PROCEEDINGS

HIGHWAY GEOLOGY SYMPOSIUM
53rd ANNUAL

Sponsored By
California State Department of Transportation
And
California Geological Survey

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HIGHWAY GEOLOGY SYMPOSIUM
53rd ANNUAL

August 13 - 16, 2002

San Luis Obispo, California

Caltrans

MINES AND GEOLOGY
CALIFORNIA
CONSERVATION

Welcome to the 53rd Annual Highway Geology Symposium. The California Department of Transportation extends a very warm welcome, and hopes you enjoy your stay in the central California coast. We are honored to be this year's symposium host, in partnership with the California Geological Survey.

California is blessed with some of the most diverse and challenging geology in the world. The Coast Ranges, created by subduction and transform faulting, the alluvial sequences of the Great Valley, the Sierra Nevada Range created through intrusion, uplift and exhumation, the North Coast still undergoing subduction, the volcanically active Modoc Plateau, and the basin and range province of the eastern Sierra, provide exceptional scenic beauty, and exciting opportunities for the geotechnical professional. We hope that your visit includes visual delight, and an increased appreciation of the geologic complexity of our continental margin state.

The Highway Geology Symposium is a unique event, bringing together the practicing engineers and geologists who support the special transportation responsibilities of public agencies. This year's symposium presents practical information from outstanding practitioners in the field of engineering geology and geotechnical engineering. We believe you will find the information interesting and immediately useful in your own practice. The opportunity to meet and share information with your peers will increase your current knowledge, and provide knowledgeable contacts for help in the future.

Sincerely

Joan Van Velsor
Chief, Geotechnical Services
California Department of Transportation

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GRAY DAVIS
GOVERNOR

Welcome to California and the 53rd Highway Geology Symposium. The California Geological Survey is proud to co-sponsor this year's symposium with California Department of Transportation. We know that you will find the symposium program and field trip informative and intellectually stimulating and hope that you can spend some additional time in our state sampling the geological wonders.

Befitting a state whose discovery, settlement, and continuing development was influenced by geologic resources and geologic hazards; California has more than its share of spectacular geologic features. The Big Sur coast, which you will see on the field trip, is certainly one of the scenic and geologic wonders of our state, but the San Andreas fault at Parkfield or the Carrizo Plain is only about 40 miles east of San Luis Obispo. Yosemite Valley and the High Sierra is about 200 miles to the northeast.

The complex and continuing geologic deformation of the state has formed these scenic areas, but also results in earthquake and landslide hazards to many of our communities and lifelines. At the California Geological Survey we provide geologic and seismic information to other state agencies, local governments and the public so that Californians can consider and mitigate the hazards that always accompany development in a geologically active region. We hope that the information you receive at the symposium and in California help you in your missions to produce safe and reliable transportation systems by considering the geologic environment in which they are built.

Sincerely,

James F. Davis
State Geologist

HIGHWAY GEOLOGY SYMPOSIUM
53rd ANNUAL

SAN LUIS OBISPO, CALIFORNIA
AUGUST 13th – 16th, 2002

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The local organizing committee for the *53rd Annual Highway Geology Symposium* would like to express its appreciation to the following exhibitors and sponsors. You are invited to visit their displays. If you have occasion to be in contact with these organizations at the Symposium or in the course of your work, please be sure to express your appreciation.

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HIGHWAY GEOLOGY SYMPOSIUM

HISOTRY, 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 (HGS) was organized and held its first meeting on March 14, 1950, in Richmond, Virginia. Attending the inaugural meeting were representatives from state highway departments (as referred to at the time) from Georgia, South Carolina, North Carolina, Virginia, Kentucky, West Virginia, Maryland and Pennsylvania. In addition, a number of federal agencies and universities were represented. A total of nine technical papers were presented.

W.T. Parrott, an engineering geologist with the Virginia Department of Highways, chaired the first meeting. It was Mr. Parrott who originated the Highway Geology Symposium.

It was at the 1956 meeting that future HGS leader, A.C. Dodson, began his active role in participating in the Symposium. Mr. Dodson was the Chief Geologist for the North Carolina State Highway and Public Works Commission, which sponsored the 7th HGS meeting.

Since the initial meeting, 52 consecutive annual meetings have been held in 32 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, West Virginia, Ohio, Maryland, North Carolina, Pennsylvania, Georgia, Florida and Tennessee serving as host state.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona where the 13th annual HGS meeting was held. Since then it has alternated, for the most part, back and forth for the east to the west. The Annual Symposium has moved to different locations as follows:

List of Highway Geology Symposium Meetings

No.	Year	HGS Location	No.	Year	HGS Location
1st	1950	Richmond, VA	2nd	1951	Richmond, VA
3rd	1952	Lexington, VA	4th	1953	Charleston, W VA
5th	1954	Columbus, OH	6th	1955	Baltimore, MD
7th	1956	Raleigh, NC	8th	1957	State College, PA
9th	1958	Charlottesville, VA	10th	1959	Atlanta, GA
11th	1960	Tallahassee, FL	12th	1961	Knoxville, TN
13th	1962	Phoenix, AZ	14th	1963	College Station, TX
15th	1964	Rolla, MO	16th	1965	Lexington, KY

17th	**1966**	**Ames, IA**	**18th**	**1967**	**Lafayette, IN**
19th	**1968**	**Morgantown, WV**	**20th**	**1969**	**Urbana, IL**
21st	**1970**	**Lawrence, KS**	**22nd**	**1971**	**Norman, OK**
23rd	**1972**	**Old Point Comfort, VA**	**24th**	**1973**	**Sheridan, WY**
25th	**1974**	**Raleigh, NC**	**26th**	**1975**	**Coeur d'Alene, ID**
27th	**1976**	**Orlando, FL**	**28th**	**1977**	**Rapid City, SD**
29th	**1978**	**Annapolis, MD**	**30th**	**1979**	**Portland, OR**
31st	**1980**	**Austin, TX**	**32nd**	**1981**	**Gatlinburg, TN**
33rd	**1982**	**Vail, CO**	**34th**	**1983**	**Stone Mountain, GA**
35th	**1984**	**San Jose, CA**	**36th**	**1985**	**Clarksville, IN**
37th	**1986**	**Helena, MT**	**38th**	**1987**	**Pittsburgh, PA**
39th	**1988**	**Park City, UT**	**40th**	**1989**	**Birmingham, AL**
41st	**1990**	**Albuquerque, NM**	**42nd**	**1991**	**Albany, NY**
43rd	**1992**	**Fayetteville, AR**	**44th**	**1993**	**Tampa, FL**
45th	**1994**	**Portland, OR**	**46th**	**1995**	**Charleston, WV**
47th	**1996**	**Cody, WY**	**48th**	**1997**	**Knoxville, TN**
49th	**1998**	**Prescott, AZ**	**50th**	**1999**	**Roanoke, VA**
51st	**2000**	**Seattle, WA**	**52nd**	**2001**	**Cumberland, MD**
53rd	**2002**	**San Luis Obispo, CA**			

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 20-25 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 contribution to the Symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. Officers, except for the treasurer, may only succeed themselves for one additional term.

A number of three-member standing committees conduct the affairs of the organization. The lack of rigid requirements, routing, and relatively relaxed overall functioning of the organization is what attracts many of the participants.

Meeting sites are chosen two or 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 protem of 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 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 on Friday. In recent years this schedule has been modified to better accommodate climate conditions and tourism benefits.

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. To cite a few examples: in Wyoming (1973), the group viewed landslides in the Big Horn Mountains; Florida's trip (1976) 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 generation 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 meeting in 1981 provided stops at several repaired landslides in Appalachia regions of East Tennessee.

In Utah (1988) the field trip visited sites in Provo Canyon and stopped at the famous Thistle Landslide, while in New Mexico in 1990 the emphasis was on rockfall treatment in the Rio Grande River canyon and included a stop at the Brugg Wire Rope headquarters in Santa Fe.

Mount St. Helens was visited by the field trip in 1994 when the meeting was in Portland, Oregon, while in 1995 the West Virginia meeting took us to the New River Gorge bridge that has a deck elevation 876 feet above the water.

In Cody, Wyoming the 1996 field trip visited the Chief Joseph Scenic Highway and the Beartooth uplift in northwestern Wyoming. In 1997 the meeting in Tennessee visited the newly constructed future I-26 highway in the Blue Ridge of East Tennessee. The Arizona meeting in 1998 visited Oak Creek Canyon near Sedona and a mining ghost town at Jerrome, Arizona.

At the technical sessions, case histories and state-of-the-art papers are most common; with highly theoretical papers the exception. The papers presented at the technical sessions are published in the annual proceedings. Some of the more recent proceedings my be obtained from the Treasurer of the Symposium.

Banquet speakers are also a highlight and have been varied through the years.

A Medallion Award was initiated in 1970 to honor those persons who have made significant contributions to the Highway Geology Symposium. The selection was and is currently made from the members of the national steering committee of the HGS.

A number of past members of the national steering committee have been granted Emeritus status. These individuals, usually retired, resigned from the HGS Steering Committee, or are deceased, have made significant contributions to the Highway

Geology Symposium. A total of 20 persons have been granted the Emeritus status. Ten are now deceased.

Several Proceedings volumes have been dedicated to past HGS Steering Committee members who have passed away. The 36th HGS Proceedings were dedicated to David L. Royster (1931-1985, Tennessee) at the Clarksville, Indiana Meeting in 1985. In 1991 the Proceedings of the 42nd HGS meeting held in Albany, New York was dedicated to Burrell S. Whitlow (1929-1990, Virginia).

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Highway Geology Symposium

Future Symposia Schedule and Contact List

Year	State	HGS Contact	Host Coordinator	Telephone Number	Email
2003	Vermont	Ingram Sweeney	Tom Eliassen	802-549-3663	Tom.eliassen@state.vt.us
2004	Missouri	Rob Henthorne	Alan Miller	316-431-1000 Ext. 47	Roberth@dt14.wpo.state.ks.us
2005	North Carolina		Russell Glass	828-298-3874	Rglass@dot.state.nc.us
2006	Colorado		Johnathan Wright		
2007	Pennsylvania		Chris Ruppen	724-495-4079	

AGENDA

TUESDAY, AUGUST 13

11:00 AM - 5:00 PM
TRB and HGS 53 Registration

TRB Special Session: Design and Construction in Melange
Chairman, Dr Edmund Medley
Location: San Luis Obispo Room

12:00 – 1:30 PM

John Wakabayashi – Geology of Melanges

Gunter Riedmüller – Geologic mapping in melanges

1:30 – 1:45 PM (Break)

1:45 – 3:15 PM

Edmund Medley – Engineering Geology of melanges

Norm Norrish - Landslide repair in melange

3:15 – 3:30 PM (Break)

3:30 – 5:00 PM

Dave Meza – Construction claims in melange

Edmund Medley – Questions and Discussion

5:00 PM - 6:00 PM
Ice Breaker Social
Location: Los Osos Room

Wednesday, August 14

53rd HIGHWAY GEOLOGY SYMPOSIUM

7:00 AM - 5:00 PM
HGS 53 Registration

7:30 AM - 8:30 AM
Welcoming Remarks
Moderator: John Duffy
Location: San Luis Obispo Room

Robert Buckley, Division Chief, Engineering Services, California Department of Transportation

Joan Van Velsor, Deputy Division Chief, Geotechnical Services, California Department of Transportation

Gregg Albright, District Director, District 05, San Luis Obispo, California Department of Transportation

Jim Davis, State Geologist, California Geological Survey

8:30 AM - 9:30 AM
Technical Session I: Rockfall
Moderator: Roy Bibbens
Location: San Luis Obispo Room

Andrew R., Arndt B., and Patterson K. *Georgetown Incline Rockfall Study*. (pages 1-12)

Carter T., de Graaf P., Barrett S., Booth P., and Pine R. *Innovative Design for Challenging Highway Widening Project in Hong Kong*. (pages 13-25)

Grassel H., Bartelt P., Ammann W and Roth, A. *Behavior, Design and Reliability of Highly Flexible Rockfall Protection Systems for Highways*. (pages 26-39)

9:30 AM - 9:45 AM (Break)

9:45 AM - 12:00 PM
Technical Session I: Rockfall Continued
Moderator: Gustavo Ortega
Location: San Luis Obispo Room

Baumann R. *The Worldwide First Official Approval of Rockfall Protection Nets Development and Experiences*. (pages 40-51)

Spang R. *Certification of Rockfall Barriers in Europe*. (pages 52-62)

Richard, M. *Rockfall Barrier Testing in France*

Viktorovitch, M. *The European Technical Approval Guideline For Rock fall Protection Net Fences - Outline of the Future Standard*

Wood D. and Morris A. *Replacement of Traditional Rockfall Detection with Innovative Catchment and Detection, A Case History at CPR's Red Rock Slide Fence*. (pages 63-73)

Priznar N., Rice A., and Humphries R. *Landslide and Rockfall Hazard Mitigation in a Multiple Intrusive Precambrian Diabase Series for US 60 in Salt River Canyon, Arizona*. (pages 74-85)

Fisher J. *Corridor H Section 16 from Elkins to Kerens Randolph County, West Virginia Rockslides Investigation & Corrective Measures*. (pages 86-96)

7:30 AM - 12:00 PM
Morning Poster Session
Location: Linda Vista Foyer

Beck B. and Zhou W. *Evaluation of a Peat Filtration System for Treating Highway Runoff in a Karst Setting.* (page 97)

Boomhower D. *Excessive Differential Settlement in the Northern Nevada Railroad Undercrossing Structure at U. S. Highway 50 Near Ely, Nevada.* (page 98) - Withdrawn

<u>Wednesday, August 14, Cont.</u>

12:00 PM - 1:00 PM (Lunch)

1:00 PM - 2:30 PM
Technical Session II: Rock/Landslides
Moderator: Zeke DeLlamas
Location: San Luis Obispo Room

LaMont D., Melita J., Peterson D., and LaFronz N. *Construction-Phase Design Changes in Response to a Compound Rockslide, Preacher Canyon Section of State Route 260, Near Payson, Arizona*. (pages 99-111)

Lane R. and Fish M. *Emergency Rock Slope Remediation at "Deadman's Curve " in Pinkhams Grant, New Hampshire*. (pages 112-123)

Norrish N., Findley D., Byers T., and Lowell S. *Ruby Creek Rock Slope Stabilization.* (pages 124-146)

Whitman T. and Heyes D. *The Stabilization of Red Top Landslide at American Canyon in Solano County, CA*. (pages 147-158)

Kovacs J., Morris M, and Schultz J. *Rock Slope Stabilization for the Boulevard of the Allies Reconstruction Pittsburgh, Pennsylvania*. (pages 159-170)

2:30 PM - 2:45 PM (Break)

2:45 PM - 4:45 PM
Technical Session III: Planning/Landslides
Moderator: Ailleen Loe
Location: San Luis Obispo Room

Rosenberg L. *Geology and GIS for Planning: An Example from Monterey County, California*.(pages 171-182)

Wills C., Manson M., Brown K., and Wagner D. *Geology and Landslide Mapping Along Highway Corridors in California: Factors Influencing Landslide Potential*. (pages 183-194)

Andrew R., and Lovekin J. *I-70 Corridor Programmatic Environmental Impact Study*. (pages 195-203)

Hapke C. *Determining Landslide Volume Input to the Coastal Zone along the Big Sur, CA Coast*. (pages 204 -215)

Dessenberger N., Harrison F., and Kracum J. *Going-To-The-Sun Road--Planning the Rehabilitation of a National Civil Engineering Landmark*. (pages 216-226)

Arndt B., Andrew R., and Pihl R. *Snowmass Canyon State Highway 82 Project*. (pages 227-239)

12:00 PM - 5:00 PM
Afternoon Poster Session
Location: Linda Vista Foyer

Mohan S., Price R., Gu R., Buell R., Thorne J., and Le T. *Soil-Pile Set-Up of Deepest Ever Large Diameter Driven Pipe Piles New East Span San Francisco-Oakland Bay Bridge*. (pages 240-251)

Wagner, D., Spittler, T., and Sydnor, R. *Landsliding along the Highway 50 Corridor: Geology and Slope Stability of the American River Canyon between Riverton and Strawberry, California.* (pages 252-263)

4:45 PM - 5:00 PM
Field Trip Briefing: John Duffy

5:00 PM
Adjourn for the day

5:30 PM
Steering Committee Meeting
Location: San Luis Obispo Room

THURSDAY, AUGUST 15

7:00 AM- 5:00 PM

Geology Field Trip: Meet at 6:45 at the front entrance of Embassy Suites Hotel. The field trip will be along the Big Sur Coast. Bring your jacket, hat and sunscreen. Food and drinks will be provided.

