figure 1).

3.0 DATA TRANSMISSION FROM FIELD SENSORS TO RECORDING STATION

Several options are available for transmission of geotechnical data from field sensors to the site where the data are recorded. The simplest and usually the least-expensive option is transmission over hard wire (cable). This may consist of transmission to a field data logger no more than a few feet from the sensor emplacement. It could also involve transmission over hundreds or thousands of feet to a central computer.

In projects where transmission distances over several thousand feet are necessary, the most cost-effective option may be radio telemetry (Figure 3). A variety of telemetry modes and bands are available (e.g., digital vs. analog, two-way vs. one-way, UHF vs. VHF), and are usually chosen on the basis of cost, distance, and anticipated interference. If the field site is near a telephone line, digital telephone transmission to the company office is an option. In truly remote field locations, satellite telemetry can be employed.

4.0 MEASUREMENT INSTRUMENTATION

The discussion thus far has focused on recording instrumentation. Proper selection of the field transducers that make the geotechnical measurements is a requirement if the overall data collection program is to operate as desired.

Geotechnical transducers fall into three basic classes: mechanical, electromechanical, and solid-state electronic. In purely mechanical systems a change in the quantity being measured moves an element in the sensor by an amount that is proportioned to the change. This movement may then be amplified mechanically or hydraulically so that the change in pressure, displacement, temperature, etc. can be read to the desired accuracy. An example of a mechanical transducer is the APPLIED GEOMECHANICS

Borehole Creepmeter (extensometer) shown in Figures 4 and 5. Wires extend from this device to different depths in a borehole. If the borehole changes length due to subsidence or if it is sheared off by a landslide, the distance between the top and the bottom of the wires changes, turning sheaves in the instrument. Changes in wire length are measured using graduations on the sheaves and, from this information, the amount of ground movement and location of the slide plane(s) can be determined. Other examples of mechanical transducers are pneumatic piezometers and load cells, standpipes, and water levels sensors. Although they are effective in many applications, purely mechanical transducers cannot be interfaced to a digital recording system because they do not produce an electrical output.

Electromechanical instruments comprise the next class of geotechnical transducers. In these instruments the measured mechanical movement excites an electrical circuit that puts out a signal, normally a voltage, that is proportional to the movement. Instruments that rely upon the movement of a wire over a sheave or capstan can be converted to electromechanical operation by adding a potentiometer that is turned by the mechanical rotation. Such instruments include most extensometers, creepmeters, strainmeters, and water level sensors. The inclinometers used to measure the deflection of slotted casings in boreholes (slope indicators) are also electromechanical devices. Their sensing element consists of a small magnet cantilevered in a magnetic field that surrounds a set of coils. The position of the magnet changes as a function of borehole inclination, inducing changes in the current passing through the coils. This current is calibrated to provide a measure of hole inclination. Most electromechanical transducers are fully suited for use with digital recording systems. What is normally required is that their output be a variable voltage within the input range of the data acquisition device, normally 0-5 VDC, $\pm$ 5 VDC or $\pm$ 10 VDC.

The final class of geotechnical instruments is comprised of solid-state electronic transducers, i.e., transducers that have no moving mechanical parts. The great advantage of these instruments is enhanced reliability stemming from the absence of mechanical elements that can malfunction or wear out. Load cells, piezometers, and other devices incorporating resistive-type strain gauges as the sensing element are in this category, as are tiltmeters (inclinometers) that use electrolytic bubble level sensors (Figures 6 and 7). These instruments all produce signals that are fully compatible with most modern digital recording systems.

5.0 CONTINUOUS DIGITAL MONITORING OF GROUND DEFORMATION: AN EXAMPLE

The previous pages have outlined the capabilities of modern field recording methods and the instrumentation requirements for interfacing to an off-the-shelf digital recording system. The following paragraphs present an example of how continuous detection of ground deformation can be used to monitor the stability of a landslide. I base my example on many years of experience in monitoring the ground deformation around wells during fluid injection. Instrumentation requirements are the same in both cases, and the abundant data from these injection tests serve to illustrate the power of this approach and the many advantages it offers to both the project engineer and the client.

Figure 8 is a diagrammatic illustration of a landslide containing an array of borehole tiltmeters that provide continuous detection of ground deformation. These tiltmeters are connected to a digital recording system at the crown of the landslide. Tiltmeters used for landslide monitoring should be selected in advance to provide the sensitivity and dynamic range appropriate for the range of ground movements that is anticipated. Tiltmeters manufactured by APPLIED GEOMECHANICS can be supplied with a variety of sensors, including those with earth tidal resolution (Figure 9) and others with a full dynamic range of $\pm$ 60 degrees.

Under stable ground conditions the tiltmeter array in Figure 8 registers no movement or only small random movements that do not correlate across the array. Stable periods appear as horizontal lines in the time series data (Figures 10 and 11). In contrast, unstable periods appear as steeply inclined segments in the time histories (Figures 10 and 11), indicating rapid ground movement. Changes in movement rates during a period of instability are also visible in the time histories (see, for example, the slope segment labeled "December 1" in Figure 11).

Ambiguities in the source of measured ground movements can normally be eliminated by correlating measurements from different tiltmeters. These correlations can be performed rapidly on a microcomputer in a variety of ways. For example, Figures 12 and 13 show the coherence among ground movements caused by different episodes of injection into a subsurface reservoir. The tilt data are presented as vectors indicating the direction and magnitude of ground tilt at each site. In Figure 12 the injection caused a bulging of the ground surface to the northwest of the injection well. In Figure 13 the injection entered the formation differently and caused ground subsidence along a northwest-southeast trending axis. Although Figures 10-13 are taken from well tests, the analogy to landslide-induced ground movement is apparent: large-scale slope deformation will produce movement across an instrument array that is correlative in both the temporal and spatial domains.

The approach to slope stability monitoring outlined above offers advantages to both geotechnical professionals and their clients. Among these advantages are the following:

- Continuous monitoring of slope stability rather than brief glimpses of slope behavior isolated in time and space.
- Low system maintenance after set-up.
- Rapid, cost-effective data gathering, processing, presentation, and interpretation, leading to high productivity.
- Improved ability to correlate slope behavior with environmental parameters, such as storms, tides, and earthquake activities.
- Performance of field transducers can be diagnosed from a central recording station.
- Ability to detect progressive slope failure and to provide an early warning of impending hazards.
- Long-term monitoring is simple and inexpensive.

6.0 CONCLUSIONS

Modern digital data acquisition and processing systems have already seen widespread use in manufacturing, exploration geophysics, and the chemical and petroleum industries. As the technology has advanced, costs have come down while capabilities have increased. Digital recording and processing systems are now beginning to appear in engineering geological and geotechnical applications. This trend will accelerate in the future. Modern digital systems benefit the geotechnical professional by increasing productivity, enhancing his interpretive skills, and broadening the range of services available to clients. Clients benefit from a more complete understanding of geotechnical conditions, rapid reporting times, and early warning of impending hazards. Furthermore, the capability for continuous stability monitoring using tiltmeters and other sensors can reduce professional liability risks by permitting problems to be detected and corrected before they occur.

The low cost and versatility of digital systems makes them accessible to most consulting firms and government agencies. The same microcomputer that controls data acquisition and performs analysis can be used in the office for standard functions such as bookkeeping and word processing.

Figure 1. (following page)

The APPLIED GEOMECHANICS "Little Aggie" data acquisition system. This high-precision 16-bit system combines all essential data acquisition hardware into a single enclosure: microcomputer, analog-to-digital converters, serial communications, and mass storage.

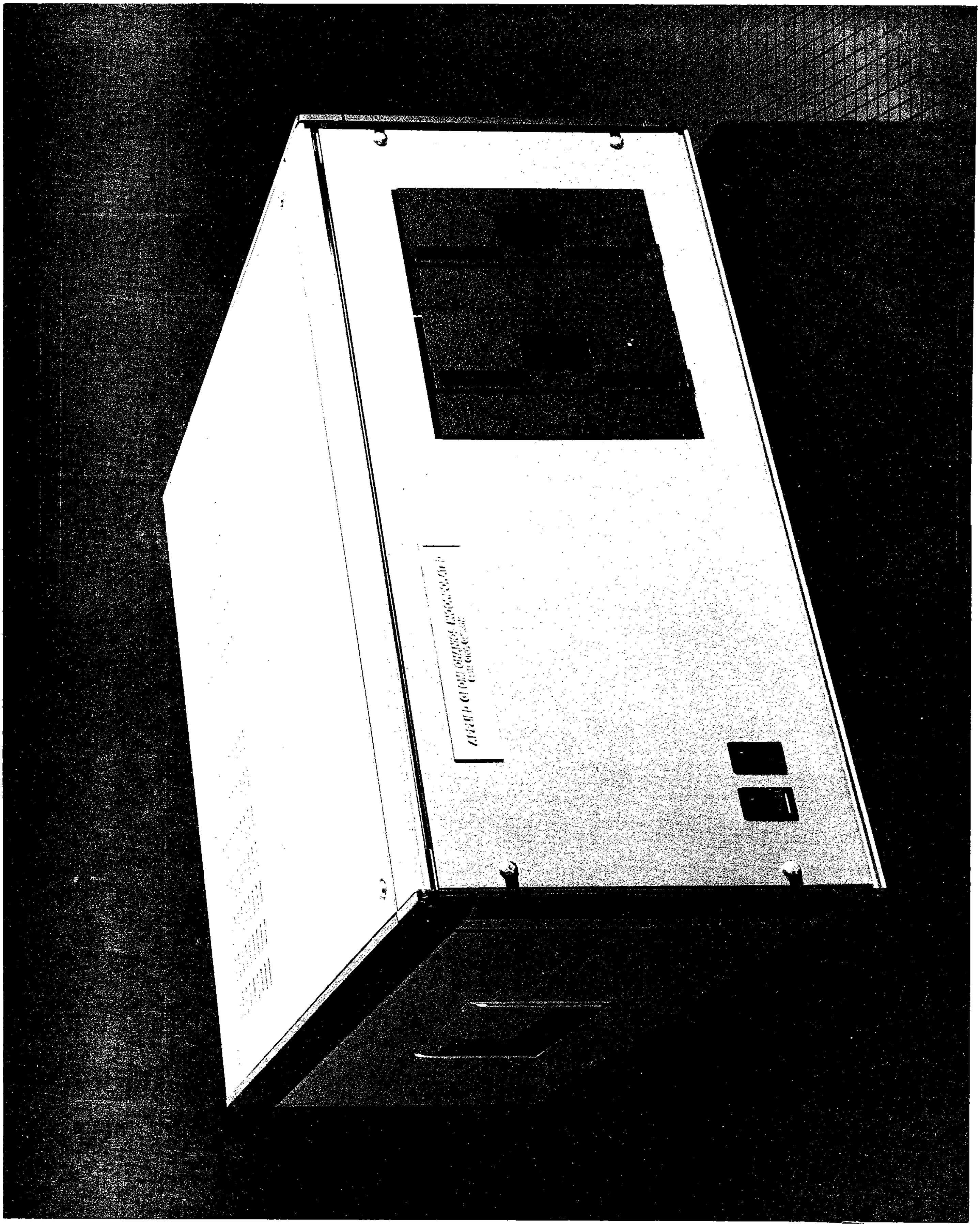

Figure 1

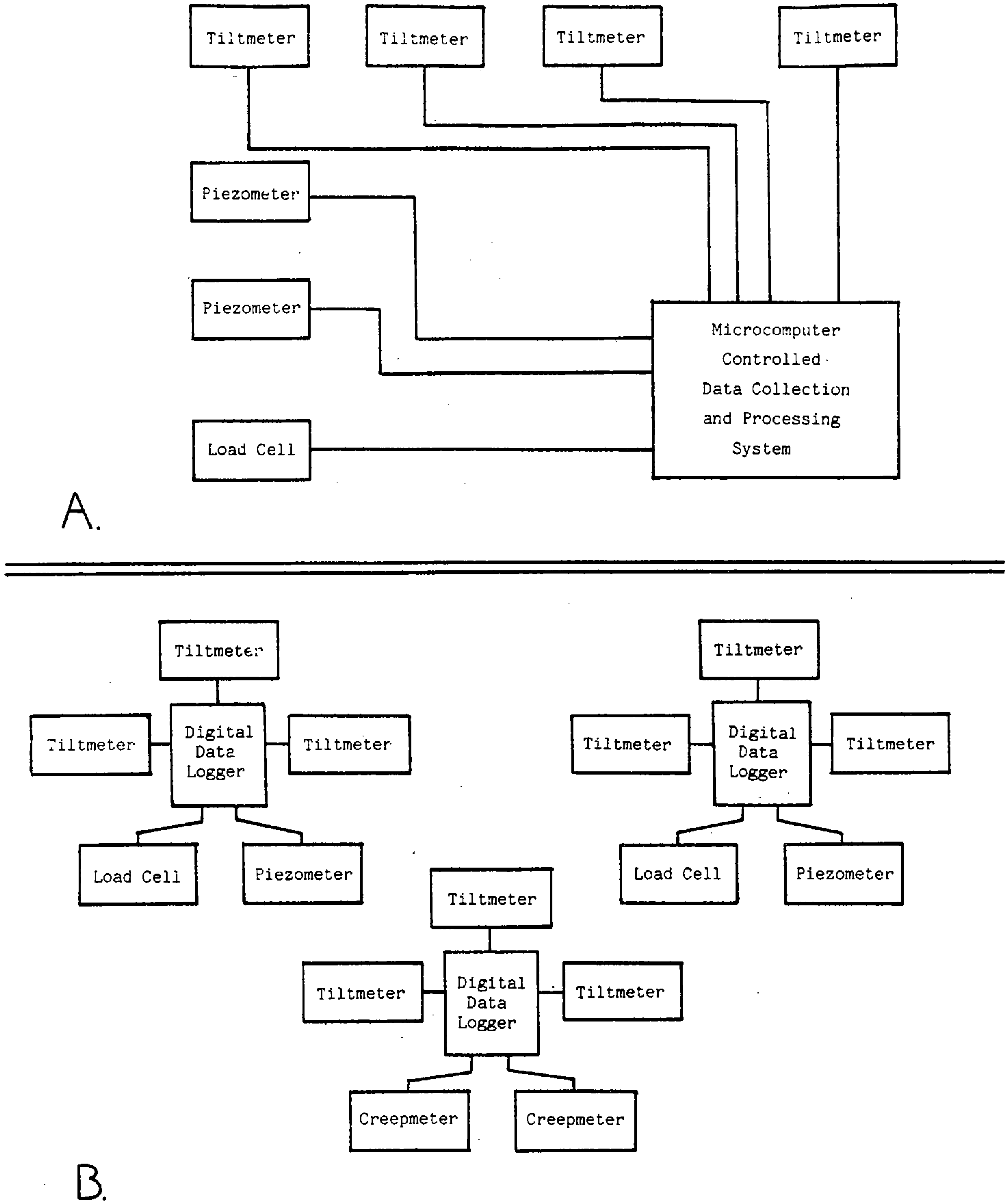

Figure 2. Flow of data to digital data acquisition units in a field program: (A) Centralized approach -- data from field transducers flows to a single microcomputer-controlled unit such as the "Little Aggie" pictured in Figure 1; (B) Decentralized approach -- data from clusters of field transducers flows to data loggers dispersed throughout the project area. In approach (B) the data are periodically transferred to a microcomputer from cassette, bubble memory, RAM pack, etc.

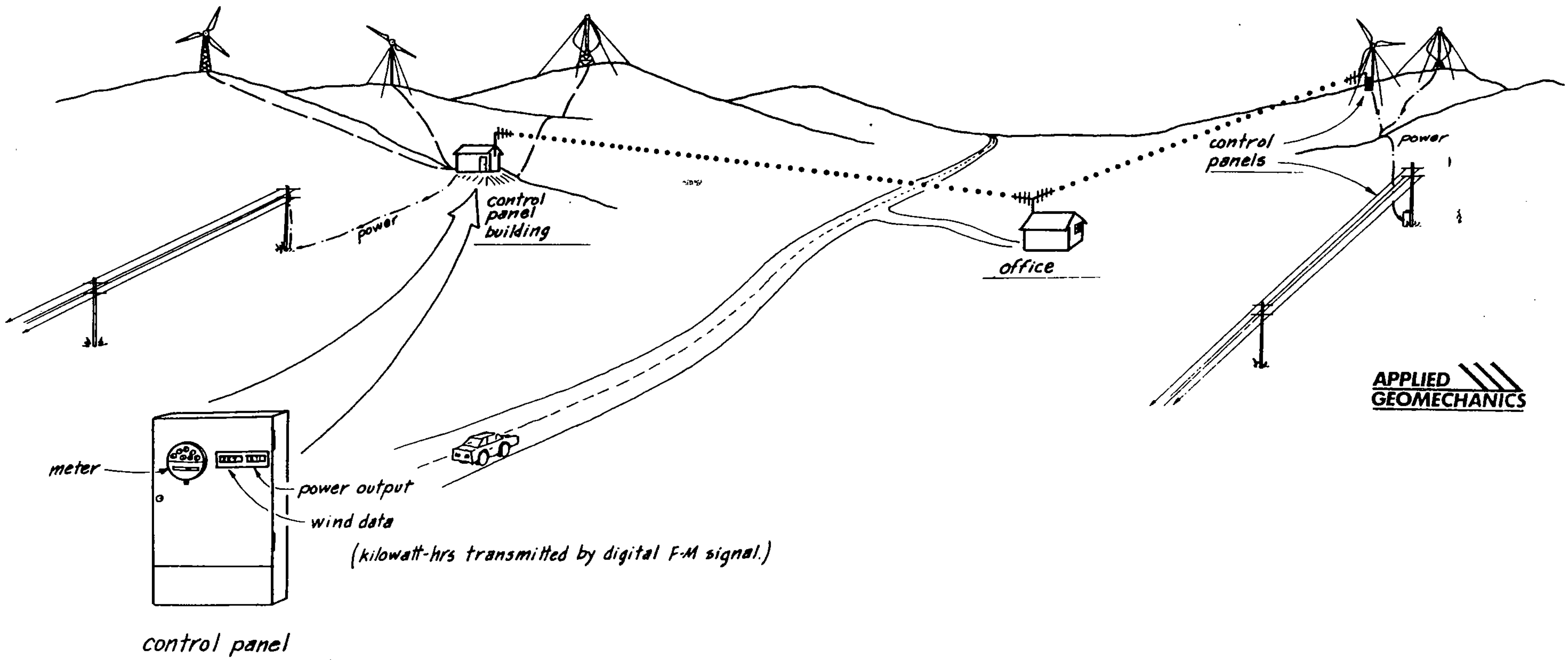

Figure 3. Schematic diagram of radio telemetry of field data from remote sites (in this case, windmills) to a central control station in a field office.

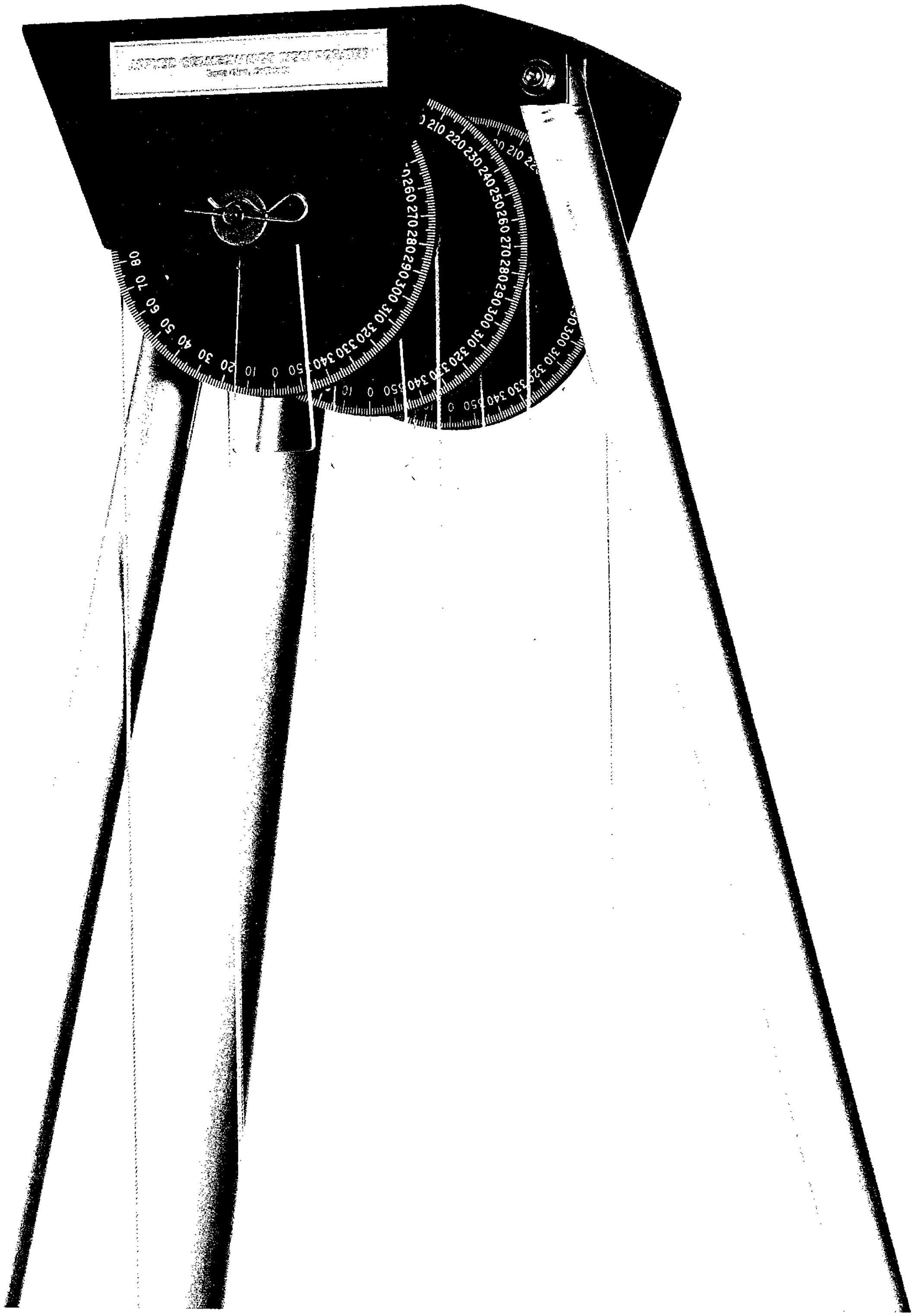

Figure 4. APPLIED GEOMECHANICS AGI-200 Borehole Creepmeter. This is a purely mechanical transducer that measures the lengthening or shortening of wires anchored at different depths in a borehole.

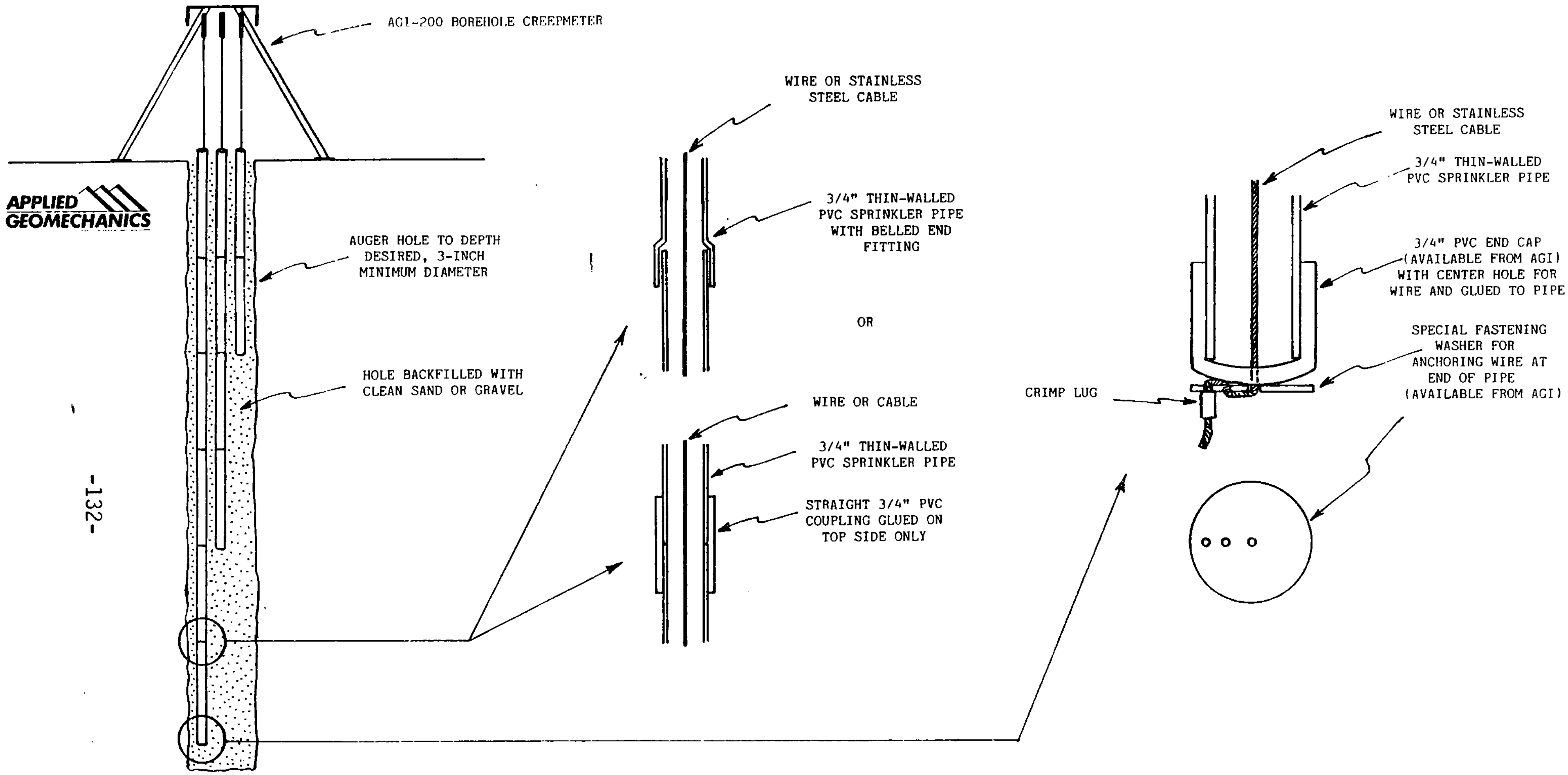

Figure 5. Installation diagram for the AGI-200 Borehole Creepmeter.

Figure 6. (following page)

APPLIED GEOMECHANICS AGI-500 Borehole Tiltmeter. Tiltmeters manufactured by Applied Geomechanics produce an electrical output that is compatible with most digital data collection systems. These instruments use solid state electronics and contain no moving mechanical parts. The optional controller box pictured in the photograph is used for adjusting sensor position after the tiltmeter has been installed in a borehole.

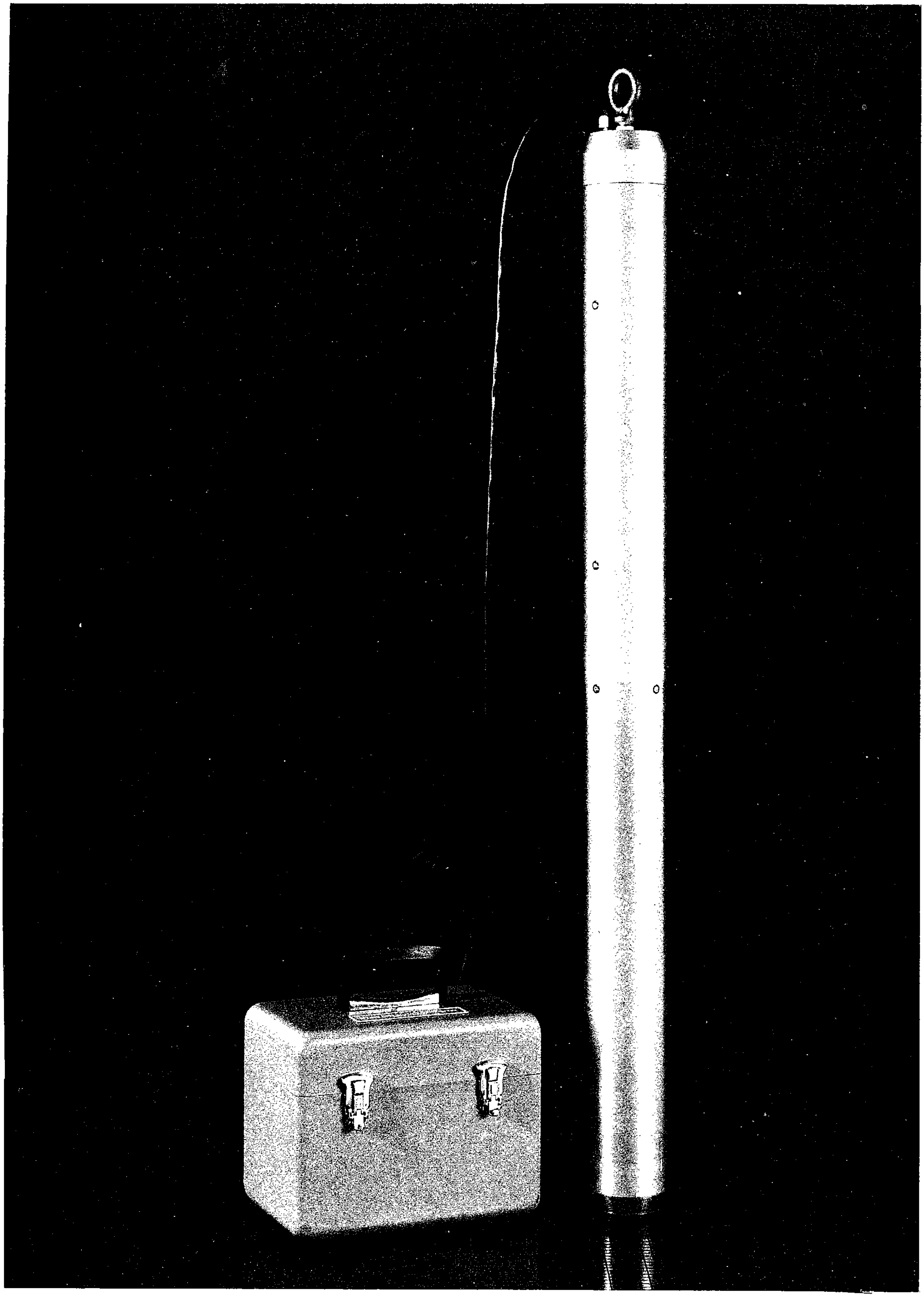

Figure 6.

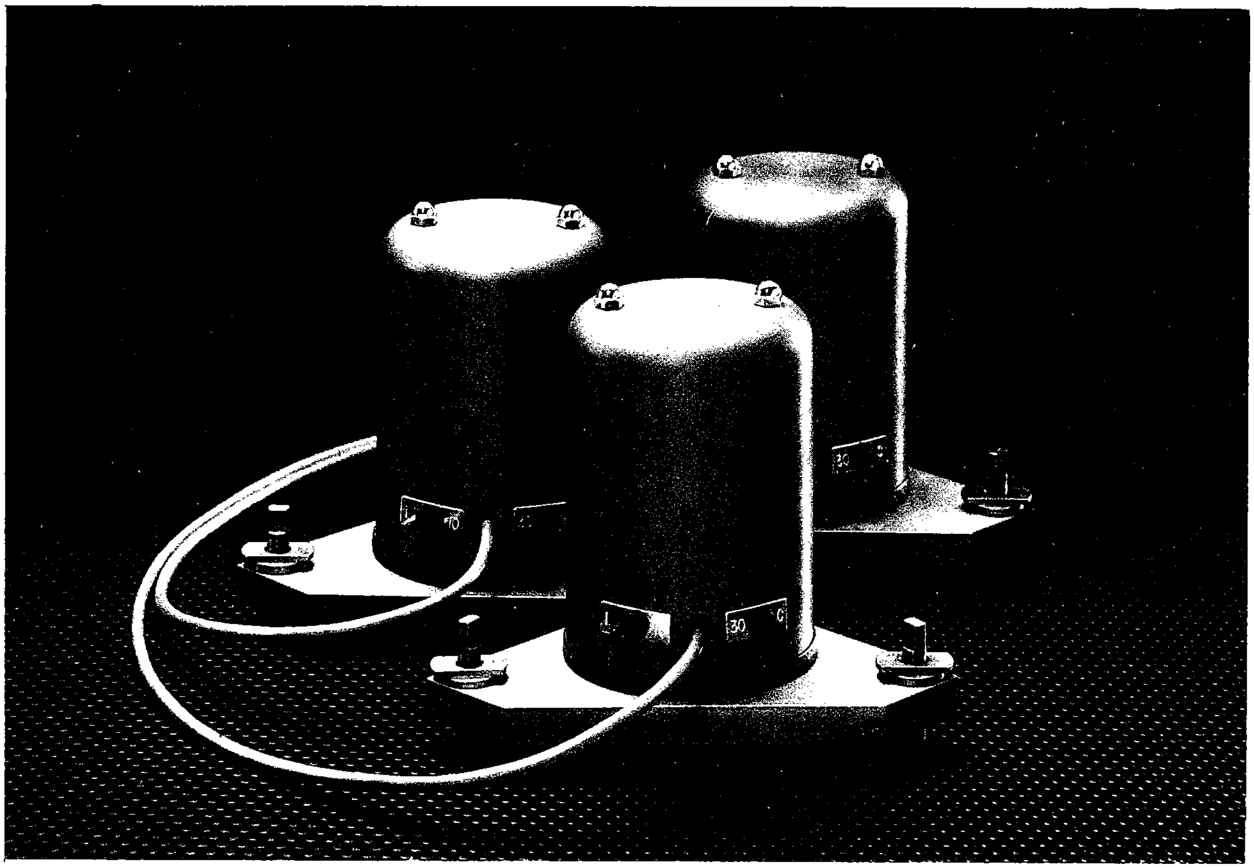

Figure 7. AGI-700 Surface-Mount Tiltmeters. Tiltmeters that mount directly on the surface of the ground or of a structure, rather than in a borehole, are required in certain geotechnical applications.

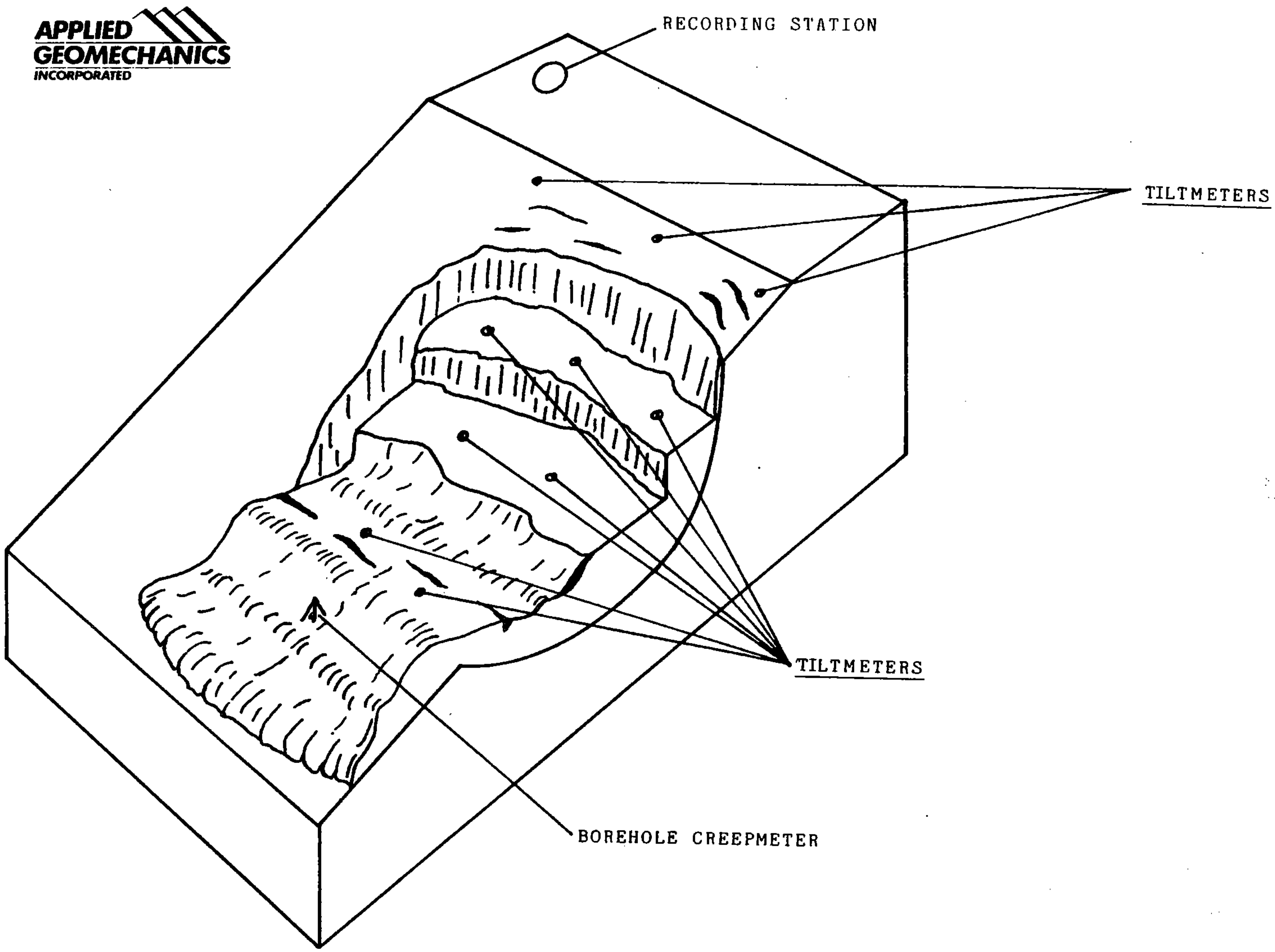

Figure 8. Schematic diagram of a landslide containing a network of geotechnical sensors. Continuous monitoring of slope stability is performed by a microcomputer at a central recording station near the landslide crown.

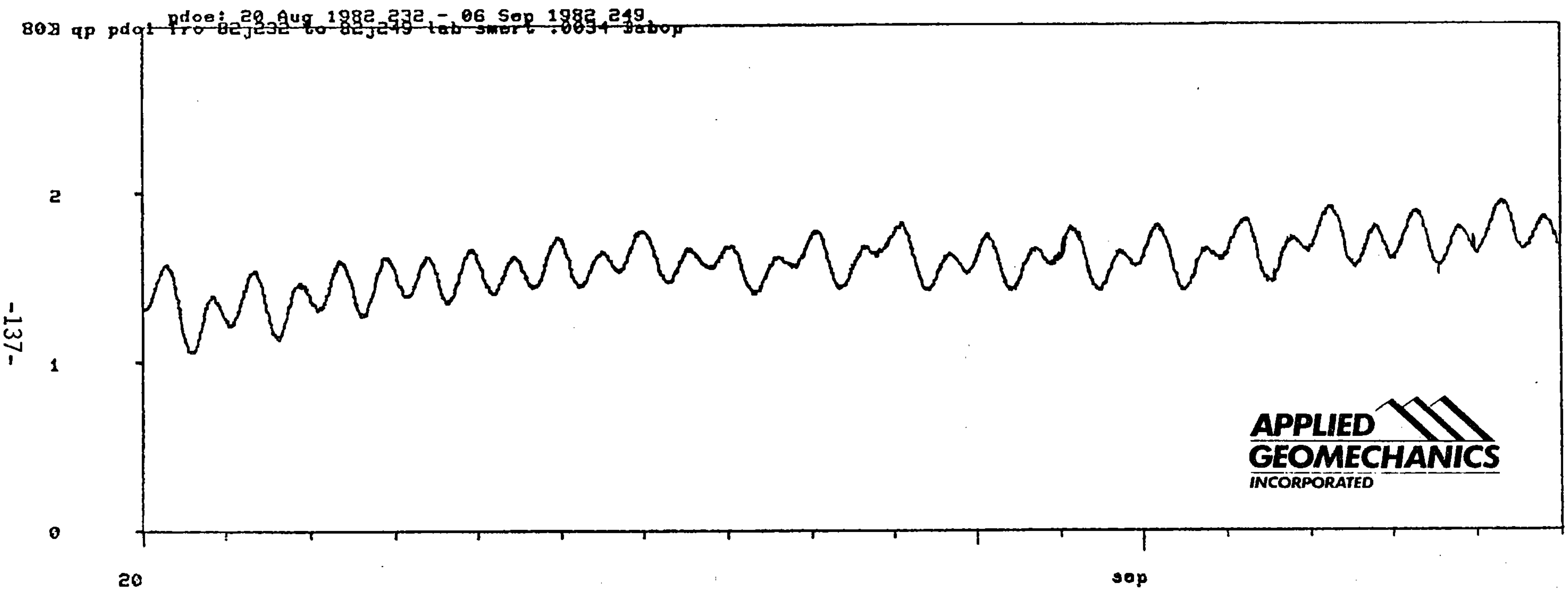

Figure 9. An example of the measurement sensitivity achievable with tiltmeters that use electrolytic bubble level sensors: Solid-earth tides measured with an AGI-500 borehole tiltmeter. Each division on the vertical axis is one microradian.