6:00 PM- 7:00 PM
Social Hour and Exhibits
Location: Los Osos Room
San Luis Obispo Room (North)

7:00 PM- 10:00 PM
Banquet Dinner
Location: San Luis Obispo Room

San Simeon Fault Zone

Cow Cliff Rockfall Protection

Gray Slip and Duck Ponds Landslides

Friday, August 16

8:00 AM - 9:45 AM
Technical Session IV: Subsidence, Modeling, and Materials
Moderator: Grant Wilcox
Location: San Luis Obispo Room

9:45 AM - 10:00 AM (Break)

10:00 AM - 12:30 PM
Technical Session V: Rock Mechanics and Foundations in Rock
Moderator: Jennifer O'Niel
Location: San Luis Obispo Room

12:30 PM Adjourn

Georgetown Incline Rockfall Study

Richard Andrew, P.G., Ben Arndt, P.E, Katherine Patterson
Yeh and Associates, Inc.

ABSTRACT

Rockfall is a common hazard along transportation routes in the mountainous terrain in Colorado. Many accidents, injuries, and fatalities resulting from rockfall events have occurred along I-70 specifically along the Georgetown Incline. In response to the rockfall hazard, Yeh and Associates completed a study for the Colorado Department of Transportation (CDOT) to evaluate the area and provide recommendations for mitigation of the rockfall hazard.

Yeh and Associates gathered geological hazard data using existing information and by conducting field reconnaissance. Outcrops and rockfall chutes were mapped and evaluated for potential rockfall. Areas requiring mitigation were also identified. These areas were evaluated using the Colorado Rockfall Hazard Rating System (CRHRS), the Modified Q-system rock mass rating system, field observations, aerial photos, and geologic maps. The Colorado Rockfall Simulation Program (CRSP) and Roc Science's RocFall software were used to evaluate rock fall behavior and assist in determining an appropriate means of mitigating the rock fall hazard.

Potential areas of hazard were identified and recommendations on mitigation methods were provided. Georgetown mitigation options included: barriers, rock reinforcement, rockfall containment, rock removal, fences and attenuators, rock sheds, and tunnels. The mitigation methods were evaluated based on feasibility, effectiveness, and cost.

INTRODUCTION

The Georgetown Incline consists of a 1.7-mile section along I-70 between Georgetown and Silver Plume, Colorado. The slopes are very steep with numerous rock cut sections which were made during construction of I-70. The rock cuts range from 50 to 100 feet in height. Bedrock outcrops are also present on the steep slopes above the interstate highway. The bedrock outcrops range in vertical height from 50 to 200 feet and are located on 1H:1V slopes. The total change in elevation from I-70 to the upper most bedrock outcrops is over 1800 feet. The combination of rock cuts and rock outcrops has contributed to numerous rockfall events along the Georgetown Incline.

Geological data was gathered along the I-70 corridor between the Georgetown Interchange and the Silver Plume Interchange, which included historical research, field reconnaissance and mapping, site photography, and aerial photography. Rock cuts, bedrock outcrops, and rockfall chutes were mapped for potential rockfall sources. The rockfall potential from these areas was then evaluated using the Colorado Rockfall Hazard Rating System (CRHRS), the Modified Q-rating system, and risk analysis methods.

The Colorado Rockfall Simulation Program (CRSP) was primarily used to model rockfall and assist in determining feasible mitigation methods for rockfall hazards. Hazard areas were identified and various mitigation methods were then evaluated.

GEOLOGIC SETTING

Rockfall generated along the Georgetown Incline originates from three sources: the highway cuts made through bedrock, the highway cuts made through overburden soils, and the natural bedrock outcrops. The Idaho Springs Formation and the Silver Plume Granite are the two rock types exposed along the Incline. The Idaho Springs Formation consists of a biotite-quartz-microcline and plagioclase gneiss with lesser amounts of lime-silicate gneiss and amphibolites. The rock is typically light gray to dark gray and fine to medium grained. The Silver Plume Granite consists of igneous rock that intruded into the Idaho Springs Formation approximately 1440 million years ago. The Silver Plume is mostly a light gray to pinkish light gray quartz monzonite. The rock is fine to medium grained, equigranular, and has a homogeneous composition.

The structure of the rock types comprising the Georgetown Incline was influenced by deformation events during the Precambrian, Laramide Orogeny, and possibly in the Tertiary Period. The foliation of both units generally strikes N to N 30° E and dips 60° - 90° NW or SE. Faults in the area mostly strike N 20° - 50° E and dip N 70° - 80° E.

HISTORICAL ROCKFALL ACTIVITY

There is a history of rockfall events, which have led to accidents and fatalities along the Georgetown Incline. Approximately 100 accidents have occurred in the past 24 years with 17 injuries and 3 fatalities. Based on physical evidence from rock filled ditches along westbound I-70 and rock covered embankments along eastbound I-70 there is a much greater incidence of rockfall activity than reported accidents.

Recent rockfall activity at the Georgetown Incline includes two fatalities in 1999 and an accident in 2001. December 9, 1999 a man from Erie, PA was fatally struck when a rock shattered the window of the shuttle van in which he was riding. This occurred at mile marker 227.35. On May 16, 1999 a women was killed after boulders struck her Toyota at mile marker 226.40. It was reported that some of the boulders were the size of bathtubs. A rockfall event occurred May 6, 2001 causing two accidents from cars impacting rock debris. The 2001 event occurred after four days of precipitation and snowfall with freezing temperatures. Rockfall from this event was observed in the ditch of westbound I-70, on the shoulder of eastbound I-70, and a bike path below the highway embankment. Rockfall debris that impacts eastbound I-70 or the adjacent bike path, must exhibit high energies and extreme bounce heights.

ROCKFALL FAILURE MECHANISMS

The combination of topography, surficial deposits, bedrock quality, and weathering processes all contribute to the rockfall events along the Georgetown Incline. The topography has been shaped by glacial activity, which has oversteepened some sections of the slope area. The overburden consists of colluvial soils with angular boulders and cobbles consisting of granites and gneiss that form talus slopes in many areas. The bedrock generally exhibits 2 to 3 discontinuity orientations, which contribute to rockfall sources. Additionally, the slopes face southeast so weathering and daily freeze/thaw cycles contribute greatly to the rockfall events.

The interstate was constructed by cutting into colluvial slopes and bedrock. This has led to rockfall from vertical rock cuts and oversteepened colluvial slopes. Many of the colluvial slopes were excavated down to bedrock, however the exposed bedrock will occasionally contribute to the rockfall event. The oversteepened colluvial slopes will also contribute cobbles and/or boulders that will create separate rockfall events.

Bedrock along the road cut section of the Incline is highly fractured with many joint surfaces that exhibit slickensides. The Idaho Springs Formation exhibits a well-defined foliation and most of the jointing either runs parallel or perpendicular to the foliation. The orientation, continuity, infilling, aperture, spacing, and roughness of discontinuities affect the shear strength and stability of the rock mass. The orientation of discontinuities in the rock mass, relative to the face of the slope, directly affects slope stability and the type of failure that occurs. Discontinuities along the Incline dip unfavorably towards the highway which, contributes to the instability of the rock mass.

Climactic conditions such as temperature variations, rain, snow, and freeze-thaw all contribute to rockfall activity. The south facing slopes of the Georgetown Incline are exposed to extreme temperature fluctuations enhancing surficial erosion, increasing freeze-thaw action, and contributing to the structural breakdown of the rock mass through expansion and contraction of the rock face. In conjunction with snowmelt, freeze-thaw cycles appear to be the primary triggering mechanism of rockfall, as shown in Figure 1. The highest accident rate occurs in March and April when the snowfall is the highest and when the temperature drops below and rises above freezing on a frequent basis.

EVALUATION METHODS

The Colorado Rockfall Hazard Rating System (CRHRS), modified Q-rating system, and risk analysis were all used to evaluate the conditions at the Georgetown Incline.

Colorado Rockfall Hazard Rating System (CRHRS)

To evaluate the rockfall hazard for the rock cuts along I-70 it was necessary to use the Colorado Rockfall Hazard Rating System (CRHRS). CRHRS was developed in 1994 by the Colorado Department of Transportation (CDOT) as a rating system for identifying, evaluating, and prioritizing sites in Colorado that produce rockfall. The methodology is based on the rating system developed by the Oregon Department of Transportation (ODOT) with some modifications. The CRHRS includes two phases. Phase I identifies and ranks segments of highways that have chronic problems with rockfall. Road segments with continual rockfall activity are identified by occurrence of vehicle accidents caused by rockfall, or identification by highway maintenance personnel as rockfall-prone areas. The information gathered from Phase I is used to determine locations where more detailed evaluation is needed. The sites are ranked from 0 to 4 with 4 being the worst or most active. Due to the numerous rockfall sites identified throughout Colorado only segments ranking 3 or higher were evaluated under Phase II.

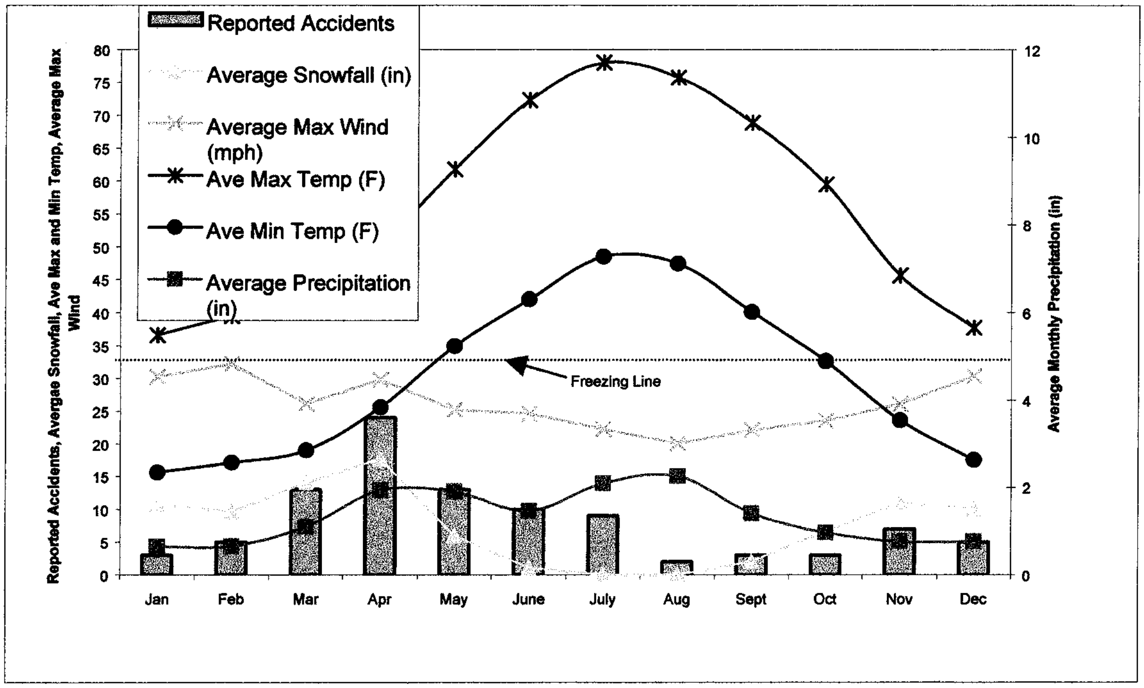


Figure 1. Reported Accidents and Climate Data

Phase II of the CRHRS hazard rating system further delineates, describes, and scores individual rockfall source areas or sites. Geologic and physical site data is collected. The evaluation and scoring criteria are used to define segments with similar geologic, slope, and rockfall criteria within stretches of highways. The rating scheme is divided into 15 categories, which represent significant elements contributing to rockfall. Points are assigned to each category with 81 indicating the greatest hazard. The sum of all the points from each category equals the overall rating for the segment of highway. The sites are then classified as follows:

Class	Rating
A	550 or higher
B	450-550
C	350-450
D	250-350
E	Below 250

Classes A and B are considered for mitigation first. Both cost and complexity of the problem are evaluated in determining which sites are practical to mitigate.

The original ratings were conducted in 1992 and updated 1998, to include additional factors that may contribute to a rockfall accident. Segments of the Georgetown Incline included 6 of the top 10 sites in the region and in the top twenty sites in the state for rockfall hazard. A summary of the ratings for the Georgetown Incline were as follows:

Table 1. Colorado Hazard Ratings for I-70 at the Georgetown Incline

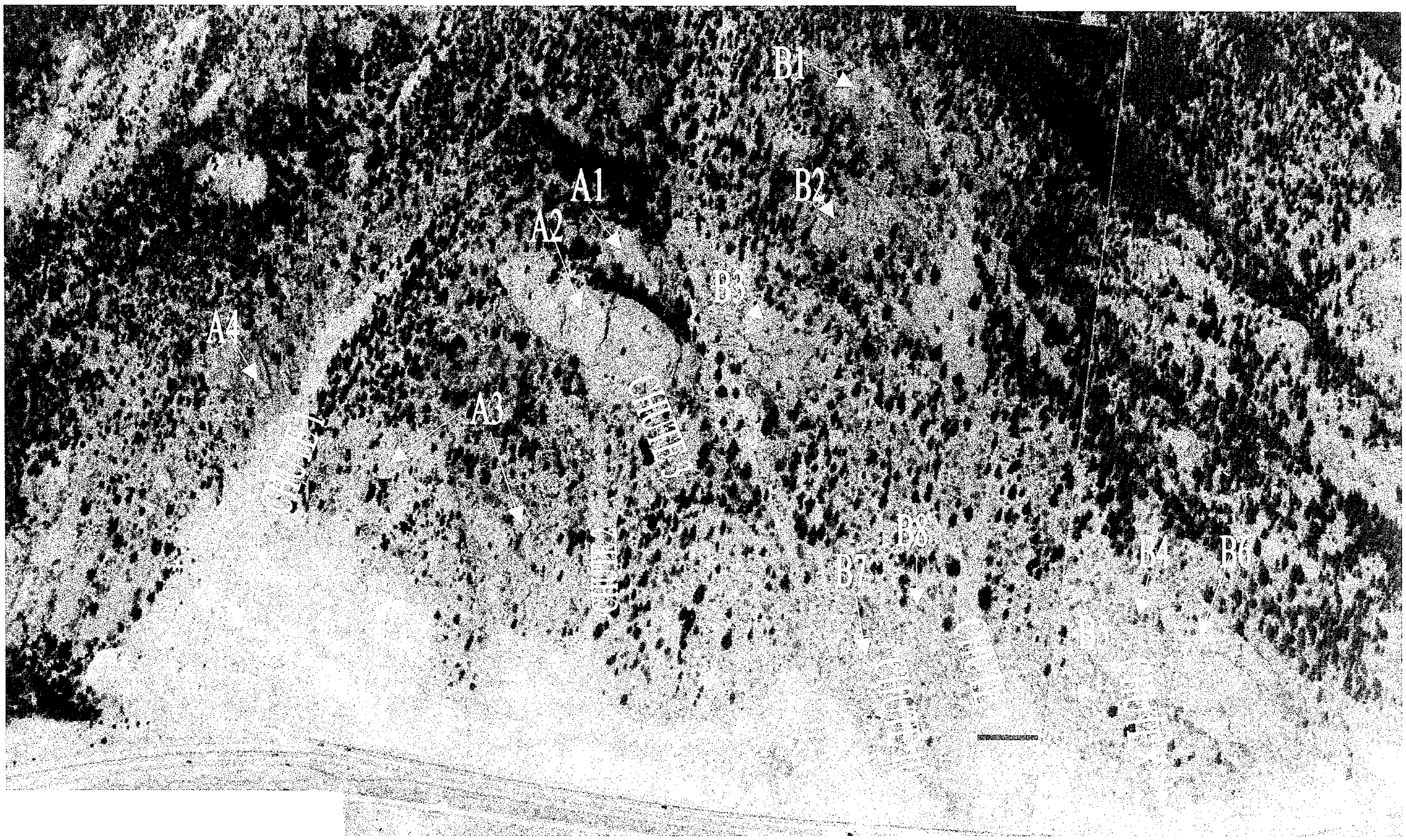
Segment ID	Mile Markers	1992		1998	
		CRHR	GT Ranking	CRHR	GT Ranking
1	226.0-226.05	435	9	534	8
2	226.05-226.29	633	1	648	1
3	226.29-226.49	615	2	594	4
4	226.49-226.57	399	10	522	9
5	226.57-226.58	222	15	405	15
6	226.58-226.70	399	11	471	13
7	226.70-226.83	549	3	570	6
8	226.84-226.94	375	12	489	11
1a	227.13-227.22	339	14	456	14
2a	227.22-227.29	531	6	552	7
3a	227.29-227.31	501	8	588	5
4a	227.31-227.36	531	5	621	2
5a	227.36-227.40	375	13	486	12
6a	227.40-227.43	501	7	612	3
7a	227.43-227.72	549	4	498	10

Modified Q Rating System

To rate the rock mass quality and evaluate the rockfall hazard of the bedrock outcrops above I-70 it was necessary to use the modified Q rating system. The original Q Rating System was developed for tunneling support design and cost estimation in mining. The system was modified for describing rock quality on slopes by Harp & Noble, 1993. Since it was not feasible to directly access the slopes above I-70, it was necessary to use this method for evaluating the quality of the bedrock outcrops from a distance or with site and aerial photographs. The outcrops were grouped and mapped according to location, as shown on Figures 2 and 3. Six factors were used to calculate Q are rock quality designation (RQD), joint set number (Jn), joint roughness number (Jr), joint alteration number (Ja), joint water reduction (Jw), and aperture factor (AF). Aperture factor replaces the Stress Reduction Factor (SRF) in the original Q rating system. Each factor has an associated rating for varying conditions. Based on these factors the value for Q is calculated as follows:

$$Q = \left[\frac{RQD}{J_n}\right] * \left[\frac{J_r}{J_a}\right] * \left[\frac{Jw}{AF}\right]$$

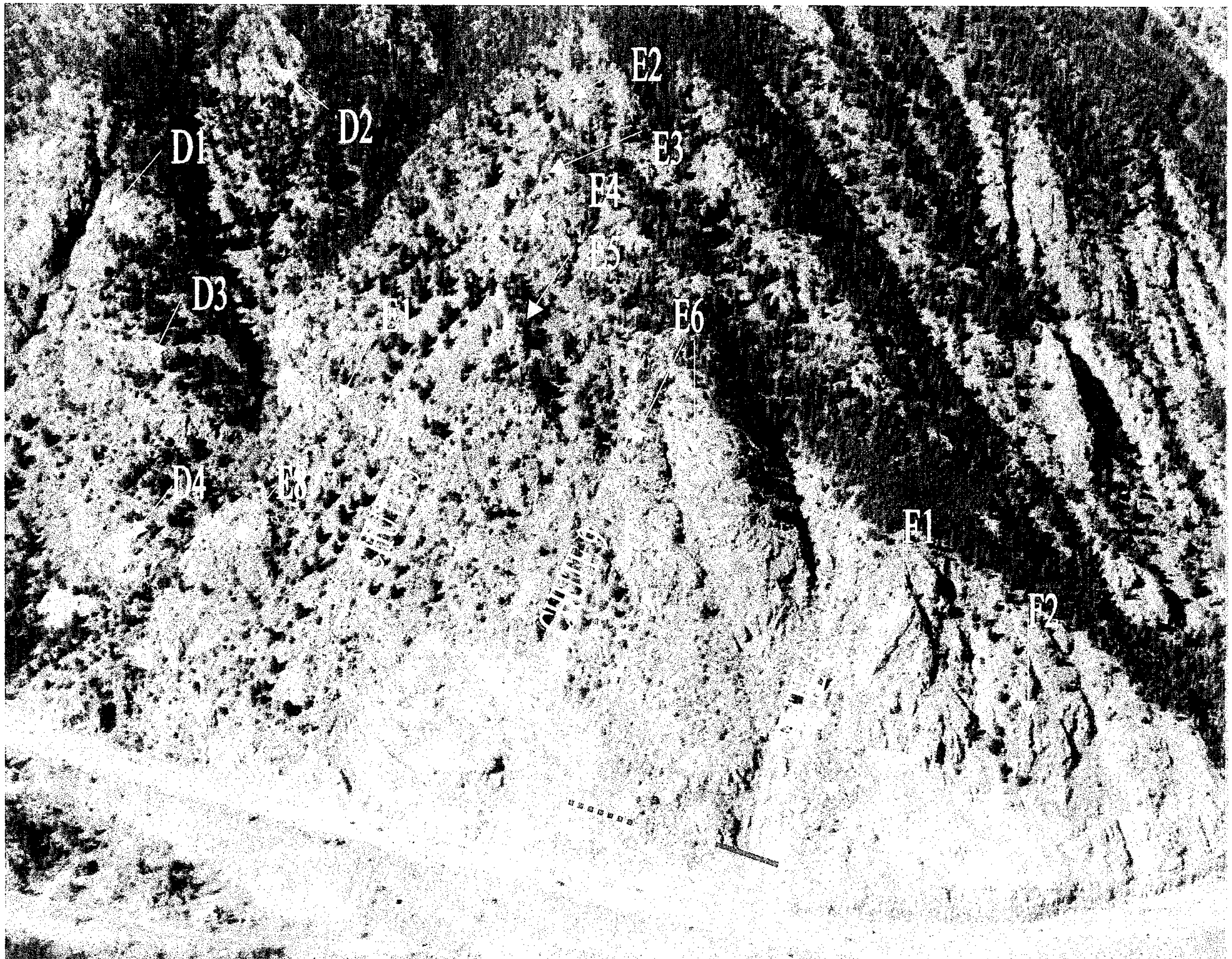
The first quotient in the equation represents the structure of the rock mass and is a measure of block or particle size. The second quotient expresses the shear strength of the block mass and represents the roughness and frictional characteristics of the joint walls or filling materials. The third quotient is a measure of the active stress conditions in a rock mass. The lower the value of Q, the more unstable the outcrop.



Existing Rockfall Fence

Yeh and Associates, Inc.
Consulting Geotechnical Engineers

Figure 2. Rockfall Chutes and Source Areas with Fence Location



Existing Rockfall Fence

Proposed Rockfall Fence

Yeh and Associates, Inc.
Consulting Geotechnical Engineers

Figure 3. Rockfall Chutes and Source Areas with Fence Location

Table 2 provides the results of the modified Q rating system for outcrops at the Georgetown Incline. The ratings were primarily done from field observation and photographs. There are many outcrops with ratings less than 1. Rockfall chutes were also evaluated to determine the chutes with the lowest Q rated bedrock outcrops. Many outcrops received low ratings, but did not appear to have direct pathways or chutes to the interstate highway. Generally the outcrops with low Q ratings and direct pathways to the interstate correlated with areas of increased reported accidents. Table 2 illustrates the outcrops with low ratings and a higher number of accidents.

The rock cuts had many of the same characteristics as the bedrock outcrops, however rockfall from the cuts generally falls into the ditch next to westbound I-70. In general many areas that received high CRHR ratings also received low Q-ratings indicating greater rockfall hazards.

ANALYSIS METHODS

There are numerous methods for analyzing rockfall energies and trajectories. Analyses can include field-testing, mathematical analysis, and empirical analysis. For this project, computer modeling was used as a tool to evaluate the rockfall and determine the appropriate mitigative measures. The Colorado Rockfall Simulation Program (CRSP) and Rocfall by Roc Science were used to model the rockfall events at the Georgetown Incline and are discussed in detail below.

CRSP

The Colorado Rockfall Simulation Program (CRSP) was developed to provide a reasonable model of falling rock from a given a set of slope input parameters. The input parameters for CRSP allow simulation of the rockfall pathway by inputting parameters for the surface roughness, the slope profile, the tangential coefficient of frictional resistance, the normal coefficient of restitution (which approximates the rigidity of the slope surfaces), and the shape and diameter of the falling rock. Field observation and studies of actual rockfall were used to calibrate CRSP on other projects.

The major chutes at Georgetown were analyzed in CRSP. The profiles were determined from topography maps, observation, and laser sighted distances. Analysis points were generally located 20 feet above the crest of the rock cuts. 3-foot diameter rocks were used in the model. The 3 ft rock size was based on rocks that had been observed in chutes 4 and 6. A 20-foot distance above the slope break was used for comparison and was considered the most feasible location for a rockfall fence. A fence located closer to the rockfall source is usually preferable, but was determined infeasible to maintain or construct. Based on the CRSP analysis Chutes 6, 4, 3, and 9 showed the highest potential energies and bounce heights for rockfall events. Analysis indicated that rockfall was able to reach the highway in the chutes that were modeled.

Table 2. Modified Q ratings and accidents for Georgetown Incline.

1	226.0-226.06	A3,A4	0.44	0.56	1.78						0.93	1
2	226.17-226.29	A1,A2	4.00	1.56	1.33	3.00	2.00				2.38	14
3	226.36-226.39	A1,A2,B3,B7,B8	4.00	1.56	1.33	3.00	2.00	1.00	1.11	3.50	1.73	2.14	9
4	226.46-226.57	B1,B2,B3,B4	0.52	0.69	0.69	0.64	1.00	1.11	0.92	2.33	1.88	1.09	9
5	227.16-227.22	E1,E2,E3,E4,E5,E8	2.22	1.11	0.56	0.56	1.67	5.00				1.85	4
6	227.37-227.44	E6,E7,E9,F1,F4	4.00	1.35	2.50	0.50	0.27	0.50	0.59			1.39	11
8	226.6-226.64	B4,B6,C7	0.92	2.33	1.88	2.31	2.00	0.98	0.93			1.62	1
9	227.25-227.32	E1,E2,E3,E5,E6	2.22	1.11	0.56	1.67	4.00					1.91	4
10	226.42-226.44	B3,B7,B8	1.00	1.11	3.50	1.73						1.84	0

Notes:Chute 8 is already meshed. Chute 4 fenced and meshed Nov. 2000. Chute 6 fenced and meshed Jan. 2001.
Accidents reported from 1976-1999

A1,A2	226.12-226.36	1.56	1.33	2.00	3.00				1.97	18
A3	226.06-226.19	0.44	0.56						0.50	8
A4	226.0-226.05	1.78	0.98						1.38	1
B1,B2,B3,B7,B8	226.36-226.57	1.00	1.11	1.73	0.52	0.69	0.64		0.95	16
B4,B5	226.5-226.6	0.92	2.33	1.88	2.73				1.96	10
B6	226.64-226.67	2.31	2.00						2.15	0
C's	226.58-226.83	1.48	1.67	1.67	1.85	1.28	0.46	0.77	1.09	6
		1.39	0.98	0.93	1.00	0.93	0.44	0.42		
D's	226.83-226.92	0.33	1.00	1.00					0.78	1
E1,E8	226.92-227.16	2.22	5.00						3.61	4
E2,E3,E4,E5	227.22-227.25	1.11	0.56	1.67	0.56				0.97	0
E6,E7,F4	227.32-227.37	1.35	0.59						0.97	1
E9	227.37-227.44	2.50							2.50	10
F1,F2	227.44-227.63	0.50	0.27	0.50	0.83	1.00			0.62	13

Note: outcrops that do not have a path to highway are not included

Rocfall
Rocfall by Rocscience is a similar program to CRSP, which is designed to assist with a risk assessment of slopes prone to rockfall. The program will generate energy, velocity, and bounce height envelopes for a given slope. Input into the Rocfall program is similar to CRSP since slope parameters, slope cell properties, number of rocks to roll, and initial coordinate points for rolling rocks can be specified. Rocfall differs from the CRSP program in that no input is provided for surface roughness of the cells, diameter of rocks to be rolled, or density of the rocks. Slope roughness in the Rocfall program is assessed by specifying a standard deviation for each of the slope segments. Rocfall then uses a normal distribution to randomly generate a maximum or minimum slope angle.

At this time, it is not possible to directly compare the CRSP and Rocfall programs since it is our understanding there is no exact way to convert the surface roughness coefficient from CRSP (version 4) into a standard deviation of the surface roughness coefficient required for input in Rocfall (v 4.02). Future versions of Rocfall will likely allow for a surface roughness input. Table 3 shows the results of comparison of CRSP with Rocfall with the understanding an equivalent surface roughness was approximated for the Rocfall input.