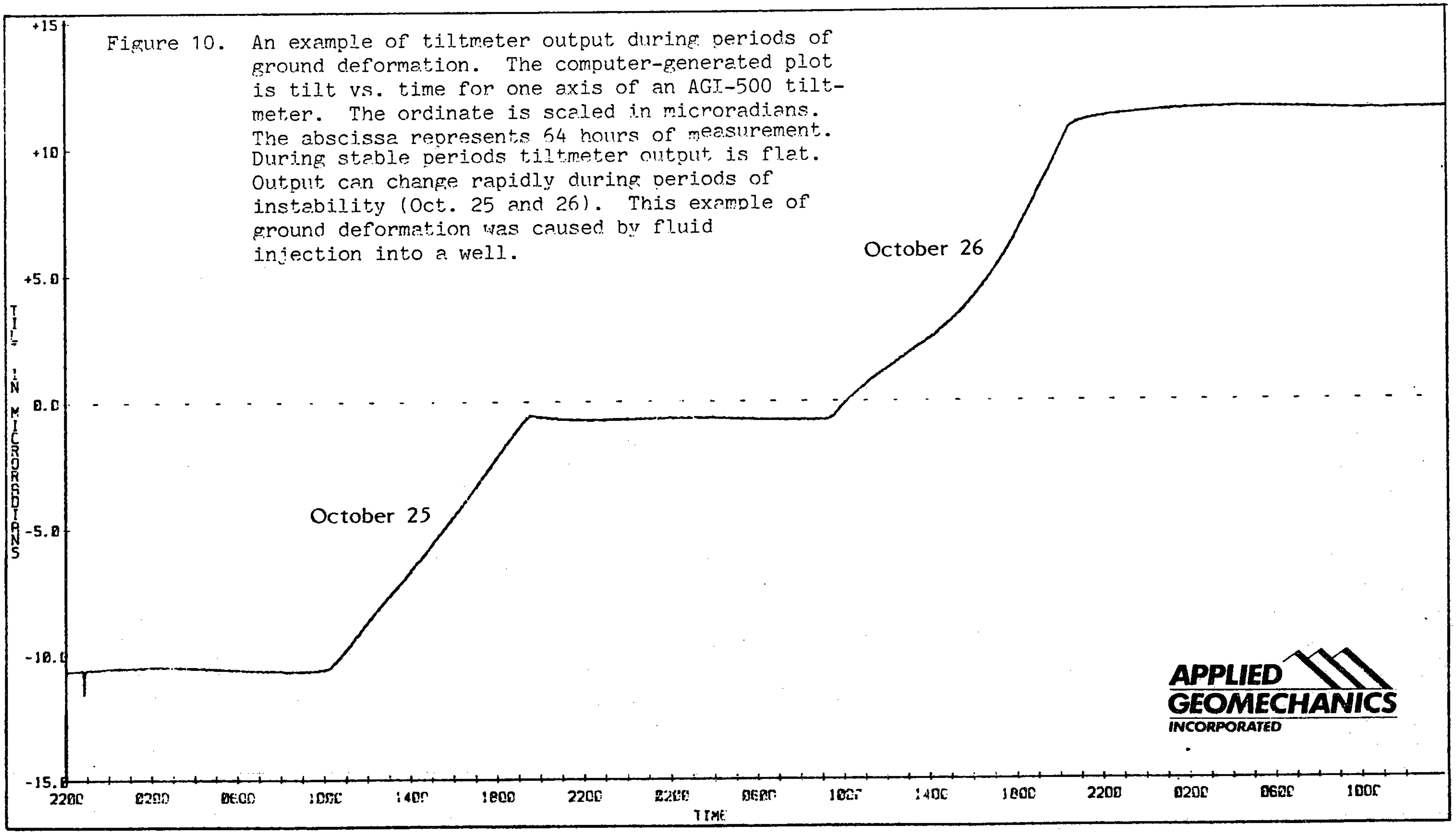

Figure 10. An example of tiltmeter output during periods of ground deformation. The computer-generated plot is tilt vs. time for one axis of an AGI-500 tiltmeter. The ordinate is scaled in microradians. The abscissa represents 64 hours of measurement. During stable periods tiltmeter output is flat. Output can change rapidly during periods of instability (Oct. 25 and 26). This example of ground deformation was caused by fluid injection into a well.

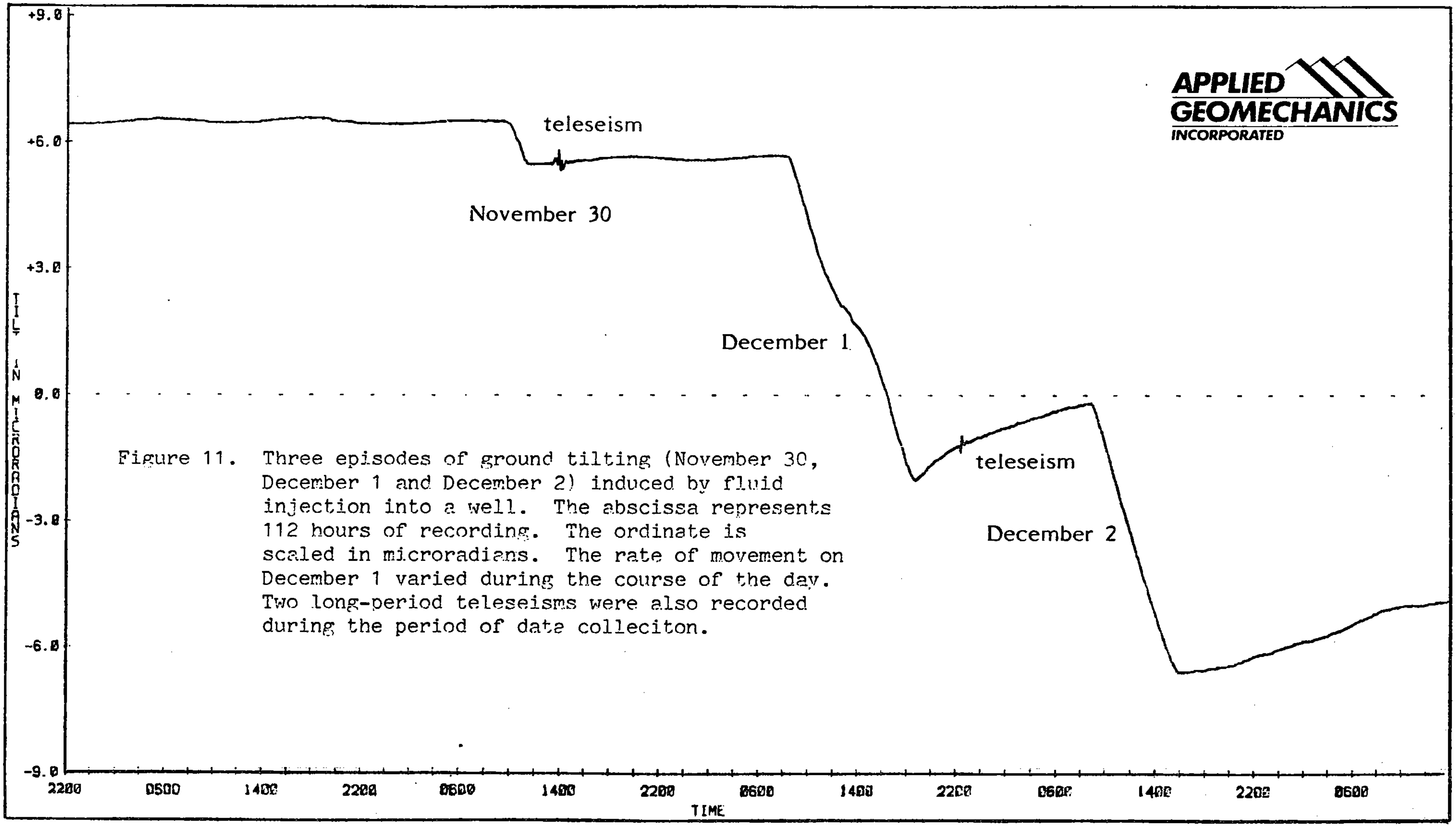

Figure 11. Three episodes of ground tilting (November 30, December 1 and December 2) induced by fluid injection into a well. The abscissa represents 112 hours of recording. The ordinate is scaled in microradians. The rate of movement on December 1 varied during the course of the day. Two long-period teleseisms were also recorded during the period of data colleciton.

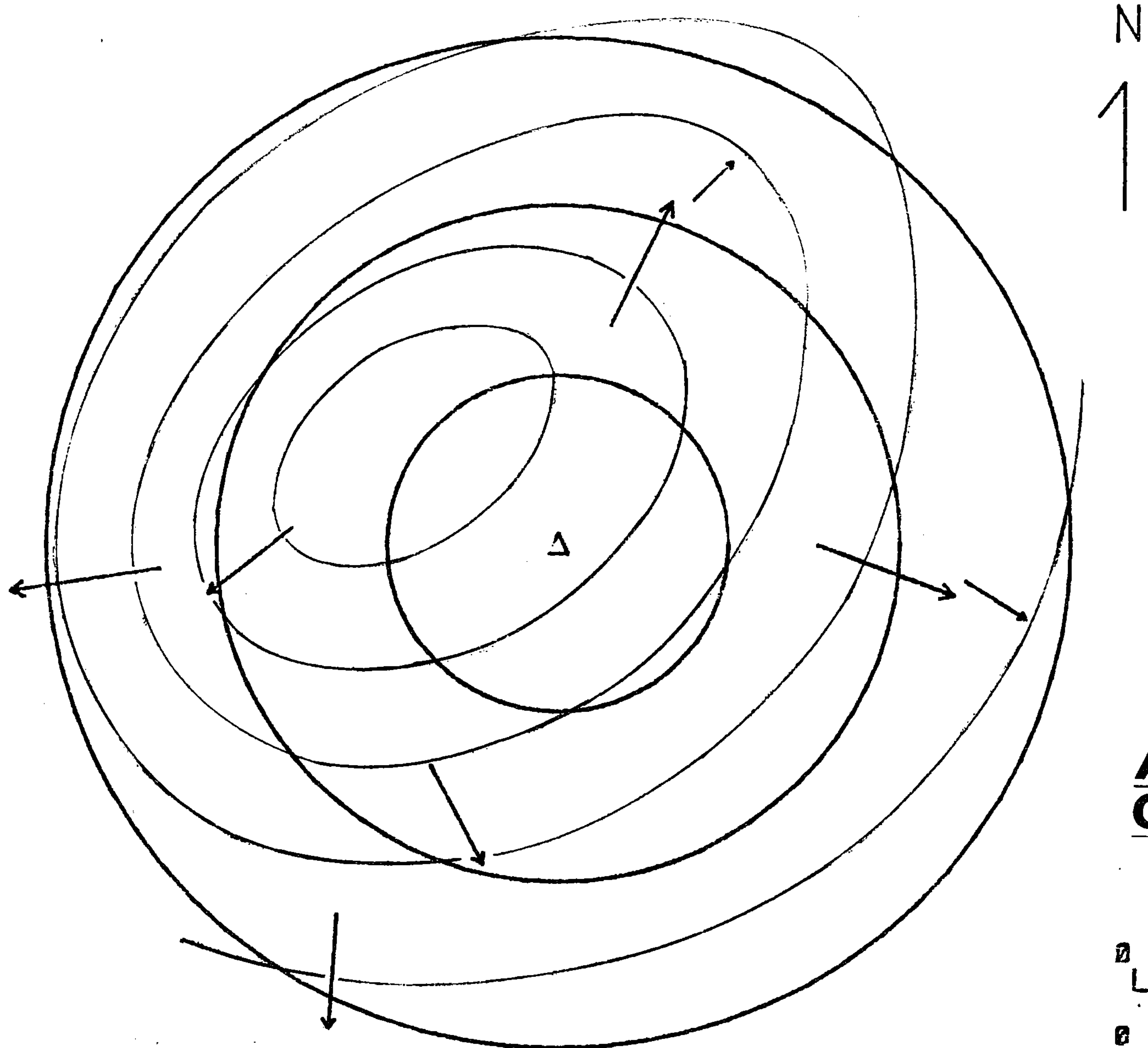

Figure 12.

Map of overall ground deformation resulting from one day of injection into test well #1 (triangle). A tiltmeter is located at the tail of each vector. Non-circular contours are lines of equal uplift (circles concentric to well are for reference only).

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0 250 FEET

0 15 MICRORADIANS

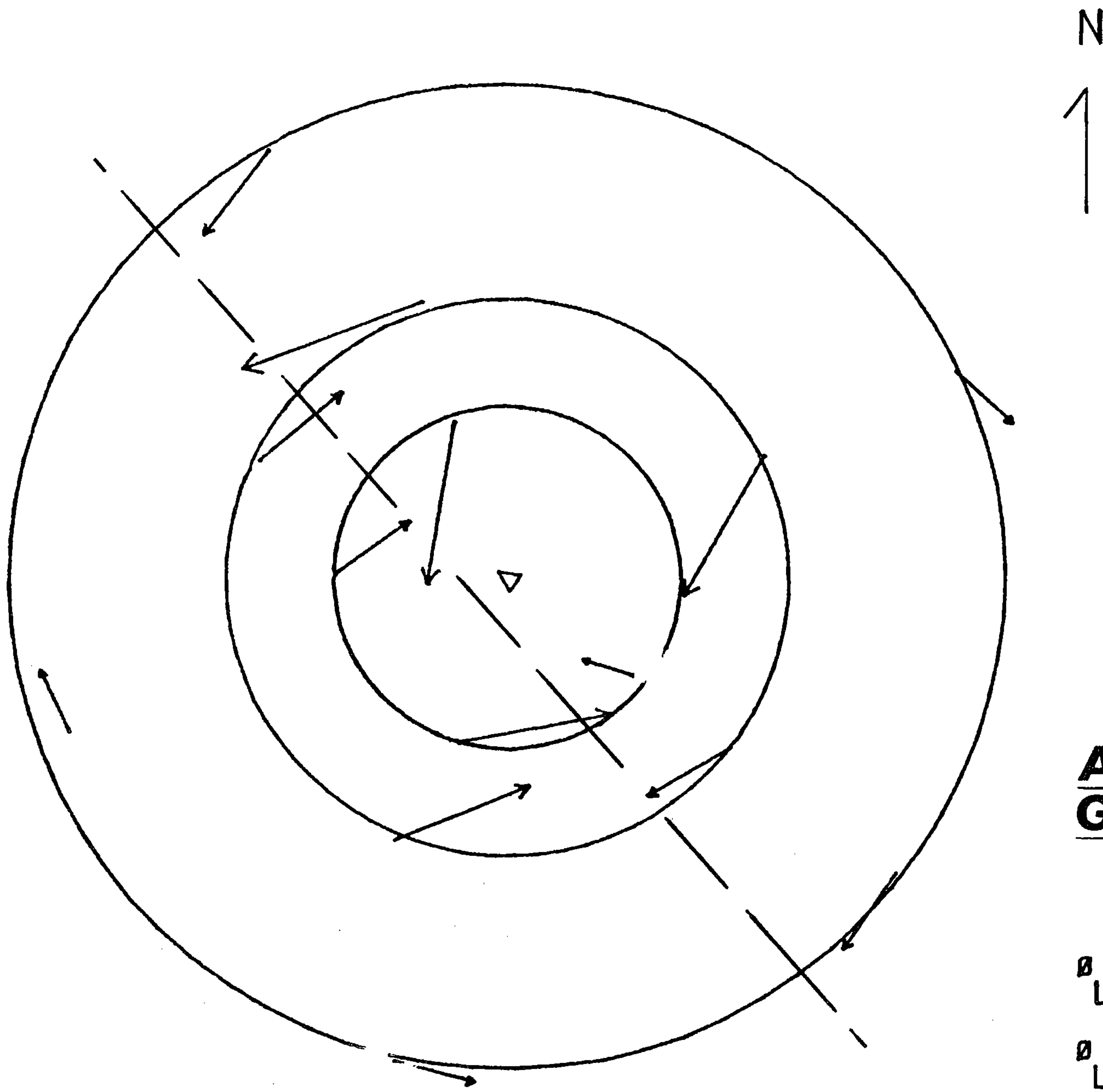

Figure 13.

Map of overall ground deformation resulting from an episode of fluid injection into test well #2 (triangle). Arrows are vectors of tilt.

APPLIED GEOMECHANICS

Ø 400 FEET

Ø .3 MICRORADIANS

GEOLOGIC AND SEISMIC CONSIDERATIONS FOR PROPOSED HIGHWAY BRIDGE SITES NEAR QUITO, ECUADOR

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The city of Quito is planning to construct a new "autopista" (highway) system that would provide a bypass of the city and extend to the proposed site of a new airport. Quito is situated in a zone of high seismic activity within a north-trending graben. The graben is flanked on both sides by volcanoes that have been active throughout the Quaternary. Thousands of feet of volcanic and alluvial deposits have accumulated in the graben. The upper 50 to 100 feet consists primarily of airfall ash called the Cangagua formation. The late Quaternary rate of deposition has exceeded the rate of down-dropping of the graben, resulting in deeply-incised river courses forming deep, narrow canyons in the valley floor.

The canyons represented significant obstacles in the planning of the autopista system in that the alignment was to some degree controlled by the location of suitable bridge sites. Although the Canagua Formation will stand for many years in vertical cuts, massive slumping can occur in high open faces or where it is being undercut, such as canyon walls. Therefore, slope stability was a major consideration at each proposed bridge site.

Seismic design was required for the five highest proposed bridges. The bridges will be made of prestressed concrete with heights varying from 130 to 400 ft. (40 to 120 m) above river level. Based on geologic mapping and review of existing data, three earthquake sources were considered in the design: 1) a magnitude (M_L) 8 on the subduction zone at a depth of at least 60 miles (100 km below the sites; 2) magnitude (M_L) 7 1/2 earthquakes on the graben boundary faults at distances ranging from 2 1/2 to 5 1/2 miles (4 to 9 km); and, 3) magnitude (M_L) 6 earthquakes on nearby faults within the graben at distances of 1/2 to 3 miles (1 to 2 km). The ground motion (0.6 g) from a maximum earthquake on the graben boundary faults predominated and was used in the design. Probability of occurrence was not considered in this investigation.

COMPILERS' NOTE: This abstract was originally prepared for a paper presented at the Highway Geology Symposium. As no proceedings contribution was received, the abstract is reprinted here.

GEOLOGIC HAZARDS AT WADDELL BLUFFS, SANTA CRUZ COUNTY, CALIFORNIA

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ABSTRACT

The Waddell Bluffs area along the coast of Northern Santa Cruz County presents a continuous challenge to highway maintenance. State Highway One was built along a bench of loose debris with high steep cliffs on one side and the Pacific Ocean on the other. Rocks fall regularly from the eroding cliffs and must be intercepted before reaching the traveled roadway. A debris trench at the base of the bluffs was effectively designed but is not always maintained at design conditions. Riprap has been required along much of the highway fill in order to protect the roadway from wave erosion.

INTRODUCTION

The rugged topography along most of the coastline of central and northern California initially presented a major obstacle to transportation. Highway One was literally carved out of rock and perched above the waves for many miles. Problems did not terminate when construction was completed, however. Protection from landslides and rockfalls, the stability of road fill, and wave erosion have created difficulties at many locations, particularly during the stormy winter months. The Big Sur coastline and Devils Slide area, for example, are both well known for their regular closures and warning signs.

The one mile long and 200 to 350 foot high cliffs immediately south of the San Mateo - Santa Cruz County line known as Waddell Bluffs have posed a nearly continuous maintenance problem. Large mudstone blocks break loose periodically from the cliffs and roll downslope. Although most of the rock debris is trapped in a roadside trench, large rocks occasionally reach the traveled roadway. During severe storms, waves attack the highway fill such that riprap has been required along nearly half of the bluff area.

GEOLOGIC SETTING

The Waddell Bluffs area lies along the Central California coast (Figure 1) and forms the seaward edge of the Santa Cruz Mountains, which is a portion of a larger structural and geomorphic unit known as the central Coast Ranges. The area is underlain by a thick prism of sedimentary rocks of late Tertiary age which overlie a granitic basement forming the backbone of the Santa Cruz Mountains.

The entire central Coast Range is tectonically active due to its location along the North American-Pacific Plate Boundary. Although the San Andreas Fault, which passes about 15 miles eastward of the bluffs area, is generally thought of as the plate boundary, it is now known that there are a number of other faults associated with the San Andreas. These branch faults are also active to varying degrees and research in recent years has focused on evaluating slip rates and activity levels along these subsidiary faults.

The San Gregorio Fault forms what is believed to be the western edge of the greater San Andreas Fault system. The San Gregorio is actually a fault zone at least 1.5 miles wide which consists of a number of individual fault traces (Weber, 1980 Figure 2). The easternmost trace passes through the seacliff just north of Waddell Bluffs as it heads southeast along the coastline. The Quaternary (the last 2,000,000 years) slip rate along the fault is about 9-10 mm/year and offset is known to have occurred in Holocene Time (Weber, 1980).

Uplift of the central Coast Ranges combined with the eustatic sea level changes of the late Pleistocene produced a series of well developed wave cut marine terraces along the northern Santa Cruz and southern San Mateo County coastlines (Bradley, 1958, and Bradley and Griggs, 1976). Due to a combination of more rapid uplift and perhaps, more rapid erosion, the marine terraces so well preserved immediately to the north and to the south, between Davenport and Santa Cruz, are absent in the Waddell Bluffs area.

CLIFF FAILURE AT WADDELL BLUFFS

Rockfalls from the steep cliffs are a major hazard for highway users along this one mile stretch of State Highway One. The Santa Cruz Mudstone, which forms the bluffs, is dominantly a thickly bedded to massive sedimentary unit which is intensively fractured and also well-jointed. Due to repeated wetting and drying, heating and cooling, and the effects of wind and salt air the mudstone is continually breaking down (through the slaking process) and sloughing off.

Two distinct lithologic units are exposed in the bluffs which weather quite differently (Figures 3 and 4). An intensively fractured grey mudstone forms the basal unit along most of the bluffs. This material breaks down into small chips and fragments half an inch to six inches in size which are continually falling and rolling down the bluff face. This material accumulates as an apron of talus 15 to perhaps 60 feet high at the base of the slope (Figure 5). Very resistant dolomitic concretions up to a meter across occur scattered through the mudstone. As the mudstone sloughs off from around these concretions they break loose and roll downslope as well.

More resistant siltstone and siliceous mudstone interbeds up to 25-50 feet thick form bold outcrops which support the steep slopes of the upper bluffs (Figures 3 and 4). As the fractured and less resistant grey mudstone weathers and breaks loose, blocks of the well-jointed more resistant beds up to three feet or more across are periodically undercut and detached, some of these are the size of small cars. These large, often equidimensional blocks which fall and roll from heights of 150 to 300 feet are moving rapidly when they reach the highway. Halting these moving boulders before they enter the traveled roadway and containing the smaller debris which accumulates continuously became a problem as soon as the highway was completed.

For years prior to the construction of state Highway One, a wagon road crossed the talus deposits at the base of the high cliffs. Old photographs show a wedge of talus extending from the beach for a considerable distance upslope (Figure 4). Although roadway maintenance at that time was a continuing problem, some degree of stability had been reached as indicated by pine trees growing in the talus.

During construction of Highway One along Waddell Bluffs in the late 1940's, substantial cutting and talus removal took place for a distance of about a mile. Fifty to 120 feet of horizontal cutting into the loose hillslope for a roadbed steepened the lower slopes on the average from 32^{o} to 45^{o} - 54^{o}, and required the removal of about 980,000 yds^3 of material. This grading produced disequilibrium along the lower slopes and rock falls and sloughing of weathered material have continued to the present.

Approximately 15,000 to 18,000 cubic yards of material are removed annually from a catch basin at the base of the slope (Figure 5). The clean out is usually done only once a year during the fall. Maintenance equipment also removes material when it reaches the highway and occasionally will clean portions of the trench at other times if they look particularly full. Ocean disposal on the opposite side of the highway of this material was practised for a number of years. The loose debris served to buffer the fill supporting the roadbed to some degree, although most of the mudstone was quickly broken down and carried offshore in suspension. In recent years a concern has been raised by the Coastal Commission regarding the impact of the fine-grained suspended material on marine life. As a result, disposal of the annual accumulated talus became a problem. Much of the weathered mudstone was, therefore, placed along the shoulder of the highway in the area to the north and south and used to form pullouts and parking areas.

The problem of rock falling and rolling off the high bluffs was recognized during planning stages in 1946. The California Division of Highways at that time realized that the bluffs would continue to slough and ravel and considered both a barrier wall and a debris trench at the base of the bluffs adjacent to the roadway in order to prevent falling material from endangering traffic. The barrier wall at that time was considered quite costly, aesthetically unacceptable and to be used only as a last resort. A trench could be cleared out periodically, or as required, with little or no inconvenience to traffic. Original traffic projections during planning stages estimated a 10 year total of 2.8 million vehicles. By 1977, however, the annual count was 1.4 million, a rate five times greater than that originally estimated.

It is uncertain how the original dimensions of the debris trench were determined, but a trench six feet deep and fifteen feet wide was included in the original plans (See Figure 6). Some years later, Arthur Ritchie (1963) of the Washington State Highway Commission performed a series of well documented field experiments in order to evaluate the control of rockfalls of this sort. His experiments produced debris trench dimensions for various slope angles and heights which have apparently been relied on and widely adopted (Figure 7 and Table 1). For rock slopes such as those at Waddell Bluffs (over 60 feet in height and a slope of about 0.75:1 (53^{o})), a trench fifteen feet wide and six feet deep would be required. Thus the original plans called for a trench of the same dimensions as were determined necessary by Ritchie.

The trench serves two slightly different purposes, 1) it collects all of the small chips and fragments of mudstone which accumulate continually as talus at the base of the cliffs, (Figure 5) and 2) it intercepts large rocks which are rolling and bouncing down the bluffs before they reach the Highway (Figure 8). Cal Trans personnel estimate that only a few percent of the 15,000-18,000 cubic yards of debris which accumulates annually is greater than 12 inches in size. It is these few percent, however, which are of concern to passing motorists.

Both Ritchie's design and the original as built plans called for a trench with a 1:1 slope of its outer wall (Figure 6). Ritchie (1968, p. 19) comments specifically on this aspect of trench design: "when gentle off-shoulder slopes, designed for the open road, are brought into rockfall areas, they provide the ramps for stones to come on the highway. Therefore, there are urgent and compelling reasons to substitute a steep shoulder design in place of a gentle one, for what was thought to be a feature contributing to the overall

TABLE 1. Relationship of variables in ditch design for various slopes. (from Ritchie, 1963)

Rock Slope

Angle	Height (m)	Fallout Area Width (m)	Ditch Depth (m)
Near vertical	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	>20	6.1	1.2
0.25 or 0.3:1	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	20 to 30	6.1	1.8[a]
	>30	7.6	1.8[a]
0.5:1	5 to 10	3.7	1.2
	10 to 20	4.6	1.8[a]
	20 to 30	6.1	1.8[a]
	>30	7.6	2.7[a]
0.75:1	0 to 10	3.7	1.0
	10 to 20	4.6	1.2
	>20	4.6	1.8[a]
1:1	0 to 10	3.7	1.0
	10 to 20	3.7	1.5[a]
	>20	4.6	1.8[a]

[a] May be 1.2 m if catch fence is used.

safety of the highway has now been proven to be in rockfall areas, a detriment and a hazard".

In actuality, the highway at Waddell Bluffs is separated from the roadway by an earth berm two to three feet high, and the outer wall of the trench after maintenance cleaning is close to vertical at most locations (Figure 10). Both represent improvements in Ritchie's design, by placing a barrier between vehicles and the trench, and by providing a more effective backstop for rocks with high angular velocity.

Cal Trans personnel estimate that the debris trench traps about 99 percent of the rocks falling from the bluffs with the remaining one percent bounding through the trench to reach the highway Figure 9). The problem at Waddell Bluffs lies principally in trench maintenance. Typically Cal Trans maintenance personnel inspect the trench briefly on a somewhat regular basis (perhaps 15 minutes is spent weekly on inspection), and then a yearly clean out is performed in the fall or when material reaches the roadway. Along the southerly half of the bluff, rock fall from the cliff is far less common such that the yearly trench cleanout is quite adequate, and design trench dimensions are usually maintained.

Slope failure and rock fall are more common along the northern end of the bluffs such that the trench in this area is commonly in a partially full or overflow condition during many months of the year (Figures 9 and 10). As the trench fills and debris piles up against the outer wall to form a ramp, large rolling rocks are able to bound through the trench and will occasionally reach the traveled roadway where they can become serious hazards to motor vehicles and their occupants (Figure 11). Ritchie's (1963) experimental work set standards for mitigating rock fall hazards through trench design. There are many variables involved which need to be considered at any particular site. Mearns (1976), for example, discusses a catchment trench constructed at the base of a rock slope in the Sierra Nevadas to reduce rock fall hazards. Although Ritchie's criteria were used, the hard granitic bedrock floor of the trench enabled rocks to bounce through the trench. A layer of sand or pea gravel (well drained to prevent winter freezing) proved to be an effective addition in reducing the velocity of rolling rocks. The floor of the Waddell Bluffs trench provides a similar cushion.

The design dimensions, however, must be maintained if the bouncing and rolling rocks are to be kept off the roadway. Trench inspection on a number of site visits in late 1982 and early 1983 showed the northern half of the trench to be over half full and that bounding rocks were regularly reaching the roadway (Figures 10 and 11). According to Ritchie's observations, rock slopes of 0.75:1 (53 degrees) and over 60 feet in height (at Waddell Bluffs maximum heights are over 350 feet) would require a trench 15 feet wide and 6 feet deep. Depths along much of the northern half of the trench in early 1983 were only two to four feet (Figures 8 and 10).

The death of a motorist whose vehicle was struck by a large airborne rock in 1976 led to a major law suit against Cal Trans. A solution to this problem lies in either a clean out schedule which regularly maintains the debris trench in its design conditions, or in the construction of a debris fence between the trench and the roadway.

WAVE EROSION

Wave erosion of the fill supporting the roadway is an additional concern along Waddell Bluffs. The threat of wave attack is an intermittent one, however, due to the fact that the simultaneous occurrence of large storm waves and high tides which produce serious erosion is irregular and infrequent. Wave approach along this stretch of coast is dominantly from the northwest such that most waves are refracted around Pt. Ano Nuevo immediately upcoast (Figure 2). The hook or spiral shape of the coastline downcoast from the point is an equilibrium configuration under these wave refraction conditions. The northwesterly wave approach produces littoral drift to the south. Normally a rocky intertidal platform, partially covered with a sand beach of variable width as well as scattered mudstone and siltstone debris, flanks the highway along the bluffs. During the winter months the sand moves offshore exposing the bedrock platform and coarser debris. With the return to less steep summer waves a protective sand beach up to 60-75 feet wide, which increases in width to the south, again forms below the highway. It is during the winter months when the sand beach is typically removed that waves can reach the highway fill itself.

The roadway which existed prior to construction of the state highway experienced recurring erosion problems. In the winters of both 1931 and 1940 much of the roadway was washed out and destroyed by wave action at the base of the bluffs.

The presence of a natural groin of resistant siltstone which extends seaward from the base of the northern cliffs had created a local erosion problem even before construction of Highway One (Figure 1). Littoral drift was temporarily trapped upcoast of this groin forming a protective beach while, immediately downcoast, wave erosion had removed the talus right to the base of the cliffs. As a result, 660 feet of granitic riprap was emplaced during initial highway construction to protect the fill needed to support the road (Figures 1 and 12). The mudstone bedrock platform in the intertidal zone provided a good foundation for the riprap. The base of the rock was placed at - 6 feet MLLW and extended to a height of +40 ft., which is considerably higher and lower than most riprap emplacement. Typically in the Monterey Bay area, riprap may extend from elevations of -2 to +1 ft. MLLW to +16 to 24 ft. MLLW. The slope of the original rock was 1.75:1. No maintenance is on record for this riprap since its emplacement in 1947-48. The structure is still in good condition although the rock has settled and shifted somewhat such that the slope is now concave up with the lower portion less steep than the upper portion (Figure 12). There has been no more than 10 to 15 feet of seaward movement of any of the rock. Over the years following installation of the rock, there has been a gradual downcoast movement of the smaller granitic rocks from the riprap. For example, 2 to 3 ft. diameter rocks have moved approximately 50 feet downcoast, 1 to 2 ft. diameter rocks have moved 100 feet and 8 inch rocks have been transported 300 feet.

The winter storms of 1983 threatened a major portion of Highway One at the southern end of the bluffs as well as countless areas along the entire coastline of California. Between January and March six major storms brought large waves from the west and southwest to the coastline. The occurrence of the large waves concided with times of extremely high tides. Sea levels along much of the California coast were up to two feet above predicted high tides due to storm surges. The tide gauge at San Francisco recorded the highest tides in 127 years of record. The importance of this simultaneous occurrence of very large storm waves (in excess of 12 feet high) and extremely high tides is that considerable erosion occurred quickly as waves attacked areas which were normally protected and out of reach. Road fill at Waddell Bluffs was removed and the roadway itself was undermined and began to collapse. Ultimately, approximately 24,000 tons of riprap was brought in a cost of $605,000 to protect two sections totaling about 1800 feet of Highway One (Figures 12 and 13). The 1983 winter was severe by any measure, over $100 million in damage to the California coast was recorded (Griggs and Johnson, 1983). If severe winters of this sort reoccur, additional riprap may well be required in order to protect the remaining exposed portions of Highway One at Waddell Bluffs.

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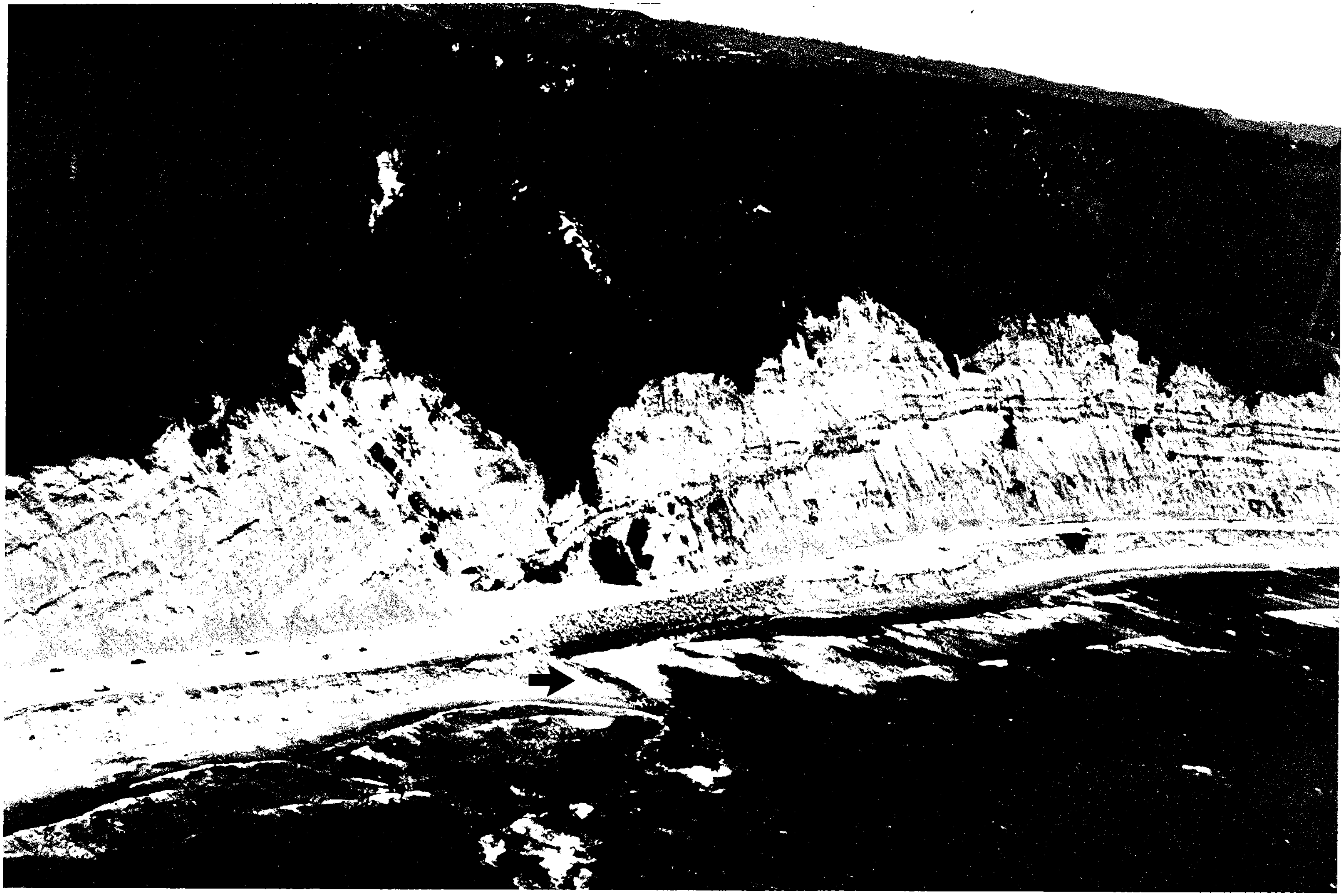

Figure 1. Oblique aerial view of Waddell Bluffs from offshore. Note natural groin in center (arrow) of photo, and protective riprap immediately downcoast.

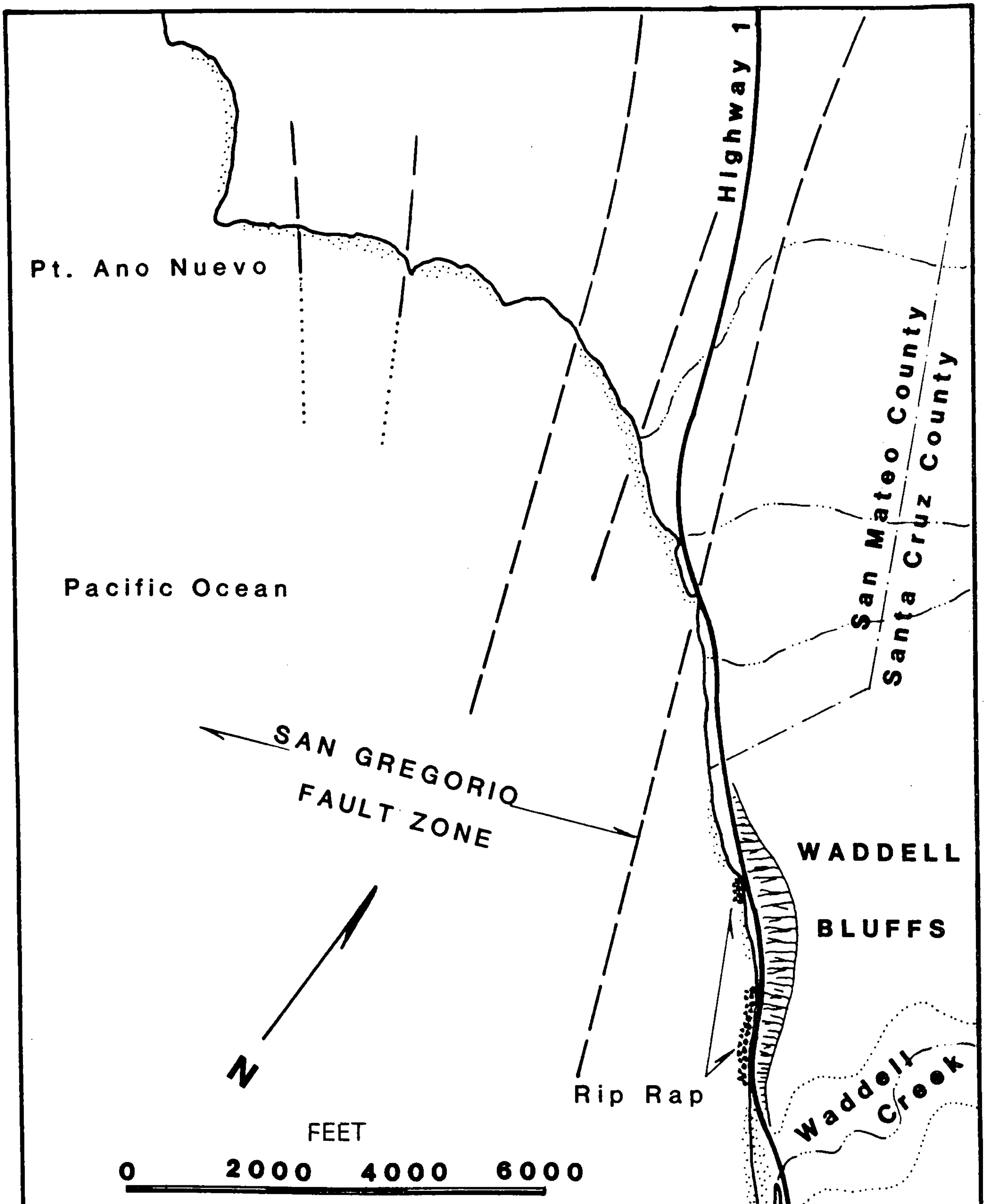

Figure 2. Location map for Waddell Bluffs area showing fault traces within the San Gregorio fault zone.