Table 3. CRSP and Rocfall Results

Chute	Kinetic Energy (ft-lb)		Velocity (ft/sec)		Kinetic Energy (ft-lb)		Velocity (ft/sec)	
	max	ave	max	ave	max	ave	max	ave
	CRSP				Rocfall			
4a	380,257	221,211	88.56	66.56	169,066	89,926	63.67	47.46
4b	469,595	320,254	99.63	80.35	377,183	132,473	91.28	55.3
6a	524,823	334,689	107.36	82.83	191,869	106,802	65.508	49.21
6b	515,310	383,139	107.45	88.96	261,157	190,532	83.06	70.5
9a	518,403	278,109	108.55	76.12	386,342	283,862	92.25	77.25

ROCKFALL MITIGATION
To solve a rockfall problem several issues need to be considered such as, physical data about the site, the acceptable level of risk, the costs to solve the problem, the level of maintenance required, constructabilty and environmental impacts. The choice of mitigation depends on the site conditions and constraints to construction. Mitigation measures are often divided into three categories: stabilization, protection, and avoidance. Stabilization keeps rocks in place and can be effective where rockfall sources are easily accessible. Stabilization methods are usually considered first since they often result in minimal maintenance and provide long term solutions. Protection methods are typically designed to prevent rocks from reaching roads or structures. These methods usually include fences that can absorb energy from a rockfall event. Avoidance and relocation allow for a complete solution but are typically the most expensive options.

The Georgetown Incline is a large site with numerous outcrops and chutes that contribute to many potential modes of rockfall. Using the CRHRS rating system, the Q rating system, and computer simulation of the rockfall events, it was possible to determine the critical areas that should be prioritized for mitigation.

The preliminary results of this study determined Chutes 4 and 6 have relatively greater potential for rockfall than other areas. Based on the results of the study and the issues briefly discussed above, it was decided that rockfall protection fences were to be used for mitigation. Rockfall protection fence systems were installed at Chute 4 and Chute 6 (see Figures 2 and 3). These chutes also corresponded to areas where two previous fatal accidents had occurred. The rockfall fences were constructed during the fall and winter of 2000 and 2001 respectively.

The rockfall protection at Chute 4 consisted of an 80 foot-ton cable net fence with draped cable net below. The rockfall protection at Chute 6 consisted of a 180 foot-ton ring net fence with draped cable net below. Figure 4 illustrates a typical profile of the fence system. Notice a 3-foot gap was left below the bottom cable support to allow for rocks and rock debris to migrate under the fence panels while still restraining the material with the draped cable net. The fences were installed close to the slope breaks to aid in self-cleaning of the fence.

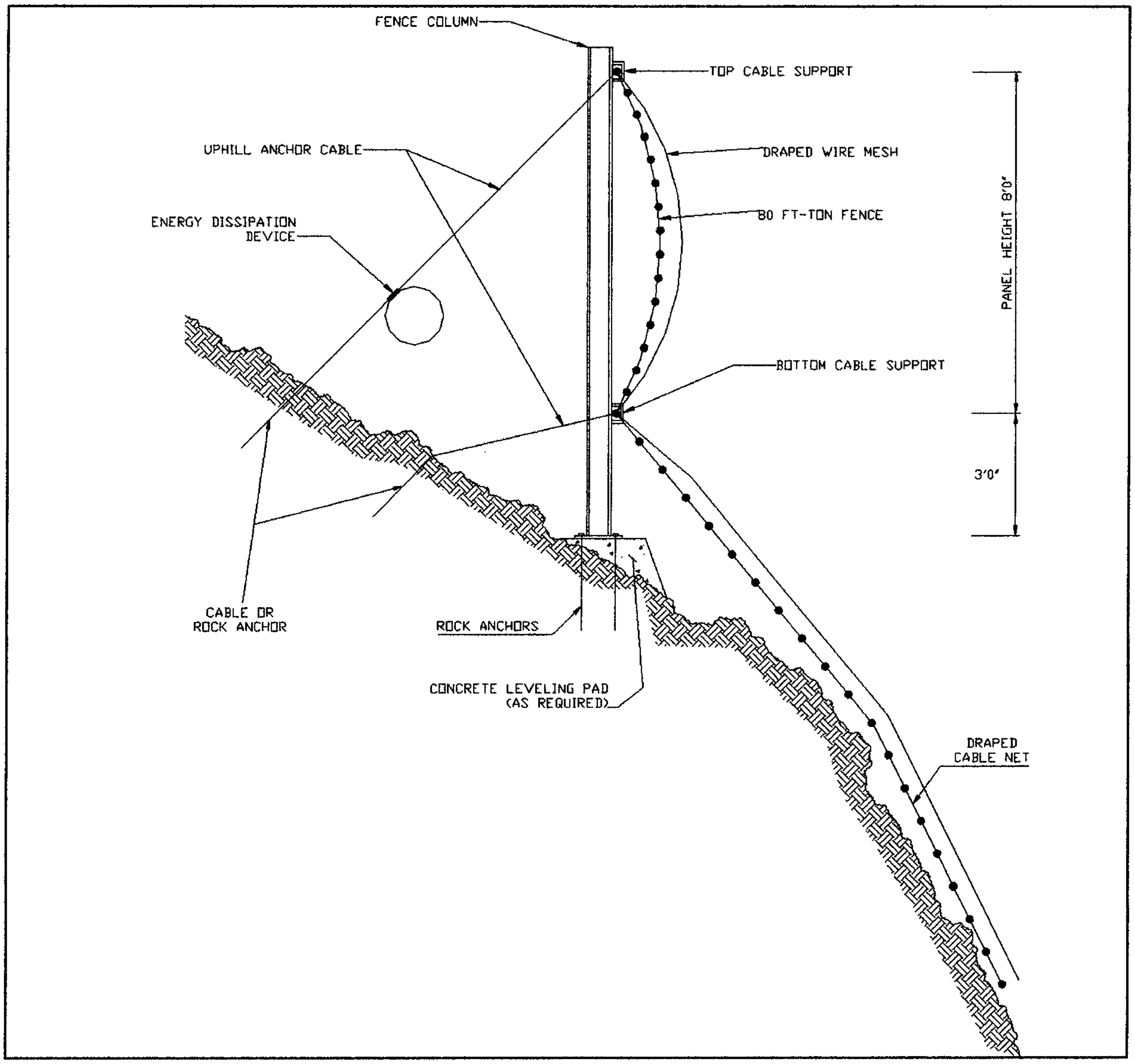


Figure 4. Typical profile of Rockfall Protection Fence

As of July 2002, both of the fences had been impacted by 2 to 3-foot diameter boulders and had experienced little or no damage as a result of the impacts. Figure 5 shows the rockfall fence at Chute 6.

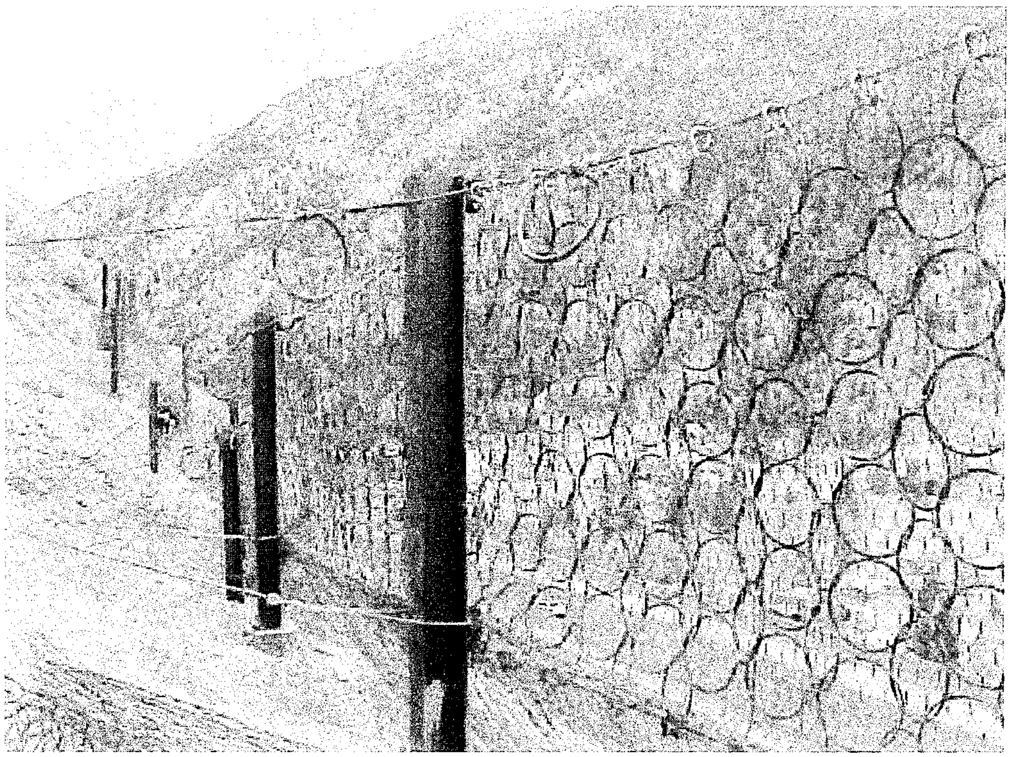


Figure 5. Rockfall Protection Fence at Chute 6

Summary

Many different methods of evaluating, analyzing and prioritizing rockfall were applied to the Georgetown Incline. An array of methods was used to establish effective locations for rockfall mitigation. Although many different evaluation methods were used, rockfall locations and behavior cannot be completely predicted. Trends based on structural discontinuities, slope aspect and climate have indicated were rockfall activity may be more likely to occur, however a rockfall event can occur at a multitude of locations along this alignment at any time. The study was conducted to identify the areas of greatest potential hazard and provide recommendations on mitigation.

INNOVATIVE DESIGN FOR A CHALLENGING HIGHWAY WIDENING PROJECT IN HONG KONG

Trevor Carter[1], Phil De Graf[1], Peter Booth[2], Stephen Barrett[3] and Robert Pine[4]

Abstract: Construction of a novel retaining wall and implementation, for the first time in Hong Kong, of high energy rock fall protection fences contributed to the successful completion of one of the most difficult highway realignment and widening programs ever undertaken in the Territory.

The Tuen Mun Highway is one of the major arterial roads linking Hong Kong to China and provides a busy commuter link between the New Territories settlements and Hong Kong Island. This highway has seen some of the region's worst traffic congestion, and in order to alleviate it, a highway realignment and widening program was selected as the most appropriate solution for increasing capacity. Initial works on the widening were initiated in 1994. However, in August 1995, a rock fall occurred resulting in the loss of life of a driver using the highway. Following this incident, the Contractor claimed the contracted design for the Tai Lam Section was inconstructable and a subsequent mediation of the claim found in favour of the Contractor.

This paper concentrates on certain aspects of the geotechnical design solutions developed to achieve constructability of this challenging highway widening. Some discussion is also presented of the design phase site investigation work, and of the key geotechnical issues that arose during construction.

1.0 INTRODUCTION

1.1 Background

The Tuen Mun Highway, initially constructed between 1974 and 1983, was Hong Kong's first high capacity highway, and still remains one of the busiest commuter links between the burgeoning New Territories settlements, the neighbouring Chinese Guangdong Province and Hong Kong Island. This highway has seen some of the regions' worst ever traffic congestion. In order to alleviate this congestion, a highway realignment and widening program was selected as the most appropriate solution for increasing capacity. In the late 1980's the need to provide climbing lanes and hard shoulders through the uphill sections was established as a key component of any traffic alleviation scheme. Initial construction contracts to achieve widening of the Kowloon bound highway, were tendered in 1992. The Highways Department awarded the works under a Design and Build Contract, with initial construction activities starting in 1994. However, in August 1995, a rock fall occurred

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resulting in a motorist fatality. Following this incident the Contractor claimed the works for the Tai Lam Section were impossible to execute in accordance with the contract, and a subsequent mediation of the claim found in favour of the Contractor. The Coroner, in evaluating the cause of the accident, specifically advised that prior to beginning any further excavation work on potentially unstable slopes (including those at Tai Lam), the areas be thoroughly mapped and analysed, and that if any negligence in execution of these tasks leading to further incidents were proven, the designers could be held criminally responsible.

1.2 The Project

In view of the fatality that occurred in 1995 and the difficulties that had already occurred in developing a workable solution, Highways Department retained Maunsell Consultants Asia Ltd. as the main consultant for the remaining works, who in turn sub-contracted Golder Associates Ltd. (Golder) as the specialist geotechnical consultant. Golder was engaged to (i) examine the stability of the existing and proposed rock cuts adjacent to the highway and (ii) to develop workable rock-engineering solutions for the difficult task of safely widening the highway. The rock engineering design was complicated by:

- the height of the rock cuts;
- the presence of unfavourably oriented "sheeting" joints in the weathered granite;
- the lack of rock fall catchment at the toe of the slopes; and by
- the occurrence of a major weak fault zone that crossed one of the slopes in a sector where high cut-backs were the only viable measure for maintaining the alignment.

In addition, the design had to maintain dual, 3-lane traffic flows, of typically 9,000 vehicles per hour throughout the entire investigation and construction periods, with only occasional lane reductions and off-peak closures permitted. Figure 1 shows the site location and the extents of the four works sections (TL/S1 to TL/S4).

1.3 Geological Backround

The rock along the Tai Lam Section of the Tuen Mun Highway is part of the Lantau Granite, which is locally heavily jointed with sheeting joints and fracture zones exhibiting deep penetrative weathering. This has resulted in onion skin slabbing and a large number of corestones and boulders on the natural slopes above the cuts.

The full range of weathering grades is evident on site. Exposed cut slopes consist mainly of moderately to slightly decomposed granite (Grade II/III) with some areas comprising highly to completely decomposed granite (H/CDG - Grade IV/V). In addition to the low angle (25° to 40°) south-west dipping sheeting joints, the rock mass is typically dissected by three other major joint sets. These include a north-west steeply dipping set, a moderately to steeply northward dipping set, and a third set which dips steeply to the north-east. As the highway trends approximately East-West, but curves slightly as shown on Figure 1, predominant instabilities are dominated by the outward dipping sheeting joints, with release surfaces created by the other three sets.

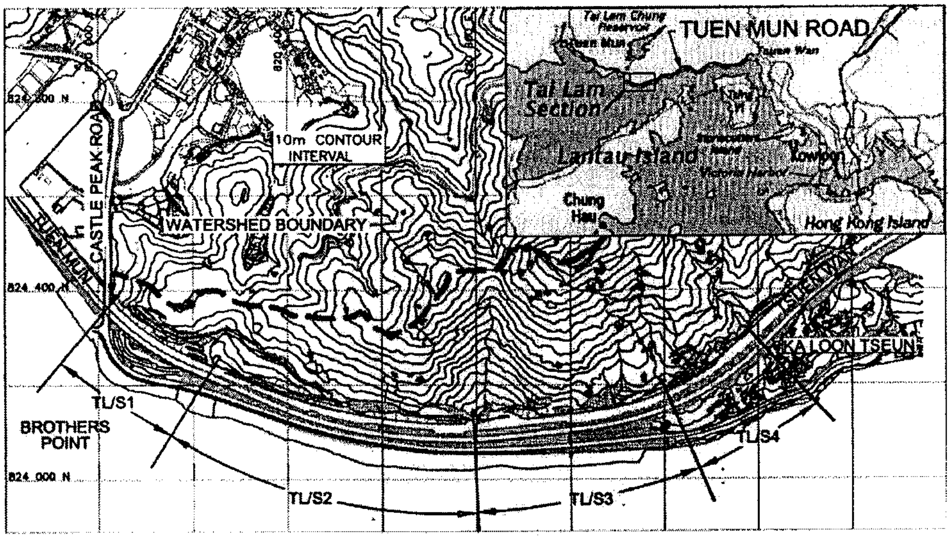


Figure 1: Site Location Map and Boundaries of the Tai Lam Sections (TL/S1 to TL/S4)

2.0 FEASIBILITY ASSESSMENT

2.1 Key Issues

Based on evaluations undertaken during the immediate post-Mediation investigations, key slope issues and required new or remedial construction measures were identified for the various rock cuts along the four work sections. These are presented in Table 1.

2.2 Design Challenges

As is evident from the above tabulation, two recurring rock mechanics themes were of most importance to achieving the design objectives of successfully widening the east-bound lanes by adding an extra climbing lane: (i) addressing the excavatability of the two slope segments where new cuts were required, and (ii) optimising reinforcement and stabilization strategies for the existing cuts and natural slopes that were to remain unaltered.

Addressing these key issues required developing a detailed understanding of the inter-relationships of major discontinuity trends with slopes and compiling data on the shear strength characteristics of the discontinuities controlling stability; both major issues that were deemed in the Coroner's hearings and during the Mediation as being significant limitations to previous construction contracts.

3.0 INVESTIGATION STRATEGIES FOR RISK MINIMIZATION

Acquisition of the data needed for undertaking the required stability analyses and completing the risk assessments to categorise hazard priorities and define appropriate risk-based safety factors for design, necessitated implementation of a very detailed site investigation (SI) program. As outlined in more detail in Carter *et al.*, (1998), key objectives of the investigations were:

1. Assessing the continuity and shear strength of sheeting and other joints, as a basis for design of rock reinforcement and rock slope excavation approaches;

Table 1: Key Slope Issues and Required Works for the Tail Lam Section, Tuen Mun Road

Tai Lam Section	Key Slope Issues	Required Works
TL/S1	High, partially unstable rock slope prone to rock falls. 10 m wide clay-infilled shear/fault zone. Likely stability complications due to high pore water pressures. Boulder zones and segments of potentially unstable natural slopes above the cut slopes.	Undertake extensive excavation to maintain optimum alignment. Cut back the toe of the 50 m high existing rock cut by about 14 m. Create tied-back retaining wall to restrain degraded material within core of shear/fault zone. Carry out boulder stabilization and install rock fall protection/barriers.
TL/S2 & TL/S3	High, unstable rock slopes prone to rock falls/slides on sheeting joints. Boulder zones and segments of potentially unstable natural slopes above the cut slopes. Undercut sheeting joint blocks and zones of buttressed sheeting joints with inadequate drainage provisions.	Excavate only a minimal segment at the west end of the TL/S2 cut to achieve required alignment. Realign highway into central median to create required space for climbing lane. Carry out substantial rock face stabilisation of existing cut slopes to reinforce sheeting joint structural instabilities (incorporating dowels, buttresses, drape mesh and drainage holes, as necessary). Carry out boulder stabilization and install high energy rock fall protection/barriers.
TL/S4	Existing cut slope comprised of completely decomposed granite (CDG) with core stones with spines of rock. Boulder zones and segments of potentially unstable natural slopes above the cut slopes.	Excavate new cut-back into existing shotcreted and supported CDG slope and remove and/or cut core stones appropriately. Install array of soil nails for slope stabilization. Carry out boulder stabilization and as required rock fall protection

2. Determining the transient response of the groundwater system (particularly in the vicinity of the sheeting joints) to heavy, short-duration rainfall events, to allow definition of adequate drainage and pore-pressure assumptions in design;

3. Defining the blocks which were kinematic free to fall from the face;

4. Defining the statistical variability, size and roughness conditions of kinematically free blocks, as well as assessing the size and distribution of boulders in the boulder fields above the cut slopes;

5. Defining the detailed topographic profiles of the slopes and cuts and determining appropriate coefficients of restitution as a pre-cursor for realistic rock fall trajectory modelling, and

6. Assessing the depth, characteristics and configuration of the CDG zones, core stones in the upper weathered zones and fault zones.

4.0 DESIGN CONSIDERATIONS

Based on the information collected from the site investigations and field assessments, four different design solutions were formulated to deal with the different field conditions encountered in various parts of the Tai Lam slopes: (i) temporary tensioned anchors to pre-support the proposed cuts and extensive permanent rock doweling to stabilise the final faces; (ii) a novel tied-back retaining wall to stabilise the 10 m wide weak fault zone that transected the centre of the TL/S1 slope segment; (iii) high capacity rock fall control fences and draped mesh to deal with potential rock fall hazards; and (iv) hybrid soil nail and rock dowel support to reinforce the CDG slope sections with interspersed large core stones and rock spines.

Although there are many individual areas of the overall project, where specific design and construction measures are noteworthy, the remainder of this paper will concentrate largely on the major works undertaken in the vicinity of the TL/S1 cut at the west end of the project alignment and the rock fall mitigation measures implemented on the natural slopes.

5.0 TL/S1 – ROCK CUT EXCAVATION AND SUPPORT

Three significant rock-engineering issues required solution for the TL/S1 section:

1. the permanent stabilization of a major fault zone exposure, of approximately 15 m width and some 23 m height, with a true thickness of about 10 m with a dip of 78° to the north;

2. reinforcement of marginally stable sheeting joints and tetrahedral and wedge shaped blocks formed by intersection of prominent sheeting and cross-joints; and

3. control of groundwater pressure heads, as data from the existing cut indicated these were unacceptably high under typhoon conditions.

The new rock cuts required at TL/S1 included removal of the previously existing rock knob on the south side of the highway and pushing back the north side of the cut into the ridge (Figure 2). This resulted in increasing the north cut height from 50 m to 60 m. Because of the requirement to keep the highway open at all times, all rock excavation had to be completed mechanically.

Design of the rock cut geometry for the new set-back required at TL/S1 was conventional, with the exception that wedge and block size geometries of controlling rock blocks were determined by a combination of field mapping and photogrammetric analysis methods. This data set was then used as a basis for discrete deterministic block size evaluation using the FRACMAN/ROCKBLOCK computer code (Dershowitz and Carvalho, 1996). Stability assessment of the 3D blocks was undertaken both in 3D using the in-house program GOLDPIT and in 2D using conventional limit equilibrium plane and wedge failure analysis methods (Hoek and Bray, 1981). Prior to and during these design analyses it became apparent that the key

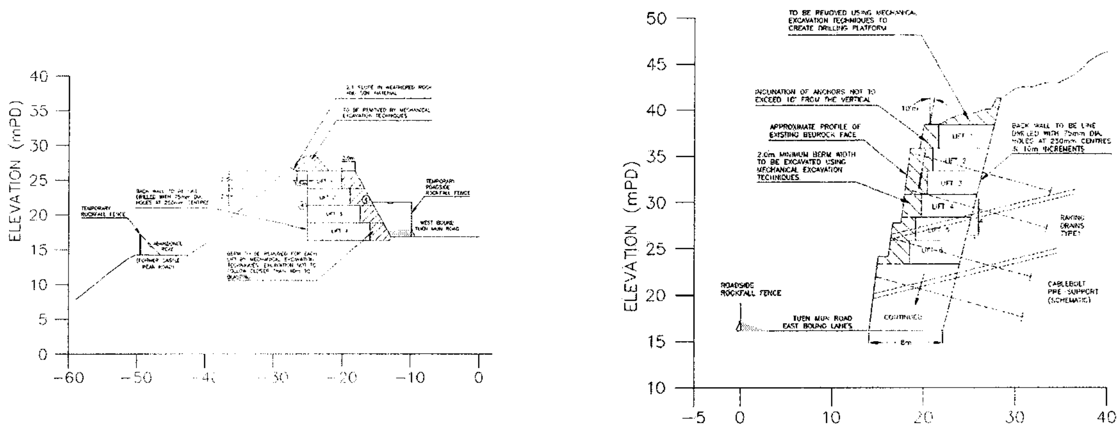


Figure 2: Typical Cross-Sections Through South and North Side of TL/S1 Showing Designed Cuts

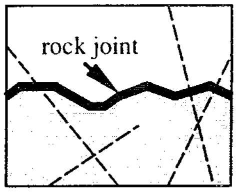
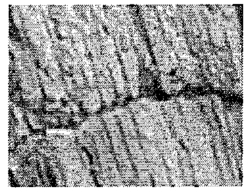
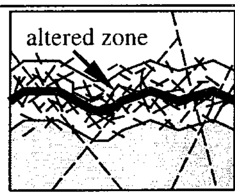
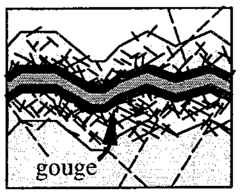
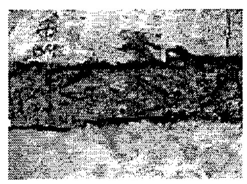
uncertainty relating to the excavation and support design was ensuring an adequate understanding of the continuity and shear strength characteristics of the controlling sheeting joints. Although a wide range of different joint characteristics could be recognized in the field, in order to simplify design analysis yet realistically ascribe appropriate design strength parameters for the weathered controlling sheeting joints, three basic joint models were defined. These were then characterised by use of two principal constitutive failure criteria, as summarised in Table 2.

The other major uncertainty affecting the rock support design, was groundwater pressure. Of most critical concern was attempting to define maximum credible design heads and pressure response and decay behaviour of porewater in the sheeting joints under typhoon style, extremely heavy, short duration rainfall events.

In order to gather this key information, the site investigation boreholes were instrumented with vibrating wire piezometer arrays and connected to a series of dataloggers. Figure 3, shows two quite different responses recorded by the VW piezometers as a result of the 280 mm single day Typhoon event of June 9th 1998. As is evident, the tip in borehole PH1A in a relatively freely draining sheeting joint shows an almost 3.5 m jump in pressure head, with about a 5 day decay period, similar to the responses recorded by Richards and Cowland (1986). The tip in borehole PH7, however shows a steady increase in pressure head with no short term decay behaviour. This head increased to a sustained plateau, which was believed to be controlled by overflow from one groundwater regime to another.

These plots highlight the marked difference in groundwater behaviour from one place to another within a fractured rock mass and emphasise the need for careful examination of such issues in any site investigation in complex rock conditions such as characterise not only the Tai Lam area but also many other parts of Hong Kong. In the TL/S1, 2 and 3 slope continuous sheeting joints created potentially unstable conditions over substantial areas. For the extreme condition design case, in addition to

Table 2: Sheeting and cross joint characteristics and strength criteria.

Strength Control	Typical Geometry	Photograph	Governing Strength Equations
Joint Strength (Rock-Rock)			Barton-Bandis Criteria for Joints with Rock-Rock contact (Barton & Bandis, 1990) $\phi_{(peak)} = JRC.\log_{10}(JCS/\sigma_n) + \phi_r$
Wall Rock & Asperity Strength (weathered joint margins, Grade II/III upwards)			Hoek-Brown Criteria for Degraded Sheet Structures (weathered wall-rock or gouge material characteristics dominate behaviour) (Hoek, 1990) $\phi = Arc\tan\frac{1}{\sqrt{4h\cos^2\theta - 1}}$
Joint Infill/Gouge (no asperity contact)			where: $h = 1 + \frac{16(m\sigma_n + s\sigma_c)}{3m^2\sigma_c}$ and $\theta = \frac{1}{3}(90 + Arc\tan\frac{1}{\sqrt{h^3 - 1}})$ and m and s are Hoek-Brown constants, with σ_c as the unconfined compressive strength

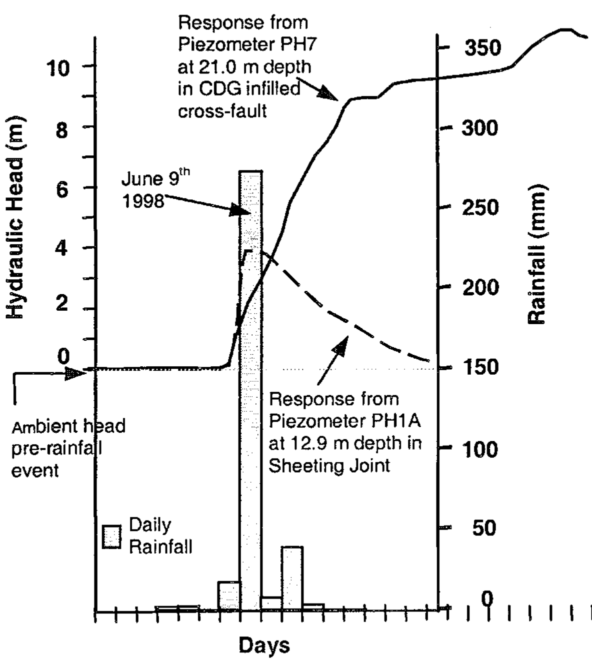


Figure 3: VW Piezometer Response to June 9, 1998 Typhoon Event

systematic dowel support, raking drains were specified to mitigate any potential groundwater pressure build-up.

Monitoring of the piezometers was automated and an effective warning system was developed through understanding of the influence of transient pore pressure responses on slope instability. Rainfall events, and subsequent transient increases in porewater pressures were quantified in terms of increased risk of failure, and an emergency response plan implemented.

Support for the rock mass in the TL/S1 rock cut sections was designed for both short term (temporary) and long term (permanent) conditions. Short term support was required to pre-support the large potentially unstable blocks, which could be exposed as excavation proceeded. It consisted of raked, tensioned cable anchors spaced on a square pattern to achieve an operational safety factor of 1.2 consistent with Hong Kong practice (GEO, 1984). Long term support consisted of passive dowels to anchor specific definable blocks to design factors of safety of 1.6 to 1.8. These risk-based factors of safety are higher than the standard values recommended by the Geotechnical Engineering Office (GEO, 1984), Tables 5.1 and 5.4, because of the large number of potential blocks and high consequence of failure.

Two types of permanent dowel were specified; dowels installed within 20° of normal to a sheeting joint which were assumed to act in pure shear, and dowels installed at more than 20° from normal to a sheeting joint which were inferred to act in combined tension and shear. Ultimate and working load capacities for design of individual dowels was based on the improved recommendations for dowel capacity and shear deformation provided by Spang and Egger (1990), rather than the Bjurström (1974) approach previously adopted in Hong Kong.

6.0 TL/S1 RETAINING WALL

The retaining wall to stabilise the major fault at TL/S1 represented one of the most significant geotechnical and construction challenges of the project. The wall structure, as finally designed comprised a 1-2 m thick vertical reinforced concrete retaining wall of up to 23 m height, tied to bored piles anchored back to the competent rock mass behind the fault zone. The anchors consisted of double corrosion protected, 36 mm diameter tiebacks, bonded 3 m into the hangingwall of the fault/shear zone. The design of the wall was carried out to ultimate limit state (ULS) criteria, with earth pressures being estimated using the Trial Wedge Method.

As illustrated in Figure 4, the fault zone strikes obliquely across the highway at TL/S1. Rock, soil and groundwater conditions associated with this fault zone were known from the 1976 original construction to be adverse (Slinn and Grieg, 1979), to the extent that a major failure of this entire zone had occurred when the initial box cut excavation had only been part completed. Consequently, multi-staged, top-down excavation and cast-in-place retaining wall construction sequences were specified.