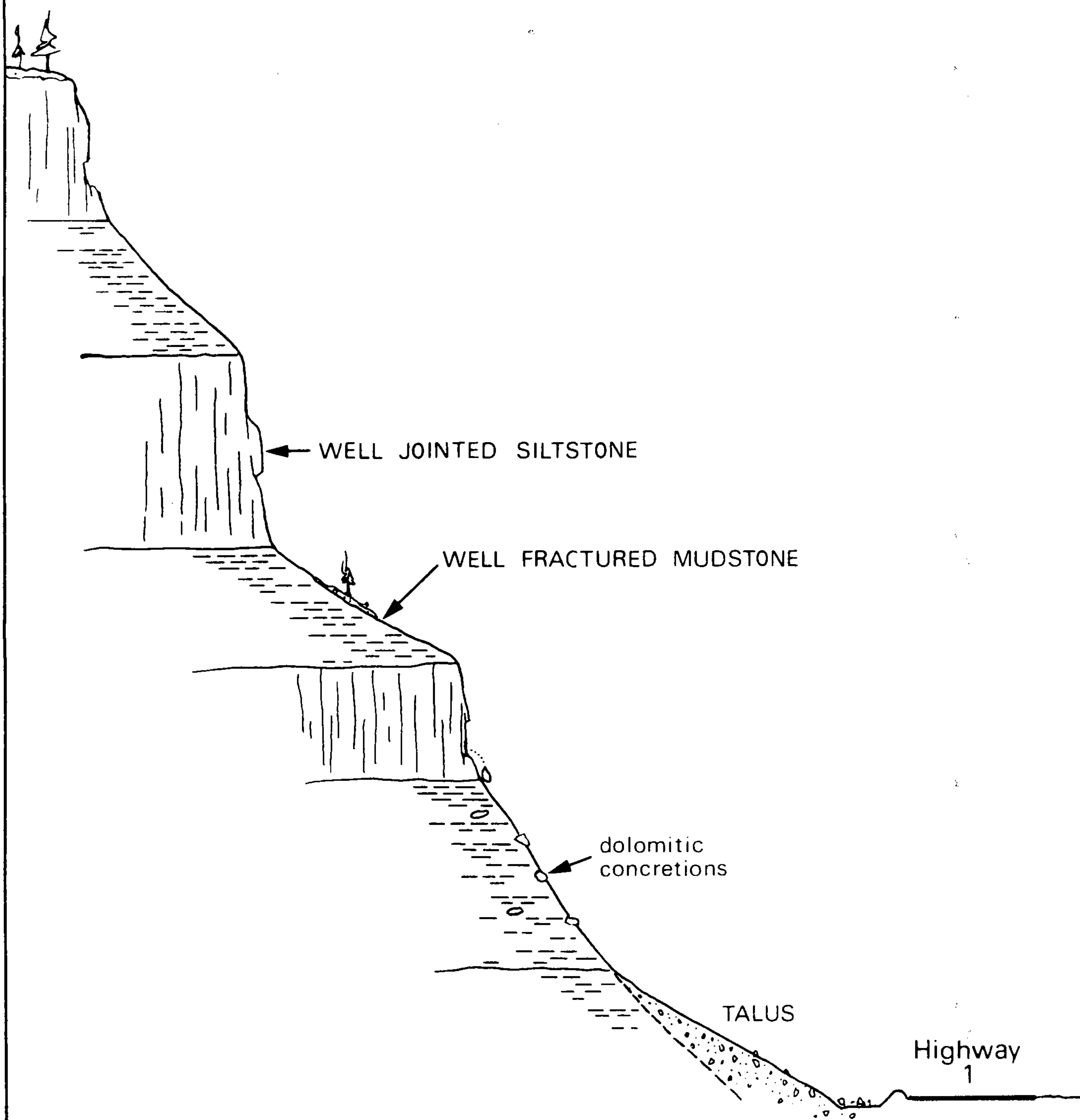

Figure 3. Typical cross-section through Waddell Bluffs.

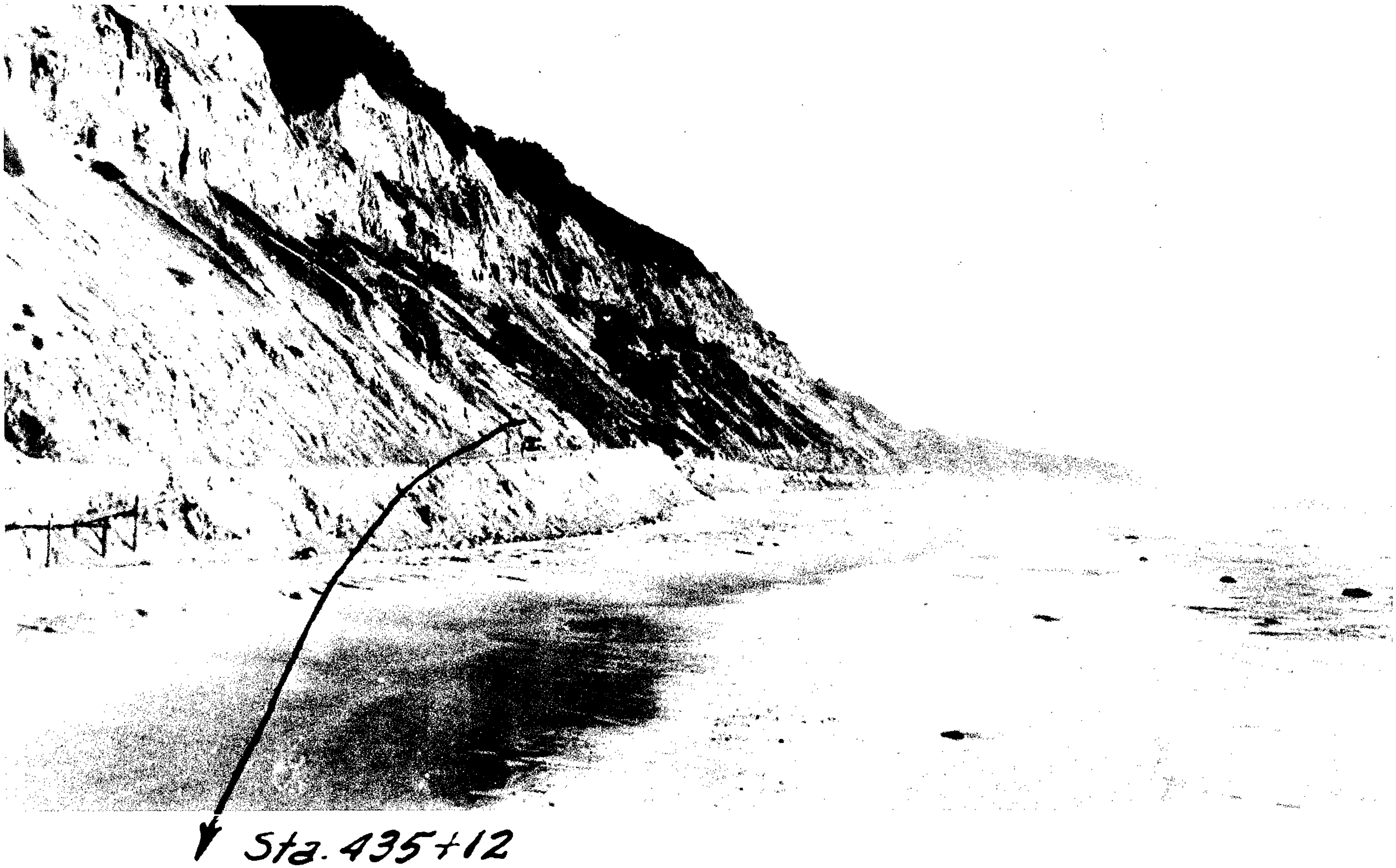

Figure 4. Photograph of bluff area in 1940's prior to construction of State Highway One. Note extensive talus deposits covered with vegetation.

Figure 5. Debris trench along base of bluffs showing upslope talus accumulation.

Figure 6. Cross sections of debris trench comparing Ritchie's design, as built plans, and field observations.

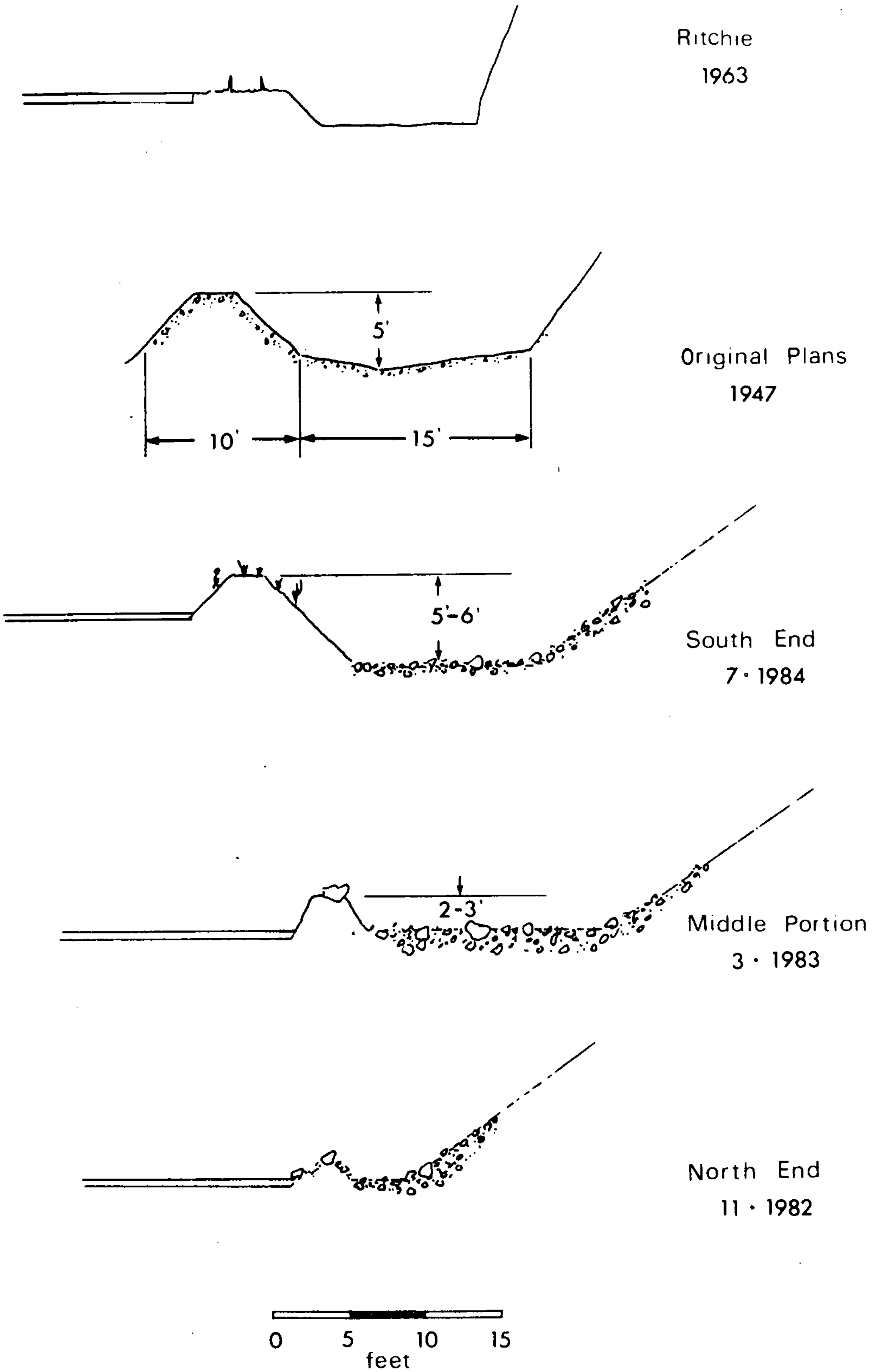

Figure 7. Rock trajectory for various slope angles and sketch of typical debris trench - see Table 1 for specific dimensions (after Ritchie, 1963).

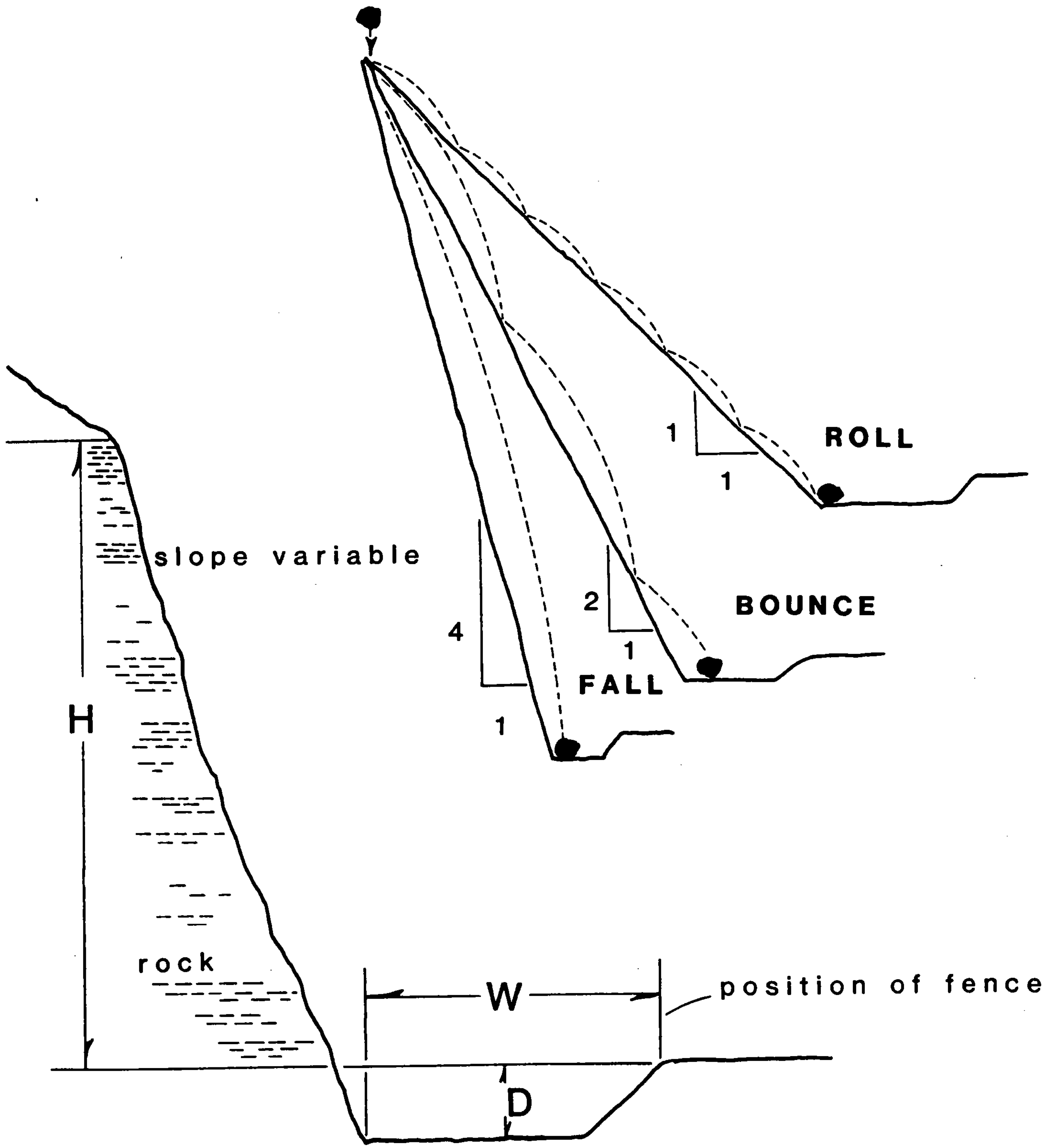

Figure 8. Debris trench along Northern bluffs in nearly full, un-maintained condition (December 1982.)

Figure 9. Debris trench in December 1982. Large siltstone blocks have rolled through trench and over berm. Largest rocks are up to four feet across.

Figure 10. Debris trench at north end of bluffs. Almost no trench exists and rock is reaching traveled roadway. Note tracks crossing road (arrow) and recently dumped rocks on shoulder (arrow).

Figure 11. Impact marks on Highway One at northern end of Waddell Bluffs where large rock bounded onto roadway. Rock was present on shoulder and weighed approximately 350-400 lbs.

Figure 12. Initial riprap emplaced in 1947 to protect eroded area downcoast from natural groin. Some settlement has occurred. View downcoast.

Figure 13. View upcoast of riprap emplaced in 1983 to save 1800 feet of Highway One from wave attack.

Wave Erosion of State Highway 1 Along the San Gregorio Fault Between Davenport and Pescadero, California

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ABSTRACT

Distinctive storm-related geotechnical problems are evident near the active(?) San Gregorio fault along the 25 miles of California Highway 1 between Davenport and Pescadero (Santa Cruz/San Mateo Counties). This portion of California Highway 1 traverses and area characterized by emergent marine terraces. Major problem sites include: (1) year-around stream crossings immediately above normal high tide marks; (2) sections with the highway near high sea cliffs; and (3) the failing Waddell Bluffs. Smaller-scale but significant problems include: (1) unconsolidated deposit/weak soil slumping and flowing in cuts; (2) gully wash flooding onto the highway; (3) beach/dune sand drifting across the highway; and (4) weak soils underlying the roadbed.

Examples include the effects of January 1982 high-tide storm-waves which (1) eroded high fill at Scott Creek; (2) undermined pavement on a low terrace near Pescadero Creek; and may have endangered both creek bridges; and (3) attacked the low Waddell Creek bridge and fill and the adjacent fill parking lot as well as the Waddell Bluffs base. Concomitantly, some fault-fractured mudstone of the high, oversteepened Bluff face ravelled, flowed, and slumped onto the highway.

Sizable cuts and fills were made across terrace remnants between Scott and Waddell Creeks during development of the present route in 1939. As a result of this development, the highway was originally situated significantly further back from the top of the 200 foot high seacliff. Aggressive wave erosion of these very steep (60-90°+, or over 150%) fractured mudstone and overlying terrace deposit cliffs has resulted in rapid retreat of the cliff face. Measurements of badly eroded sites reveal several hazardous retreats to less than 60-70 feet of the pavement edge and, in one case, to 10 feet of the pavement edge! Some highway sections currently are in jeopardy of failure.

Wave attack of the seacliff further north, near the intersection of Highway 1 and Pescadero Road, has caused slumping and endangered the highway. Slump blocks there can be seen to include the 50-75 feet from the seacliff to the highway. These blocks are bordered by fractures and have dropped two to five feet vertically. The seacliff here is relatively low, however, being approximately 30 feet high.

INTRODUCTION

A 25-mile long segment of California Highway 1 between Davenport on the south and Pescadero on the north in Santa Cruz and San Mateo Counties demonstrates rather diverse geotechnical problems. The highway roughly parallels and lies near both the ocean and the active(?) San Gregorio fault (Figure 1). It is situated from a few feet to several hundred feet back from the ocean, mainly crossing uplifted marine terraces. The San Gregorio fault lies approximately a mile offshore at Davenport. It trends northwest and proceeds onshore near Waddell Bluffs/the County line and back offshore at the mouth of Pescadero Creek and/or the more northerly San Gregorio Creek.

The large-scale geotechnical highway problems result from wave erosion or a combination of wave erosion and faulting. Where the highway is low in elevation and crosses the mouths of the major creeks (mainly Scott, Waddell, Gazos, and Pescadero) storm wave action can directly attack the road bed, its fills and its bridges. Terrace deposit and even bedrock road bed is similarly vulnerable where the highway nears the ocean, whether close to or considerably above sea level. Where bedrock is badly fractured by fault-related movements, it is particularly susceptible to erosion. The Waddell Bluffs present a particularly complex and difficult problem; the roadbed consists of a fill placed just above the narrow beach and the road is backed by the high and steep cliff face cut along the San Gregorio fault in highly fractured mudstone which dips oceanward.

In addition to the large-scale hazards, there are others which cause lesser maintenance problems. These include slumping and associated debris flowing, mainly in saturated terrace and/or other surficial deposits and

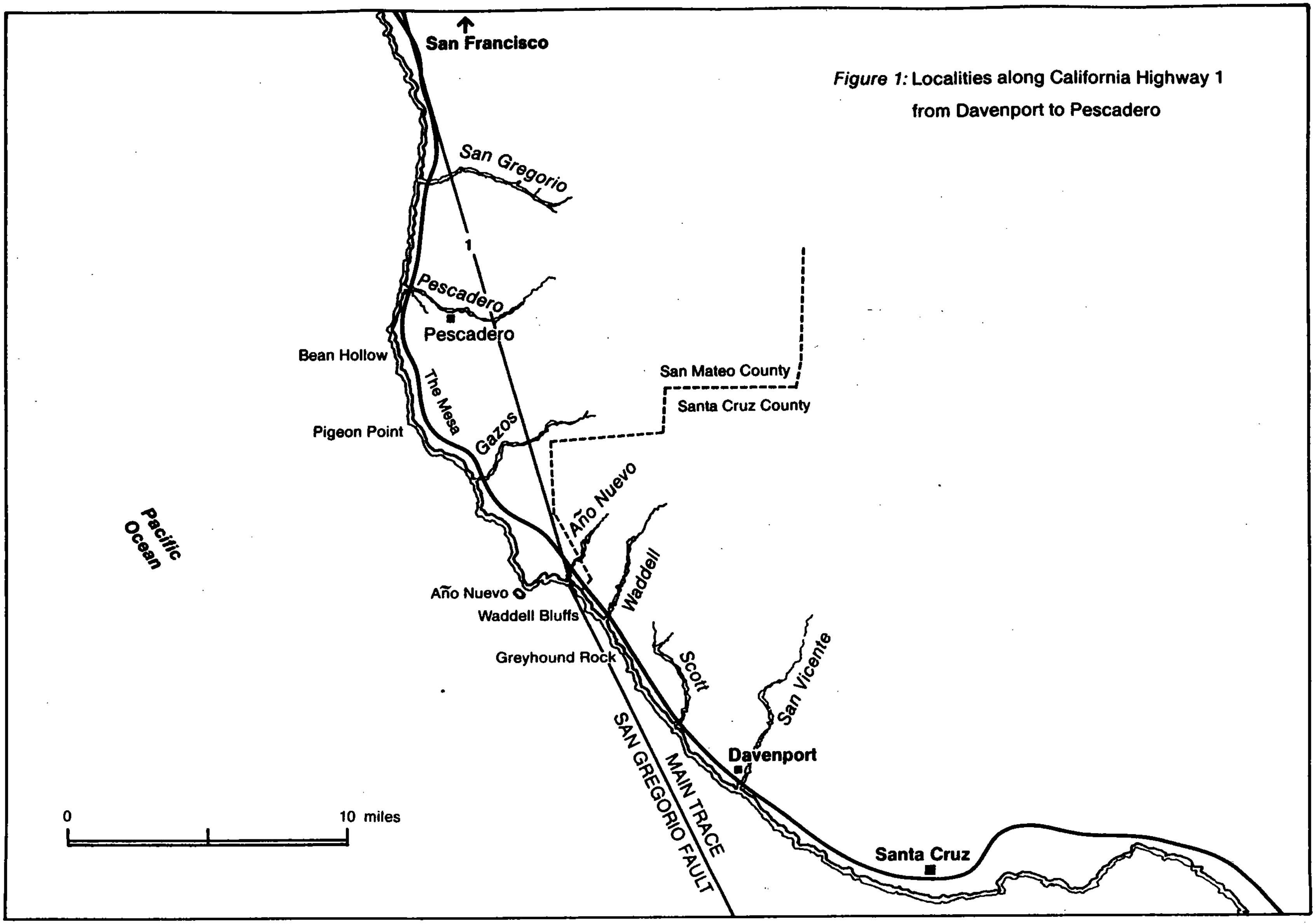


Figure 1: Localities along California Highway 1
from Davenport to Pescadero

soils. Gully erosion also can cut into the roadbed. Existing or new gullies and sheetwash can channel considerable debris onto the highway from the upslope areas. Locally, sand can blow and drift across and onto the highway. Weak soils underlying the roadbed can also necessitate considerable bedwork and surface repairs.

GEOLOGICAL SETTING

Geomorphology From South to North

Davenport to Scott Creek

The 3 miles of Highway 1 between Davenport and Scott Creek are situated well back across a broad, moderately dissected wave-cut platform--the first marine terrace--about 80 feet above sea level. The dissecting features are ephemeral creek drainages, many from the higher land forms to the east. Many of these creeks are also very steep-sided as they transect the two or three existing marine terraces, having initially been surge channels.

Scott to Waddell Creek

The highway descends from the marine terrace to cross the mouth of Scott Creek on a 25 foot high bridge and fill. The highway then climbs in elevation and traverses dissected terrace remnants, which are probably of the first marine terrace, at a maximum elevation of about 200 feet above sea level. When the highway was straightened and the present route constructed in 1939, this segment involved the most cutting (excavation) and realignment. The route was relocated oceanward from the valley of Scott Creek to its present position along the near-vertical sea cliffs. The marine terrace here is widest and least dissected at its north end near Big Creek Lumber Mill. At its northern extremity, above the mouth of Waddell Creek, the terrace is 120 feet above sea level.

Waddell Creek and Bluffs

The highway descends to the bottom of the steep canyon of Waddell Creek where it crosses the creek mouth on an exposed low (15 foot high) bridge close to sea level. Then, beginning at approximately 15 feet in elevation on fill, the highway passes inshore of a fill parking area and crosses the toe of the steep, ravelling Waddell Bluffs. The fill is raised increasingly above beach level to about 70 feet at its north end. Some sections of the outer fill slope are buttressed with large boulder rip-rap.

Año Nuevo Creek to Gazos Creek

The highway crosses the back of a wide fairly flat marine terrace from the north end of the Waddell Bluffs and Punta Año Nuevo to Gazos Creek. The terrace reaches approximately 100 feet in elevation near the highway. Just east of the highway are gentle slopes, low hills, and wide drainages. These are backed by a fault scarp with hills abruptly rising higher inland. The highway descends slowly as it nears the ocean at Gazos Creek Beach. It lies just east of a vegetated dune field and then crosses the creek mouth over a low (15 foot high) fill and bridge.

Gazos to Pescadero Creek

North of Gazos Creek the highway lies at approximately 80 feet elevation just inshore from a narrow marine terrace which dips gently oceanward. A moderate slope rises inland of the highway to a dissected nearly flat-topped marine terrace feature appropriately named "the Mesa." The roadbed passes inland of Pigeon Point light, crossing the base of the gentle ocean-facing slopes behind the narrow, lowest marine terrace. These slopes extend upward to the top of the Mesa, with step-like breaks for two intermediate sets of marine terraces (Lajoie et al., 1974). Much of the mesa top lies at approximately 500 feet elevation. Ephemeral to year-round minor stream drainages and inactive and active gullies have dissected the Mesa and its side slopes and ridges. At the location of the highway, however, the relief mainly is gentle and road cuts through these ridges are low (less than 10 feet). North of the creek mouth at Bean Hollow the highway closely approaches the low (±15-30 foot) sea cliffs. Just south of Pescadero Road the highway traverses the bedrock terrace, approaching to within 60 feet of the narrow beach.

Pescadero Creek Vicinity

The highway crosses the mouth of Pescadero Creek on a bridge, from the bedrock marine terrace at ±30 feet elevation north to beach and dune sand. The bridge itself is about 25 feet in elevation on the rock, grading down to 20 feet at its north end, where it rests on fill with large-boulder riprap. The roadway crosses the stream valley mouth between a large fresh-water pond inland and a beach dune ridge oceanward. Then it climbs onto rolling hills and continues north. The soils and underlying terrace and colluvial deposits in these hills are subject to severe gully erosion. Thus, the road cuts themselves are sites of severe erosion and upslope gullies can intersect the highway.

Rock Types

Bedrock from Davenport north to Waddell Creek is the blocky, bedded upper Miocene Santa Cruz Mudstone (Clark, 1970). This rock weathers to a cream color and crops out extensively. Some difficulty can be encountered distinguishing it from the overlying Pliocene Purisima formation. This latter unit is less homogeneous over its vertical and areal extent. In the general area under discussion, it tends to be softer, shalier and less siliceous than the Santa Cruz mudstone. Shearing of the Santa Cruz Mudstone produces a rock superficially very like the Purisima formation. At Waddell-Año Nuevo, both units are represented, along with the underlying (middle Miocene) siliceous Monterey mudstone.

North of Punta Año Nuevo, the terraces and hills west of the main trace of the San Gregorio fault are underlain by the Purisima formation and, oceanward, the Pigeon Point formation of Cretaceous age. This latter unit includes conglomerates, sandstones and mudstones. It is not seen north of Pescadero Creek; there the hills are underlain by the Purisima formation.

Surficial Deposits

Locally marine terrace deposits and channel fills overlie the bedrock and underlie the soils. These can be seen in road cut and sea cliff exposures. There are also some alluvial and colluvial deposits (Lajoie et al., loc. cit; Weber, 1980). Some beach and dune sands are evident near creek mouths.

Soils

Soils on the lowest marine terrace in the Davenport area are loams and sandy loams of the Elkhorn and similar Watsonville series (Bowman and Estrada, 1980). These soil series are up to several feet thick and have well developed sandy loamy A and more clayey B horizons. These series and sandy loamy Baywood and Elder soils, and, predominantly, thin Bonny Doon loams mantle the terraces and ridges from Scotts to Waddell Creek. The low terrace north of

Año Nuevo features Elkhorn soils, with the similar Watsonville soils on the gentle slopes east of the highway (Wagner and Nelson, 1961). In this area both Elkhorn and Watsonville and thinner upland Tierra soils are widespread; they are very unstable and prone to failure when saturated because of the expansive characteristics of their clays. Thus, severe gullying is common. Steeper slopes and mature, vegetated gullies tend to be mantled by the thin stony Gazos and related soils.

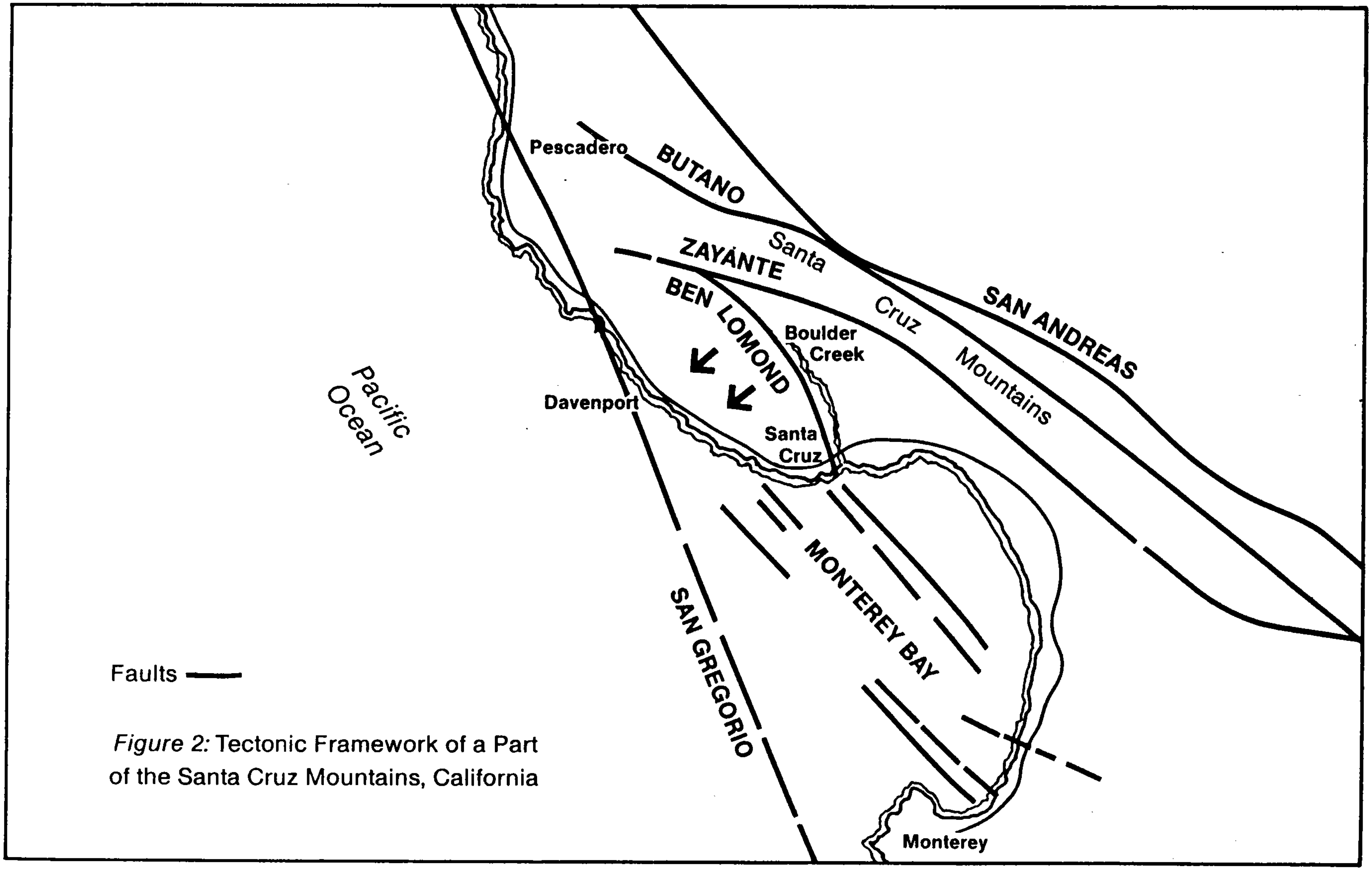
Tectonic Framework

Davenport to Waddell Creek

The siliceous mudstones exposed along the coast from Davenport to Waddell Creek directly overlie the massive Mesozoic granitoid basement rocks of the Santa Cruz Mountains (Clark, loc. cit.). The mudstone beds dip gently seaward. This dip reflects uplift with a general oceanward dip of a block of metasedimentary, granitoid and overlying Tertiary marine rocks along the Ben Lomond fault. The fault trends inland up the San Lorenzo valley from Santa Cruz (Figure 2). The Neogene rocks at the coast show little other folding structure.

Although the block dips oceanward, coastal emergence is reflected in the set of three marine terraces seen along this coastal strip. The uppermost terrace is very dissected and represented mainly by remnants. The lower two are well developed.

The main trace(s) of the possibly and potentially active San Gregorio fault lie(s) approximately two (and three) miles offshore at Davenport (Hall et al., 1974). Pronounced sets of fault-related fractures can be seen in the



Faults

Figure 2: Tectonic Framework of a Part of the Santa Cruz Mountains, California

Santa Cruz mudstone exposures there. From this point what is taken to be the main trace or zone of the fault angles shoreward, trending northwest. Although topographic features suggest that branches of the San Gregorio fault may lie as much as three miles inland and several fault traces come onshore in the vicinity of Punta Año Nuevo (Weber, op. cit.) suggesting that branches may lie further offshore as well, the main trace apparently comes onshore at the north end of Waddell Creek. The mudstone at Waddell Bluffs is severely fractured and sheared by the fault. Faults can be seen clearly in bedrock just to the south across Waddell Creek and dips in the mudstone on Greyhound Rock, across a fault trace, are gently landward, reversing the usual local oceanward dips.

Waddell or Año Nuevo Creek to Pescadero Creek

The main trace of the San Gregorio fault trends northwest across land for approximately 15 miles from the mouth of Año Nuevo Creek to near the mouth of San Gregorio Creek, which is about five miles north of the Pescadero Creek mouth (Brabb and Pampeyan, 1972; Weber, op. cit.). Several branches of the San Gregorio fault system angle across the Pigeon Point block oceanward of this main trace. Some are strongly expressed topographically, such as the northwest trending Frijoles fault. It lies in the steep-sided Arroyo de los Frijoles just east of the Mesa. This arroyo probably was formed in fault-weakened rocks by marine erosion. With a southern projection coming onshore on or just north of Punta Año Nuevo, the Frijoles fault appears to extend northwesterly into the ocean at Pescadero Creek. Many other fault traces are recognized only by detailed subsurface studies (Weber, op. cit.). Bedrock exposures are poor and other structural complexities of the block have not been determined.

The rocks of the Cretaceous Pigeon Point formation occupy the western half of the San Gregorio fault block, west of the Frijoles fault system (Brabb and Pampeyan, loc. cit.). Inland of this fault trace are rock of the Pliocene Purisima formation. Sedimentary rocks of Cretaceous age are not found between the San Gregorio and San Andreas faults. The Pigeon Point may represent an exotic terrane.

GEOTECHNICAL PROBLEMS

Marine Erosion

Creek Mouths

San Vicente Creek: This creek enters the ocean at the town of Davenport. Such highway-crossing marine erosion problems as may once have existed here have been corrected or deferred. The creek was rechanneled through a tunnel cut in mudstone bedrock to permit the railroad to cross the mouth on a high fill. This railroad lies immediately oceanward from the highway which is situated on a separate, lower fill. Thus, the railroad fill acts as a barrier for the highway fill. The railroad similarly protects the highway at some pocket beaches and surge channels in the immediate Davenport area--where the railroad line now ends. The railroad fill at the back of Davenport Beach is subject to attack by high storm waves. A localized fill-slope failure occurred when waves cut the base in 1982.

Scott Creek: The base of the fill slope north of the Scott Creek bridge was attacked by storm waves in 1982. An alert citizen called this to the attention of California Department of Transportation. A truckload of large crystalline limestone boulders destined for the riprap of Waddell Bluff was diverted; it was dumped over the outside of the Scott Creek fill slope in just enough time to prevent the highway from being undermined. At the same time, high tide and storm waves sent salt water with logs and other debris back under the bridge to the marshy area behind. These events illustrate the problems which can occur at this site. The sand in the core of the fill could be replaced with less erodible material and the entire fill slope faced with riprap on the ocean side.

Waddell Creek: The fill and associated bridge at the mouth of Waddell Creek are very low in elevation (15 feet). They were endangered by the 1982 storm waves (at high tide) as water was driven back under (and over) the bridge. Much of an adjacent parking area underlain by fill was removed by these waves. In previous years waves had hurled logs over the highway at the bridge (McCrary, 1984). The bridge at this site could be raised and perhaps lengthened.

Gazos Creek: The bedrock at the mouth of Gazos Creek has less relief than bedrock at Scott and Waddell Creeks. Yet because the highway crosses slightly further up the channel, the bedrock and the overlying dunes offer more protection to the highway bridge and fill than at Scott and Waddell Creeks. Nevertheless, the creek at the bridge is barely above highest tide levels and high-tide storm waves can rush up the channel and endanger these structures. The bridge, presently 15 feet high above the water, could be raised. Some stream bank protection is offered by sand bags and willows under and just downstream from the bridge. Riprap could be placed here.

The stream crossing at Bean Hollow Beach, to the north of Gazos Creek at the mouth of Arroyo de los Frijoles, is also protected by bedrock. In addition, the highway there is on a higher fill, 25 to 30 feet above the creek level. There is some riprap on the ocean side of the fill. The entire fill could be faced with riprap.

<u>Pescadero Creek</u>: The southern bridge abutment at the present mouth of Pescadero Creek is founded on bedrock of the Pigeon Point formation. Although the bridge and bedrock platform here may not be above highest storm waves, the bedrock extends out along the south edge of the stream mouth into the surf zone. Thus, the force of the waves is diminished as wave energy is dissipated against the protruding rocks. Some large boulder riprap also is seen at the base of the outcrop under the bridge.

The bridge is approximately 25 feet above the water at its south end and 20 feet above it as its north end. The roadbed at the north end of the bridge rests upon fill which overlies beach and dune sands. Although protected by large-boulder riprap, this area is more susceptible to wave erosion than the south end of the bridge.

Sand dunes presently protect the rest of the highway from wave erosion as it crosses the back of the beach at the mouth of Pescadero Creek. Shifting sands here may tend to cover the highway. Vegetation stabilizing the dunes may need more protection. A change in dune or sand-source conditions could expose the highway to ocean attack but that does not seem to be an immediate problem.

Waddell Bluffs

The highway runs across the base of the actively ravelling Bluffs. It is raised above sea level on fill. The outer margin of this fill is subject to wave attack at all times, especially by high-tide and/or storm waves. During the winter of 1982 the situation was critical. Volumes of large-boulder riprap were required to retain the highway. At present, only segments of the ocean-facing fill slope are fronted with riprap; this entire fill slope could have riprap emplaced on it. The main face of the Bluffs could be cut back and reshaped, but this would be a very large-scale and difficult endeavor. It is not practical to reroute the road.