Ensuring adequate temporary pre-support in advance of excavation and retaining wall construction was key, due to the low strength and significant thickness of the fault zone. As shown on Figure 4, the temporary support was achieved using the bored pile wall covering virtually the entire interpreted exposure area of the fault zone. Piles were bored from wall crest elevation to 4 m below the excavation toe. Pile diameter was restricted to 457 mm due to rig size limitations, related to the extremely difficult access to the drilling location 30 m up on the existing cut slope. Most of the drilling went smoothly, however some of the challenges that had to be overcome in the pile drilling included (i) cutting down in a couple of cases through the steel bar tie-back anchors restraining the existing toe buttress, and (ii) dealing with collapsing pile holes due to the introduction of drilling fluids

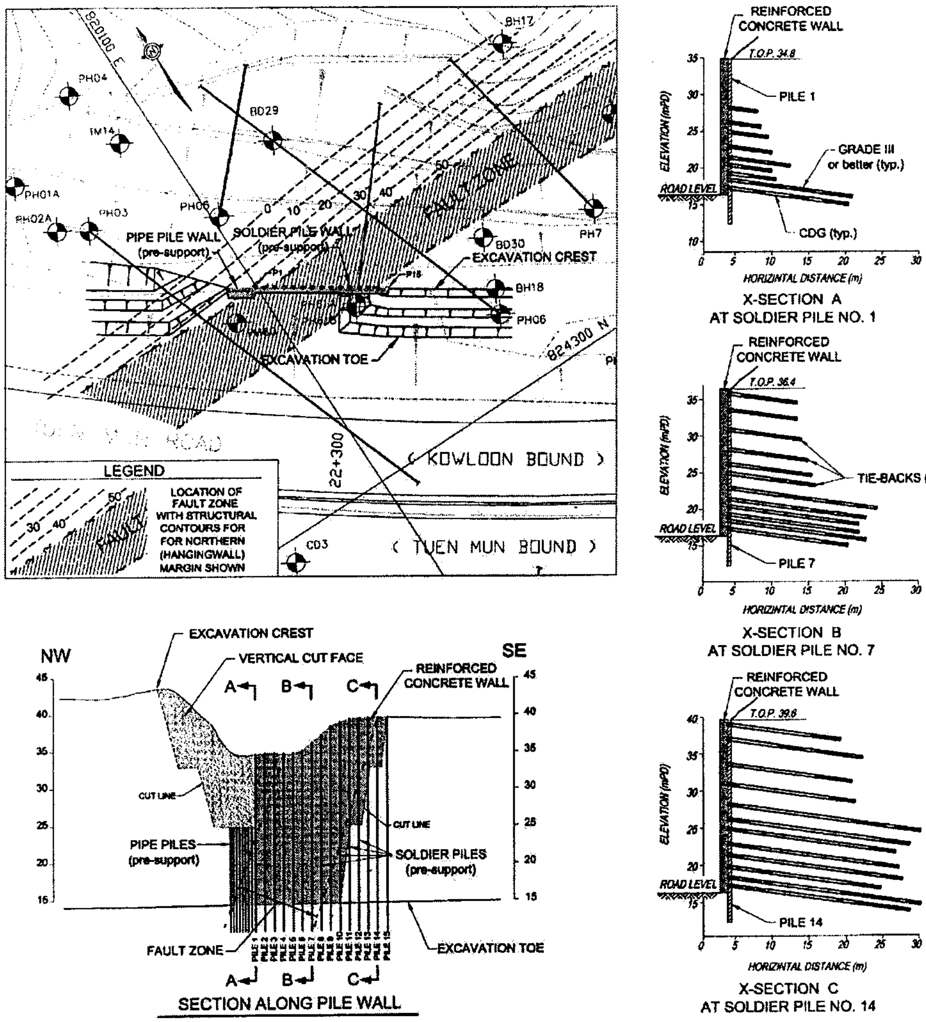


Figure 4: Schematic View of TL/S1 Tied-Back Retaining Wall

into the already low strength fault zone CDG material. These challenges were overcome through persistence and flexibility in the construction process, although often substantial delays were endured.

As shown in the photographs in Figure 5, at each stage of the top-down construction sequence, a 2 m high segment of the bored pile wall was exposed, allowing drilling of the tie-back anchors prior to in-place casting of the RC wall itself. Each of the tie-backs was drilled at a nominal 10° down angle into sufficient length of competent grade I-II granite to allow adequate anchor bond length (as shown in the three typical sections included within Figure 4). As excavation and wall concreting was completed on a lift-by-lift basis, drilling of 15° upward inclined raking drains was undertaken to attempt to maintain control of any potential groundwater build-up behind the fault zone. In this way, wall stability was maintained and the construction process simplified.

7.0 ROCK FALL CATCH FENCES

Although most of the hazardous sections of the high rock slopes along the Tai Lam segment of the Tuen Mun Highway had been covered with draped mesh either as part of the original 1970's construction works, or had been draped in subsequent

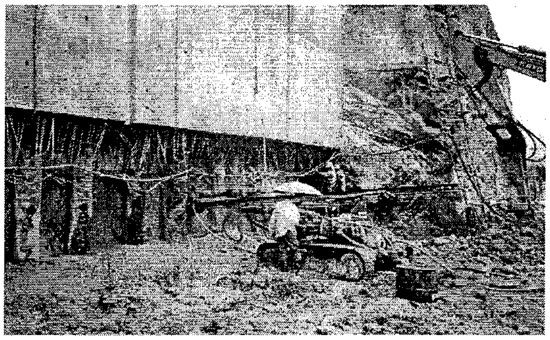


Figure 5: Drilling of Tie Back Anchors Through Bored Piles Prior to Placing Reinforced Concrete Facing

remedial works campaigns, little attention had been previously paid to possible large scale rock fall issues. In view of the long-term hazards posed by possible dislodgement of large, partially stable sheeting joint slabs and/or relict core-stone boulders, the risk-based design approach to the stabilization program demanded much higher levels of rock fall protection than had hitherto been practised in Hong Kong. This also applied to the short-term hazards posed by construction of the works themsleves.

The core-stone boulders above the cuts posed a particular problem. Although some of the larger, more easily accessed boulders could be stabilized by construction of site specific concrete butresses and tie-backs, because of the large number of relatively small boulders, it was deemed necessary for both the short-term construction period as well as the permanent works, that high capacity rock fall fencing would be required. For the construction period it was also recognized that, even though the cut slopes below the proposed new high capacity rock fall fences were going to be stabilized (bolted and meshed during the works), any rock falls or equipment falling from the cut slopes would have to be prevented from reaching the highway.

Due to the large number of boulders present on the slopes and in particular to assist in laying out the rock fall fences, the natural slopes above the slope cuts were initially classified in terms of the overall risk of boulders reaching the highway. Moderate and high risk areas were further assessed to determine the general frequency, size and stability of boulders in those areas.

Assessments of the potential trajectory paths of probable boulder and or rock fall releases from high on the existing slopes, highlighted the need for much higher capacity fences than had previously been used in Hong Kong. Discussions were therefore held with GEO and other Government agencies to allow use of such measures, based on demonstrated precedent in Europe and North America. As shown on Figure 6, which illustrates the permanent Geobrugg fence design actually employed along the TMR slopes, these high capacity fences are very different from the conventional rock fence arrangements included in CED drawings C 2501A and C 2502B, (ref. CED 1994).

Optimising the positioning and sizing of the Geobrugg fences for both short-term construction control of rock falls and for long-term protection, required multiple computer simulation runs of falling, rolling and bouncing boulders. To undertake the analyses using a program which could meet GEO's rigorous verification requirements, Golder developed an in-house program (Rockfal3) which could address all key parameters required for modelling the most critical bouncing boulder trajectories, namely: boulder diameter, shape, density, release velocity and

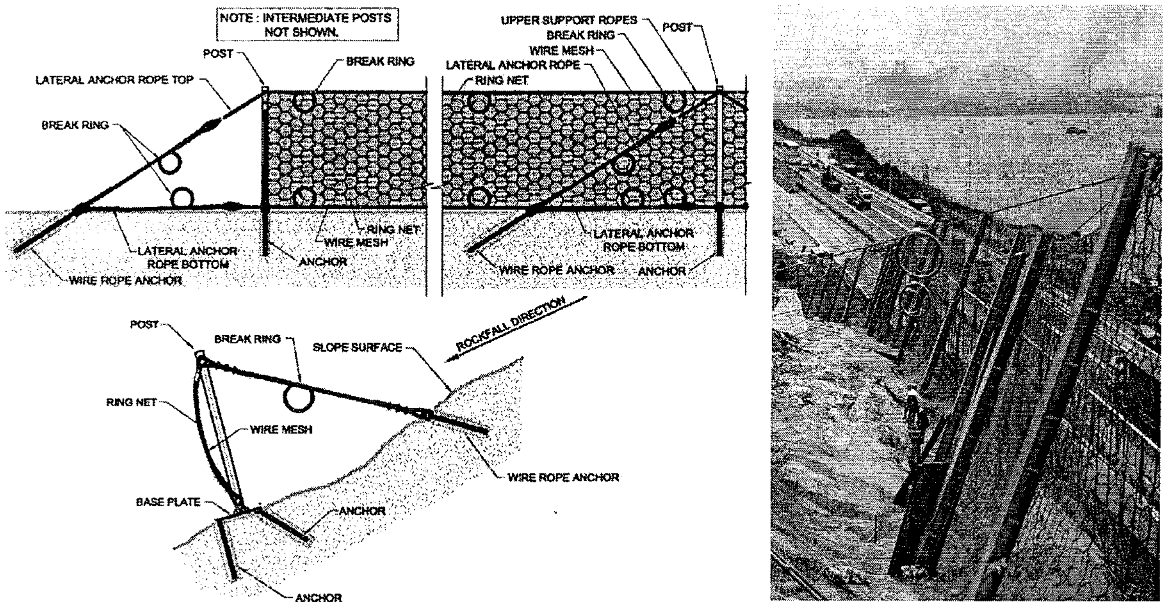


Figure 6: Rock Fall Fences Installed at the Crest of the Existing Cuts at TL/S2

slope profile, type and roughness and coefficients of restitution for impacts between boulders and the substrate (Golder, 1997). As shown in the typical output plot included within Figure 7, this probabilistic program predicts not only the boulder trajectories, but statistically quantifies boulder bounce height and total kinetic energy along the trajectory paths and thus provides all the output parameters critical for adequately sizing and positioning rock fall fences.

For the short-term construction condition, a critical aspect for layout of any temporary roadside fences was the limited rock fall catchment between the fence location and the slope toe. Based on the trajectory modelling it was determined that minimum temporary fence height and capacity should be typically 3 m and 250 kJ, except at the east end of TL/S4 where fence heights had to be increased to 5 m to prevent overtopping. Initial trajectory modelling and precedent design reports had demonstrated the beneficial effects of a "bounce cushion" in front of the roadside fence to help minimise impact magnitudes. However, because of the limited catch zone width at the toe of the slope, the contract was tendered with slightly higher capacity fencing and no bounce cushion. The Contractor however opted to incorporate a sand/CDG cushion on top of the highway inside the roadside fence for additional safety.

For the long-term permanent fencing, the output from Rockfal3 was used to plot optimum design layouts and determine appropriate fence capacities. For all slope areas, the basic procedures were: i) determine fence plan positions relative to slope crest and toe based on predicted trajectory patterns, ii) establish required fence heights needed to capture the 99% probability level of trajectory impacts, and iii) determine required fence capacity for this 99% probability level. Based on these analyses, slope positioning of the optimal fence locations were re-arranged to allow use of just three standard permanent fence types: a 3 m high, 250 kJ fence; and two 5 m high fences with 250 kJ and 750 kJ capacities. However, in some instances it was found that fall path length needed to be minimised further

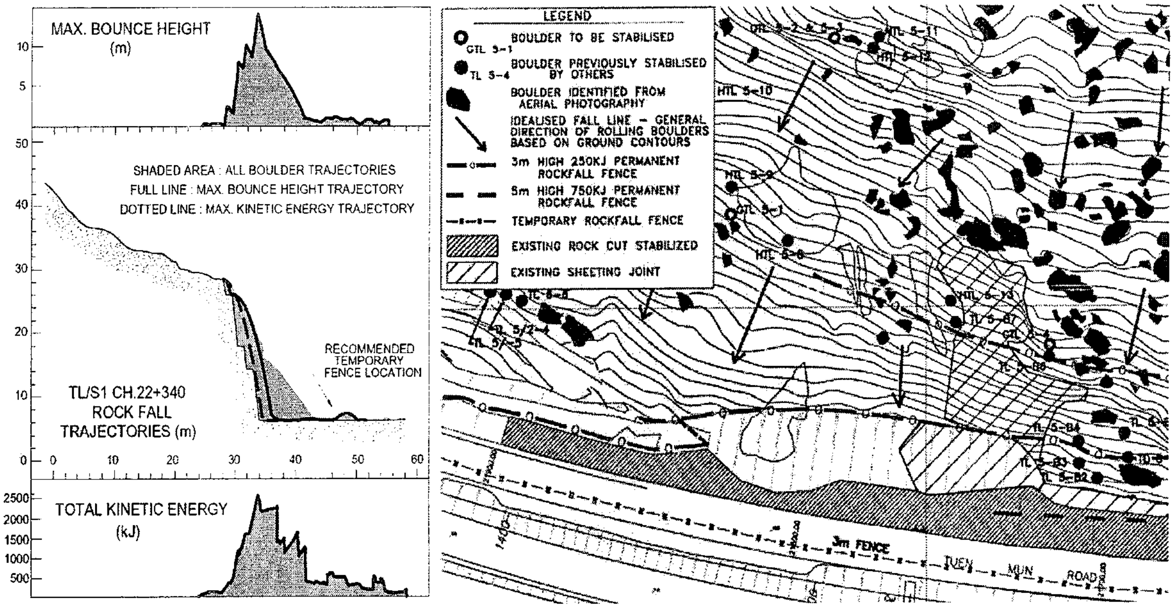


Figure 7: Typical Output from Rockfal3 and Example of Fence Layouts for Natural Slopes Above TL/S3

still, by using upper, intermediate and lower slope fences. This, in turn provided a substantial measure of redundancy in critical locations.

8.0 CONCLUSIONS

Demanding conditions for the widening of the Tuen Mun Highway were overcome through detailed geotechnical investigations, the implementation of innovative slope design solutions, and stringent risk-based safety management requirements. As with many other of Hong Kong's problematic high slopes this project has unquestionably raised public awareness and increased the local profile of geotechnical and geological engineering. The project has gone from attracting negative newspaper headlines due to potential litigation and accusations of professional negligence to a model example of a well executed construction project. This is attested to by the impressive 23 m high tie-back retaining wall at TL/S1 and Hong Kong's first ever high energy rock fall protection fence system.

ACKNOWLEDGEMENTS

The authors would like to thank the Highways Department for their kind permission to publish this paper. We would also like to acknowledge the contributions made to the effective execution of the investigations and design, by the staff of the Hong Kong Highways Department and Geotechnical Engineering Office, Maunsell (Asia), Lam Geotechnics, and particularly to the authors' many former and current colleagues at Golder Associates (HK) involved through the several years that were required to successfully complete the work. In particular we would like to acknowledge the extensive contributions made by Dr. Alan Kwong, Neil Ng and Raymond Leung to the design of the retaining wall, and to Chris Mah and Drs. Bill Roberts and Kauntsai Lee on aspects of the block sliding, risk assessment and rock fall analyses respectively. In addition, specific thanks are due to Dr. Laurie Richards for his insightful reviews and guidance throughout the design phase.

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Behavior, Design and Reliability of Highly Flexible Rockfall Protection Systems for Highways

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Abstract

Highly flexible steel-net protection systems are a cost efficient method to protect inhabitants and infrastructure of mountain regions. Laboratory tests and real 1-to-1 field tests are used to optimize these structures. This paper presents the quasi-static laboratory experiments with net-rings as well as the dynamic rockfall experiments with single and multi-field nets. The field tests use a high performance reinforced concrete boulder containing a three dimensional acceleration measuring device. The deceleration of the boulder is recorded and therefore the braking response of the system is quantified. The tests are back-calculated with an explicit finite element model consisting of nets, cables and brake elements. The model is integrated in a probabilistic simulation program based on the first order reliability method to evaluate the structural safety of the systems.

1 Introduction

Rockfalls pose a serious danger to mountain highways in Switzerland. Annual investments on defense measures are of the order SFr. 7.0 ($ 4.2) million. Steel-net protection systems have become a cost efficient method to protect highways from rockfalls as opposed to concrete galleries. The systems consist of wire-ring nets or diagonal-cable nets supported by steel cables with inelastic brake elements and columns. They are installed like fences above transportation lines and residential areas to stop falling rocks with masses of up to 10 tons and velocities of up to 60 mph. Within the last twelve years the energy absorbing capacity of the systems has been improved by a factor ten [1].

Engineers must design these light weight steel net systems to stop falling rocks with kinetic energies of over 5000 kJ within about 10 m. The amount of energy that needs to be absorbed and the stopping distance are a function of the geology and the topography of the rockfall site. Since the nets are often placed in steep terrain, another design criteria is the maintenance cost. To date, there is neither a reliability concept nor a calculation guideline for steel-net systems. This is surprising considering the safety requirements of these protective systems. To overcome practical design problems standards for testing and certification of flexible rockfall barriers have been applied in Switzerland since 2001 [2].

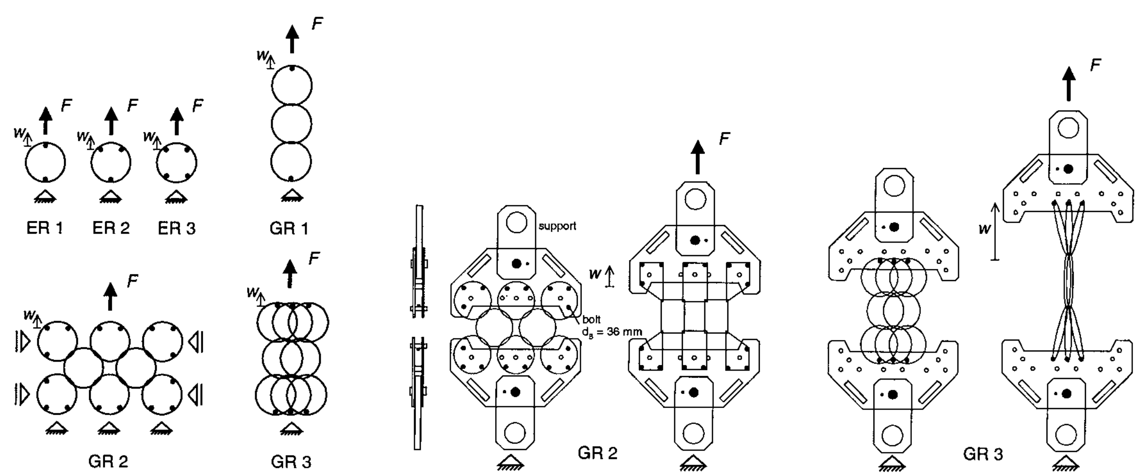


Figure 1: Experimental set-up of quasi-static ring experiments, test series with single rings *ER 1* to *ER 3* and with groups of rings *GR 1* to *GR 3*.

In addition to the certification of protection systems advanced simulation models are now being developed to reduce field testing and to improve the design of the structures and their foundations. A research project with the goal to combine specialized field experiments and numerical modeling was started in year 2000. This project is jointly carried out with the Federal Research Institute WSL, the manufacturer of the ring nets and the Federal Institute of Technology, ETH in Zürich. Two different simulation tools have been developed. Both tools are based on the finite element method and use an explicit time integration method. One is a simplified model which applies cable and bar elements suited for reliability analysis. This cable element model is treated within the present paper. The other numerical model, developed at the ETH, is an detailed finite element program combining special purpose ring elements with large deformations, nonlinear materials, contact and sliding effects as well as friction. The application field of both models is to optimize product development and the protective capacity of the structures. To calibrate and to verify the numerical simulations a multistage testing programme with new measuring methods was devised.

Thus, experimental research, numerical modeling and reliability theory are combined to quantify the influence of the predominant parameters on the safety of rockfall protection systems. Each will be discussed in the following.

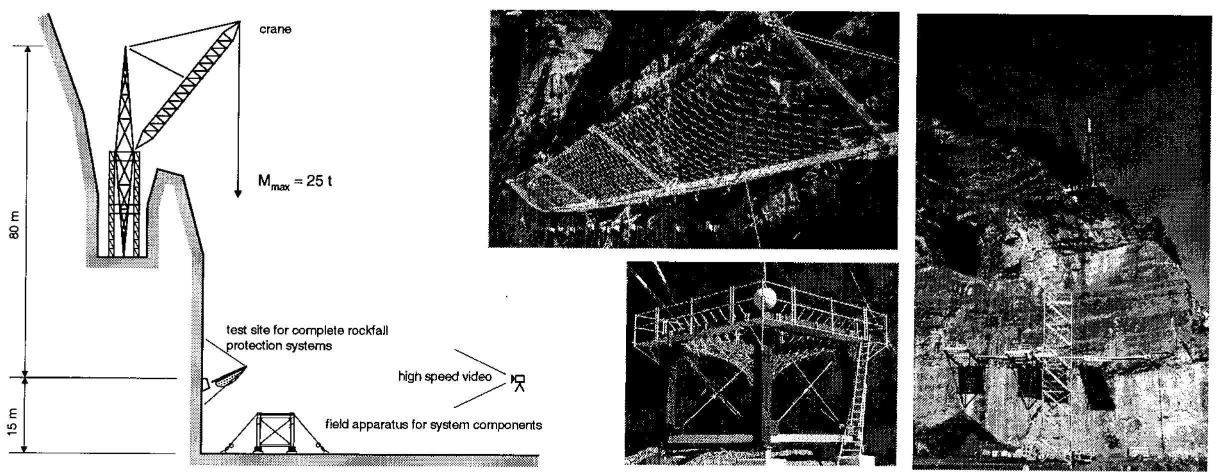


Figure 2: Real 1-to-1 rockfall test-site in Walenstadt (Switzerland) for three field systems and single spanned nets.

2 Experimental research

The experimental programme consists of quasi-static laboratory experiments with wire rings and rockfall field experiments with single spanned nets and complete systems. Within the present paper the focus is on those experiments which were used for the development and the validation of the cable element model. More complex experiments have been carried out to validate the full finite element program, but these are not treated in detail. See [3] or [4] for more details.

2.1 Quasi-static laboratory experiments

To determine the non-linear behavior and the energy capacity of the wire rings, quasi-static tensile tests were performed in the laboratory. Different configurations of single rings and groups of rings were investigated (see **Fig. 1**). The results of the tensile tests *GR 2* and *GR 3* are used to validate the material law of the cable elements modeling the net behavior. The 300 mm diameter net ring consists of 3 mm steel wire windings. The number of windings depends on the desired ring resistance. The yield stress of the steel is 1770 N/mm^2. Typical results are shown in **Fig. 8**. The force curves show a period with little resistance. Afterwards, when the elements begin to elongate, the resistance force increases dramatically. In the first loading phase, bending is the primary deformation mode.

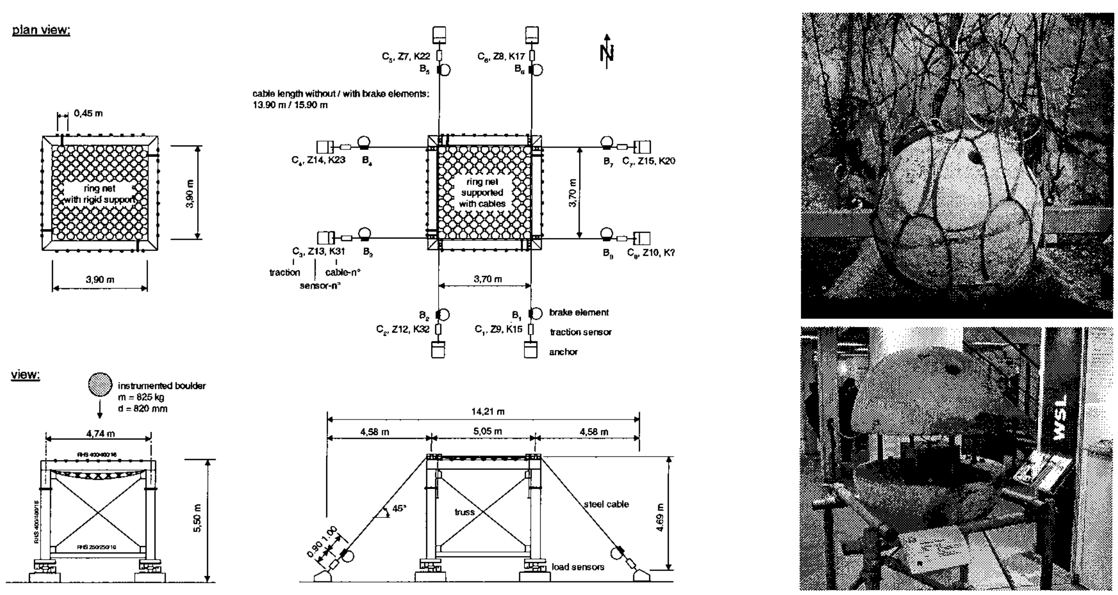


Figure 3: Plan and elevation of the field apparatus for experiments with single spanned nets (left); instrumented concrete sphere with a diameter of 820 mm and a mass of 825 kg (right).

3 Rockfall experiments

3.1 Test site

The field tests were performed in the Swiss Federal Rockfall Test Site near Walenstadt in Canton St. Gallen (see **Fig. 2**). The site is used for type-testing, research purposes and product development. The boulders have a maximum mass of 16 tons and are released from a crane. They can be dropped a vertical distance of up to 60 m before the impact with the protection systems. To prevent ground contact during the deceleration phase the complete systems with three fields are installed in a vertical rock face 15 m above the ground. The free fall test guarantees the reproducibility of the velocity, the (vertical) trajectory and the location of the impact.

3.2 Rockfall experiments with single spanned nets

The dynamic behavior of the single components is investigated using a specially constructed field apparatus. The experiments on the field apparatus, shown in **Fig. 3** are realized with system components in a simplified arrangement with reduced degrees of freedom. Three test systems were defined. During the first phase, square nets with blocked displacements at the boundary were performed. During the second phase, square nets were suspended from cables and during the third phase, square nets were suspended from cables with integrated brake elements. Thus, in the first test phase, the behavior of the nets was determined, in the second phase the supporting cables and in the third phase, the brake elements. This component-wise testing procedure was selected in order to understand the behavior of the single elements within the system. Thus, single element models could be developed and tested. Furthermore, the simplified configuration reduces the number of model parameters and makes integral and accurate measurement possible. The experiments with system components are used for the validation of the detailed finite element program.

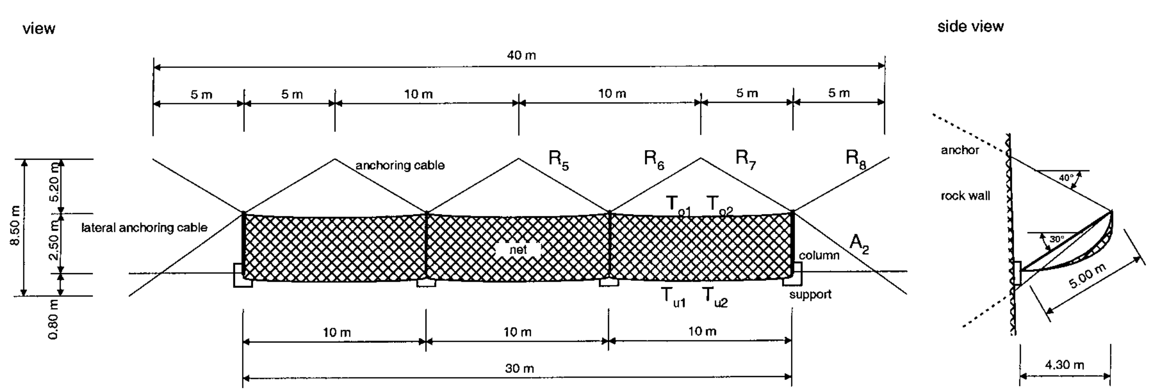


Figure 4: View and side view Installation of complete rockfall protection system.

3.3 Rockfall tests with complete protection systems

Rockfall field tests with complete systems were additionally performed. The results of the experiments with complete systems were applied for the verification of the cable element model.

The tested system shown in **Fig. 4** has three spans of 10 m and a column height of 5 m. The loading corresponds to approximately 80 % of the energy absorbing capacity of the system. The geometric configuration of the tested system is similar to the configuration in practice. The static system, the location of the single components and the measuring devices are indicated in **Fig. 5**.

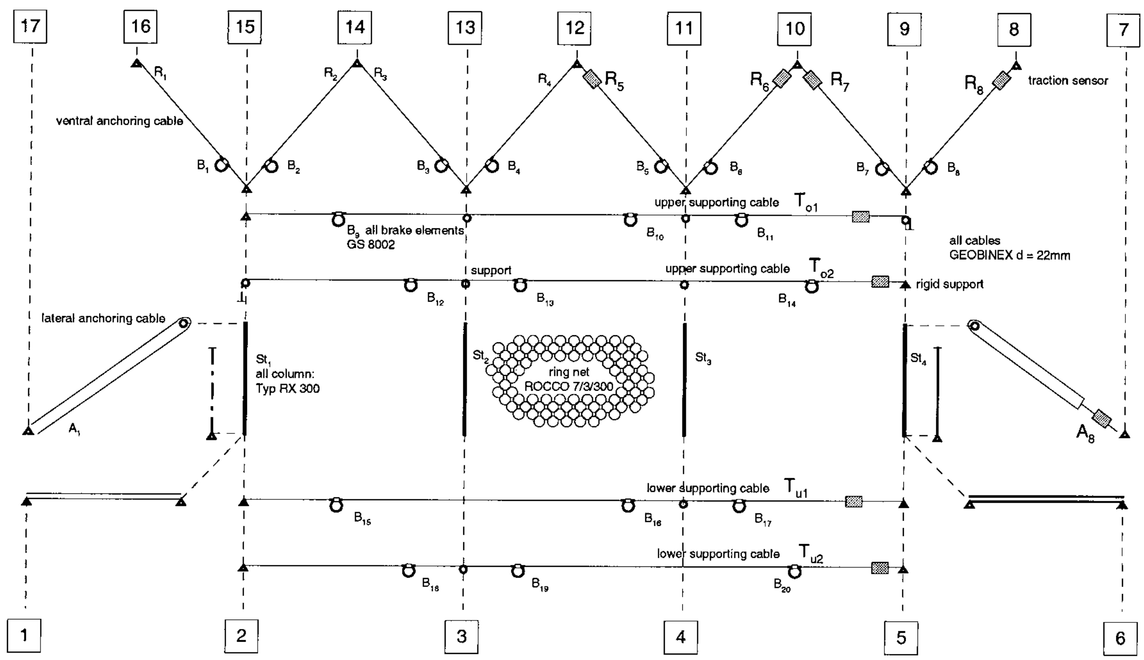


Figure 5: Construction of rockfall protection system and location of the traction sensors.