Historically, the roadbed was on the sand and rock of the narrow beach. It was subject to regular closures both from tide and wave action and landslide debris. Although a great inconvenience, no loss of life was ever reported. Griggs (1984) describes Waddell Bluffs slide problems elsewhere in this volume.

Sea Cliff-Edge Sites

Scott to Waddell Creek Highway Segment: Along this segment there are several sites where the highway closely approaches the edge of the high (200 feet) sea cliffs. The mudstone bedrock here has been weakened by fracturing and shearing along the San Gregorio fault. Where the beach below is narrow, the sea cliffs are particularly subject to wave erosion. In addition, thick unconsolidated deposits can overlie bedrock on these terraces. At some sites, gully erosion, in the unconsolidated marine terrace deposits abets this marine erosion. The gully erosion proceeds both headward from the cliff edge and downward from the highway edge.

It is interesting to observe, oceanward of the highway, that a few sections of the roadbed of the projected old Oceanshore Railroad remain intact. Many have been undermined and collapsed into the sea (McCrary, 1984). Although not constructed as close to the cliff edge in 1939, sections of the highway are in increasing danger of doing likewise.

Several cases in point may be seen. At most the beach is narrow or absent. The cliffs are approximately 200 feet high; oceanward slopes are usually more moderate in the terrace deposits than in the underlying bedrock. Bedrock slopes are very steep. Locally, especially at "pocket beaches," some parts of the bedrock slopes are reversed/concave.

Observations were made at five near-critical-appearing sites. In all cases the steep lower slopes extend down to the ocean or beach at the percents cited insofar as can be observed. Some 180 feet south of milepost $32^{\underline{30}}$ the highway pavement edge approaches to within 11 feet of the sharp break in slope. It was not possible to obtain measurements of slope angles at this site; however, the slope immediately below the highway appears to average approximately 90-100% and lies above a near vertical slope.

Two sets of measurements were made near milepost $33^{\underline{06}}$. Twenty-four feet south of the milepost a width of 32 feet separates the edge of pavement from a sharp break in slope. Below that break in slope, a 90% slope extends downward for a linear distance of 45 feet, then breaks off to a near-vertical slope which continues down to the ocean. Sixty-four feet south of the milepost only a 10 foot width separates the pavement edge from the sharp break in slope. An 85% slope then descends for approximately 50 linear feet to another break in slope below which the cliff is nearly vertical.

One hundred feet north of milepost $33^{\underline{90}}$ is a site where the road is cut in bedrock which forms a small promontory oceanward of the highway. Sixty-nine feet out through a notch from the edge of pavement, the slope drops abruptly off to 160%. Just north of milepost $34^{\underline{00}}$ a slope extends outward 62 feet from the edge of pavement at 15%. Then it drops off abruptly to 162%.

At milepost $34^{\underline{68}}$, above Greyhound Rock Beach, a gully is cut into the terrace deposits. It slopes oceanward at 57% for 110 linear feet from the edge of the pavement. Below that the slope in bedrock nears vertical but could not be measured. Another very similar gully can be seen approximately 50 feet to the north.

Action should be taken soon to protect the highway in at least some of these cases. Considering the height and slope of the cliffs, it seems likely that realignment will be required at some sites and at many ultimately. Construction of support for the outside (oceanward) of the highway does not appear feasible at many sites.

<u>Bean Hollow-Pescadero Road Highway Segment</u>: Along this highway segment some expensively developed homesites on small acreages lie on the low (20-40 feet elevation) bedrock terrace between the highway and the ocean. Some stretches are bare, however, and at some sites between Bean Hollow and Pescadero Road and Creek, the highway approaches to within less than 100 feet of the sea cliffs. The top of the marine terrace can be reached by storm waves.

A case in point is found immediately south of Pescadero Road. In the winter of 1982 waves apparently undermined and weakened the the seacliff. An area 75 feet wide at the top of the cliff, including the outer lane of the highway, was imperiled. A slump initiated and the outer lane began to collapse. At the same time heavy rain and salt spray were being driven across the highway. Repairs are hampered under these demanding and dangerous conditions.

Just south of this failure site, a larger, rather broken-up slump block can be seen. It occupies the slope between the highway at the cliff-edge.

Waves induce this slumping. This and similar sites are very unstable and may involve the highway soon. The highway may require rerouting an appropriate distance inland. Development of homesites on the gentle slopes immediately above (east of) the road may hamper rerouting slightly further south.

Slope Failures

Waddell Bluffs

The large, actively-ravelling, fractured ridge face of Waddell Bluffs is truncated and being eroded and undermined by the ocean. It poses the greatest ongoing highway maintenance problem along the coast between Davenport and Pescadero. Indeed, it is the largest problem between Big Sur and Devil's Slide. As this feature is discussed elsewhere in this volume (Griggs, op cit.), it is not appropriate to treat it in detail here. As mentioned, however, reshaping the cliff face would be a very large-scale undertaking.

Slumps, Flows, and Floods

A few surficial slumps and debris flows take place in the higher road cuts between Scott and Waddell Creeks. These are initiated in unconsolidated, heterogeneous but clay-rich terrace and channel deposits during especially wet weather. The saturated material fails and slumps or flows onto the highway. This necessitates clearing but poses no serious structural problems. It is nevertheless notable that these failures can seriously endanger passing motorists.

Soil slumps and flows on lower slopes of various angles are very common in the Gazos Creek-Pescadero Creek area. A few of these move out and down over cut slopes onto the highway. Failures can also occur in unconsolidated deposits and soft or weakened bedrock where oversteepened by cutting. Many failures of these types were seen in the Pescadero area in the winter of 1982. Few of these events actually impinge upon the highway, however. Usually, only minor road clearing is required when they do.

Besides gullying initiated by road-cutting, the sheetwash and severe gullying caused by agricultural practices (Lajoie, 1984) in the Gazos Creek-Pescadero area can initiate debris floods. A site 0.5 miles north of Pescadero Beach provides excellent examples. Here a preexisting major gully extends upslope perpendicular to the highway. A bench or diversion ditch just above and paralleling the highway may also have critically concentrated some slope and gully wash. During heavy rains this preexisting gully was most likely already at capacity and the debris flooded (through the diversion ditch) onto the highway. Two other major gullies appear to have been created in road cut and/or diversion ditch failures--both with debris floods onto the highway. Diversion of these water courses, and others of their type, is a problem. Care must be exercised not to cause further gullying downslope (or upslope).

Drifting Sand

This is a serious problem only at Scott Creek and Pescadero Creek. Off-road vehicular use at Scott Creek has removed much vegetation and destabilized the beach and dune sands. Sand blows across the highway just south of the bridge. The extensive dune field at Gazos Creek State Beach has been stabilized by vegetation which is little disturbed. Thus, it is not presently a highway problem. The highway just north of Pescadero Creek traverses dune sand. Dunes lie oceanward. Destabilized sand becomes available to drift onto the road, especially with recreational use.

Weak Soils

The highway crosses a low weakly dissected marine terrace between Año Nuevo and Gazos Creek. This terrace is mantled by fairly thick soils underlain, in part at least, by unconsolidated marine terrace, alluvial and colluvial deposits. Somewhat boggy conditions can develop because of rainfall, agricultural irrigation, and clayey subsoils. Locally, these conditions can produce road bed problems. The highway is built up several feet on fill across these surfaces. Where the highway passes through low cuts, drainage ditches are maintained alongside of it.

Where slopes intersect the highway, saturated weak, clayey soils are also evident very locally between Scott and Waddell Creeks. They are especially common between Gazos and Pescadero Creeks. Although prone to failure, these soils have limited opportunity to slump onto or undermine the highway.

SUMMARY

The 25-mile long segment of California Highway 1 between Davenport on the south and Pescadero on the north demonstrates a diverse set of geotechnical problems. The major problems result from wave erosion. Fracturing by past movement along the San Gregorio fault has weakened bedrock and makes it more susceptible to erosion. Uplift may also be occurring inshore and easterly from the main trace of the fault.

The highway itself is endangered at stream mouth crossings where storm waves can undermine fills and bridges. Where it nears sea cliffs, wave erosion can and is undercutting the highway. This is seen both where sea cliffs are low and, even more seriously, where sea cliffs are high. The truncation of a ridge by wave erosion and the San Gregorio fault has caused active ravelling and flows down the steep face of the Waddell Bluffs.

Besides these major geotechnical problems, there are smaller-scale problems of different origins. Mainly, the failure of surficial materials causes these problems. Slumps and debris flows are initiated in unconsolidated or weakened materials in road cuts. Very common in the general Pescadero area, soil slumps and flows can move onto the highway. They usually take place on slopes not adjacent to the highway, however. Similarly, severe gully erosion and sheet wash are common in the general Pescadero area. Only occasionally, however, do the gullies intersect the highway. When they do, debris can flood the highway. Highway cuts and ditches aid in gully formation also--a process which should be mitigated insofar as possible. In addition to these erosion and failure processes, weak soils may cause minor local highway foundation problems. Very locally, drifting sand must be treated.

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DETAILED OFFICE, FIELD AND LABORATORY ANALYSIS TO DISCERN ROCK SLOPE STABILITY, INTERESTATE HIGHWAY 287, NORTHEASTERN NEW JERSEY

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Stability of highway rock slopes is a major concern adjacent to or some distance from plate boundaries. Seismic effects intensify the forces acting on rock mass discontinuities.

Analysis of small scale and large scale aerial photographs of the New Jersey Highlands study area delineated overall fault and shear zone locations and detailed joint patterns respectively. Consisting mostly of Precambrian gneiss the study area lies about 40 miles west of New York City and involves a five mile extension of Interstate Highway 287 through massive, rocky terrain.

The field survey was accomplished in two parts, structural geology mapping within 1/4 mile of the highway and production of 23 detail lines obtained in four study areas. Detail line data were processed using the electronic notebook procedure and microcomputer system described by Watts and West (1981).

Unconfined compression and direct shear strength tests were performed. Direct shear tests on natural joints and cut joints were tested at low confining stresses. Some joints were tested repeatedly at increased normal loads obtaining additional data.

RQD values, fracture density and condition of joint surfaces were considered in evaluating failure aspects of discontinuities.

Evaluations were made using Barton's stability equation.

Combined analysis yielded evaluation of the four sites for slope stability. Local failure zones were delineated.

COMPILERS' NOTE: This abstract was originally prepared for a paper presented at the Highway Geology Symposium. As no proceedings contribution was received, the abstract is reprinted here.

CONTRACTING FOR AND USING TIEBACKS FOR LANDSLIDE STABILIZATION

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ABSTRACT

A tieback is a construction element that is anchored in the ground to support a structure. Tiebacks have been used to support structures in Europe since the mid-1960's, and since the mid-1970's in the United States. They are a unique solution for correcting certain types of landslides; this paper describes a landslide stabilization project for a State Highway on the San Francisco Peninsula.

In conjunction with the project described, a review has been made of various contracting procedures used on public works projects to accommodate proprietary systems. The impact on design, installation, and cost of the various contracting procedures is discussed, and recommendations are made for design approaches for landslide stabilization, using tiebacks, and for specifications of the work.

INTRODUCTION

In developed areas landslides cause numerous problems and great economic hardships. Historically, there have been three solutions to stabilize landslides: (1) regrading; (2) drainage; and (3) relocation. Because of the cost and the enviornmental problems associated with relocation, it is rarely a viable option today. Regrading and drainage, while economical, have not proven to be long-term solutions because slopes must be periodically regraded, and drains must remain free-flowing to prevent reactivation of slides.

Relocation, regrading, and drainage are relatively passive attempts to correct slope failures when compared to the use of tiebacks. A tiedback wall is a positive correction, because it applies a horizontal force to resist the driving forces of the landslide. This feature is unique to the tiedback wall.

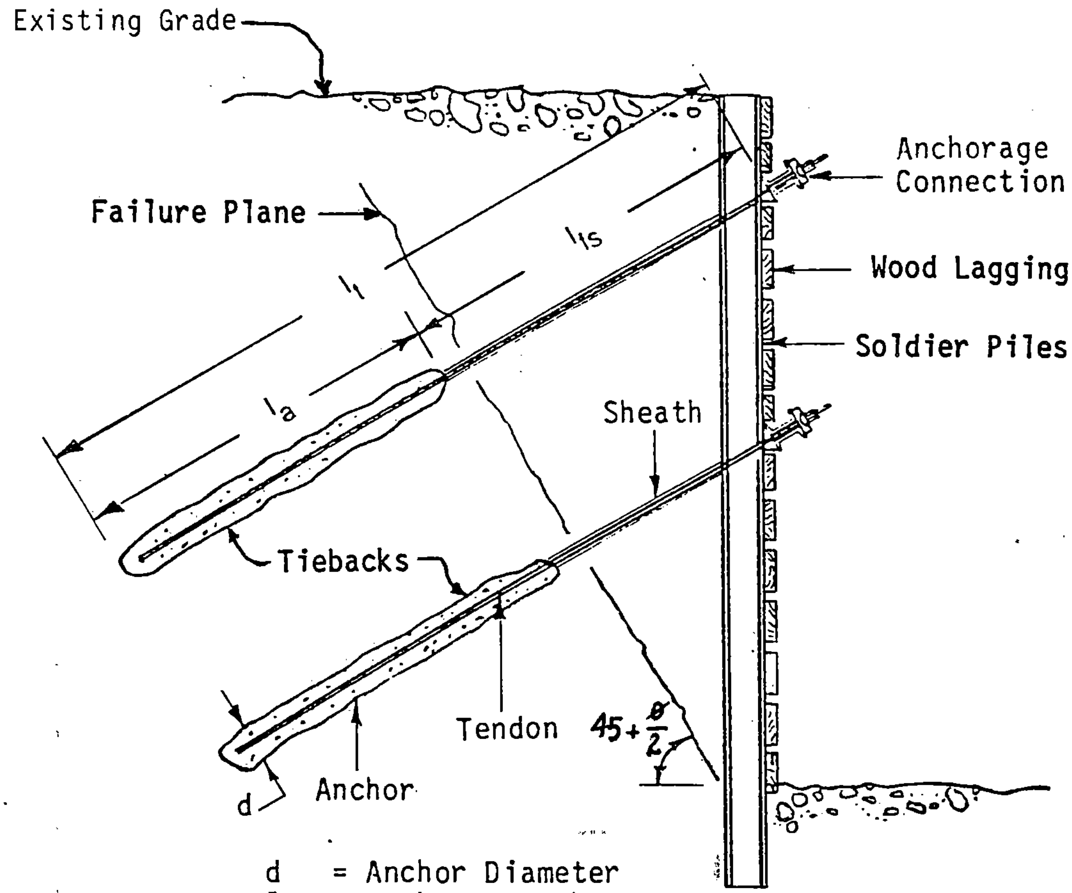
The design and installation of tiebacks has been developed primarily by specialty contractors; each contractor has evolved its own methods of performing the work, and consequently many of the techniques are proprietary.

In this paper on tiebacks, I will discuss:

- o what a tieback is and how a tiedback wall is normally constructed;
- o a tiedback landslide control wall located on Route 84, (Woodside Road) just east of I-280 in Woodside, California; the project was designed by the California Department of Transportation with tieback installation by Schnabel Foundation Company;
- o a potential performance problem with the Woodside Road landslide wall;
- o alternate designs for tiedback walls for landslides similar to the Woodside Road slide that are more enonomical and have a history of superior performance; and
- o contracting for tiedback walls using performance specifications.

TIEBACKS

Figure 1 shows a cross section of a tiedback wall. A tieback is a construction element that transfers load from a structure and anchors it to the ground behind a theorectical or measured failure plane. The typical tiedback wall is built from the top down and is normally used in cut situations. In Figure 1 the soldier beams are installed from the existing grade. The excavation is then taken down to the first tieback level as lagging is installed. The tieback is installed, tested to verify its capacity, and locked off at a predetermined lock-off load. This sequence of excavation, lagging, and tieback installation is then repeated at each tieback level until subgrade is reached. When subgrade is reached, a finished wall face of some type may be applied to the tiedback soldier beams. This construction sequence from the top down has an advantage of stabilizing the slide early in the construction sequence.



d = Anchor Diameter
l_a = Anchor Length
l_t = Total Length
l_{fs} = Unbonded Length

FIGURE 1

Figure 2 shows a partially completed tiedback wall, built in a slide condition similar to that of Woodside Road. The wall was built from the top down, and the picture shows the testing of the final level of tiebacks. When testing is finished the wall will be completed. This tiedback wall, like the Woodside Road wall, consisted of soldier beams and wood lagging supported by three levels of tiebacks, but utilized a more appropriate sequence of construction.

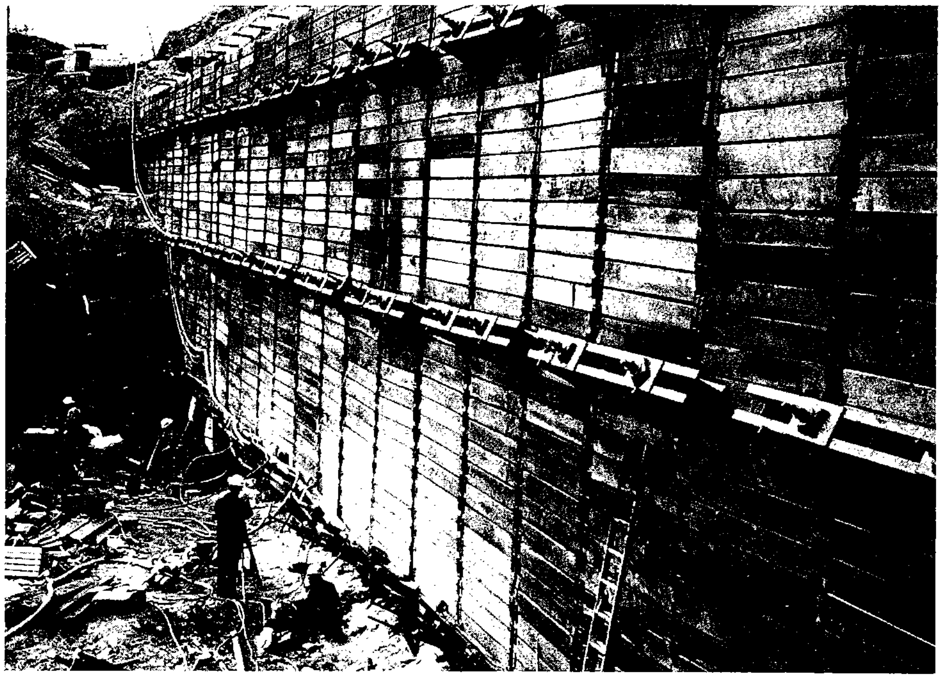


FIGURE 2

WOODSIDE ROAD WALL

The slide at Woodside Road occurred in the Spring of 1984 after long periods of heavy rain. The material through which the slide occurred was compacted fill. Previous slides had occurred along this road, and two cantilevered retaining walls consisting of drilled piers and treated wood lagging already had been constructed in an attempt to stabilize the slides. The 1984 slide removed parts of the two cantilevered drilled pier walls. For the Woodside Road slide, the California Department of Transportation designed a tiedback wall similiar to the ones constructed on twenty to thirty other California highway projects.

The California Department of Transporation prepared the detailed design for the tiedback wall and specified a tieback testing procedure, but left the tieback installation method up to the contractor. A cross section of the wall is shown in Figure 3.

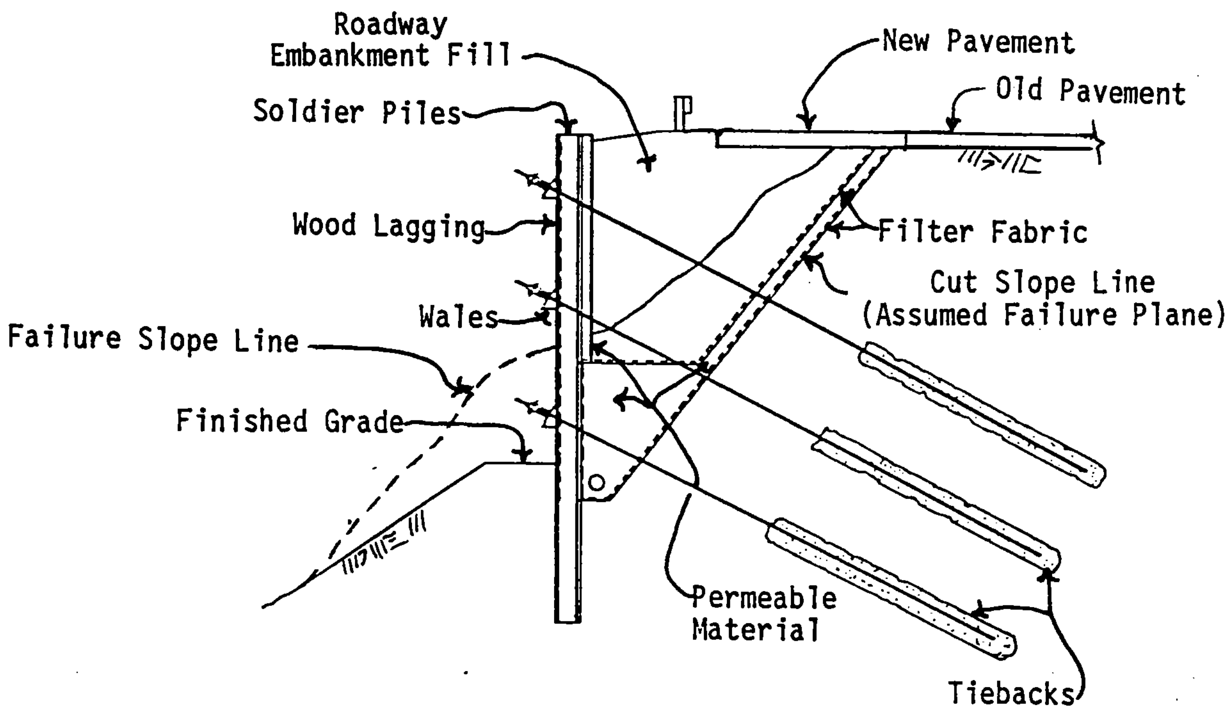


FIGURE 3

The design was based upon removing all slide material back to the assumed failure plane, and then placing controlled backfill behind the wall as the wall was built from the bottom up, the opposite procedure from the usual construction of a tiedback wall. The backfill soil parameters used in the design were $\phi = 33^{\circ}$ and $\gamma = 120$ pcf. The final design consisted of:

- HP 10 x 42 soldier piles on five-foot centers;
- four-inch thick treated lagging between the soldier piles;
- C15 x 33.9 galvanized wales;
- corrosion-protected one-inch diameter 150-ksi bar tiebacks with minimum unbonded length, minimum anchor length, design load, and maximum tieback testing load specified; and
- a combination of permeable backfill with filter cloth and compacted excavated roadway embankment material.

As the contractor, we were allowed to select the tieback installation procedure and the drill hole size. The specification was essentially a prescriptive specification for the wall and a performance specification for tieback installation and testing.

CONSTRUCTION

The sequence of construction was generally as follows:

- the loose slide material was removed to the location of the assumed failure plane, and a bench was established at the bottom of the wall;
- the bottom level of tiebacks were installed;

- o the soldier piles were installed;
- o backfill was placed up halfway to the middle level of tiebacks;
- o the middle level of tiebacks were installed;
- o backfill was placed up halfway to the upper level of tiebacks;
- o the upper level of tiebacks were installed;
- o backfill was placed up to finished grade; and
- o pavement was placed.

As mentioned previously, typical tiedback wall construction proceeds just the opposite, from the top down, even in landslide repair work. Although the tiedback wall on this project and on other California highway projects have performed satisfactorily, they have a potential performance problem which will be discussed later.

Figure 4 shows the slide after the loose material was removed and during the installation of the bottom level of tiebacks.

FIGURE 4

The drill rig is operating on a twenty-foot wide bench which continued to settle and move downslope while installing this level of tiebacks. The drilling for the tiebacks indicated that the anchors were being placed in very compact claystone with seams and lenses of sandstone and cemented sand. Resistance to drilling in the claystone and cemented sand was about the same, while resistance in the sandstone was much greater. After each tieback shaft was drilled, the corrosion-protected tieback tendon and a grout tube was placed to the bottom of the hole. Neat cement grout was then tremied through the bottom of the pipe until clean grout came out to the surface of the drill shaft. The grout was allowed to achieve an initial set of 15 to 30 minutes, then grout was again pumped from the bottom of the hole as a second-stage of pressure grouting. Grout pressures up to 300 psi were achieved until grout began to come up to the surface around the grout plug, at which time the second-stage grouting operation was stopped.

After the tiebacks each had set a minimum of four days, they were tested against the earth, as shown in Figure 5.

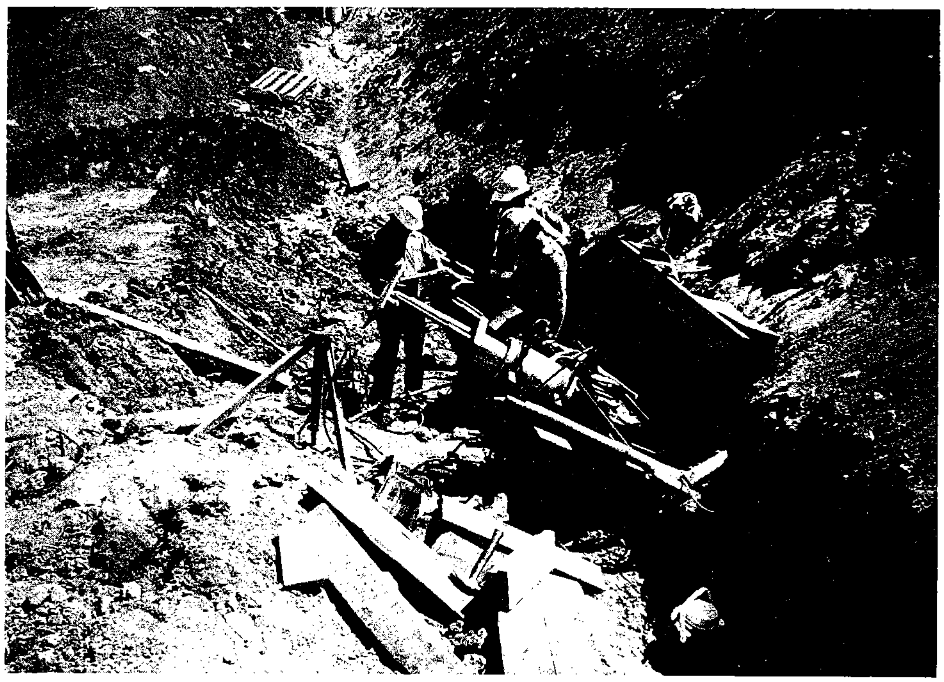


FIGURE 5

Design loads were typically 45 kips, maximum test loads were 90 kips, and lock-off loads were 15 kips. Performance or proof tests, as described by Schnabel[4], were performed on each tieback. The second-stage grouting procedure was established after 30 tiebacks had been installed, and was necessary because 17 of the first 30 tiebacks failed to achieve the required test loads. The replacements for the 17 failed tiebacks, and the remaining 115 tiebacks were grouted with primary tremie grout and second-stage pressure grout, and no further tieback failures occurred.

It is normal to have some tieback failures during the early stages of tieback installation and testing, however the failure rate on the first 30 tiebacks on this project was excessive. These failures, however, point up the advantage to the owner of using a performance specification. Had the owner specified the tiebacks to be installed a certain way, and they then failed, it would be the owner's responsibility to pay for both the original and the replacement anchors and any required changes in the installation procedure. With a tieback performance specification as used on this project, the contractor is responsible for successful performance. On this job, after the tieback failures occurred, the grouting procedure was modified and the failed tiebacks were replaced and tested, with no change in contract amount, and no claim submitted to the owner.

After the lower tiebacks had been installed and tested, the soldier beams were installed, as in Figure 6.

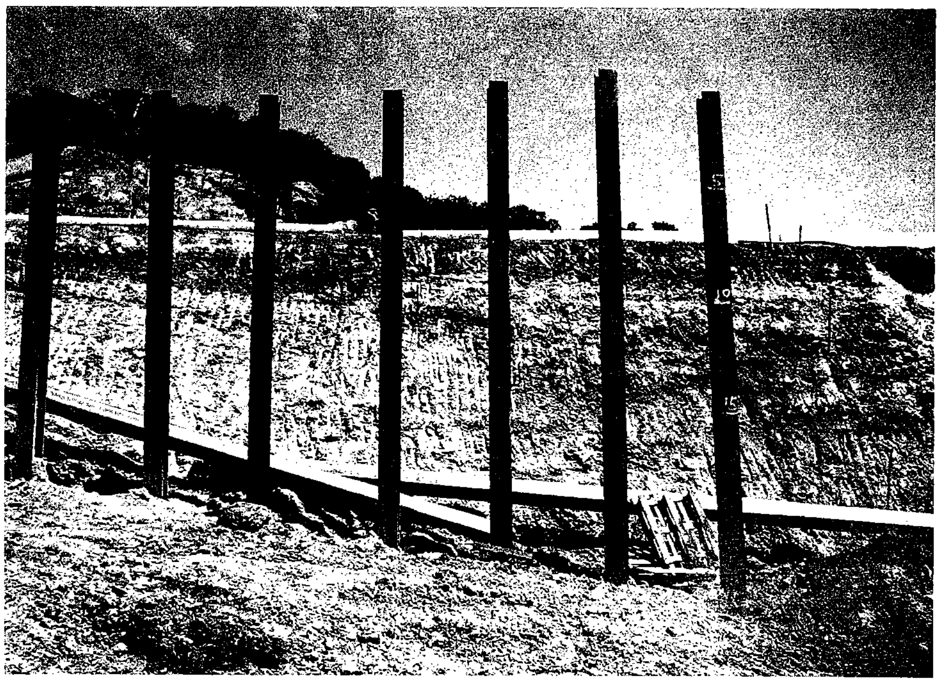


FIGURE 6

Couplers were attached to the lower level tieback tendons and extended past the front of the soldier piles, Figure 7. As can be seen in Figure 7, backfilling around the extended bars will be very difficult and can cause bending in the bars.

FIGURE 7

Figure 8 shows the front of the wall with the lower tiebacks extended through the wale prior to being locked off at 15 kips.

FIGURE 8

Most of the middle and upper levels of tiebacks were installed from behind the wall as shown in Figures 9 and 10.

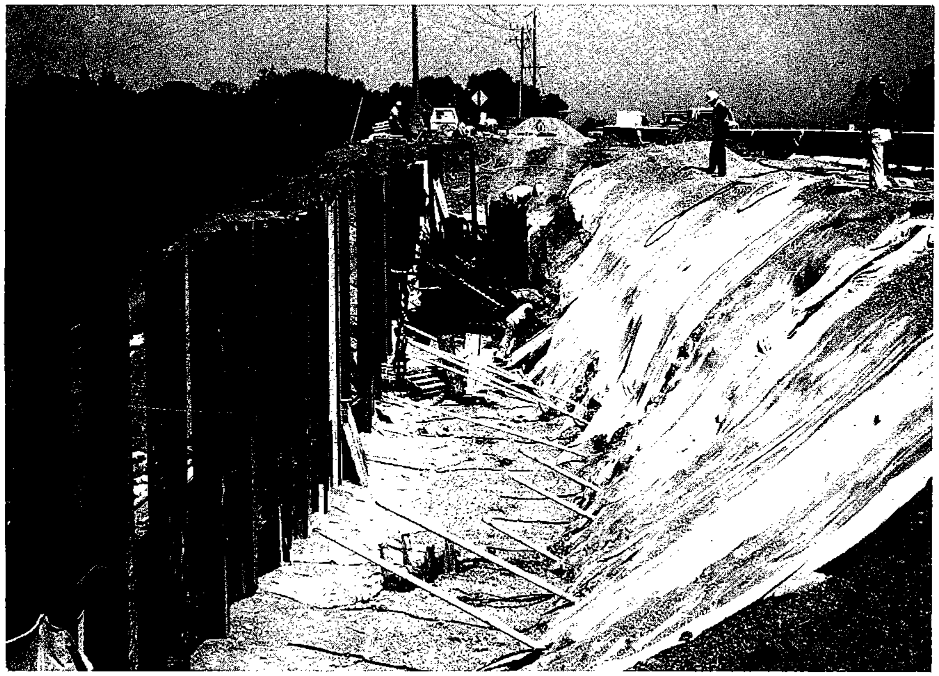


FIGURE 9

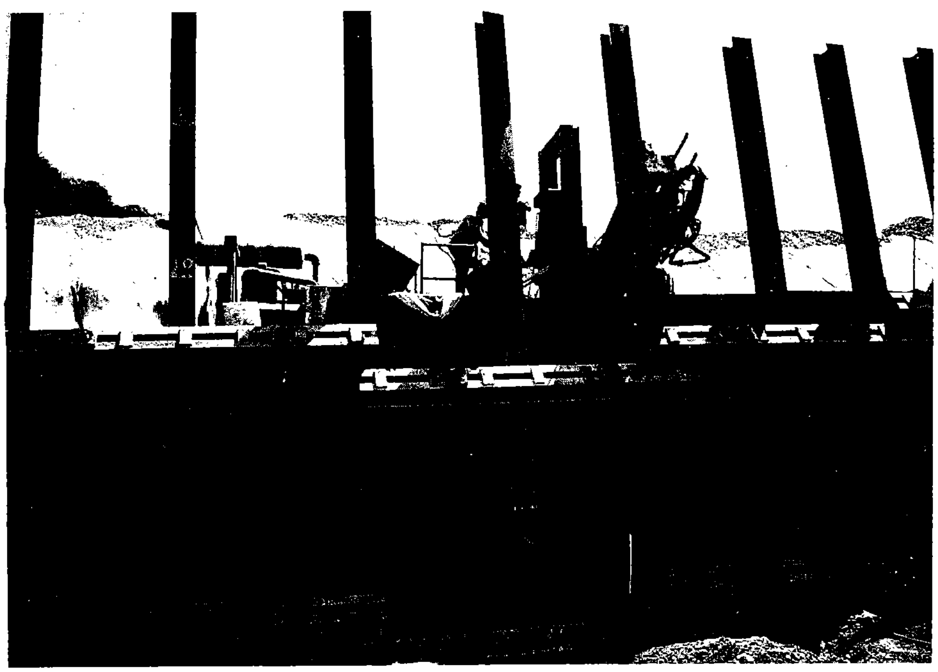


FIGURE 10

Some of the tieback extensions to the front of the wall are very long, up to 20 feet, see Figure 11. Backfilling of the lower portion of the wall was accomplished by end dumping crushed rock from the top of the slope to the bottom of the wall. Backfilling around the upper level of tendons was accomplished by end dumping roadway embankment soil from the top of the slope, then compacting the soil with hand-held compactors around the tendons. Because the tendons were at an angle, good compaction under the tendon was hard to achieve and some bending in the bars was observed. Figure 12 shows one of the completed walls with a maximum depth of approximately 28 feet.

PERFORMANCE

Tieback lift-off tests were performed on all the tiebacks approximately five months after the wall was completed. In all cases, the lift-off load was within five percent of the original lock-off load of 15 kips, which is within expected tolerances.

There are potential performance problems with the system designed for the Woodside Road project. The problem occurs if bending is induced in the bar tieback tendon. The bar tendon is a high-strength post-tensioning tendon with large capacities in tension, but very little capacity in bending. Bending develops when fill is placed over the tendons. The fill tends to be poorly compacted under the bar, and highly compacted over the top of the bar. Bending also develops when the fill settles after completion of the entire backfilling operation. Theoretically, the compacted fill should not settle, but in most cases it will settle or consolidate to some extent. A minor amount of bending over a large radius of curvature would not be a problem necessarily, if both ends of the bar were free to rotate. In the design for this wall, however, one end of the tendon was fixed at the wale, and any rotation would tend to kink and bend the bar against the side of the wale. The other end of curvature was where the tendon left the fill and entered into the natural material. Here it was tightly grouted in a small diameter hole. In general, if the natural material is reasonably soft, then the problem is not too great; however, if the natural material is considerably harder than the fill, which it usually is, then a point of fixity occurs. Carey[1] reports a failure of a tiedback wall built from the bottom up. The failure occurred due to bending in the bar tendon at the contact between very loosely placed fill and very hard rock.

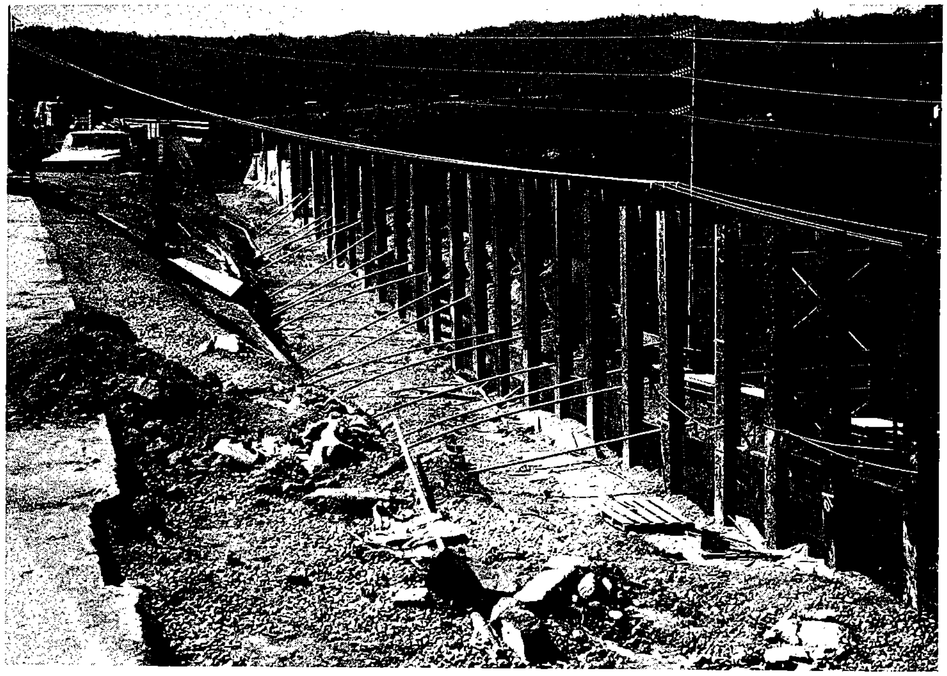


FIGURE 11

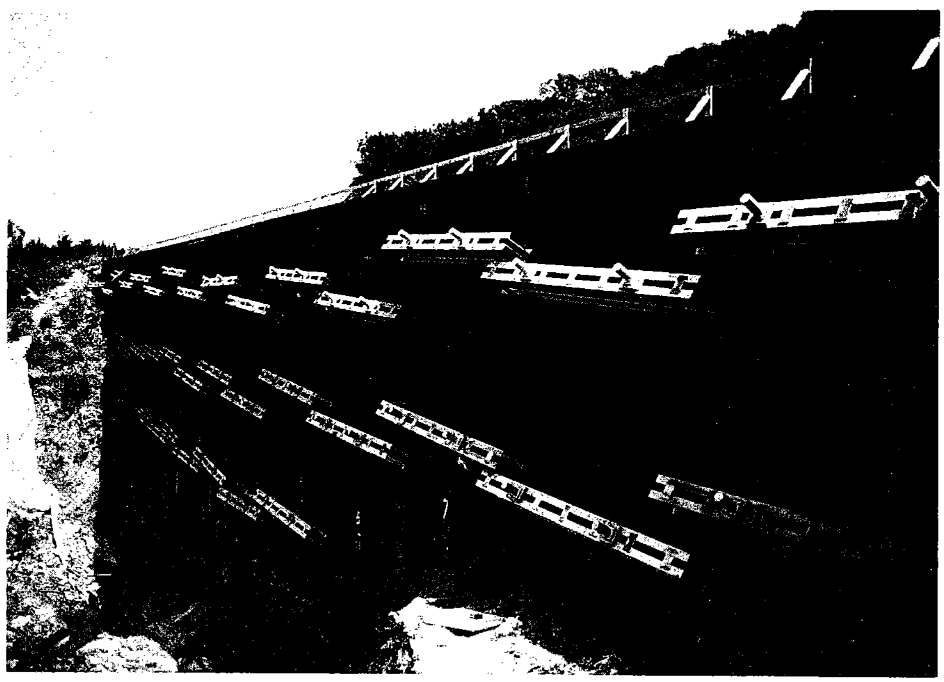


FIGURE 12

So, why have all the projects built from bottom up performed well in California? I believe it is because, typically, the design working loads and lock-off loads for the tiebacks on California projects have been well below the allowable working load of the tendons. In the case reported by Carey[1], the design working load and lock-off load in the tieback was equal to the allowable working load of the tendon. In the Woodside Road project, the design working load was 45 kips and the lock-off load was 15 kips, while the allowable working load of the tendon was 76 kips. Rephrased, bar tiebacks with design working loads and lock-off loads well below the allowable working load have more margin for error, and some capacity in bending. This is a costly way to achieve a factor of safety, as it greatly increases the number of tiebacks required to support a slide.

There were several other items specified that significantly increased the cost of the wall without comparable increases in the quality level. The first was the spacing of the soldier piles at five feet when the industry's standard is to install piles on eight-foot centers. Also, the use of galvanized wales, as opposed to coal tar epoxy or elimination of the wales through the use of a through-beam connection, had an adverse affect on the total cost of the wall.

DESIGN ALTERNATIVES

How can a tiedback wall can be designed to correct a landslide without the hazards of failure due to bending of tiebacks, as reported by Carey[1]? There are two design approaches that have been proven to be technically and economically preferable to the wall construction at Woodside Road. The first approach can be used when the slide debris has not moved down the slide. The second approach is used when the slide debris has moved far downslope.

The first design is illustrated in Figure 13, which shows the wall being built through the slide debris along a track owned by the Clinchfield Railroad Company. Prior to the construction of the wall, the railroad had been placing fill on the top of this slide for over 50 years, and the slide material had crept downslope some 1500 feet. Maintenance crews placed fill weekly during the rainy season and monthly during the dry season. Through discussions among the owner, the geotechnical engineer, and tieback contractors, it was determined that a tiedback wall was the most technically suitable and cost effective type wall to control the slide. The owner prepared a performance specification that required the finished wall to consist of steel soldier beams and wood lagging, and further specified that construction could not disrupt the main line rail traffic at any time. He also provided geotechnical data for the contractors use in preparing design/construct proposals.

Three specialist firms submitted proposals for design and construction of the wall, and Schnabel Foundation Company was selected to perform the work. As a part of our design, we utilized our patented corrosion protection for the tiebacks. Figure 14 shows a view of the completed wall. This landslide stabilization wall was built from the top down, right through the fill. Drainage material was placed behind the wood lagging as the lagging was installed.

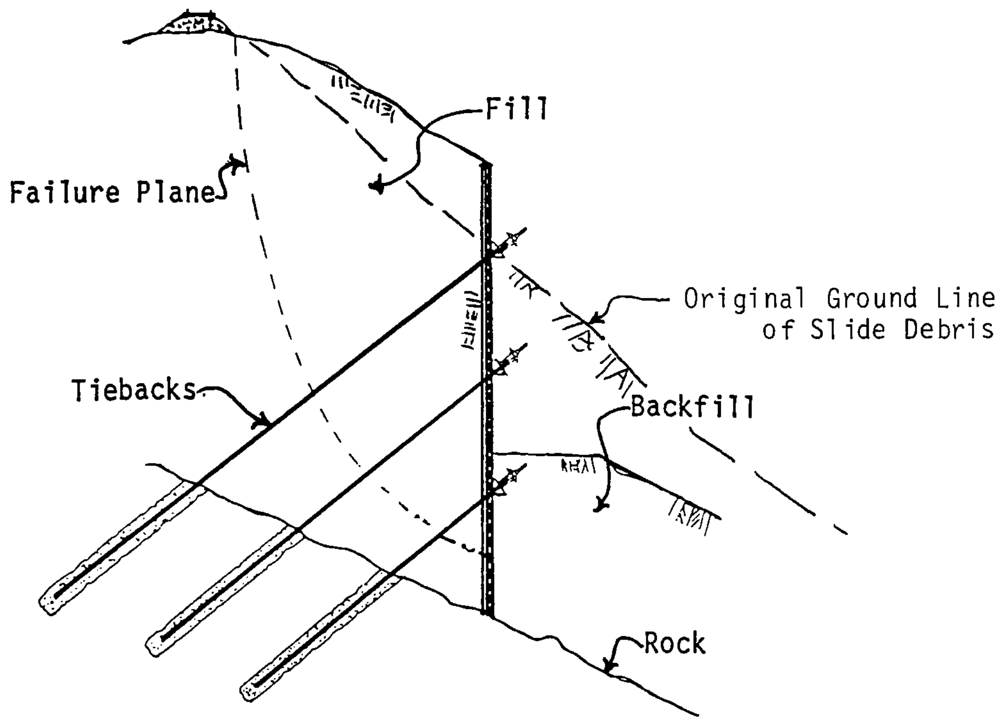


FIGURE 13

FIGURE 14

Figure 15 shows the soldier beams being driven in place. Figure 16 shows the last row of tiebacks being installed.

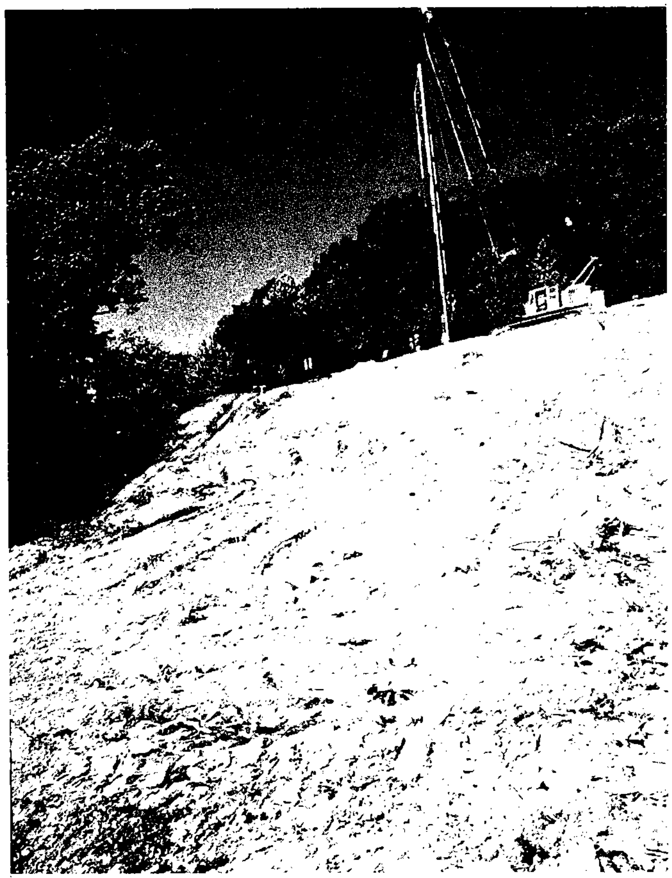


FIGURE 15

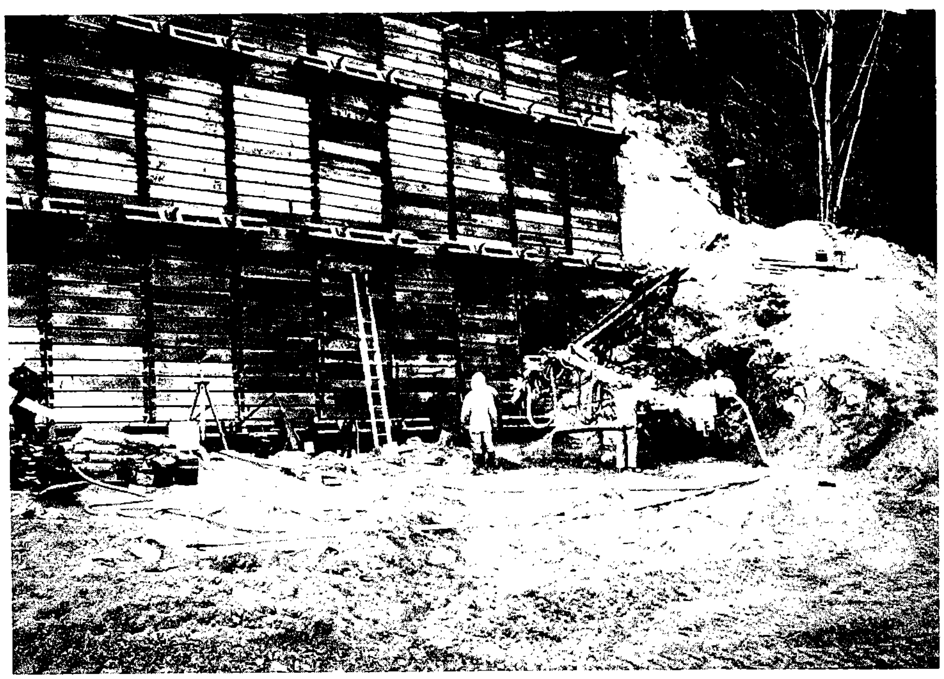


FIGURE 16

The main advantage of the design approach used in this project was that once construction started, every step provided increased stability to the slope. The soldier beams were installed quickly by driving and provided the beginning of movement resistance. After the first row of tiebacks were installed, tested, and locked off, the slide stopped its movement downslope and has not moved since that time (Tysinger[5]).

The key to the success of this project was the owner's use of a performance specification for the entire tiedback wall. It enabled the owner to benefit from the knowledge of several specialized design/construct tieback firms. The owner determined the desired end product, and then the contractors were able to compete with the most cost-effective of several proprietary systems that met the specification.

A second illustration of the design approach of placing a tiedback wall directly through fill is illustrated in Figure 17.

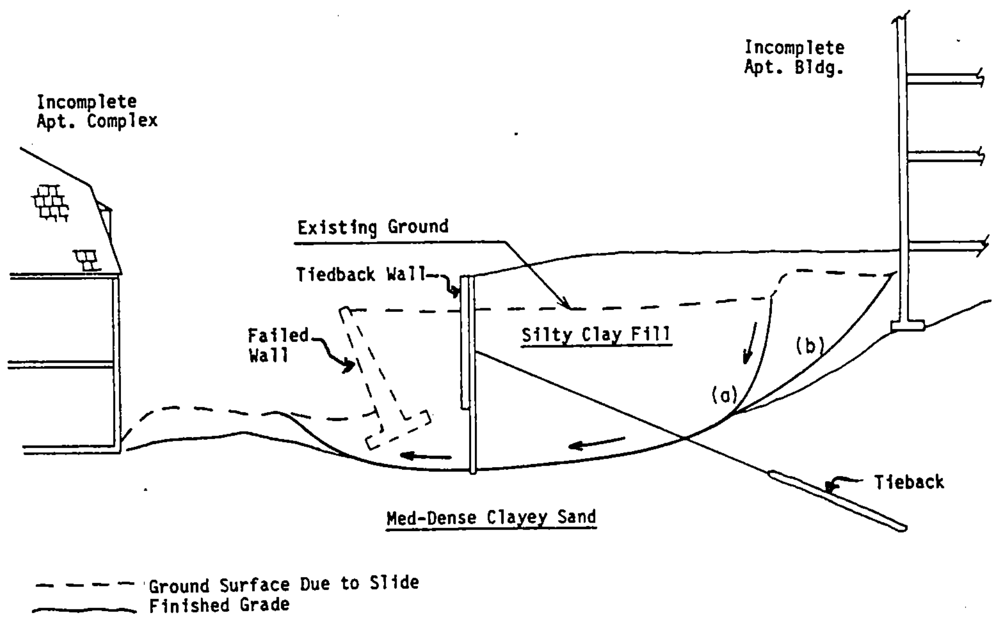


FIGURE 17

On this project, an existing cantilevered retaining wall was failed by a slide. The failure threatened a partially completed apartment complex. Excavation of the slide debris would have caused the head of the slide to move closer to the upslope building and possibly undermine its footings. A rapid construction solution was required. A tiedback wall built through the fill met that requirement and resulted in a successfully performing wall.

Figures 18 and 19 show the patented double-zee soldier beam, wood lagging, and tiedback wall being installed behind the failed cantilever retaining wall. Figure 20 shows the tiedback wall complete with a thin concrete face. The concrete face was the desired end product required in the performance specifications prepared by the owner.

FIGURE 18

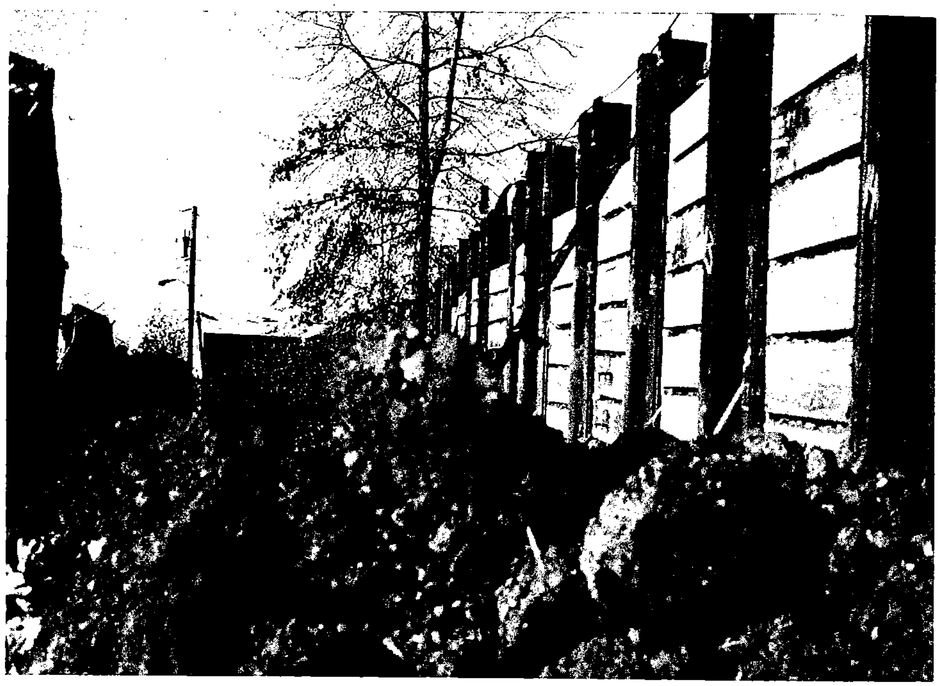


FIGURE 19

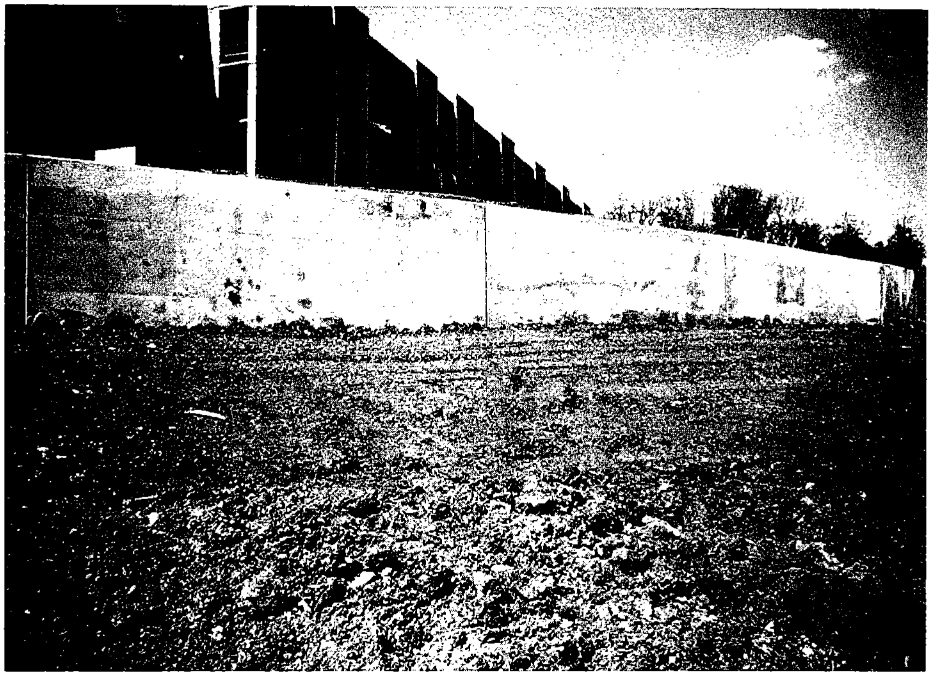


FIGURE 20

The advantage of the design approach on this project was that the wall could be built rapidly behind the existing failed wall, without disturbing the slide debris and/or causing further movement of the slide. Again, the use of a performance specification by an owner gave him the benefit of specialized knowledge and proprietary systems, and resulted in an economically and successfully performing wall.

I would like to digress here a moment and mention that various wall facings can be placed on a tiedback wall. We have discussed two already, wood lagging and concrete. We do not recommend wood because of the potential for continued maintenance problems. Other types of wall facings an owner may want to specify include precast concrete, Figure 21 or cinder block, Figure 22.

The precast boards in Figure 21 were attached to the face of the soldier beams, in front of the wood lagging. The wood lagging had been installed between the soldier beams as the site was excavated to subgrade. The cinder block wall in Figure 22 is a patented system called Ivany Block and is installed from the bottom up in front of the wood lagging.

FIGURE 21

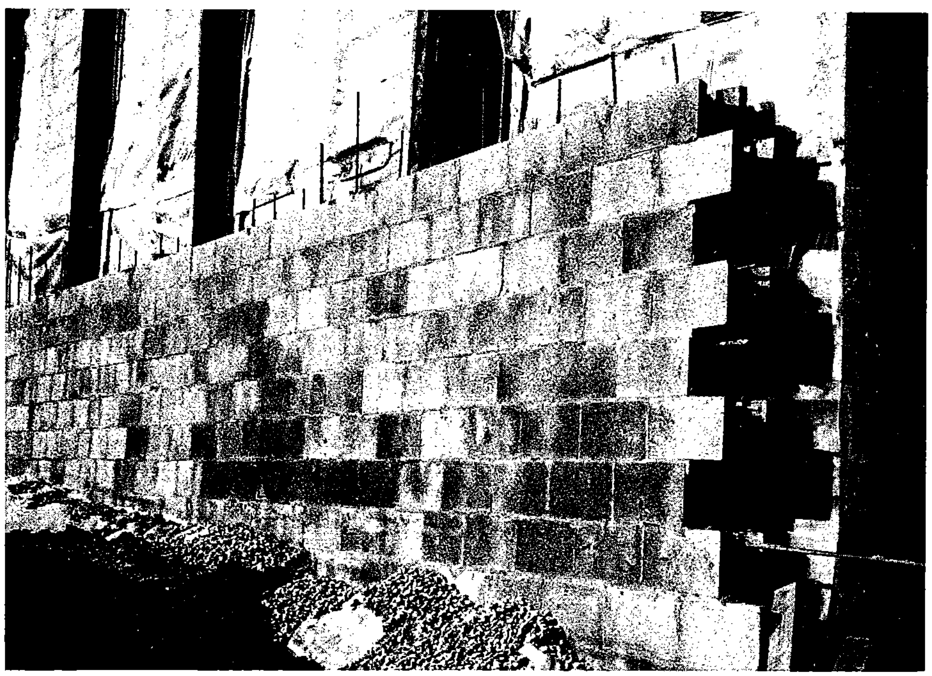
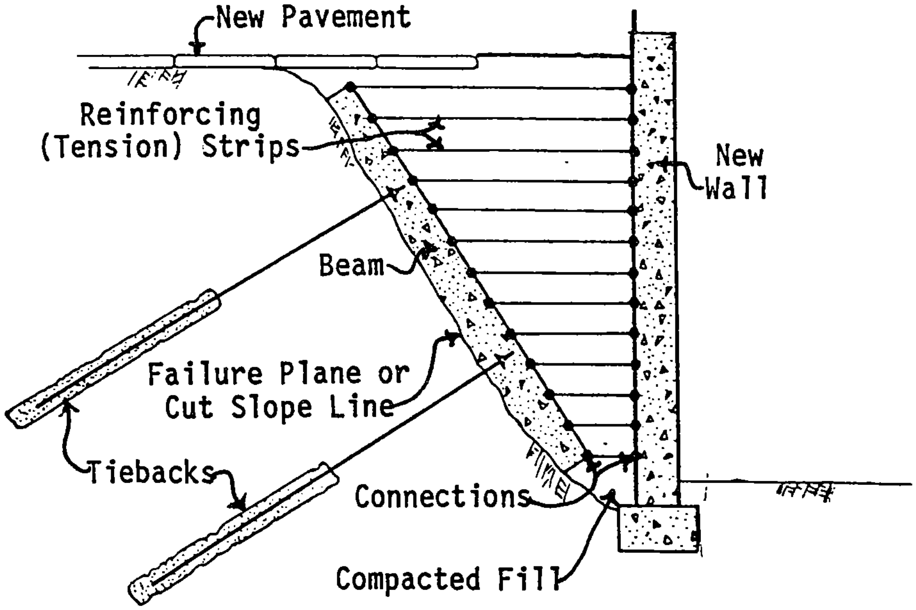


FIGURE 22

Returning to the subject of design approaches, the second design approach is a proprietary system called "Tiedback Earth Fill Wall", Figure 23.



"Tiedback Earth Fill Wall"
(pat. pend.)

FIGURE 23

This approach is used when the slide debris has moved downslope, and there is no material in which to build. With this system, tiebacks are installed on a failure plane or a cut slope and anchored to a transfer beam. A wall is constructed, and horizontal tension strips are attached from the transfer beam to the wall face as backfill is placed from the bottom up. The advantage of a "Tiedback Earth Fill Wall" is that it combines the ability of tiebacks to stabilize a failed slope with the ease of constructing an earth fill wall, while avoiding the potential problems of backfilling around tieback tendons in a fill situation.

CONTRACTING PROCEDURES-PERFORMANCE SPECIFICATION

The tieback industry in America has been developed principally by specialty contractors. Most of the development has been as a result of the contracting procedures used for tieback work in the private sector. The normal procedure is that the specialty contractor is completely responsible for both the design and construction of a tiedback wall. "With this procedure, proprietary products can be used easily, and as a result almost all tiebacks used for this work are proprietary", Schnabel[4].

This procedure is somewhat different from that used in the public agency where the normal contracting procedure is to design and specify all details of construction (prescriptive specification). Tiedback wall construction is not incorporated easily into prescriptive specification work because of its proprietary nature; however, tiedback walls can be incorporated economically into publically funded construction with the use of a performance specification. Contracting agencies will benefit from using a performance specification because it will allow specialty contractors with equal but different proprietary systems to compete with those systems, and still give the agency its desired end product. In several cases, this has been done by public agencies, and performance specifications have been used successfully for tiedback wall construction.

A performance specification can establish a quality level for a finished product without eliminating suitable proprietary tieback systems. It will require prequalification of the tieback contractor based on experience; or a list of acceptable contractors could be included in the specifications. Both methods of contractor prequalifications have been used successfully by different public agencies. When using a performance specification, the designer/specifier (agency) should provide certain information as follows:

- provide a detailed geotechnical site investigation;
- specify a tieback testing procedure and acceptance criterion, and any other testing to be used to check compliance;
- specify the minimum unbonded and total tieback length;
- list the standard specifications to be used in designing the wall;
- indicate the type of wall facing and finish that will be acceptable;
- specify material properties and requirements;
- describe the type of corrosion protection required;
- establish monitoring requirements, and;
- specify an equitable method of payment.

The contractor should then do the following:

- design the tiedback wall;
- select the tieback type and determine its design load;
- select the anchor length;
- design the corrosion protection system;
- be responsible for contract compliance of materials used;
- be responsible for obtaining the tieback load-carrying capacity;
- obtain the required unbonded and minimum total tieback length, and;
- perform the specified tests.

The performance specification method of contracting for tieback work has been accepted in the private sector and is beginning to gain acceptance with public agencies. Schnabel[4] states, "This contracting procedure (performance specification) has gained acceptance because it not only is more economical, but also has resulted in better performance. This should not be too surprising. Tieback specialists have everything to gain from good projects; with control over both the design and construction, they have accepted responsibility for good performance".

But, what is the effect of using a prescriptive specification for tiedback wall work? "When using a prescriptive specification, the designer/owner selects and designs the tieback, the corrosion protection systems, the tieback capacity, the installation methods, the testing procedures, and the wall. The contractor is required to submit material certifications and build the wall in accordance with the specification. When this contracting method is used, the owner is responsible for performance, if the contractor complies with the specifications. This form of specification does not enable the experienced contractor to make best use of his patents or expertise, and frequently contractors unfamiliar with the work are the successful bidders. When an inexperienced contractor obtains the work, then the owner must be prepared to direct the contractor's work if the specification cannot be met. This type of specification does not encourage competition or guarantee low prices; in fact, higher prices, change orders, and delays often result when the wrong tieback systems, drilling methods, tendon type, corrosion protection, or tieback capacity are specified", Weatherby[6].

There are several articles available in the literature that will help contracting agencies in the preparation of performance specifications. Weatherby[6] compares prescriptive specifications to performance specifications, and presents a thoughtful discussion on various scopes of performance specification. Weatherby[7] again reviews requirements of a performance specification and presents a very helpful sample specification. PTI[3] also presents a performance specification. Klinedinst and Dimaggio[2] discuss some of the problems public agencies have in utilizing proprietary systems and suggest methods to overcome those difficulties.

Two of the most important aspects to be included in performance specifications are tieback testing procedures and corrosion protection requirements. Detailed and helpful information concerning these aspects can be found in the works of Schnabel[4], Weatherby[7], and Weatherby and Nicholson[8].

CONCLUSION

- Tiebacks are a positive solution for landslide stabilization because they provide a force to resist the landslide's driving force.

- Tiebacks have been used successfully on many projects and have proved to be technically sound and cost effective.

- Tiedback walls have been built successfully through slide debris. Tiedback walls are more economical and perform better when built from the top down in cut situations.

- Since most tieback installation methods and design systems are proprietary, owners will benefit from using performance specifications that allow specialist contractors to compete on the basis of their own established expertise.

- A performance specification has proven to be the most economical way to provide the owner with a tiedback landslide control wall of the quality desired.

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GEOTECHNICAL DESIGN PARAMETERS FOR CUT-AND-COVER STATIONS AND TUNNEL SEGMENTS OF L.A. METRO RAIL PROJECT

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ABSTRACT

Since June 1980, Southern California Rapid Transit District (SCRTD) has been engaged in the preliminary engineering of the Los Angeles Metro Rail Project. A project team is currently completing a major portion of the preliminary design of an 18.6-mile initial segment of the proposed ultimate rapid transit network.

Collection and synthesis of the general regional geological and geotechnical information were started since early 1960's.

The preliminary geotechnical design parameters and criteria for the project was developed in 1981. More definitive geotechnical information was obtained by 1983 and 1984. The 1983 and 1984 geotechnical design parameters are compiled for a cut-and-cover construction method for underground stations with invert elevations of generally around 25 to 50 feet below street surface, and a twin tunnel line section to be constructed principally by bored tunneling method varying in depth from 25 feet to approximately 125 feet. Up to 700 feet of tunnel depth will be involved beneath the Santa Monica Mountains.

Pertinent geotechnical design parameters were developed after careful examination of data gathered through the basic disciplines of geology, engineering geology, geophysics, seismology, soil and rock mechanics, hydrogeology, petrology, gas chromatography and petroleum analysis.

INTRODUCTION

The initial stage of the geotechnical investigation undertaken in 1980 was performed under a philosophy that before the selection of an appropriate line and grade, investigation endeavors to define exact prediction of all the geologic conditions to be encountered during construction is not economically feasible nor necessary. For a major underground propject, the primary goal of the geotechnical study is to attain some indications of the subsurface conditions and engineering properties to permit reasonable engineering design and cost estimates to enhance planning and alternative analysis. The initial report of 1980 was prepared for that purpose. A second stage investigation was followed in 1983 after SCRTD had adequate assessment of the station locations and rough track line and grade. Additional investigation was undertaken to identify site specific problems to assist the final design. The second stage report addressed typical solutions to a limited selection of potential methods of construction in each segment of the design units of the line. An individual report was prepared for each of the 16 design units selected by the SCRTD based on the need of the urban environment. Special design and test reports were also developed for areas of special concerns identified to be inherent with the geographical and geological characteristics of the alignment. Separate reports addressing seismic design, gas and methane evaluation, and oil-sand (tar-sand) field testing results were further prepared to provide information and guidance for adequate design and successful construction. Groundwater conditions have been continuously monitored to provide more understanding of the seasonal and transient trends.

The 16 design units are shown on Figure 1, Metro Rail Project Design Units. The geotechnical reports should satisfy one of the major basic needs of the owner SCRTD, the designer (Metro Rail Transit Consultants), and the contractor who will construct the

Southern California Rapid Transit District
Metro Rail Project

DESIGN UNITS

PHASE 1

A100 – CENTRAL YARD & SHOPS, INCLUDING YARD LEAD TRACK FROM UNION STATION
A135 – UNION STATION
A140 – TUNNEL SEGMENT BETWEEN UNION STATION & 7TH / FLOWER STATION, INCLUDING CIVIC CENTER & 5TH / HILL STATIONS
A165 – 7TH / FLOWER STATION
A170 – TUNNEL SEGMENT FROM 7TH / FLOWER TO WILSHIRE / VERMONT, INCLUDING THE STATION AT WILSHIRE / ALVARADO
A195 – WILSHIRE / VERMONT STATION

PHASE 2

A220 – TUNNEL SEGMENT BETWEEN WILSHIRE / VERMONT & WILSHIRE / LA BREA, INCLUDING THE WILSHIRE/NORMANDIE & WILSHIRE/WESTERN STATIONS
A240 – WILSHIRE/CRENSHAW STATION
A245 – WILSHIRE/LA BREA STATION
A250 – TUNNEL SEGMENT FROM WILSHIRE/LA BREA TO FAIRFAX/BEVERLY, INCLUDING THE WILSHIRE/FAIRFAX STATION
A275 – FAIRFAX/BEVERLY STATION
A310 – TUNNEL SEGMENT FROM FAIRFAX/BEVERLY TO HOLLYWOOD/CAHUENGA, INCLUDING THE FAIRFAX/SANTA MONICA & LA BREA/SUNSET STATIONS
A350 – HOLLYWOOD/CAHUENGA STATION AND POCKET TRACK
A410 – TUNNEL SEGMENT FROM HOLLYWOOD/CAHUENGA TO UNIVERSAL CITY STATION
A415 – HOLLYWOOD BOWL STATION
A425 – UNIVERSAL CITY STATION
A430 – TUNNEL SEGMENT BETWEEN UNIVERSAL CITY STATION & NORTH HOLLYWOOD STATION
A445 – NORTH HOLLYWOOD STATION & END LINE TAIL TRACK

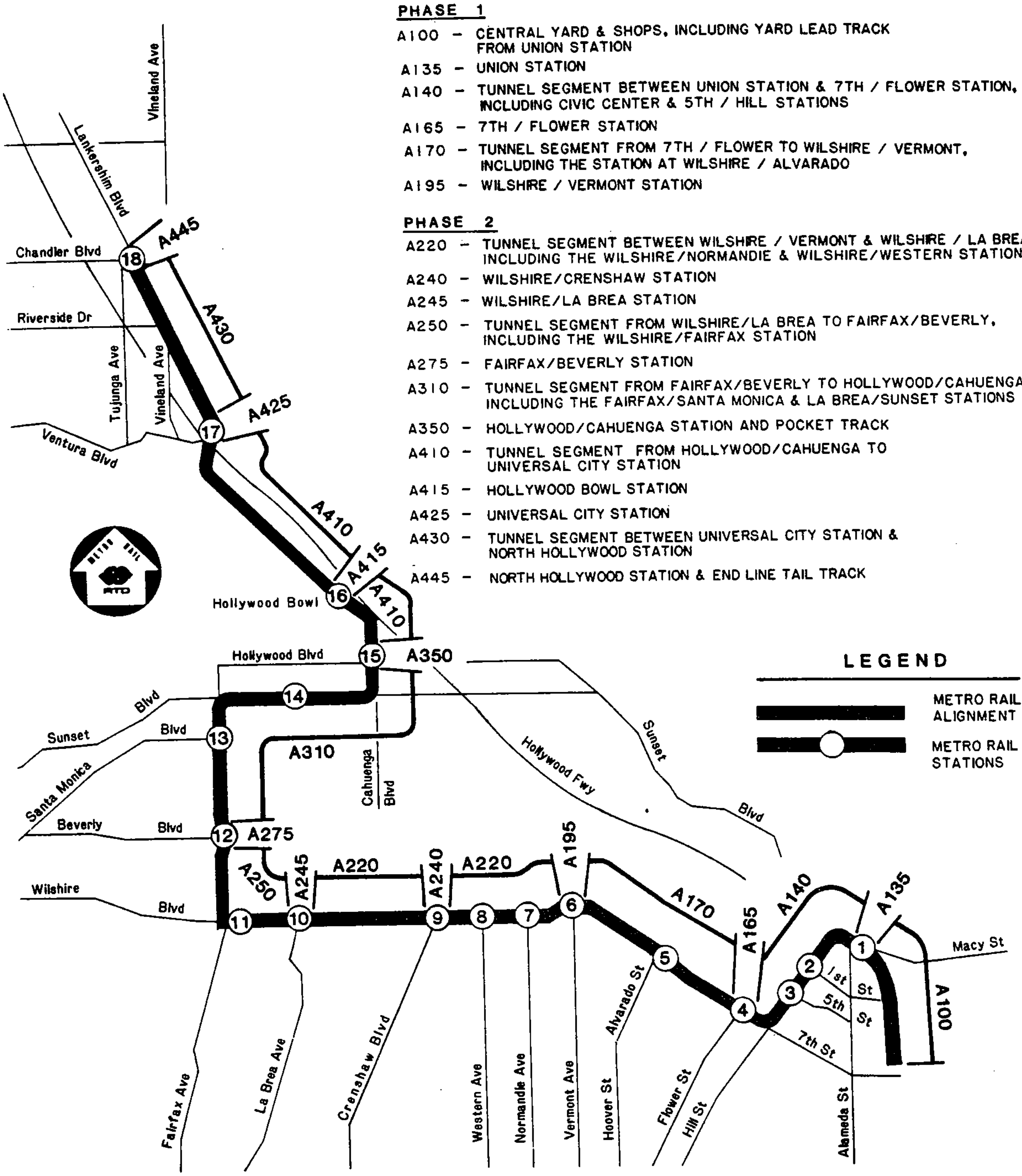

Figure 1 - Metro Rail Project Design Units

structure with the latest methods and equipments available.

DESIGN PHILOSOPHY AND ENVISIONED CONSTRUCTION METHODS

The geotechnical investigation, design, and construction are inextricably correlated. The extent of the geotechnical explorations performed depends heavily on the design philosophy and probable construction methodology envisioned. Conversely, the design also depends heavily on geological and geotechnical conditions found during the exploration. The designer gives considerations of the best and most economical construction methodology available. The combined product of the geotechnical investigation and the design plans provide relevant and specific data for the contractor to prepare the optimal estimate for the bid.

After repeated failures to implement a rapid transit system in Los Angeles since 1925, SCRTD has adopted the philosophy that an optimal system to win the voters' approval has to satisfy the need of the urban problems, the approval of public opinion through community participation, and a balanced transportation system of automobiles, buses, and rail rapid transit to assure socio-economic success. Toward that end, SCRTD charted 12 milestone reports, each corresponding to a vital, interrelated decision point of project development. The station design philosophy which was developed is that all stations in the starter system are to be constructed in below-ground structures using the cut-and-cover method in existing streets or in off-street locations.

Based on local and world-wide experiences of underground constructions, the fundamental construction methods envisioned for the tunnel is the mechanical tunnel excavation by using tunnel boring machine (TBM), ripper, or drag cutter. The conventional drilling and blasting method is not deemed desirable in an urban developed area such as Los Angeles. Noise and structural vibrations would not be tolerated. Controlled blasting in the less urbanized areas, such as the Santa Monica Mountains, may be considered if environmental conditions permit the method. Special provisions are developed for seismic design of the system crossing the known Santa Monica fault zone. Construction difficulties through the tar-sand laden zones and methane-gas rich areas are identified. Groundwater conditions have been constantly monitored. General soil and rock properties in the proposed alignment influence zones have been assessed.

CUT-AND-COVER STATIONS

The presently proposed 18 stations along the 18.6 miles of underground starter line will be constructed by cut-and-cover method. Generally, the station cut-and-cover will be 60 feet wide, 550 to 880 feet long, and 45 to 80 feet deep. Two above grade yard and shops facilities are also planned. The geotechnical parameters check list developed for design and construction use for the cut-and-cover stations and yard and shops included the following:

1. Material properties selected for static design:
   * dry density and moisture content
   * total and effective stress strength and permeability
   * unconfined compressive strength
   * vertical compression modulus
   * subgrade reaction modulus

2. Material properties recommended for dynamic design:
   * average compressional and shear wave velocities
   * Poisson's ratio
   * Young's modulus and constrained modulus
   * shear modulus

3. Underpinning guidelines for structures adjacent to shoring systems

4. Lateral loads on temporary shoring systems:
   * braced shoring conditions
   * cantilevered shoring conditions
   * building surcharge effects
   * construction surcharge effects
   * slope behind wall conditions
   * earthquake loading condition

5. Groundwater conditions, dewatering requirements and induced ground subsidence

6. Lateral loads on permanent walls:
   * end of construction loading condition
   * long term loading condition
   * sidesway of box culvert effect

7. Vertical capacity of piles for shoring and decking

8. Soldier pile passive resistance pressures

9. Straight shaft tieback anchor capacities

10. Shallow foundation bearing capacities and settlements

11. Excavation heave and settlement of station structures

12. Seismic considerations and liquefaction potential

13. Corrosion potential

14. Subsurface gas

15. Tar sand

16. Faults and ground movement

The field exploration and testing programs utilized state-of-the-art applications to deduce the geotechnical parameters and assessment listed above.

LINE CONSTRUCTION

The line sections of the Metro Rail system will be constructed principally by bored tunneling method. Twin bored tunnels will be constructed using mechanized tunnel boring machines (TBMs) which continuously support the ground during the tunneling operation. At the rear of these machines are tunnel liner erection devices that erect precast segments that make up the permanent lining to the tunnels in the form of rings of precast concrete between 3 and 4 feet wide and approximately 17 feet 6 inches inside diameter. These rings serve to carry the earth and rock loads and to prevent groundwater, gas, and oil from entering the tunnels.

Upon completion of tunnel excavation and lining, the crossovers and pocket tracks between the twin tunnels will be constructed by hand mining methods from openings found in the tunnel liners.

The Metro Rail tunnels will involve soft-ground tunneling, hard rock tunneling, and mixed face tunneling. Generally, the ground behavior and tunneling requirements are differentiated by the shallow depth soft-ground and deep rock-tunneling.

The forces that develop on the lining of tunnels in soft-ground units is a function of many complex factors. The major structural factors considered are as follows:

1. ring loads which is a function of earth and and water pressure on lining and the radius of tunnel
2. the bending moment induced in the lining due to the flextural rigidity of the lining
3. surcharge from adjacent structures or construction loads
4. other factors such as buckling, pillar width and jacking forces

The geotechnical factors for soft-ground tunneling are more judgmental, and the parameters established in the cut-and-cover stations section would be very helpful in evaluation of methods to be used. Since the new methodologies are still improving, generally, the following parameters would be very helpful in the selection of the most optimal method:

1. groundwater conditions
2. soil classification, gradation, and properties
3. shear strength and unconfined compressive strength
4. overconsolidation ratio
5. stress-strain-time relationship
6. geologic features

Due to the abundant groundwater and the alluvial material properties found along the Metro Rail alignment, shielded excavation of both open face and closed pressurized face would be feasible.

The Metro Rail project tunnel will involve hard rock tunneling in the Santa Monica Mountain area. The parameters for assisting the selection of a rock tunneling equipment are:

1. detailed geologic mapping of rock type, strike and dip, hardness, space of jointing, faulting, shearing
2. rock quality designation (RQD)
3. unconfined compressive strength
4. seismic velocities
5. groundwater

The geotechnical parameters developed from an investigation program, at best, provide a guide and feeler for the conditions to be anticipated. Actual conditions during construction should always be expected to vary somewhat and modified as construction progresses. The amount of modification necessary should also be anticipated within a known quantitative range. The author believes that the experience factor of the engineers and geologists involved in the L.A. Metro Rail Project has a very sound base to complete the complex project successfully.

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Overcoming Difficulties Encountered During Geotechnical Field Investigations Along Urban Transportation Corridors

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ABSTRACT

A subsurface exploration program along a major urban transportation corridor can only be effective when it is carefully set up and meticulously carried out. The problems of locations of borings, utility clearances, traffic control, scheduling of multiple drill rigs, site safety, and cleanup can be compounded by the distances involved in corridor exploration. Examples of overcoming the logistical problems while obtaining reliable geotechnical data are drawn from experiences in Los Angeles, San Francisco and Seattle. Various projects have involved coordination of as many as four drill rigs operating simultaneously along socially and geologically diverse corridors. Auger, rotary wash and air rotary techniques have been employed in varied urban settings that include vacant lots, busy downtown sidewalks adjacent to highrise buildings, and exclusive residential neighborhoods. Each type of setting requires a particular approach. The geologic conditions have ranged from hard volcanic and sedimentary bedrock to weak clay, loose sand, overconsolidated glacial sediments, gravel, tar sand, shear zones, and of greatest interest, active fault zones. Difficulties encountered while drilling in some of the more troublesome materials may be overcome by proper selection of drill bits, drilling fluids and soil sampling equipment.

INTRODUCTION

A large subsurface exploration program along a major urban transportation corridor is many-phased. Much logistical and technical behind-the-scenes planning is necessary before the field program begins. Initial research helps outline the areas to be investigated by the drilling program. Once these areas are identified budgets and programs are worked up, negotiated, modified, and renegotiated. After a contract is secured, detailed research of available geotechnical and geological reports is necessary so that precise boring locations may be chosen. Many things need to be considered in this phase. The borings must provide the greatest amount of information possible. Borehole sites are chosen where information is needed and no existing data on subsurface conditions are available. Once the technically preferred locations are determined, the logistical aspects of readying these locations for drilling must be evaluated. This involves examining utility structure locations, getting permits and determining the physical constraints and potential safety problems at each site. The potential problems are compounded by the great lengths of transportation corridors. Working out preliminary logistics may become a significant proportion of the entire project effort.

Once the boring locations are determined and all the preliminary logistics are worked out, field forms, formats, and equipment need to be selected. A schedule should be set up with a reasonable timetable. An ideal schedule may be worked out but a more realistic (or perhaps pessimistic) schedule should also be set up. Typically several drill rigs are used to meet schedule constraints, which can place a great deal of pressure on the coordinator. Time and expense budget tracking formats must be set up to monitor the progress. It's important to keep a good handle on the budget especially for the large jobs, as they have greater potential to get out of control. The components of a field exploration program are shown in the flow diagram, Figure 1.

Many of California's geological and geotechnical hazards are related to the state's position along an active tectonic plate boundary. Landslides, subsidence, differential settlement, fault rupture, and other hazards occur in the urban areas. Investigations of these types of hazards generally involve drilling and sampling or coring in materials that are difficult to recover samples from and may even be difficult to drill in. Drilling problems include, but are not limited to, circulation loss, earth movement during drilling operations, caving or running sands, and gas pockets. Many methods are used in an attempt to overcome these problems. Each problem requires a different approach. Selecting the proper drill bits, drill fluids, soil sampling equipment and drilling methods generally solves most problems encountered.

RESEARCH & CONTRACT APPROACH

Preliminary Research

Initial response to the Request for Proposals for an extended subsurface geotechnical investigation along an urban transportation corridor requires some preliminary research. In many urban environments there are geological or geotechnical reports available to the public. A cursory examination of these informs one of the types of materials and potential problem areas that exist. This research is especially important in a long corridor investigation due to the liklihood of variation in the type of material over the length of the alignment.

From the available information a general preliminary subsurface

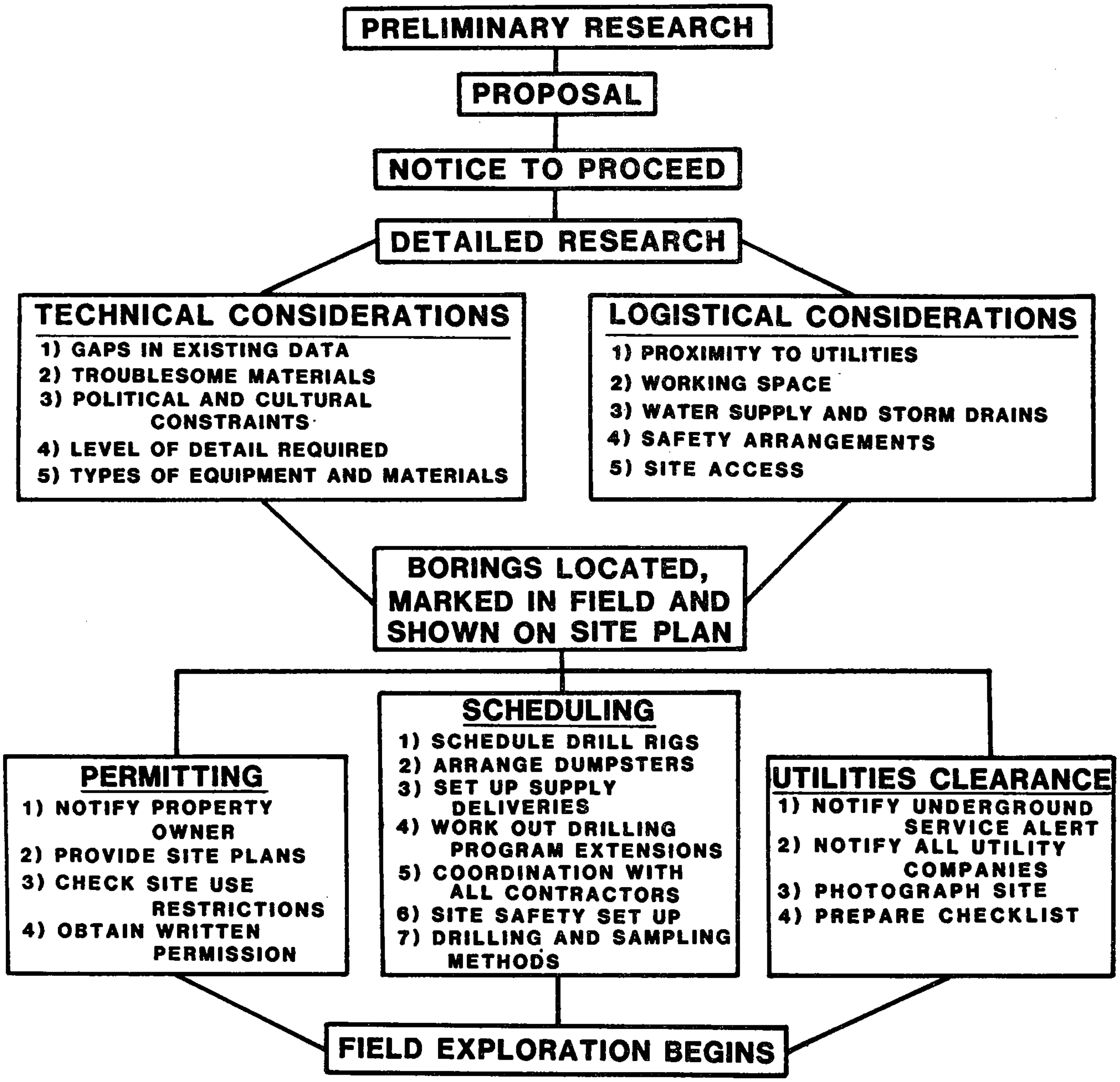

Figure 1: Flow diagram showing temporal overlap of components of a geotechnical field program.

exploration program can be planned to examine those areas of primary concern and areas in which there is a lack of information. Often, in addition to drilling at the locations where more information is needed, the exploration program includes evenly spaced borings to provide uniform subsurface control along the alignment. Ideally, the available data yield enough information to set up a realistic budget for the field exploration program.

Budget Calculations and Schedules

Working up a budget for an extensive field exploration program is complicated. On a large project with several firms involved, each provides a specific expertise. Coordination of the firms to develop a scope of work accomodating each of their needs is necessary to provide a complete investigation. Several route options may be considered at this stage. Rough cost estimates for the project, including those for field investigation, design, construction and operation, are worked up for each option. The client evaluates the estimates in light of the cultural, plitical and geological constraints on the project. Once one or more options are selected, budgets are drawn up and negotiated.

Selection of the location of an alignment for subsurface mass transit or another facility is based on many variables including cost and cultural constraints. Alignments are generally planned to follow existing surface transportation routes. Once a general plan is worked out and a scope of work drawn up, contract negotiations begin. Contract negotiations generally involve interaction with the city, protesting citizens and all the contractors and subcontractors. Typically the client wants state-of-the-art service at discount prices. Consequently terms must be modified and renegotiated several times.

Geotechnical and geological considerations that affect the cost of the investigation, design, and construction include: shallow ground water, fault zones, landslides, types and thickness of problem materials such as gravel and boulder zones, location and extent of toxic waste dumps, tar sands, and oil and gas fields. Field work under some conditions is complicated and safety provisions involving additional time and equipment must be implemented. Investigating toxic waste sites, for example, involves a great deal of additional expense. All these variables must be considered in developing the final scope of work and budget.

TECHNICAL PLANNING OF AN EXPLORATION PROGRAM

Prior to exploration a detailed program must be planned. Borings are located so that geological problems may be resolved and a general understanding of subsurface materials and conditions is gained. Politics plays a large role in organizing an extensive exploration program. On occasion, large government-funded projects become political pawns.

An extensive coordination period is generally required prior to actual field work. Types of equipment and supplies and field forms must be selected based on the information needed.

Setting up a field exploration program is basically a simple procedure until the stumbling blocks begin to pile up. Boring locations are laid out according to geological and political constraints. Borings are widely spaced for a preliminary investigation, but a design investigation requires closer spacing for more detailed information. For a project spanning more than five miles the number of borings can add up.

Often more than one geotechnical firm is involved and each has its own field forms. The forms used should be discussed and modified so that all personnel are satisfied. Field forms to be completed and turned in daily include progress reports, boring logs and piezometer installation reports. Once a week, outside service reports and time and expense sheets should be given to the coordinator. Daily reports to the coordinator allow him or her to modify the drilling program promptly if needed.

Geological Constraints

Detailed research usually provides the geologist with general and specific reports pertaining to sites along the alignment. Available geological and geotechnical information provide a framework on which the important aspects of the field program rest. There are generally many foundation investigation reports that the city allows the public to inspect. Of particular interest are those for large structures (highrises, large office buildings and apartment complexes). A section on geology is usually included. Urban areas, particularly in California, have been thoroughly investigated by both governmental and private concerns. Detailed investigation is ongoing in areas of active faulting, landslides, subsidence and liquefaction. Much of this information is useful in working out a subsurface exploration program along a transportation corridor. Existing conditions, potential problem areas and information gaps can be identified.

Once detailed research is completed, the boring locations are chosen. Initial borings are located where subsurface information is unavailable and design problems are anticipated. These borings yield data that will allow the program to be modified as more detailed information is needed. Any active or potentially active fault zone should be investigated to determine lateral extent and impact on ground water conditions, to examine sheared material and to determine the nature of offset. Borings may be drilled on each side of the fault and/or directly on the fault trace. Major slope movements that could impact design or construction should be investigated, and slide planes should be thoroughly examined. At least one boring should be located where the slide mass crosses the alignment.

Previously identified problem areas should be investigated. Such areas may include tar sands, natural gas pockets, materials exhibiting liquefaction potential (saturated loose sand), areas of shallow ground water, toxic waste disposal sites, bouldery gravel, etc. If the budget allows, at least one boring should be drilled into each area exhibiting troublesome characteristics. If these are not thoroughly investigated, improper design or construction methods could lead to higher construction costs and danger of personal injury. The geotechnical/geological investigation is generally a small portion of the whole project and its importance may be downplayed. Actually the geotechnical/geological work is the cornerstone of a successful project.

Political and Cultural Constraints

Politics often control aspects of public works projects. Political considerations should be (but rarely are) worked out before the field investigation begins. Politicians and citizens groups may impede the project's progress. Many delays and subsequent cost overruns are due to political conflicts. The field season may be compromised by delays, resulting in unpleasant and perhaps unsafe working conditions.

Neighborhood citizens action groups may attempt to change a field program after exploration has begun. Often, people don't think about a project until it is highly visible, and a drill rig is hard to hide. A typical response is, "sure, we need the planned facility, but we don't want it in our neighborhood."

Citizen action groups can be both beneficial and detrimental to a project. They help express the public mood. But they may speak only for a small group that does not represent the majority. The interest of the public at large may be compromised.

FIELD SUPPLIES AND EQUIPMENT

Before a subsurface exploration program begins, decisions must be made concerning drill rigs, piezometer pipe, backfill material, and other supplies. The proper drill rigs should be selected before the budget is negotiated. Costs vary for different types of drill rigs and different drilling companies. Drilling companies and individual drillers' expertise should be considered. Some drillers are challenged by a job bid on a footage basis and are eager to complete it as quickly as possible. A driller's fee based on footage rather than time can save money and keep a project ahead of schedule.

Drill Rigs

There are many types of drill rigs and drilling methods. Most common are flight auger, hollow stem auger, bucket auger, rotary wash, air rotary, and percussion or air hammer. The material anticipated, information desired and environment of exploration sites must be considered in the evaluation of type of drilling method to be used. Each method has its advantages and drawbacks, briefly discussed below.

Flight auger is generally used in fine to medium grained materials in which accurate ground water level readings are needed. They are best used where the surface is soil, as augers tend to break a large area of asphalt, as shown in Figure 2. Concrete can be penetrated slowly, but only with the proper drill bit. If thick concrete is anticipated, additional time should be budgeted. Flight auger generally can't be used below 50 or 60 feet. It is a relatively clean, quick method for shallow dry borings. However, it can be quite messy when ground water is encountered. Flight auger is commonly used for foundation investigations requiring relatively shallow borings.

Hollowstem auger is also used in fine to medium grained materials. The advantage hollowstem has over flight auger is that sampling is faster since samples are taken through the center of the drill pipe and the auger remains in place. As with flight auger, it is best to start a borehole on a soil surface. This method can be used to a depth of approximately 100 feet if no troublesome material is encountered. Hollowstem is also a clean, dry method for shallow borings. Neither auger method works well in coarse gravel or cobbles.

Bucket auger is often used when a large diameter boring is required for downhole examination of in-place material or when installing a well. The volume of cuttings is very large and in an urban environment arrangements must be made to accommodate them.

Rotary wash is used most often for borings deeper than 40 to 50 feet. Almost any material may be drilled. Several types of bits and drilling fluids may be used to overcome difficulties. (Bits and drilling fluids are discussed below.) A "garbage barrel" bit cuts with relative ease through all types of unreinforced concrete and asphalt. Rotary wash methods can be used to depths of 700 to 800 feet or greater for both rock coring and soil sampling. The primary drawback of this method is the mess created by the introduction of fluids (see Figure 3). Setup and cleanup take longer than with other drilling methods.

Air Rotary is similar to rotary wash except air is used as the drilling fluid. This method can be used in situations that are also appropriate for rotary wash. Once ground water is encountered, a foam is used. Both air and foam drilling can make an atrocious mess and the compressors are exceedingly noisy. This method is not recommended for urban environments.

Percussion drilling is sometimes called air hammer. This method is used primarily in hard rock or bouldery conditions and is much faster than rotary drilling, but only provides sand

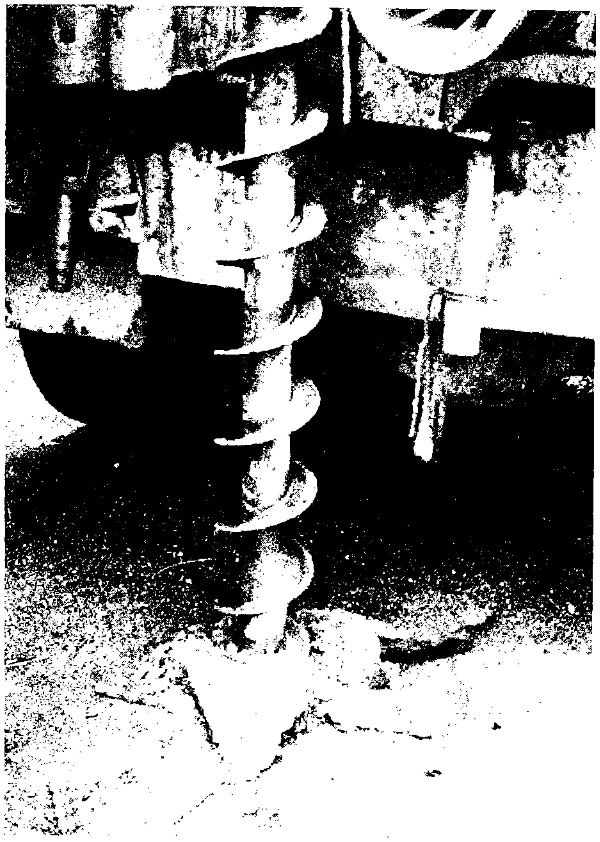

Figure 2: Beginning a new borehole with hollowstem auger, note condition of pavement.

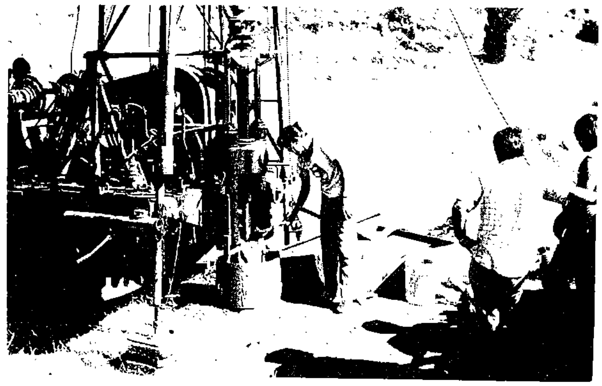

Figure 3: Drilling with rotary wash equipment.

and silt sized cutting samples. Coring cannot be performed by this type of drill rig. Like air rotary, percussion can be noisy and dirty, and is best to use out of the urban environment.

Piezometer Pipe

Piezometer pipe is sized as required for piezometers or water quality sampling. Multiple piezometers are often installed in a single boring where there is perched ground water or multiple aquifers (see Figure 4). Multiple piezometer installations in a standard five inch borehole generally require 3/4 to 1 inch diameter piezometer pipe in order to have enough space. For piezometer development, 3/4 inch pipe is the minimum size easily worked with. There must be enough space to allow a blow-out hose to extend down to the perforated section and to let water return in the annulus. Often when gluing the pipe sections together, excess glue gathers on the inside at the joint. This blocks the blow-out hose from smooth travel to the proper depth. Thus, it is important to insist on careful installation.

When water samples will be collected, larger diameter pipe is required. A two inch pipe could be used, however, three to four inch pipe is easier to work with. If it is necessary to install multiple piezometers and take water quality samples, then the two inch pipe must be used unless a larger hole is drilled. It is usually better to keep the borehole less than six inches in diameter when drilling in water-bearing gravel to minimize caving.

Backfill Materials

Sand or gravel should be used for backfill. Drill cuttings do not make good backfill material. When streets and sidewalks in urban areas are to be cemented or asphalted over following drilling, a suitable backfill must be used to prevent subsidence. If subsidence were to occur and an accident happened, there would be a question of liability. Piezometer boreholes require good quality pea gravel or 4x8 sand for proper backfill. These borings typically have surface casing and a cap, so subsidence shouldn't be a problem. Fine grained

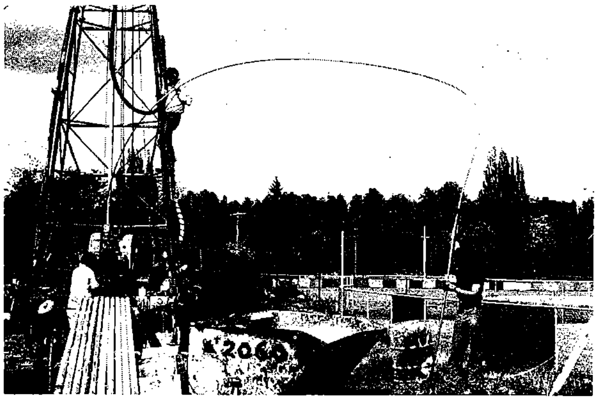

Figure 4: Installation of 3/4-inch PVC pipe in boring with multiple piezometers.

material used as backfill may accumulate in the perforations and damage the piezometer.

LOGISTICAL PLANNING OF AN EXPLORATION PROGRAM

Planning the logistics of a large exploration program is often just as time consuming and exacting as planning the technical aspects. Large exploration programs involving several drill rigs operating along a major urban transportation corridor require a lot of behind-the-scenes coordination. Advance planning is needed to insure that each component of the program will be ready to go exactly when it is called for. Selecting drilling sites, obtaining utility clearances and drilling permits, and mobilizing drill rigs and support equipment all take time and good organization.

The first step is a field reconnaissance along the exploration corridor. Pertinent borehole site restrictions such as limited access, no parking, or an exclusive neighborhood should be noted. Arrangements must be made to obtain water (especially for rotary wash borings). City water may be purchased and drawn from fire hydrants when a permit is obtained and a meter is used. Each municipality has its own procedures to follow.

Drilling fluids and cuttings have to be properly disposed of. Cuttings and viscous liquids are treated as solid waste and should be disposed of in large dumpsters. Thin fluid may be pumped or gravity fed to a storm drain. If storm drain catch basins are clogged with debris, request ahead of time that the city clean them. The sanitation company needs ample notice to keep up with the exploration program.

Most of the boreholes are likely to be drilled on city property. Many cities require the drilling contractor to furnish a certificate of insurance before exploration may begin. The certificate is kept on file with the city, and checked each time a new drilling permit is issued.

Temporary "no parking" signs will be needed to reserve space at drilling sites. Arrangements should be made for a safe place to store the drill rig and other equipment during work breaks.

At some sites special rules apply. Heavily travelled lanes may not be permitted to be blocked during "rush hour", work may be allowed only at night or during weekends, or drilling may be banned in popular shopping neighborhoods between Thanksgiving and New Years Day. The exploration program runs smoothly as long as this type of special consideration is known and planned for.

Plans showing the location of buried utility structures should be obtained as early as possible. Where proposed borehole sites are on private property, the land owner, once identified, must be contacted to arrange for permission and site access.

The director of the field exploration program is responsible to see that adequate supplies are on hand and are delivered to each borehole site as needed. Depending on arrangements with the drilling contractor, supplies may include drilling mud, casing, piezometer pipe, gravel, grouting compounds, sampling equipment, borehole covers, etc.

An exploration program runs most efficiently when all of the drill rigs are operating close to each other along one portion of the corridor. Water meters and dumpsters may be used by more than one rig, and tools and supplies are easier to share. If needed, logging geologists can compare notes, support services can be better coordinated and the field director can cover less ground and maintain better communication with all personnel.

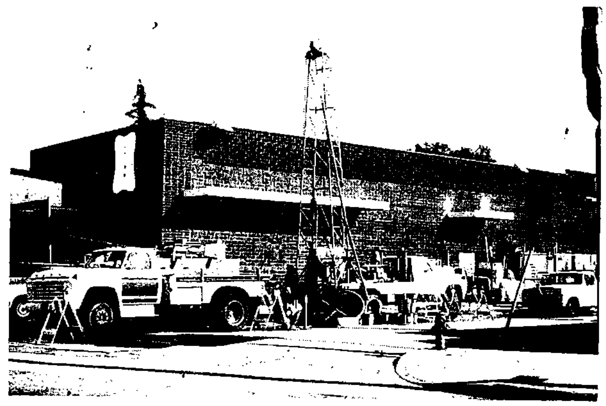

Figure 5: Crowded working conditions in urban area.

Choosing Borehole Sites

Borehole sites are selected to satisfy many logistical requirements as well as to provide pertinent technical data. Once a target area satisfying the technical needs has been identified, the area is visited to choose a feasible site. In the urban environment most drilling sites are located on the parking strips of city streets and sidewalks.

The borehole site should be on level ground or a relatively gentle slope. Adequate site access and working room are needed for the drill rig, logging geologist's vehicle, and associated equipment (see Figure 5).

A storm drain catch basin is desirable down slope from the drilling site in case of overflow and for ease of cleanup. If arrangements have been made to use municipal water, the site should be near a fire hydrant or other source. All parts of the drill rig must be at least ten feet from overhead power lines and at least five feet from other overhead installations. (These requirements may vary somewhat from state to state.) Driveways and traffic lanes may be blocked if permission is granted. Whenever possible, a borehole site is chosen away from important thoroughfares and sensitive residential and commercial property so that drilling operations do not create a nuisance.

The locations of underground utility structures are an important factor in choosing drilling sites (see Utility Clearance below). Considerable hazard and expense are involved in drilling through and repairing buried utility lines. Whenever available, plans showing locations of utility structures should be used in the field to select unobstructed sites. Plans may be outdated, inaccurate, or incomplete, so the locations must be double checked with the utility companies. When plans are not available, careful observation provides clues to the whereabouts of buried structures. The following features usually indicate underground utility structures: 1) Backfilled trenches; 2) Repaved sections of streets and sidewalks; 3) Manhole covers and other service vaults; 4) "Risers" on telephone or power poles (covered conduits that extend from the pavement to the overhead wires); 5) Linear reaches of slight subsidence (where trenches were not properly backfilled and compacted), and; 6) Older street marks painted by utility company personnel. Throughout some areas, utility structures are located in the same relative positions block after block. For example, in a certain neighborhood, water lines may be consistently located 16 feet north of property lines and telephone cables may be buried five feet north of property lines.

When a proposed drilling location is chosen, an identifying mark is made. A circle surrounding a cross of bright colored, fast drying spray paint works well on pavement, and a stake and surveyors flagging identifies the site in an unpaved area. A tape or other measuring device is used to pinpoint the proposed drilling site with respect to property lines and other relevant reference points. The exact location of the proposed site is then plotted on a site plan. If no site plan is available, a sketch map is drawn showing the proposed site (see Figure 6). The sketch map should show the borehole locations and numbers, property lines, street names, the closest street address, other pertinent landmarks, and a north arrow. If alternate boring locations at the site are identified, time may be saved during the utility clearance process if the preferred location is not feasible.

For drilling sites located off of city property, the land owner can be identified on maps in the city or county assessors office. It may take longer to begin drilling at these sites since a different permitting process is involved and utility companies' plans may be less complete and accurate than for sites on city property.

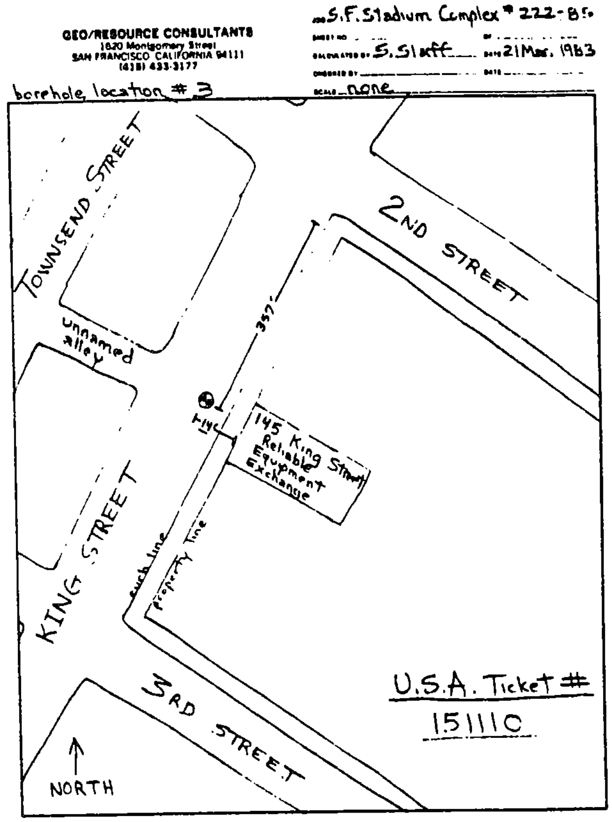

Figure 6: Borehole location sketch map.

Utility Clearance

After the proposed borehole sites have been chosen and marked on the site plan, the utility clearance process begins. The first step is to call Underground Service Alert (U.S.A.), a service organization that reports locations of proposed excavations to its member utility companies. Some cities, such as San Francisco, require U.S.A. to be contacted before they will issue a drilling permit. All calls to U.S.A. are tape recorded to provide a permanent record in case a conflict develops concerning a reported drilling location. The caller pays no fee, since the utility companies support the operation. The caller reports the proposed borehole locations, and the time and date the work is to begin. U.S.A. issues a "ticket number" identifying each proposed site. Note the ticket number on the site plan (or location sketch map) and verify which utility companies U.S.A. will contact. They generally cover those that operate gas, electric, telephone, and water lines. Direct contact between the firm in charge of the exploration program and one or more of the utility companies may be necessary. If so, the procedure described below can be followed.

Utility companies, other agencies, and private companies that are not members of U.S.A. might also own subsurface facilities in the target areas. Fire departments, municipal engineering departments, transit districts, communications companies, sanitary engineering departments, and others should be contacted directly. Those without subsurface facilities in the area can be cleared with a single telephone call or letter. For all phone calls, get the name and title of the person contacted and keep a written record of the conversation. This kind of documentation could be very important if a conflict develops. If an organization does have underground equipment nearby, send them a copy of the site plan showing the proposed borehole location and request that they plot the location of their equipment on the site plan, sign it, and return it. Ask them to mark the location with paint or stakes in the field. They should mark a large enough area so that if the borehole must be moved, they won't need to return to the site. If there is a chance for a problem to develop, set up a field meeting and go over the locations with a representative of the utility company.

Techniques for locating buried structures and the willingness to do so vary considerably from one organization to the next. At best, the indicated locations are a very close approximation. Get a feeling for the quality of the agency's maps and note field locating equipment used. Then, establish reasonable set-backs based on the degree of certainty of the buried utility location. Some drillers swear by their own "witching rods", but they can not be trusted in lieu of proper utilities clearances. Mistakes may result in personal injury and extremely expensive law suits.

The utility clearance process works most efficiently when batches of approximately 15 to 20 proposed borehole locations are handled at once. A table showing each borehole, all the utility companies and other organizations involved, and the status of each clearance and permit helps avoid mistakes and shows at a glance which sites are ready to be drilled. A photograph of each site taken before the drill rig arrives can be a useful record. It shows the borehole location with respect to property lines and landmarks, marked locations of buried structures, and the condition to which the site must be returned after drilling.

Drilling Permits

Drilling permits should be obtained while the utility clearance process is under way. It is important to apply for permits as early in the exploration program as possible, since many agencies process applications slowly. After the first batch of permits is obtained, the process becomes routine once contact has been made with personnel in the appropriate divisions. Subsequent permits are generally easier and quicker to obtain.

Most of the borehole sites will be on city property, so the following discussion refers to a city permitting agency. Some desirable drilling locations will be on county, state, federal, or private property and each has different requirements for obtaining permission to drill. If the procedures prove to be too slow or costly, proposed borehole locations may have to be shifted to city property. In some cases, the land owner will allow exploration on his property if he is provided with a copy of the borehole log. If the client is willing, this arrangement benefits all parties.

Unlike smaller projects, in which several borings may be drilled on a site covered by a single permit, the wide spacing of drill sites along a proposed transportation corridor generally requires a separate permit for each borehole. Permit fees may be waived if the exploration is being conducted for the city. Similar to utility clearances, it's more efficient to obtain permits in batches of 15 to 20 at a time.

The engineering division of the city department of public works is frequently the lead agency in handling permit applications. The traffic and building inspection departments may also be involved. These agencies generally require the following items before a permit will be issued:

1) Site plan or sketch showing the proposed boring location;

2) Cover letter identifying the project and drilling contractor, the schedule, and the type of equipment to be used;

3) Certificate of insurance from the drilling contractor and, in some cases, the engineering firm;

4) Statement releasing the city from liability for the project;

5) Fee payment, or waiver letter.

The permit, in some cases, restricts the work periods. A copy of the permit must be kept on the site throughout the course of the work, although in practice the police, city inspectors, and private property owners virtually never ask to see it. In Los Angeles, city inspectors visit the drilling sites almost daily whereas in San Francisco there are no inspections.

Site Setup

Before the drill rig moves to a new borehole location, parked cars must be cleared from the site. Temporary "no parking" signs attached to safety barricades should be placed prominently around the site 24 hours before work is scheduled to begin. At that time, notify the police of the locations of the signs so that cars may be legally towed away if they are blocking the site when drilling is supposed to begin. Some drillers prefer not to have a car towed because vandalism may occur in retaliation. The period of time in which the rig is expected to be on the site should be written on the "no parking" signs. The geologist should keep the time interval as brief as possible to minimize the inconvenience but allow ample time to complete the boring and testing in spite of unforseen delays.

Dumpsters should be provided for removal of drill cuttings, fluids and garbage that accumulates around a drill site. There are several choices for dumpster locations and to a certain degree it is based on the type of dumpster that can be provided by the disposal company. For single widely spaced boreholes, small watertight dumpsters may be provided at each drilling location. The primary drawback of this system is scheduling dumpster movement. If the borings are very shallow then the dumpster must be moved too frequently.

A large, centrally located watertight container is good for widely spaced clusters of boreholes. We used this arrangement at proposed subway stations along the Los Angeles corridor. The drill fluids and cuttings are loaded into barrels, then emptied into the dumpster. The dumpster can then be emptied if full or moved to the next cluster location. The drawback of this arrangement is the time and effort spent moving the cuttings twice (into barrels then into a dumpster). However, if borings are very shallow, it's the only way to go.

Site Safety

It is the logging geologist's responsibility to see that safe conditions are maintained at the drill site. All field personnel should be provided with a memo describing safety procedures.

Before work begins at a new site, the logging geologist verifies that no utility structures have been identified at the proposed drilling location. If a location needs to be shifted or a utility company's markings are ambiguous, drilling should not begin until the person responsible for utility clearance has been contacted.

All field personnel wear hard hats, boots with steel toes, and protective clothing. Bright colored vests are used when working in a street. Traffic lane closures should be made with warning signs, safety cones and barricades far enough in advance of the borehole site that motorists merge early. The public is restricted by cordoning off the work area with surveyors flagging or rope, but an ample path around the site has to be provided for pedestrians. Tools, equipment, drilling fluid and cuttings should be kept within the immediate drilling site. Clean the site if dangerous conditions develop, and insure that it is isolated and secure over night.

A good driller will "feel his way down" carefully for the first 20 feet or so just in case unmarked utility structures are present. The logging geologist should watch for trench backfill material and tell the driller if he finds any in the cuttings. The field personnel should know where the nearest public telephone is, and the geologist needs to have a list of pertinent telephone numbers, including the fire department, police, ambulance, utility companies' emergency numbers, and the city department of public works. The driller's helper and geologist need to know how to shut off the drill rig. Communication devices such as C.B. radios and electronic pagers improve site safety. If a serious situation develops, shut down the rig until it is safe to resume drilling.

DRILLING AND SAMPLING METHODS

Often materials are encountered during subsurface exploration that present a challenge to the driller. There are many ways to deal with a troublesome boring. A number of drilling fluids on the market are designed to overcome circulation loss and caving. These are the two most common problems. If all else fails, the driller uses intuition and can try a variety of other materials in an attempt to solve the problem.

Trying a different drill bit or sampling method may help to recover an ellusive sample. A large variety of drill bits and sampling equipment is available. Each piece of equipment works best in certain types of earth material. Before drilling begins the drilling contractor should be told what kinds of soil and rock the geologist expects to encounter. Then the driller can choose from an appropriate array of trouble-shooting equipment.

Drill Fluids

A rotary wash drill rig is generally used in urban areas for large geotechnical projects requiring samples from moderately deep boreholes. If problems with caving and circulation loss occur, the first corrective measure is usually to mix a drilling mud. Bentonite and an organic polymer are the two most common drilling muds used for geotechnical investigations.

The organic polymer is a powdered additive that is mixed with water to increase viscosity. This is helpful when piezometers are to be installed because it breaks down over time (approximately four days). A mixing funnel should be used to properly shear the powder to get a thorough mix. Viscosity will only increase if the fluid is within a limited range of pH. The organic polymer breaks down by bacterial action. Additives are available that kill the bacteria, preventing or slowing the breakdown. The polymer is not recommended for drilling into a confined producing aquifer since the bacteria could contaminate the water.

Bentonite is a natural clay that is powdered. It can be added to water without a mixing funnel, but it combines more completely when a funnel is used. Bentonite is a swelling clay that easily forms a wall cake, filling in and binding caving materials. It helps keep the borehole open and discourages water loss. Adding barite to bentonite adds weight to the fluid and helps bring up larger particles in the cuttings. In some formations barite will help hold back artesian water. There is no way to break down bentonite chemically. It remains permanently unless the wall cake is physically knocked off the wall. It should not be used if piezometers or water pressure tests are scheduled. The advantages bentonite has over the organic polymer are cost and noncontamination. Pound for pound, the polymer is approximately 20 times more expensive and bentonite is a natural noncontaminating clay.

Many other drill fluids are on the market, including synthetic polymers. The polymers are broken down chemically by adding a breakdown agent or drastically changing the pH. Some are expensive and not readily available. This, in addition to not being well understood, precludes their use by many drillers.

A driller has several options when he is using a thick, heavy drilling mud and still losing circulation. He can drive the casing deeper, if drilling conditions permit. Lost circulation materials may plug the voids. Among the most common materials used are oatmeal, hay, rags, paper and horse dung. If that doesn't work, grouting the hole is usually attempted. When all else fails he can abandon the borehole, move the rig a short distance, and try again.

Soil Samplers

For good sample recovery in rock and soil the proper sampling method or drill bit must utilized. On occasion a combination of rock and soil sampling methods may be the best solution. Descriptions of the most common soil sampling methods used in geotechnical investigations follow:

Pitcher barrel sampler: a push-drill method in which a tube or inner barrel is advanced ahead of the bit to prevent weak materials from washing away. The spring-mounted inner barrel retracts into the core barrel in direct proportion to the firmness of the material. It works best in clays, silts and fine sandy clay or silt. Gravel and clean sand are hard to recover using a Pitcher barrel. Granular materials lack the binder necessary to create friction against the walls of the sampler. Since the sampler has no catcher to retain the sample, gravel and clean sand are especially hard to recover below the water table. In coarse gravel the tube tends to get mangled, which can cause mechanical problems in addition to poor sample recovery.

Shelby tube: this method utilizes a push technique. The sample tube is the same type used in the pitcher barrel sampler. Hydraulic force advances the tube into the soil. Ideally the push is a steady continuous pressure. Shelby tubes are most effective in soft soils, primarily clay silt and bay mud. This method, in the proper soils, yields the most nearly "undisturbed samples." If soils are too firm the tube will bend. This method is useless in sand or gravel.

Drive sampler: a steel-walled cylinder driven by a hammer or slip jars. "Undisturbed samples" are recovered from a sampler containing rings or tubes as liners. Blows of the hammer or jars frequently cause minor disturbance of the outside edge of the sample. A surface hammer or a downhole hammer may be used. A downhole hammer is more efficient at sample depths greater than 20 to 30 feet and weighs approximately 300 pounds. A drive sampler is used in granular material and is the best method for sampling clean sand and gravel. A catcher may be installed at the bottom of the sampler just inside the drive shoe to prevent the sample from falling out. Spring and drop leaf catchers are most commonly used.

Standard Penetration Test (SPT): a drive method used for density determinations. The sampler has an unlined split barrel two inches in outside diameter. It is driven by a 140 pound hammer with a 30 inch free fall on drill pipe stiffer than A-rod (1-5/8 inches in diameter). The sampler should be driven 18 inches. The number of blows required for each six inch increment is recorded. Samples recovered are disturbed but may be examined for a thorough field description and kept for grain size analysis. SPT's should be taken in all granular materials, particularly those anticipated to be susceptible to liquefaction.

Core Barrels and Bits

Core samples of fresh bedrock are collected for detailed lithologic description and strength testing. Coring rock can be complicated. Overburden must be cased off or grouted prior to coring. To drill without obstructing the core barrel, clear water or thin drilling fluids must be used. Uncased boreholes may cave when the drilling fluid is thinned.

The basic types of core barrels used are conventional and wire-line. A conventional barrel fits on the end of a standard drill string. With this equipment the whole string must be removed to retrieve each core run. A wire-line barrel is attached to a special rod and contains an inner barrel that can be removed by lowering a snatching device on the end of a cable. The drill string remains in place while the sample is removed. This allows more complete core recovery and faster progress than with a conventional core barrel. Wire-line is a must for holes deeper than about 75 to 100 feet. A split inner barrel can be installed in either type of barrel to facilitate recovery of soft formations. The inner barrel, commonly called a triple-tube, is used with a smaller diameter drill bit and is usually extruded from the core barrel with pump pressure.

The nature of rock can be extremely variable both from one borehole to another and within a single borehole. Many types of drill bits are used for different formations. The old standby is a bottom discharge bit with surface set diamonds. Selection of the proper bit allows faster penetration and more complete recovery. Bits vary in size, composition, fluid discharge portal location, and size and distribution of abrasive material.

Side discharge bits are used in soft granular formations that wash out easily. Face discharge bits are used in material that tends to wash out but has a little more binder. Face and side discharge bits are generally used with a split inner barrel. Most carbide bits are the face discharge type. They are used in fault gouge and deeply weathered rock that lacks hard layers and concretions. Bottom discharge bits are the type commonly used for coring. They are used in competent rock of fairly consistent hardness and strength without layers or blocks of very soft material.

Diamonds are used in most coring bits to improve penetration and to lengthen the life of the bit. There are two principal types of diamond coring bits: surface set and impregnated bits. Different bits of each type are designed for use in hard, moderate, or soft formations. Surface set bits have industrial diamonds attached to the cutting surface. Most surface set bits meant for hard formations have small diamonds, and soft formation bits have larger diamonds. The larger diamonds prevent the bit from "gumming up" in soft material while allowing somewhat harder zones also to be penetrated. Impregnated bits have diamond chips embedded throughout a metallic matrix. As the bit wears, new diamonds are exposed. For a soft formation or coarse grained rock, an impregnated bit with a harder matrix is used. A bit with a relatively soft matrix works better in hard formations since more diamonds are exposed for cutting. Impregnated bits are generally superior to surface set bits for drilling core in very hard formations. They cut faster because the diamond chips are not flattened or knocked out of the matrix as easily as those in surface set bits.

CONCLUSION

There are so many technical and logistical considerations involved in running a geotechnical subsurface exploration program that careful planning is just as important as a working knowledge of drilling and sampling methods and of the local geology. An exploration program is more likely to remain within its time and budget constraints and to supply the best possible data when each aspect of the program is planned and coordinated long before field work begins. Exploration along urban transportation corridors requires the geologist to operate in a variety of geological and political settings. Technical know-how is integrated with attention to logistics in a successful exploration program.

ACKNOWLEDGEMENTS

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EARTHQUAKE GROUND RESPONSE STUDY FOR THE CENTURY FREEWAY, LOS ANGELES, CA

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ABSTRACT

The Century Freeway alignment, located in the Los Angeles basin, traverses an active fault zone and will require 200 structures to complete. This paper presents the method used to predict seismic ground surface motions for structure design.

The Los Angeles basin, the site of near continuous deposition since the Late Cretaceous, contains from 5000 to 30,000 feet of sediment over the basement rock. The surface deposits traversed along the alignment include Upper Pleistocene dune sand, Upper Pleistocene marine and alluvial deposits, and Recent alluvial deposits.

A study of the seismicity of the Southern California Region indicates that there are 11 active faults which could affect the project. The maximum credible earthquake for a given fault is chosen as the design earthquake.

The alignment is divided into six segments, each centered on a site of major structures. For each of the segments, a shear wave velocity profile was determined using downhole shear wave velocity measurements. Since the bedrock motions vary with magnitude and distance from the source, four different design earthquakes were used to give the range of bedrock motions expected at the site. Recorded accelerograms, one dimensional shear wave propagation theory, equivalent linear soil properties and the SHAKE computer program, are used to develop an Acceleration Response Spectra (ARS) curve for each of the six segments along the alignment. After a review of the analysis procedure, it is concluded that the ARS curves are conservative based on present knowledge, however, the main uncertainties are "near-field" ground motions and predictions of maximum credible earthquake magnitudes.

OUTLINE OF CONTENTS

INTRODUCTION

The Century Freeway will be located in the Los Angeles Basin and will extend for 18 miles (see Figure 1). The freeway will be located predominantly on embankment with an occasional depressed section. Small bridges will be used at locations of through cross streets. Major bridge structures will be required at four freeway to freeway interchanges. At these locations, tall (40 to 60 foot) single column bent connector ramp structures are among the bridges planned. In addition, structures will be required at two viaduct sections (1000 and 400 feet in length) and the two major river crossings along the alignment. A total of 200 bridges are planned. The freeway will be eight lanes wide, three lanes for traffic and one lane for transit and high occupancy vehicles in each direction. A 64 foot wide median is planned for separate transit facilities.

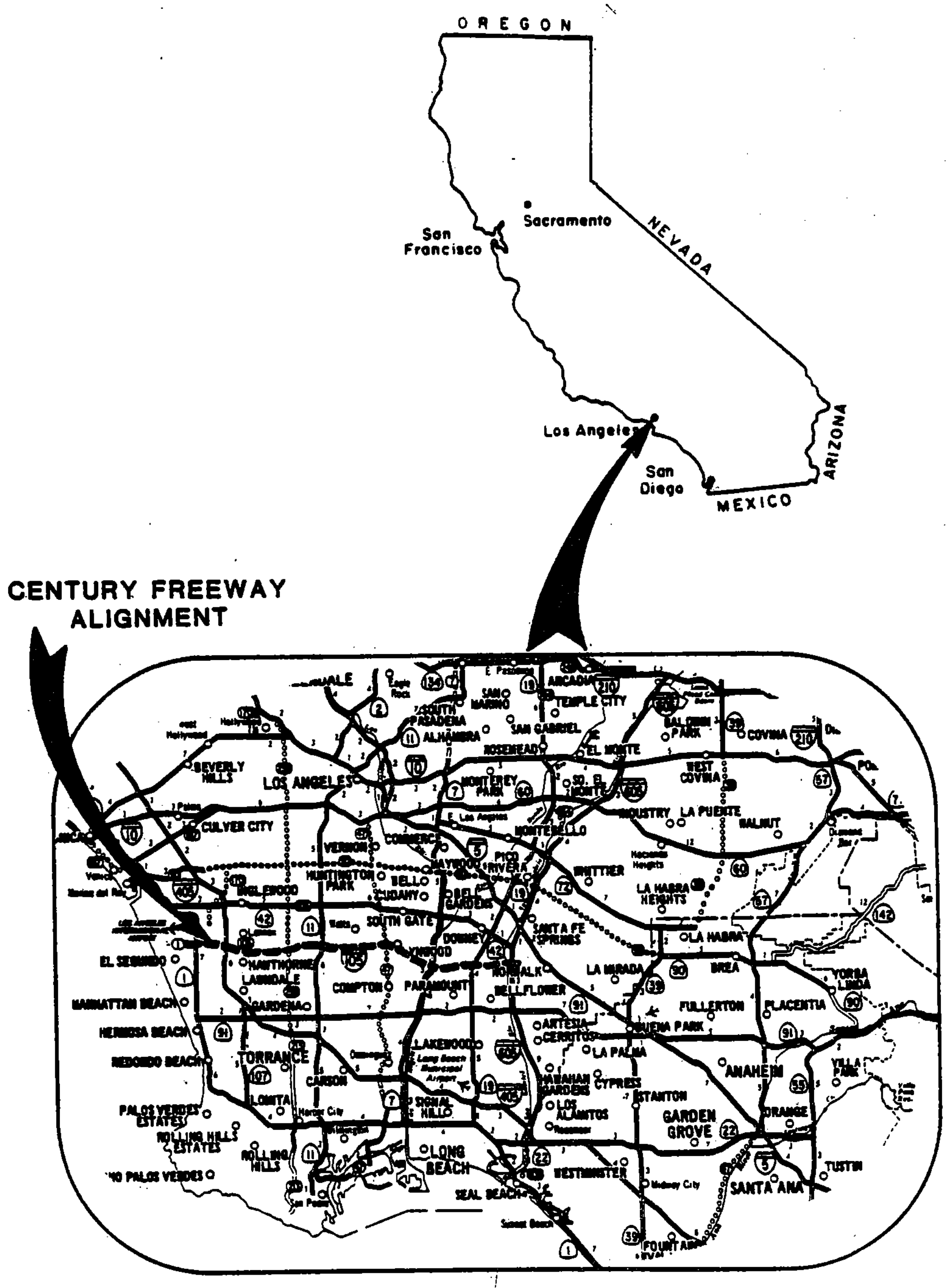


Fig. 1. PROJECT LOCATION

The project cost is estimated to be 1.5 billion dollars. Of this, $450 million is for structures and $600 million for right-of-way and housing relocation costs. The projected completion date is late 1992. (Gates [1984]; and District 7 Materials Section [1974])

Due to the enormous cost of the structures, the importance of the transportation corridor and the complex seismicity of the Los Angeles area, a site specific ground response study for structure design was deemed necessary. The study considers only seismic ground response for structures design, other hazards such as liquefaction, etc., will require a separate investigation.

GEOLOGY

The L.A. basin is underlain by a deep structural depression. The geologic structure is described in terms of four structural blocks which have contrasting basement rocks. The margins of each block have been zones of faulting and flexure since at least Middle Miocene time. The boundaries of the structural blocks are shown in Figure 2.

Due to the location of the alignment near the central portion of the basin and the basin structure, the earthquake ground response can be predicted considering only the local geology, that is, the upper 200 to 300 feet of the deposit. The following discussion briefly describes the basement rock, basin formation and surficial deposits found along the freeway alignment. For a more complete description of the geology of the L.A. basin see Yerkes, et al. (1974) and Jahns (1954).

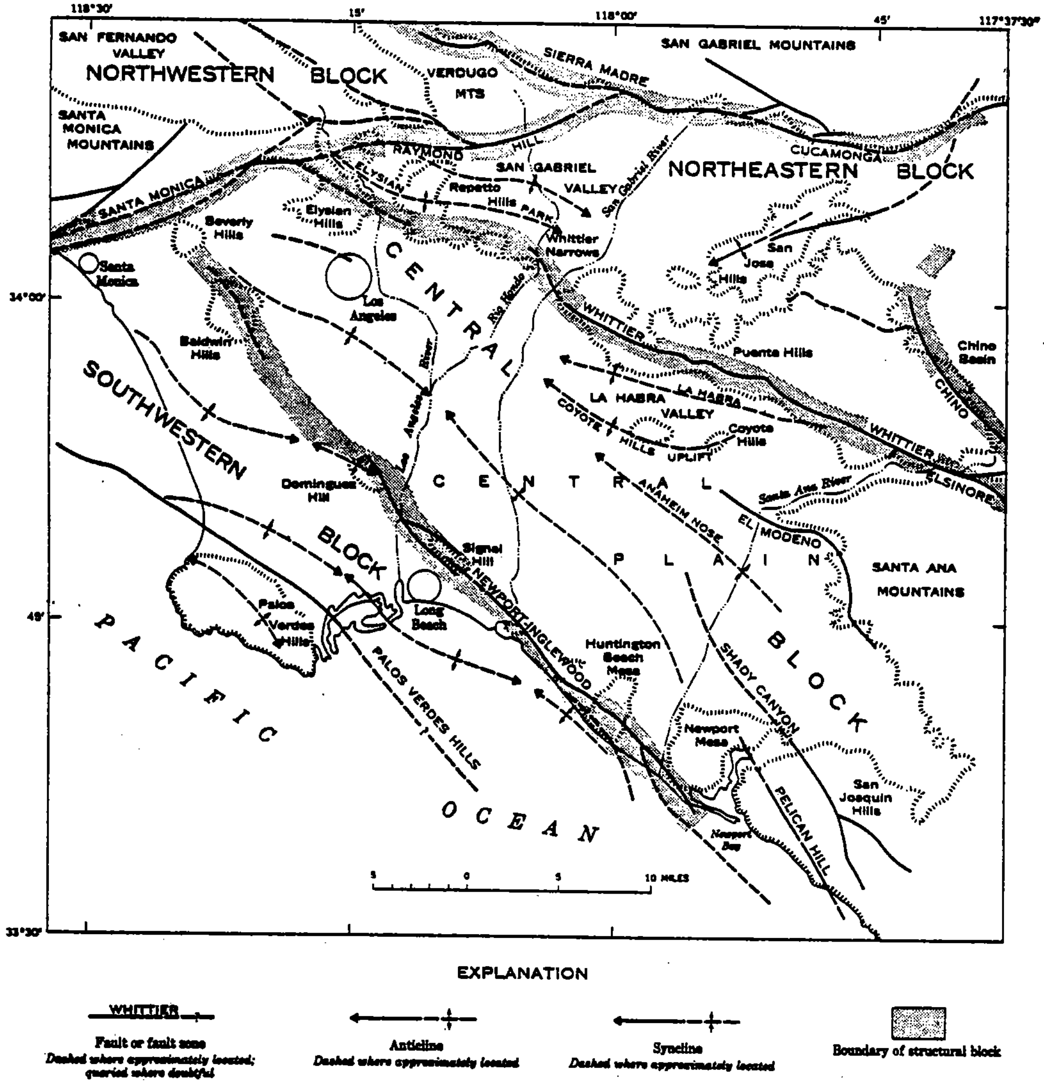


Fig. 2. PHYSIOGRAPHIC FEATURES AND MAJOR STRUCTURAL FEATURES ON THE BASEMENT SURFACE IN THE LOS ANGELES BASIN (From Yerkes et. al., 1965)

The basement rocks underlying the L.A. basin are of two distinct types. Underlying the Southwest Block is the Catalina Schist (green chlorite and blue glaucophane schists). Neither the age nor stratigraphic position of these rocks are known. The basement rocks of the Central Block are questionably described as slightly metamorphosed sedimentary rocks of Triassic (?) and Late Jurassic age which have been intruded by Late Cretaceous granitic plutonic rocks of the Southern California Batholith. This is based on exposures in the Santa Ana Mountains and scattered well penetrations along the margins of the block.

The depositional history of the sedimentary rock and deposits overlying the basement rock is complex. Portions of the Central and Southwest Blocks have been sites of discontinuous deposition since Late Cretaceous time, and sites of continuous subsidence and marine deposition from Middle Miocene to Late Pleistocene time. During most of the Pleistocene, the rising anticlines along the Newport-Inglewood Fault Zone probably formed small islands. In the Late Pleistocene, the sea level began to drop exposing the entire coastal plain. The Recent alluvial deposits in the Central Block were formed from the the Los Angeles, Rio Hondo, San Gabriel and Santa Ana Rivers. See Figure 3 for the present day depth to the basement surface of the L.A. basin.

There are three different surficial geologic formations found along the freeway alignment. They are (from west to east, see Figure 8).

1. Upper Pleistocene dune deposits to Station 190±.

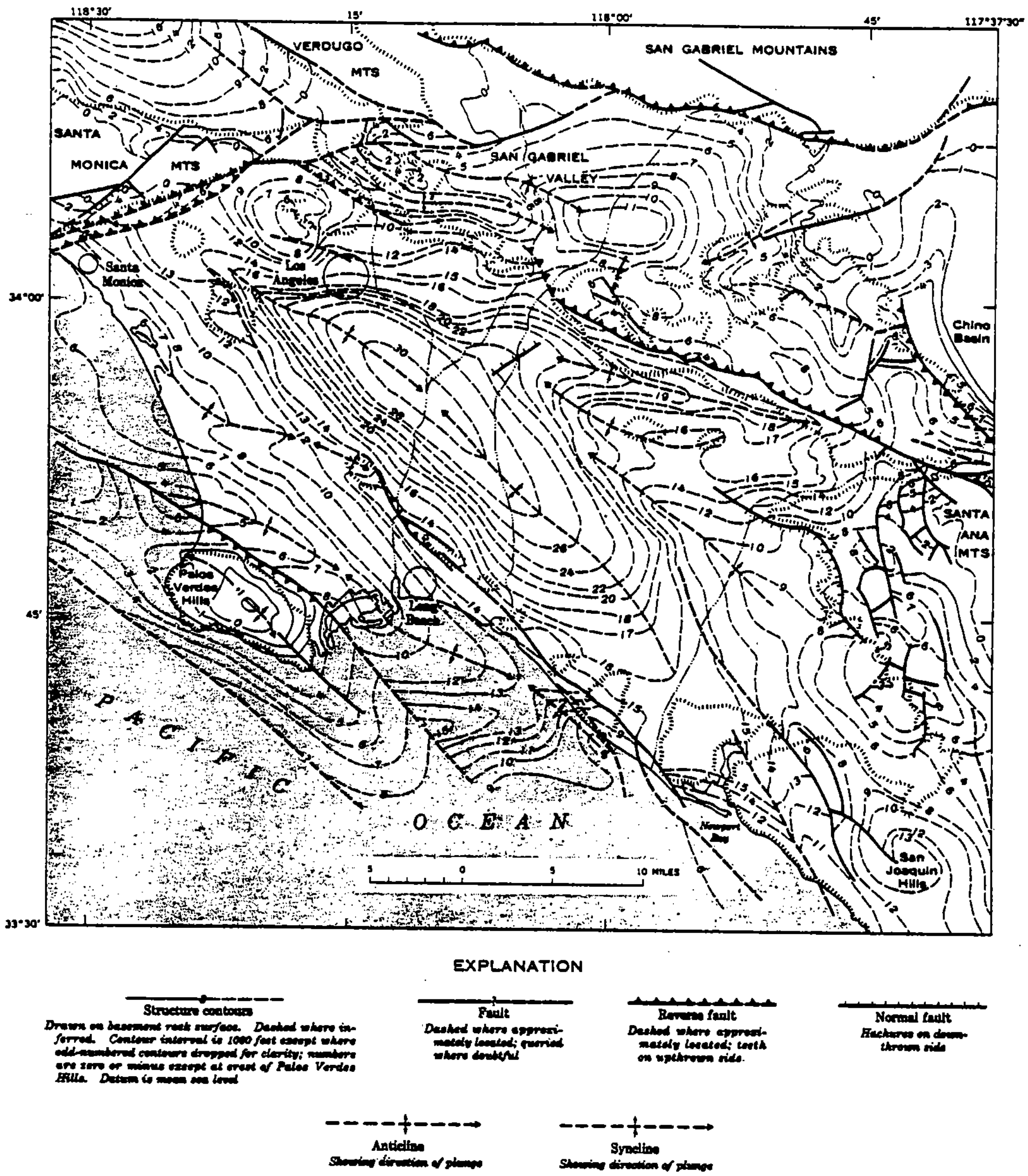


Fig. 3. MAJOR STRUCTURAL FEATURES AND DEPTH TO THE BASEMENT SURFACE IN THE LOS ANGELES BASIN (From Yerkes et al., 1965)

2. Upper Pleistocene marine and alluvial deposits to Station 620$\pm$.

3. Recent alluvial deposits to the end of the project.

The Upper Pleistocene dune deposits are poorly sorted, poorly consolidated, fine grained cohesionless deposits ranging from silt to medium sand. They have a thickness of approximately 100 feet and are underlain by the more consolidated Upper Pleistocene marine and alluvial deposits.

The Upper Pleistocene deposits are consolidated marine and alluvial deposits showing a great variability of materials. In general, the deposit can be described as an 80 to 100 foot cap of alluvial flood plain deposits over shallow marine or littoral deposits. The alluvial flood plain deposits are composed of interbedded sands, silts and clays. At several locations, thick deposits of clay, clayey silt and peat were found. The shallow marine or littoral deposits are composed chiefly of sand with scattered gravel and silt to clay lenses.

The Recent alluvial deposits are poorly consolidated deposits of sands, gravels and silts. Thin scattered layers of clayey silt to clay are also present. Very loose to slightly compact layers are present near the surface becoming denser with depth; however, at many locations, loose to slightly compact layers were encountered at depths to 80 feet.

SEISMICITY

The seismicity of the Southern California Region, like the geology, is very complex. There are 11 active faults which could produce ground motions along the alignment. Only a brief review of the historical seismicity and a discussion of the method used to assess the regional seismicity are presented. For a more thorough discussion of faulting and seismicity in the Southern California Region, see Albee and Smith (1966); and Allen, et al. (1965).

Historic earthquake records in the Southern California Region date back approximately 150 years. Prior to 1932, earthquake magnitudes and epicenters were estimated from published damage reports. In 1932, modern seismographic stations of the Caltech network were established allowing routine and systematic epicentral determinations (Hileman, Allen, and Nordquist, 1973). Figures 4, 5 and 6 show the historical earthquakes which have occurred in the Southern California Region.

Due to the importance of transportation routes in the event of a large scale emergency, the policy of Caltrans Structures Design is to specify the maximum credible earthquake along a particular fault as the design earthquake. The recurrence interval is neglected. The choice of active faults and assignment of maximum credible magnitude is based on work by Greensfelder (1973). All the faults which could affect the project have known or suspected Holocene (last 10,000 years) offset. The maximum credible magnitude was assigned on the basis of length of fault rupture. Table 1 shows a listing of all the active faults which could affect the project. The magnitude of the maximum

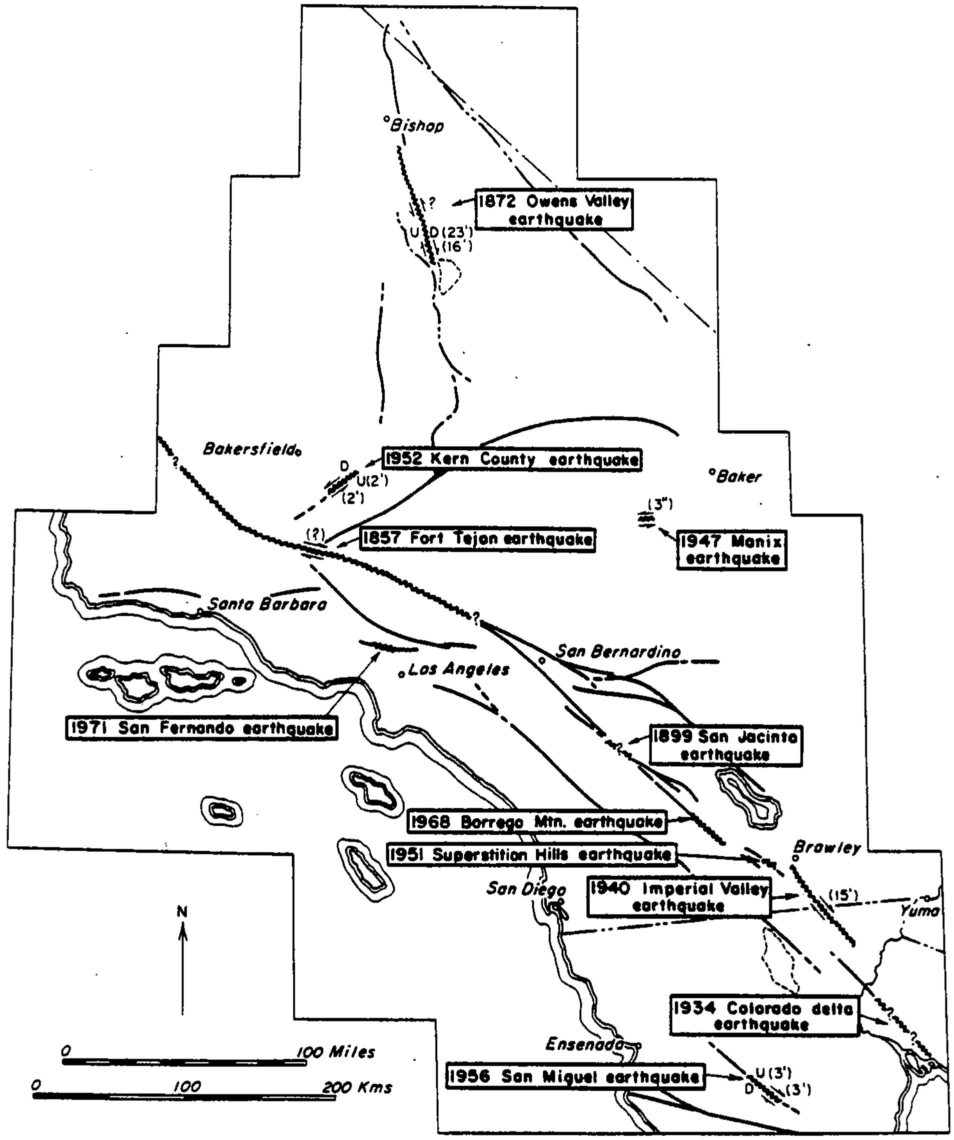


Fig. 4. HISTORIC FAULT BREAKS AND ASSOCIATED EARTHQUAKES IN THE SOUTHERN CALIFORNIA REGION (From Hileman, Allen and Nordquist, 1973)

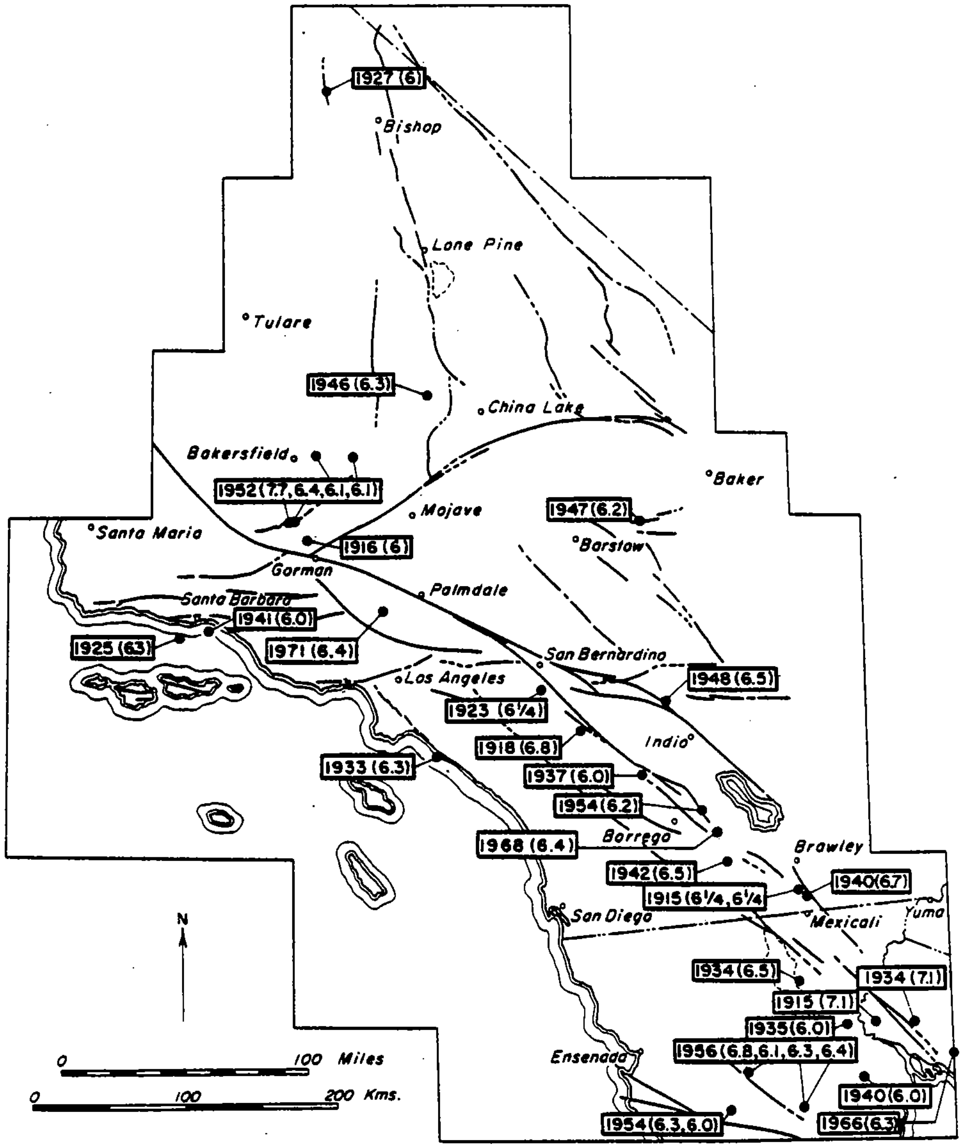


Fig. 5. EARTHQUAKES OF MAGNITUDE 6.0 OR GREATER IN THE SOUTHERN CALIFORNIA REGION, 1912–1972 (From Hileman, Allen and Nordquist, 1973)

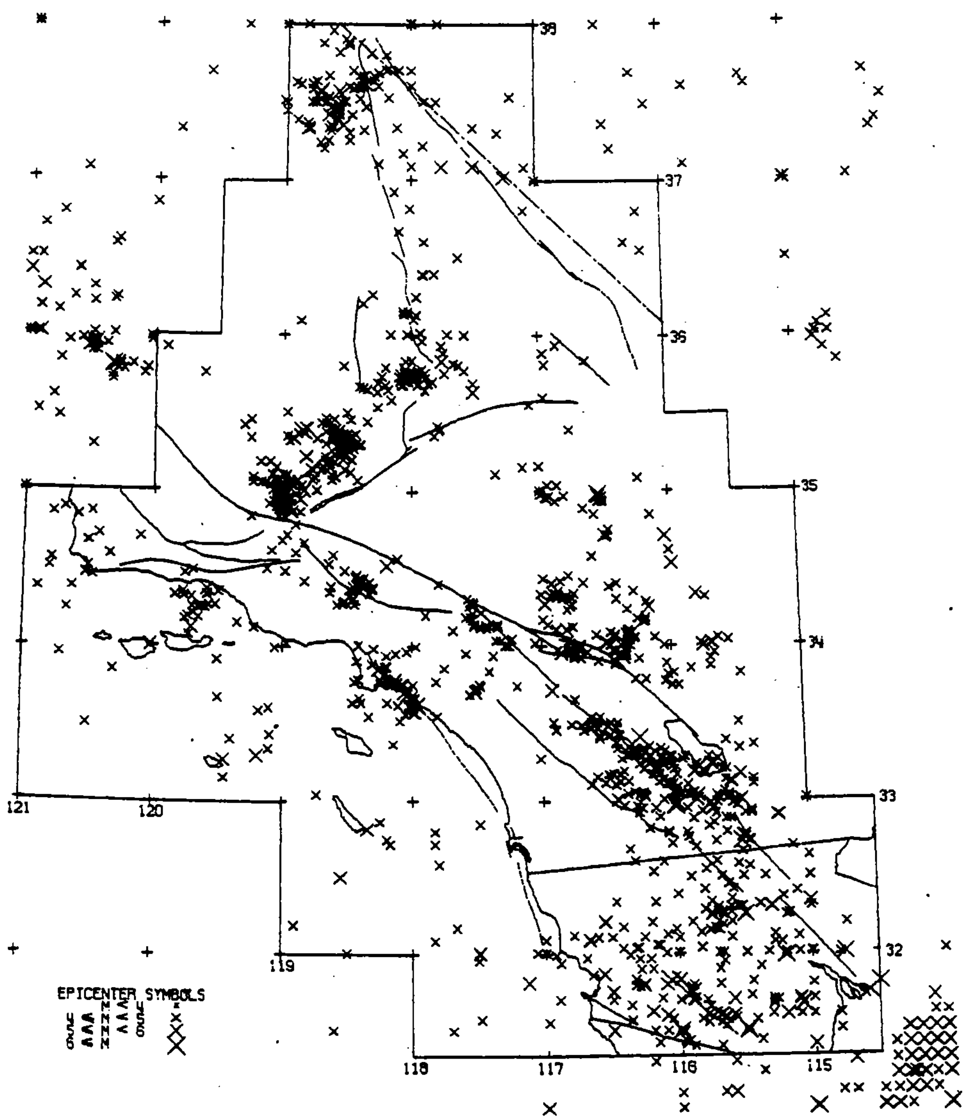


Fig. 6. EARTHQUAKES OF MAGNITUDE 4.0 OR GREATER IN THE SOUTHERN CALIFORNIA REGION, 1932–1972. (From Hileman, Allen and Nordquist, 1973)

credible earthquake for the Newport-Inglewood Fault Zone has been decreased from Greensfelder's original estimate based on recent work by Woodward Clyde Consultants for the San Onofre Nuclear Power Station.

Fault	Distance from Project * (miles) West End	Distance from Project * (miles) East End	Maximum Historical Magnitude (Richter)	Maximum Credible Capability Magnitude (Richter) **	Maximum Credible Capability Duration (Sec)
Cucamonga	40	24	---	6.6	18
Malibu-Santa Monica	8	18	5.2	7.5	30
Newport-Inglewood	3	12.5	6.3	6.5	24 or more
Palos Verdes	8	14	---	7.2	24 or more
Raymond Hill	14	12	---	7.5	30
San Andreas	44	37	8.3	8.3	40 or more
San Fernando	23	23	6.4	6.6	18
Santa Susanna	26	26	---	6.7	18 or more
Sierra Madre	24	18	---	6.6	18
Simi-Northridge	22	22	---	7.5	30
Whittier-Elsinore	14	8	---	7.6	30

* This distance is measured from the indicated end of the project to the nearest point on the fault trace.

Grantz, A. and A. Bartow. 1971. Active Faults of California. United States Department of Interior, Geological Survey.

** Greenfelder, R. W. 1973. A Map of Maximum Expected Bedrock Acceleration from Earthquakes in California. California Division of Mines and Geology, January 1973.

TABLE 1. ACTIVE FAULTS ASSOCIATED WITH THE PROJECT

GROUND RESPONSE INVESTIGATION

General

The purpose of this ground response study was to develop an Acceleration Response Spectra (ARS) for the structural analysis of the bridge structures. This is the maximum acceleration a single degree of freedom structure of varying fundamental period of vibration will attain due to the seismic ground motions. The level of internal damping can be specified that models the actual structural material that will be used. An ARS curve and its evaluation is shown in Figure 7.

In general, there are two methods of developing ARS curves, they are:

1) A statistical analysis of a number of ARS curves developed from ground surface motions recorded at similar sites subject to similar seismic motions.

2) A wave propagation analysis using recorded bedrock motions and the properties of the site materials.

For this study the latter method was used. The approach to the investigation was as follows:

1) The proposed alignment was broken into six segments, each containing a site of major structures. See Figure 8.

2) At each of these sites, a field study was completed to determine the shear wave velocity and unit weight of the various soil strata.

3) Using the data from the seismicity study, bedrock motion characteristics were predicted for the site.

4) Finally, a wave propagation analysis was completed to develop an ARS curve for each site.

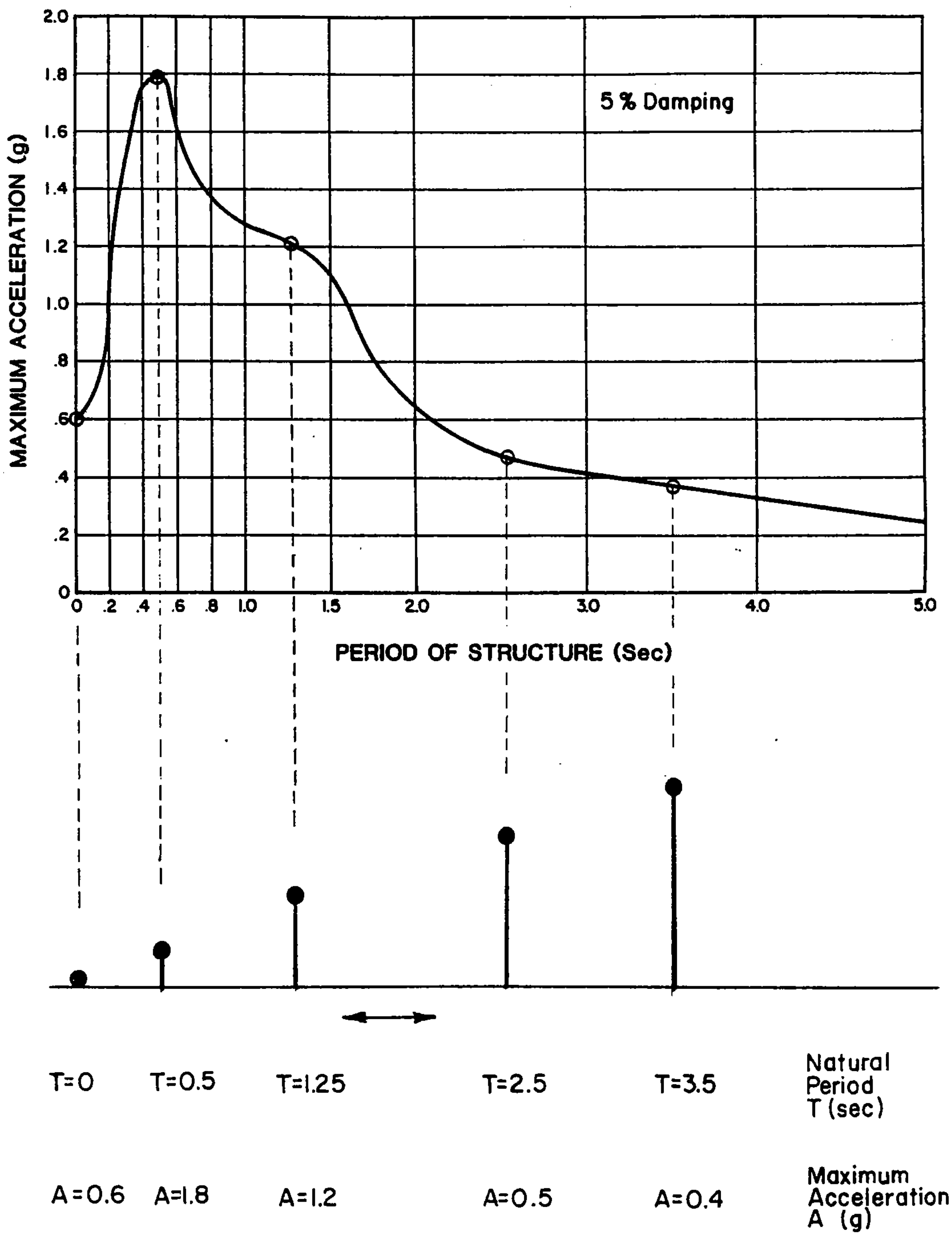


Fig. 7 EVALUATION OF AN ACCELERATION RESPONSE SPECTRA CURVE

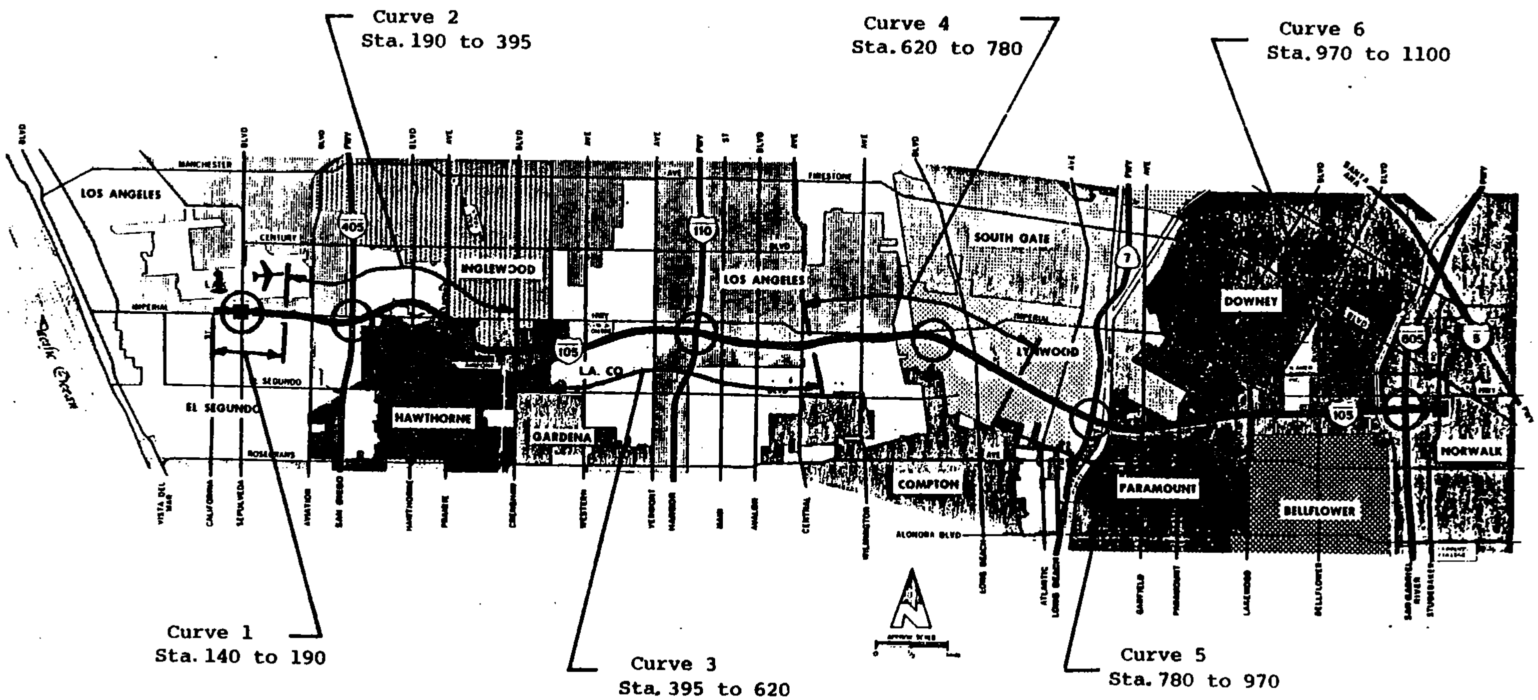


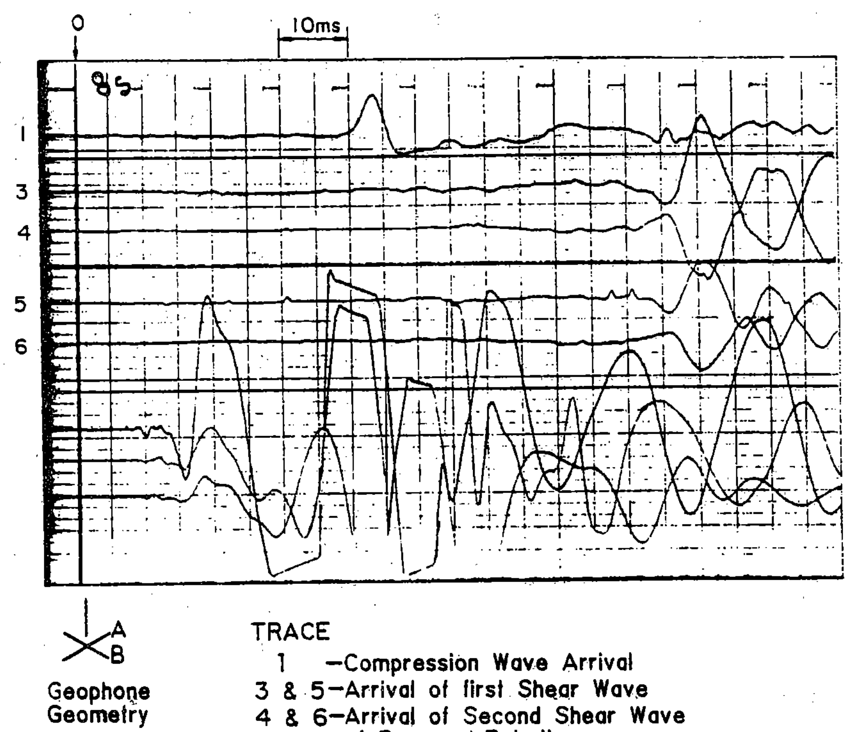
Fig. 8. SITES OF DETAILED FIELD INVESTIGATION AND A.R.S. CURVE STATIONING LIMITS

Shear Wave Velocity Determination

The shear wave velocity study done at each of the six sites used down-hole measurements. One or more holes were drilled at each site and cased. The soil profile was carefully logged during this operation. A three-component geophone was lowered into the hole and wave travel times were recorded from hammer blows at the surface. The interval velocities were then calculated. The method has sufficient resolution to pick out a five to ten foot layer with a velocity contrast of 20 percent. For more information regarding the method or equipment used, see the papers by Mooney (1974), and Beeston and McEvilly (1977).

At each of the selected sites, one to four boreholes were drilled to determine both the soil and shear wave velocity profiles. In all, 13 holes were drilled; three were drilled to depths from 300 to 350 feet to investigate the shear wave velocity at depth, the rest were drilled to a depth of 200 feet. The holes, six inches in diameter, were drilled using rotary drilling methods. A Failing 1500 Holemaster was used for all holes. For some of the boreholes, a small Concore drill rig was used to drill a pilot sampling hole to a depth of 100 feet. The Failing was then used to drill the six inch diameter hole and sample at depths below 100 feet. This method was faster and provided more sensitivity to changes in the soil conditions near the ground surface. The hole was cased with three inch plastic pipe, then gravel-packed. The sampling was done using a 1.4 inch split spoon sampler driven with a 140 pound hammer. A two inch Modified California sampler fitted with brass tubes was also used to determine wet densities of the various soil strata.

The equipment set-up used to record the shear wave travel time is shown in Figure 9. A 12 channel signal enhancing seismograph was used to record the wave arrivals. The three-component geophone (1 vertical, 2 octagonal horizontal) was lowered to the bottom of the hole and the hole clamp released. The wave travel times were then recorded from a series of hammer blows at the ground surface. Wave travel times were measured at five foot intervals up the hole. The shear (S) waves were generated by striking the end of the plank with a wooden hammer, the polarity of the S wave was reversed by striking the opposite end of the plank. The compressional (P) wave was generated by vertically striking a small metal plate on the ground surface. Generally three to five hammer blows were necessary to enhance the recorded arrivals. The wave arrival times were picked to an accuracy of ± 0.5 ms for P waves and ± 1.0 ms for S waves. A typical wave arrival record is shown in Figure 10.



TRACE
1 —Compression Wave Arrival
3 & 5—Arrival of first Shear Wave
4 & 6—Arrival of Second Shear Wave of Reversed Polarity

TYPICAL SEISMOGRAPH RECORD

FIGURE 10

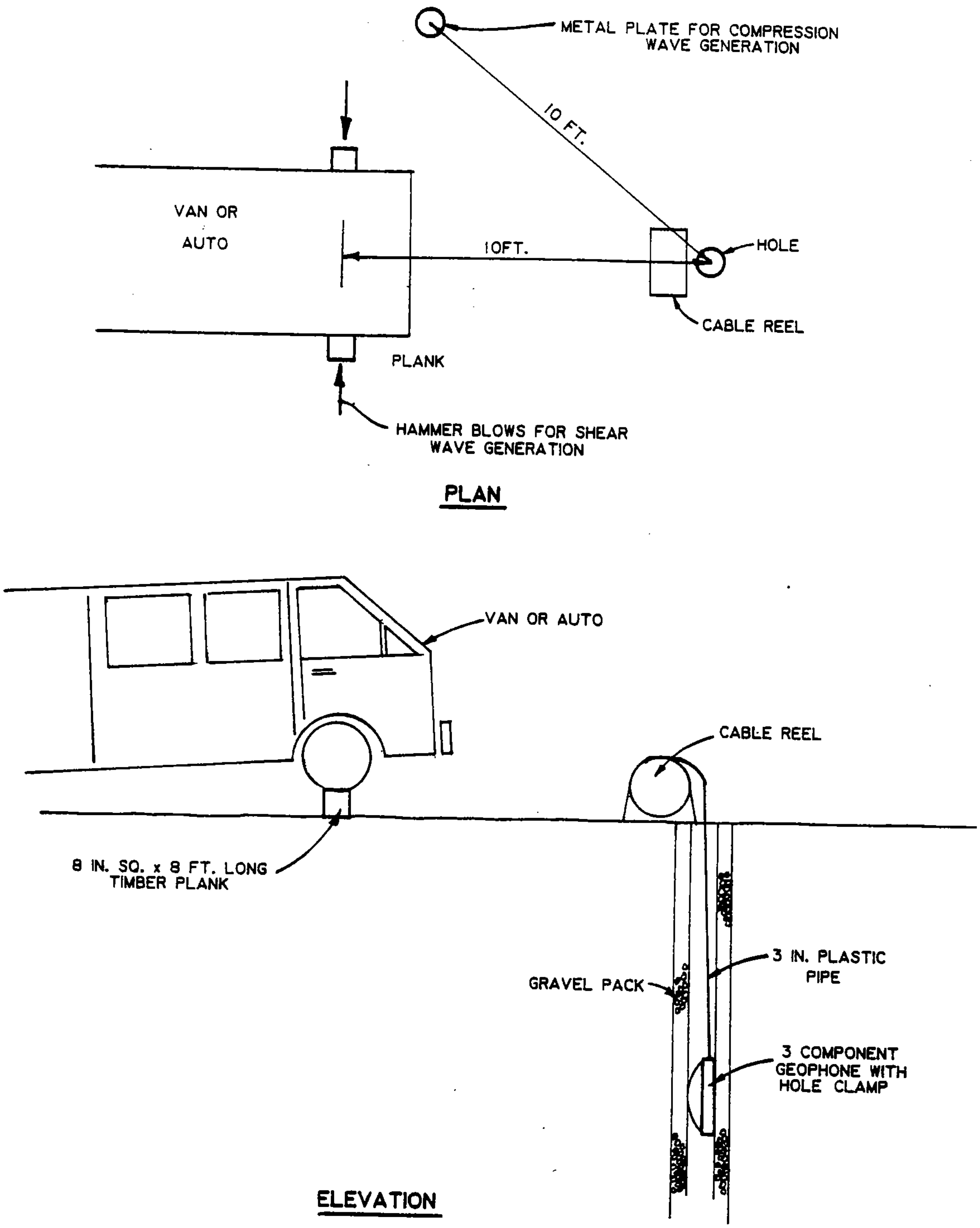


Fig. 9. FIELD MEASUREMENT OF SHEAR WAVE VELOCITY

The equations used in the data analysis are shown in Figure 11. Note that the velocity calculation uses linear regression analysis assuming error only in the arrival time data. Figure 12 shows the results of the completed shear wave velocity study for one of the sites.

The final step in the shear wave velocity study was to check other boring records within the segment to make sure the shear wave velocity profile developed would be representative of the entire segment.

Ground Response Analysis

The ground response analysis combines the results of the shear wave velocity study and the seismicity study to predict ground surface motion due to bedrock motions. The steps in the analysis are as follows:

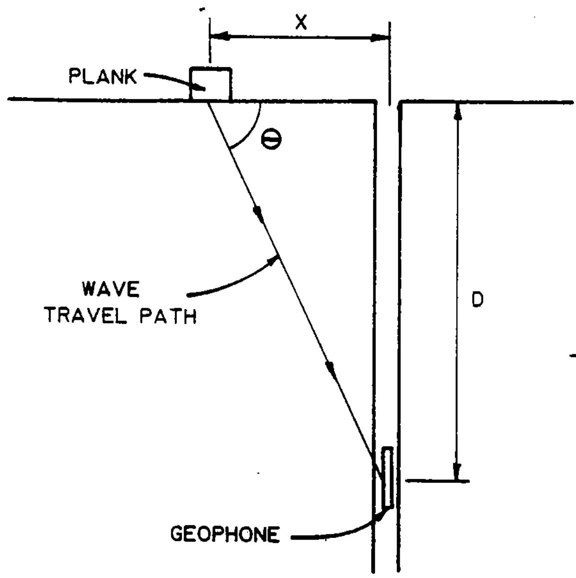
1) The depth to assumed bedrock must be specified.

2) The design earthquake must be chosen and the bedrock motions expected at the site determined.

3) The acceleration time history or accelerogram is then selected and used as the input bedrock motions for the wave propagation analysis.

4) The acceleration response spectrum is then calculated.

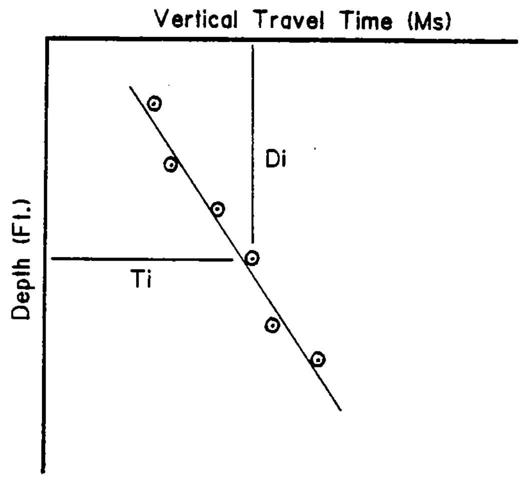
For this study, a depth to bedrock was chosen at 300 feet. This assumes that the sediments below this depth act like a weak rock when subject to seismic motions. The 300 foot depth is supported by shear wave velocities measured in the field investigation (>2000 fps).



$$\Theta = \tan^{-1}(D/X)$$

$$T_{VERTICAL} = T_{MEASURED} \sin\Theta$$

a) CALCULATION OF VERTICAL WAVE TRAVEL TIME



FROM STD. LINEAR REGRESSION ANALYSIS ASSUMING ERROR ONLY IN THE MEASUREMENT OF T.

$$V(ft/sec) = 1000 \frac{\Sigma(Di - \bar{D})^2}{\Sigma(Di-\bar{D})(Ti-\bar{T})}$$

$\bar{D} = \Sigma Di / n$
$\bar{T} = \Sigma Ti / n$

n = NUMBER OF POINTS ALONG A GIVEN VELOCITY INTERVAL.

b) CALCULATION OF INTERVAL VELOCITY

FIGURE 11

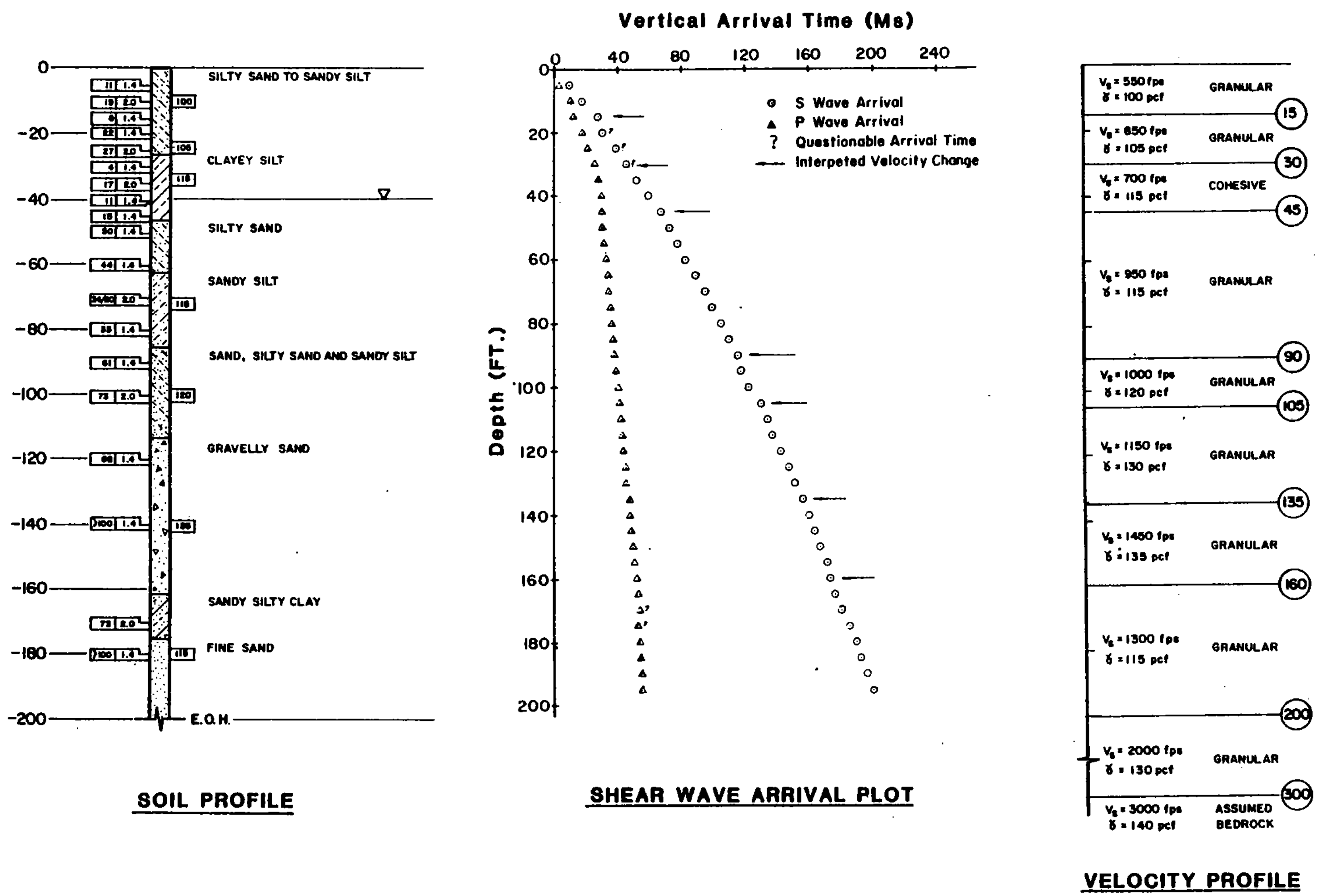


Fig. 12. RESULTS OF THE SHEAR WAVE VELOCITY STUDY

A large number of faults are located near the project which could cause significant ground motions at the site. In general, the ground motions which can be expected at the site are related to the magnitude of the earthquake and the distance from the zone of energy release. An earthquake located only a short distance from the site will cause high frequency, intense ground motions affecting short, stiff structures. A more distant earthquake will cause low frequency, low intensity motions which will affect tall flexable structures. Thus, a wide range of ground motions could occur at the site depending on the causative fault. For this study the concept of an "envelope" ARS curve was used in that ARS curves were developed for both local and distant earthquakes then combined to give an envelope of the ground motions which could be expected at the site.

Illustrating the envelope concept, four different earthquake events were identified which envelope the range of ground motions which could be expected at site 2. They are:

Earthquake Location	Maximum Credible Magnitude	Distance (mi.)
San Andreas	8.3	40
Whittier-Elsinore	7.5	15
Palos Verdes	7.0	8
Newport Inglewood	6.5	2.5

The bedrock motion characteristics which must be specified for the analysis are the predominant period of the motion and the maximum acceleration. By knowing the magnitude of the earthquake and distance of the zone of energy release from the site, the predominant period and maximum acceleration can be estimated using charts developed by Seed, Idriss and Kiefer (1969) and Schnabel and Seed (1972). The charts were developed by analyzing a large number of earthquake accelerograms recorded in the western United States (see Figure 13).

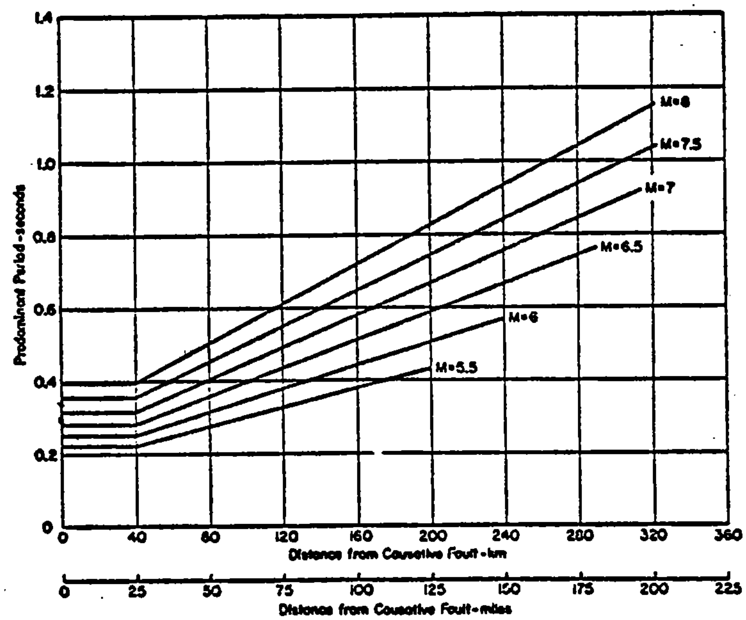
For the analysis, an accelerogram must be used for the input of bedrock motions. Usually the predominant period and maximum acceleration of the accelerogram are scaled up or down slightly to match that expected at the site. At the present, a large number of accelerograms have been recorded and are available for use. The accelerogram should meet these criteria:

1) The magnitude of the recorded earthquake and distance from the causative fault should be very close to that expected at the study site.

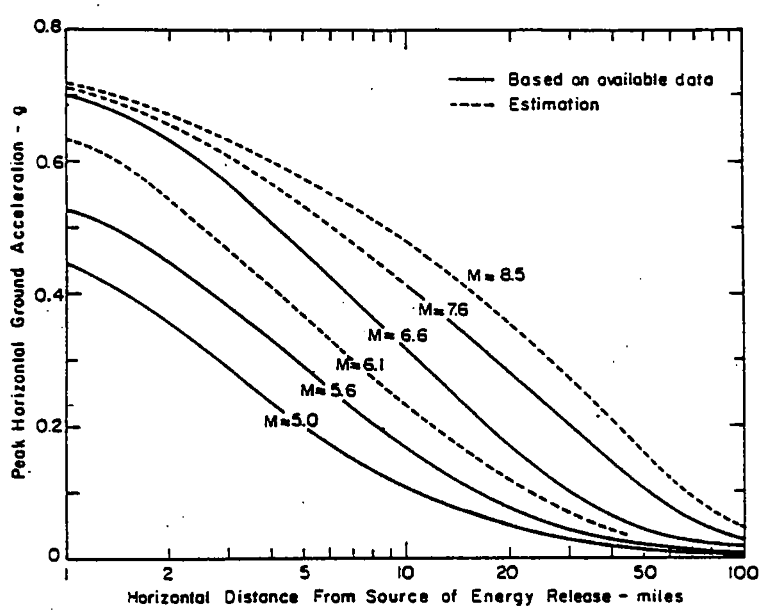
2) The accelerogram must be recorded on bedrock or modified if recorded on soil to reflect bedrock motions.

3) It should have been recorded in the same general region as the study site and ideally, the type of faulting should match that expected for the fault which will cause ground motions at the site.

4) The maximum acceleration and predominant period of the recorded motion should be very close to that expected at the study site.



a) Predominant Periods for Maximum Accelerations in Rock (From Seed, Idriss and Kiefer, 1969)



b) Average Values of Maximum Accelerations in Rock (From Schnabel and Seed, 1972)

FIGURE 13

Finally, knowing the soil and shear wave velocity profiles and having selected an input bedrock motion accelerogram, the response of the soil deposit to the input bedrock motions is determined using a wave propagation analysis. For the analysis, only vertically propagating horizontal shear waves are considered. The response of the soil deposit to the input bedrock motions was analyzed using the computer program, SHAKE (Schnabel, Lysmer and Seed, 1972) which uses one dimensional wave propagation theory and equivalent linear soil properties.

To simplify the analysis, a study was conducted to compare the envelope ARS curve considering the four earthquake sources with an ARS curve derived using only an event on the Newport-Inglewood Fault. Two accelerograms were used to produce the ARS curve which considered only an event on the Newport-Inglewood Fault; 1) an accelerogram recorded during the El Centro Earthquake in 1940 and (2) an accelerogram artificially generated by Romstad, Bruce and Hutchinson (1978). The El Centro accelerogram was chosen because the magnitude, distance from the fault and fault type were similar to that expected from an event on the Newport-Inglewood Fault Zone. The Romstad accelerogram was chosen because it has a smooth response spectra in terms of frequency content and distribution. The comparison of the ARS curves is shown in Figure 14. It is evident that the ARS curve derived using rock motions for a maximum credible event on the Newport-Inglewood Fault Zone is a conservative envelope of the response spectra for the range of rock motions expected at the site. Thus, for the ground motion study, ARS curves were generated for each site using the El Centro and Romstad accelerograms as the input bedrock motion.

A.R.S. CURVE 2
M=6.5 Dist =2.5 mi.
Newport-Inglewood

Accelerograms:
El Centro (1940)
A=0.6g T=0.3sec

Romstad
A=0.6g T=0.3sec

COMPOSITE A.R.S. CURVE
M=8+ Dist =40 mi
San Andreas

Accelerogram - Seed 8+
A=0.2g T=0.5sec
....
M=7.5 Dist =15 mi
Whittier-Elsinore

Accelerogram - Taft
A=0.4g T=0.35sec
....
M=7.0 Dist =8 mi
Palos verdes

Accelerogram - El Centro (1940)
A=0.45g T=0.35sec
....
M=6.5 Dist =2.5 mi
Newport-Inglewood

Accelerogram - El Centro (1940)
A=0.6g T=0.3sec

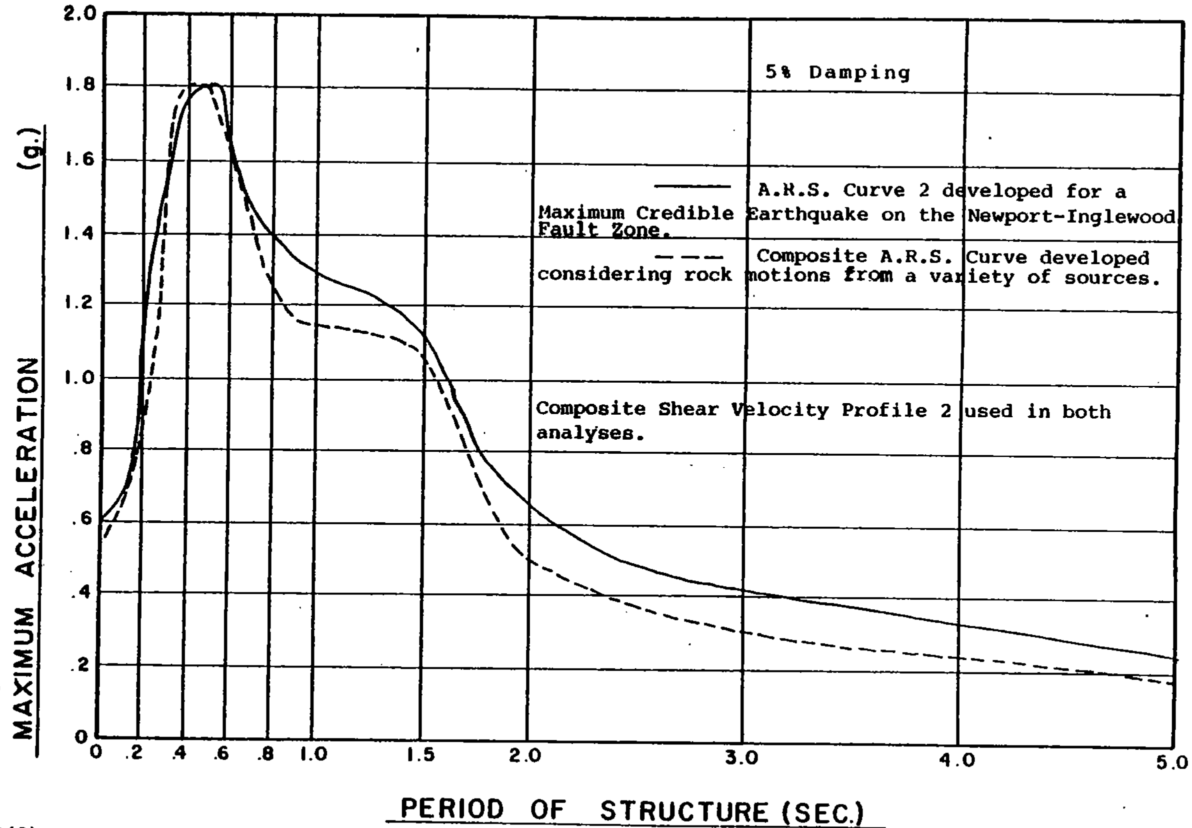


Fig. 14. COMPARISON OF A.R.S. CURVE 2 DEVELOPED FOR A MAXIMUM CREDIBLE EVENT ON THE NEWPORT-INGLEWOOD FAULT ZONE AND A COMPOSITE A.R.S. CURVE DEVELOPED CONSIDERING A VARIETY OF POSSIBLE SOURCES

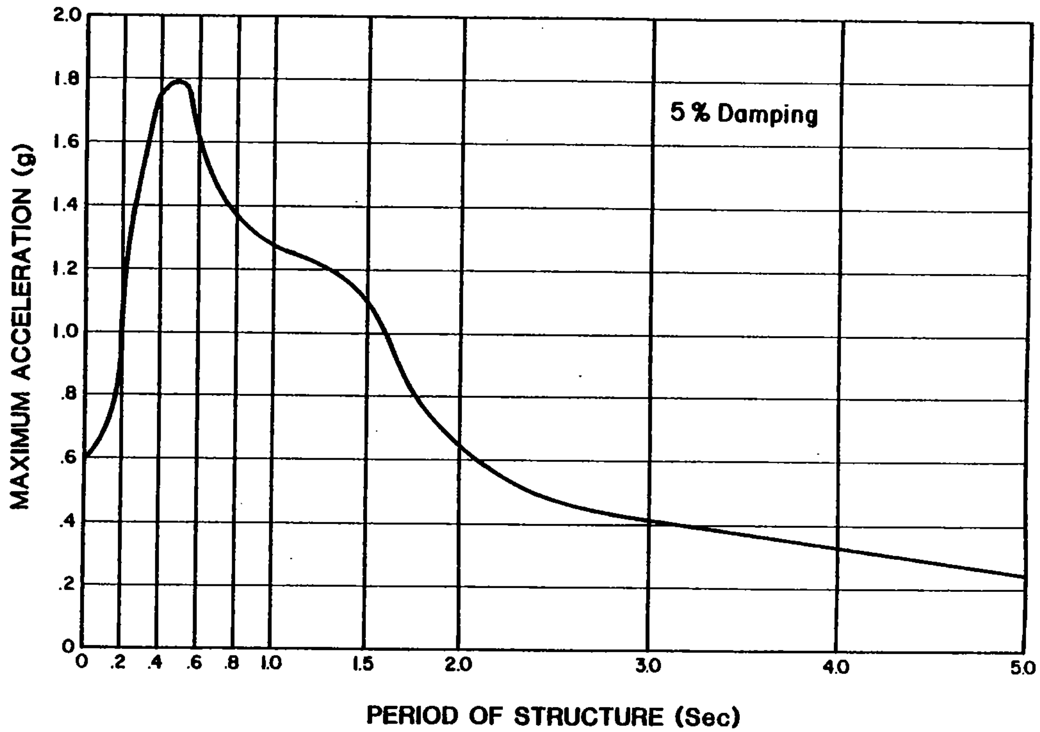
EARTHQUAKE DATA

MAXIMUM CREDIBLE EARTHQUAKE:

FAULT: NEWPORT-INGLEWOOD
MAG.: 6.5
DIST.[1]: 2.5 MI.
PRED. PERIOD[2]: 0.3 SEC. (ROCK)
MAX. ACCEL.[3]: 0.6 G. (ROCK)
FAULT TYPE: REVERSE IN PROJECT AREA
STRIKE SLIP ELSEWHERE

DIGITIZED ACCELEROGRAMS USED IN THE ANALYSIS

1. EARTHQUAKE: EL CENTRO (1940)
   MAG.: 6.5
   DIST.: 6.0 MI.
   PRED. PERIOD: 0.2 SEC.
   MAX. ACCEL.: 0.3 G.
   RECORDING SITE GEOL.: DEEP ALLUVIUM[4]
   FAULT TYPE: RIGHT LATERAL STRIKE SLIP
2. EARTHQUAKE: ROMSTAD[5] (ARTIFICIAL)
   PRED. PERIOD: 0.2 SEC.
   MAX. ACCEL.: 1.0 G.



NOTES:

1. DISTANCE TO CLOSEST POINT ON FAULT TRACE
2. (SEED, IDRISS AND KIEFER, 1969)
3. (SCHNABEL AND SEED, 1972)
4. DECONVOLUTED TO APPROXIMATE ROCK MOTIONS BY SEED (FROM SHANNON AND WILSON)
5. (ROMSTAD, BRUCE AND HUTCHINSON, 1978)

PROJECT ENGINEER:				DRAWN BY:		TRANSPORTATION LABORATORY State of CALIFORNIA DEPARTMENT OF TRANSPORTATION	CENTURY FREEWAY EARTHQUAKE GROUND RESPONSE STUDY	SCALE:
M. Sorensen				Leony L. Lopez 6/83			A. R. S. CURVE No. 2	SHT. 3 OF 3
DIST.	COUNTY	RTE.	STATION	CHARGE DIST.	EXPENDITURE AUTHORIZATION			
07	LA	105	190 TO 395	07	053201			

Fig. 15. FINISHED A.R.S. CURVE

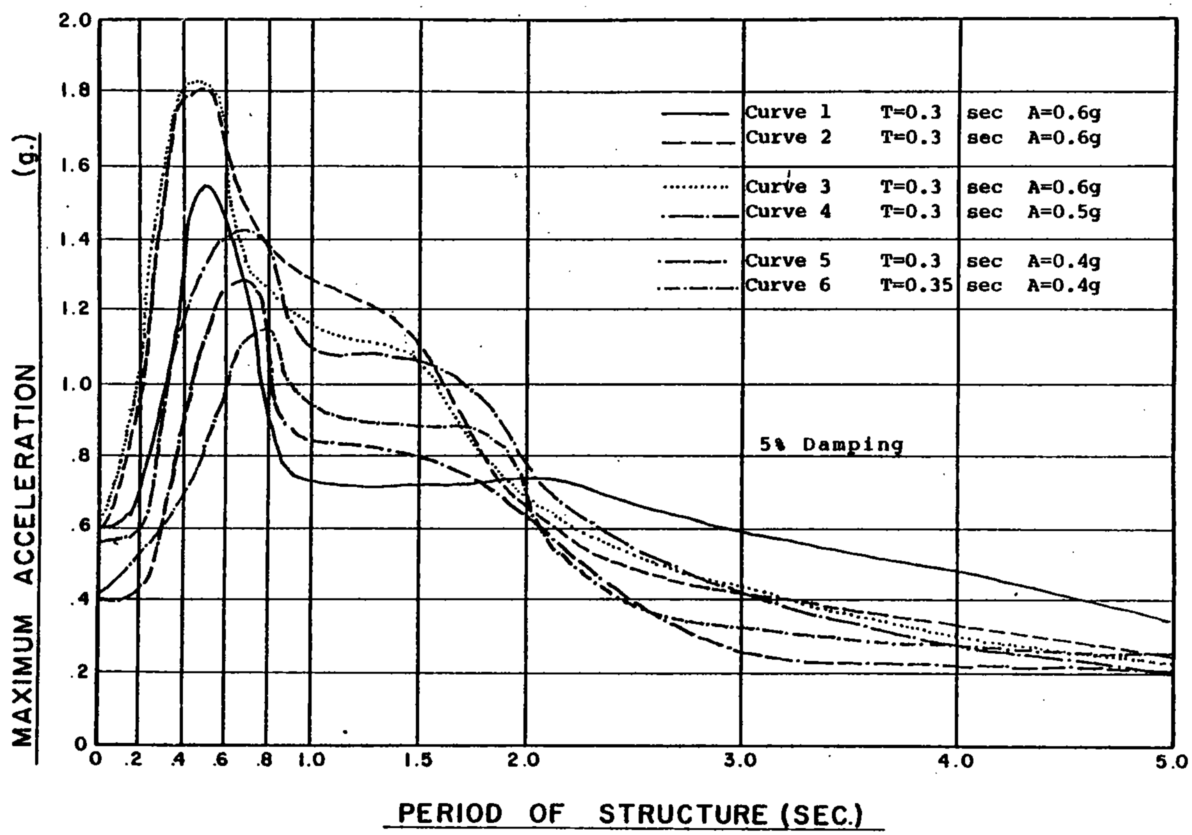


Fig. 16. COMPARISON OF ALL A.R.S. CURVES DEVELOPED FOR THE PROJECT

A sensitivity study was conducted to determine the effect on the ARS curve of the use of 600 feet as the depth to bedrock. The results showed only a slight variation. Since the ARS curve for a depth of 300 feet to bedrock would be more conservative in the natural period range of the structures for this project (<1.0 sec), this depth was selected for use in the analysis.

An ARS curve developed for one of the study sites is shown in Figure 15. Figure 16 shows a comparison of all the ARS curves developed for the project.

DISCUSSION OF THE RESULTS

It must be remembered that the prediction of earthquake induced ground motions is not an exacting science. Any analysis must be reviewed for possible sources of error and degree of conservativeness. In reviewing this analysis, it is expected that the shear wave velocity profiles are fairly accurate. The ground response analysis has been shown to produce excellent results if the proper bedrock motions are input. Thus, it appears the major uncertainty is the bedrock motions used in the wave propagation analysis.

For this project, an active fault zone is traversed and two sites of major structures are located within three miles of the fault zone. Bedrock motions within three miles of a fault are termed "near-field" motions. Unfortunately very little data has been recorded for near-field motions. At this distance, the faulting mechanism might have a great influence on the ground motions. There is also evidence to suggest that vertical motions may even exceed horizontal motions (Shakal and Bernreuter, 1981). Only time will solve these uncertainties, since a number of near-field events must be recorded before reliable correlations of characteristic ground motions can be made.

Another uncertainty is the maximum credible earthquake magnitude assigned to a particular fault. At the present, there is considerable debate as to the actual maximum credible earthquake which the Newport-Inglewood Fault Zone is capable of. The use of a magnitude equal to 6.5 is assumed to be conservative, however, as with near-field motions, only time will tell.

Thus, the analysis is considered conservative based on present knowledge. Further safety factors are included in the structural design. However, if time does show this analysis is not conservative, the structures can be modified (as unpalatable as it seems) at that time in the light of the new information.

ACKNOWLEDGEMENTS

A number of individuals deserve credit for their part in this study. I would like to thank:

Marv McCauley - for his help in shaping the study scope, objectives and methodologies.

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Ralph Fitzpatrick - for his help with the shear wave velocity study and a finicky seismograph.

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This paper was reviewed by Marv McCauley and Duane Smith; their comments and suggestions were greatly appreciated.

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FINAL RESULTS OF EMBANKMENT PERFORMANCE AT DUMBARTON

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San Francisco, CA

The results of embankment performance, settlements and strength gain of the bay mud foundation soils at Dumbarton are reported. Special construction features were utilized that allowed embankment placement to proceed on schedule without major foundation failures. Reinforcing fabric, lightweight fill (sawdust) and vertical "wick" drains composed the system of special features which led to successful embankment placement for a test fill and construction of embankments on three separate contracts. Construction was required over open water and soft bay mud soils with depths varying from 10 to over 40 feet. The reinforcing fabric provided initial support over the soft bay mud; lightweight fill reduced loading to assure ultimate stability; and vertical wick drains accelerated foundation consolidation. Instrumentation was installed to monitor performance of the embankment and foundation system during embankment placement. Four different commercial wick drains were installed to consolidate the soft compressible bay mud foundation. Field performance of each of these drains is compared. A laboratory test procedure was also utilized to evaluate wick drains. The laboratory test proved valuable in predicting field performance. Up to seven feet of settlement occurred in the bay

mud foundations over periods of one to three years. Considerable strength gain was also achieved.

COMPILERS' NOTE: This abstract was originally prepared for a paper presented at the Highway Geology Symposium. As no proceedings contribution was received, the abstract is reprinted here.

35th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM FIELD TRIP

The preface, table of contents, and field trip route map are included in this Proceedings Volume so that the reader can know and understand the theme of the field trip, the nature of the stops, the general route, and the trip leaders involved. The entire field trip guide has not been included because of its length.

PREFACE TO FIELD TRIP

This field trip which is an integral portion of the 1984 35th Annual Highway Geology Symposium has two primary functions. The first is to afford the participant the opportunity to get a general overview and appreciation for the geologic setting of the central California coastal area with its great diversity in geology and topography. This will introduce some of the challenges that the engineering geologist and engineer face when attempting to design and maintain a transportation system in the region. The second function of the trip is to permit the study of specific sites where geotechnical problems have been encountered. These stops are designed to present information on the history of a particular problem, its potential hazard, and the corrective techniques that have been undertaken to reduce the detrimental impact. The four scheduled stops allow the study of major slides in urban areas, active faults, coastal landsliding, and coastal erosion in relatively unpopulated areas.

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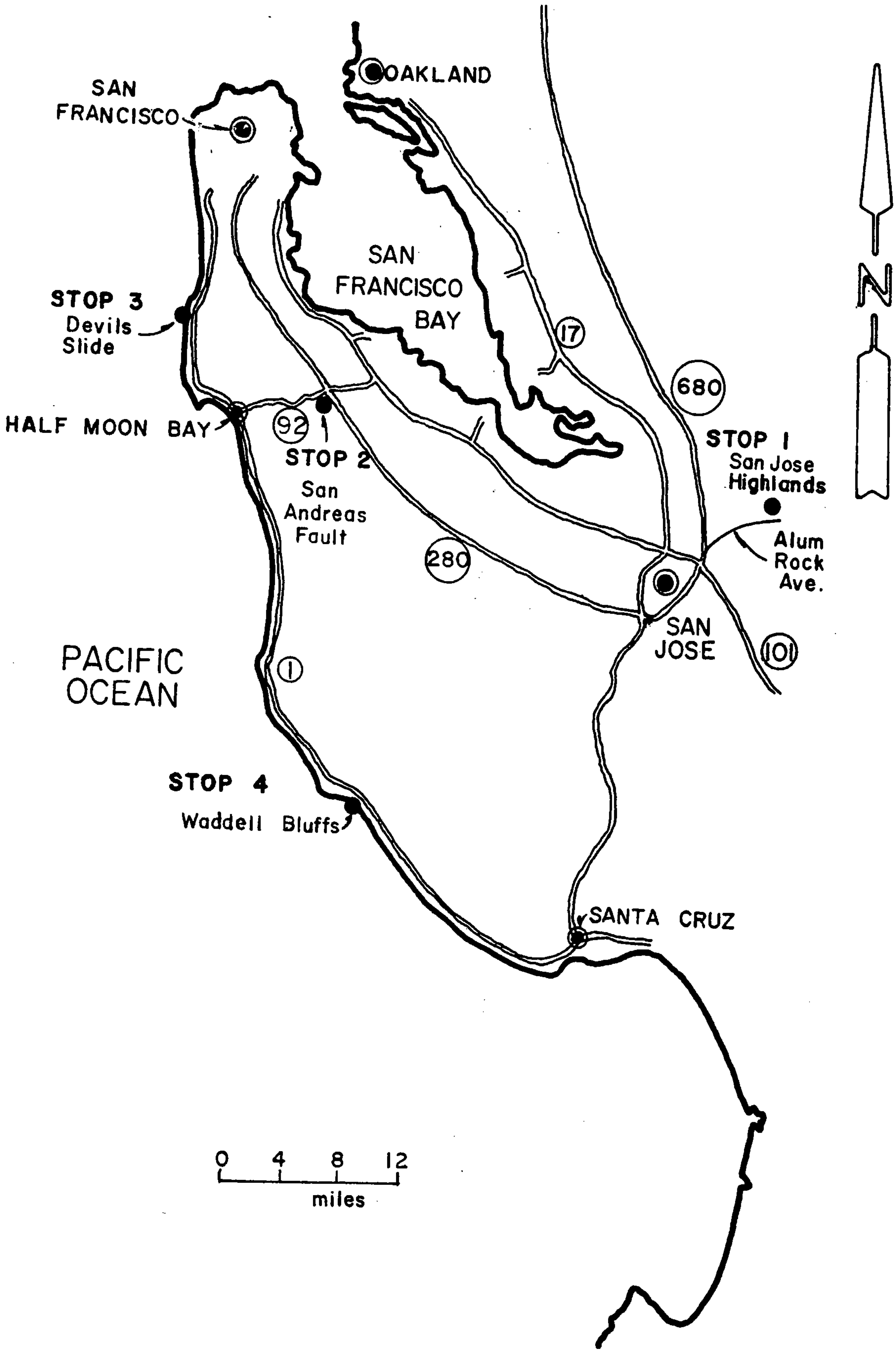

FIELD TRIP ROUTE