3.4 Measuring techniques

Several measuring techniques are used to record the braking process of the boulder in the net. During the experiments with single spanned nets the horizontal and vertical bearing reactions were measured with load cells fixed at the four columns of the field apparatus. Eight load sensors measured the tensile forces in the cables. To avoid dynamic effects these sensors are directly fixed to the eight anchoring points surrounding the frame.

The forces in the anchoring and in the supporting cables of the complete system were measured with nine load sensors, see **Fig. 5**.

To obtain the resulting braking force which acts on the net in a direct manner, accelerometers and a computer control unit have been integrated into the boulder. The boulder consists of two semi-shells of fiber reinforced high performance concrete with a total mass of 825 kg (see **Fig. 3**). This instrumented rock is dropped from heights of up to 32 m and can withstand impact decelerations of 100 g. The integral parts of the data acquisition system is a micro-controller and a 12 bit A-D transformer. The data are sampled with 20 kHz and written continuously in a ring buffer. After triggering, the data are retained in the static RAM-memory until down-loading is completed. This deceleration is tracked with eight capacity accelerometers. The range of the sensors is ± 50 g with guaranteed overload of maximum 1000 g. The accelerometers measure the stone's deceleration over time. A special integration procedure has been developed to obtain the velocity and position of the rock from the measured deceleration.

Beside the force sensors and the accelerometers, the third independent measuring technique is a high speed video system with two cameras recording 250 frames per second. The video system is synchronized with the force sensors. These recordings document the entire braking process.

4 Simple Finite Element Model

The cables, the brake elements and the net of the protection system are modeled with lumped masses which are connected with tensile elements having the mechanical behavior of each component. The lumped masses are located at the intersection points of different element types as well as at the support of the cables and columns (see **Fig. 6**). A cable consists of different cable elements. The columns are modeled with two lumped masses which are connected via a truss element.

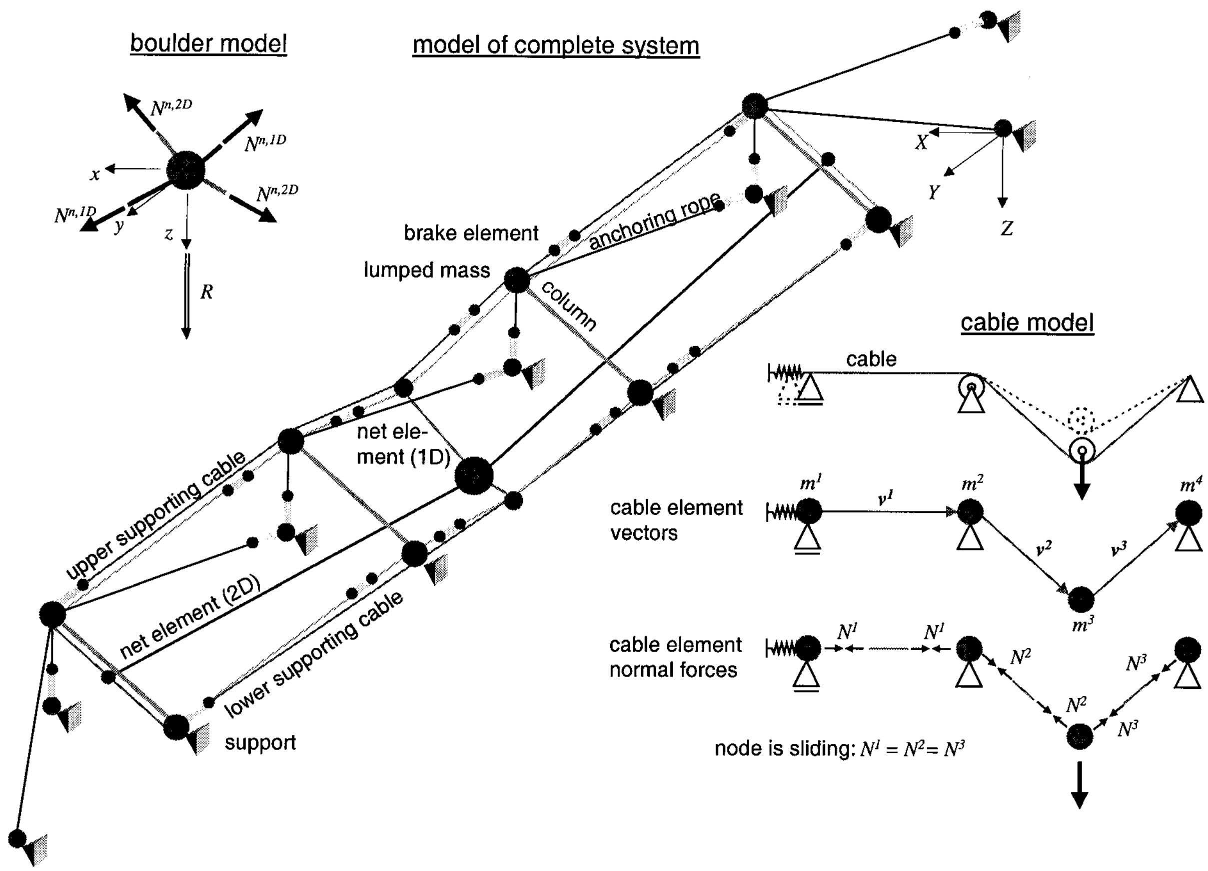


Figure 6: Simple finite element model of a three field system.

The impacting boulder is considered as a single mass without a mass moment of inertia. The impact between net and boulder is neglected. In the first time-step of the calculation the boulder mass has the impact velocity $\dot{\boldsymbol{d}}_0$.

4.1 Non-linear dynamic analysis

The equation of motion of a node n with the lumped mass m, the resulting node force $\boldsymbol{q}(t)$, the gravity $\boldsymbol{g}$ and the displacement vector $\boldsymbol{d}$ is.

$$m\ddot{\boldsymbol{d}} + c\dot{\boldsymbol{d}} = -\boldsymbol{q}(t) + m\boldsymbol{g}$$

For the solution of the nonlinear equation, an explicit time integration algorithm based on the central difference method is applied. Due to the lumped masses and cable elements the global system of equations is decoupled and can be inverted easily to find the deceleration at a single node, n. In the numerical model the damping is $c = 0$.

The displacement increment vector $\Delta\boldsymbol{d}$ of a node n is obtained from the equation

$$\Delta\boldsymbol{d} = \left(\frac{m}{\Delta t^2}\right)^{-1}\left(-\boldsymbol{q}(t) + m\boldsymbol{g} + \frac{m}{\Delta t^2}\Delta\boldsymbol{d}'\right).$$

$\Delta\boldsymbol{d}'$ is the node displacement between time $t - \Delta t$ and time t. The normal force N^s of a cable element is calculated with the sum of the elongation of the cable elements $\boldsymbol{v}_t^s$ connected to one cable.

$$N^s = EA\varepsilon^s \quad \text{with } \varepsilon^s = \frac{\sum_s \left\|\boldsymbol{v}_t^s\right\|}{\sum_s \left\|\boldsymbol{v}_{t_0}^s\right\|} - 1$$

$\boldsymbol{v}_t^s$ and $\boldsymbol{v}_{t_0}^s$ are the element vectors at time t and t_0. A node can slide if his two neighbor elements belong to the same cable ($N^s = N^{s+1}$). The resulting node force $\boldsymbol{q}(t)$ is obtained from the sum of the normal forces of the neighboring elements.

$$\boldsymbol{q}^n(t) = \sum_s \frac{\boldsymbol{v}_t^s}{\left\|\boldsymbol{v}_t^s\right\|} N^s$$

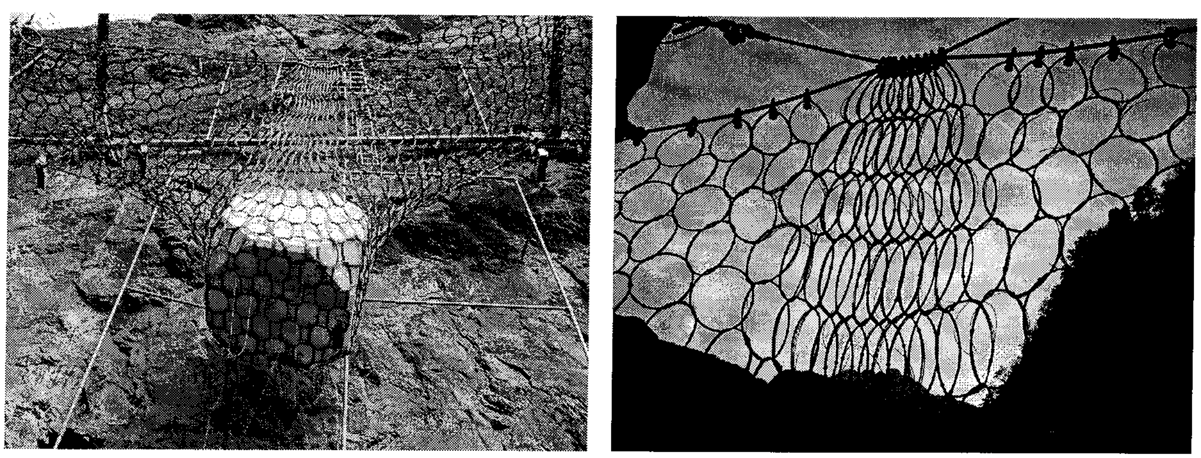
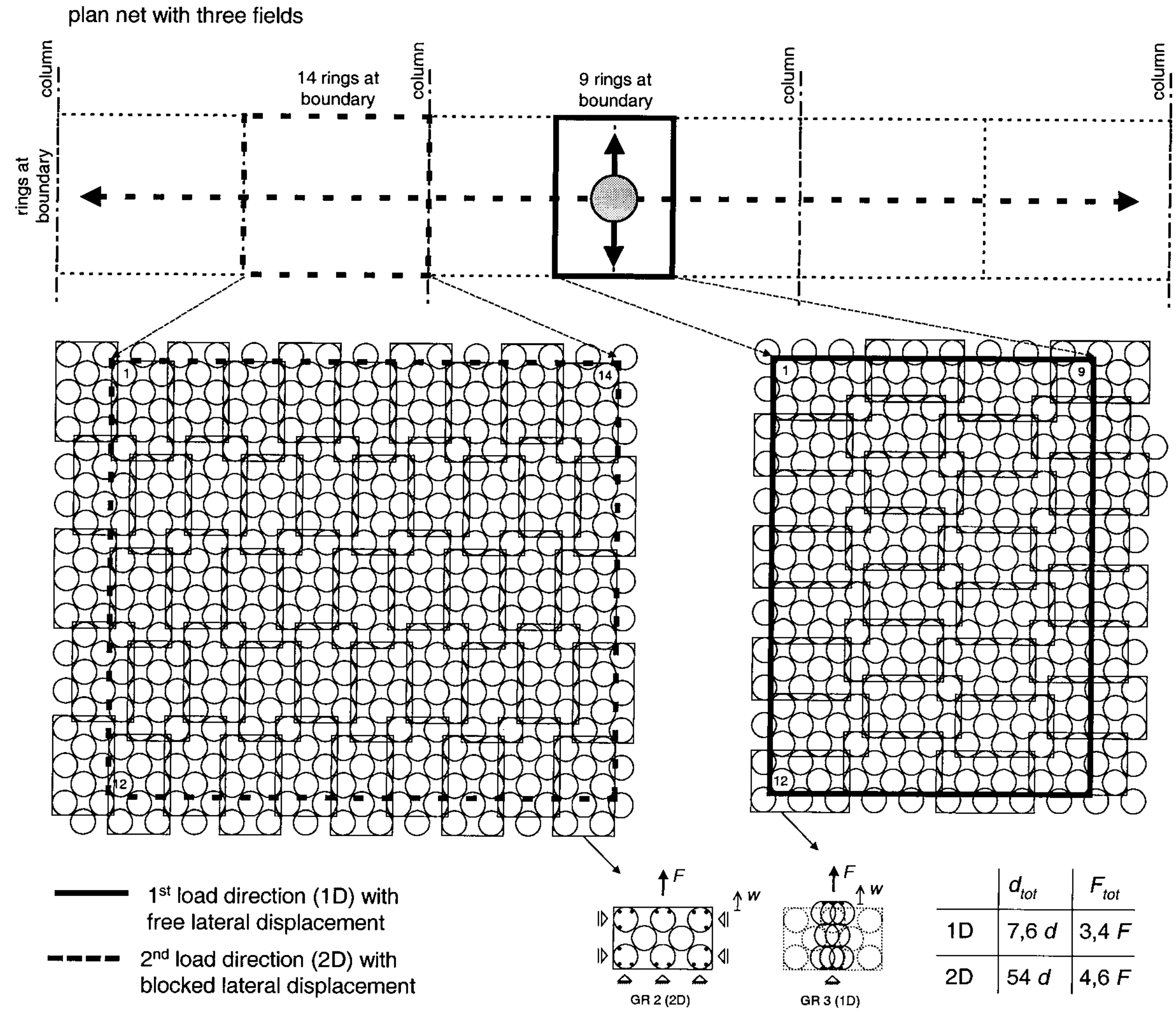


Figure 7: Rockfall protection system after dynamic field test: concentration of the ring net at the position of the maximum cable sag.

4.2 Material laws

During the braking process of a boulder in the protection system, the rings, which are connected to the upper and the lower supporting cable, slide together at the position of the maximum cable sag (see **Fig. 7**).



	d_{tot}	F_{tot}
1D	7,6 d	3,4 F
2D	54 d	4,6 F

Figure 8: Determination of the stiffness factor for the 1D- and 2D-net-element.

In the vertical direction the rings are strained with free lateral displacement (1D). In the horizontal direction the lateral displacement is blocked due to the upper and lower supporting cable, the loading acts in two axis (2D). This kind of mechanism has been studied with the quasi-static test series *GR 2* and *GR 3.* The factors from the table in **Fig. 8** indicate how many series and parallel connections of the experimental set-up fit into the loaded net zone.

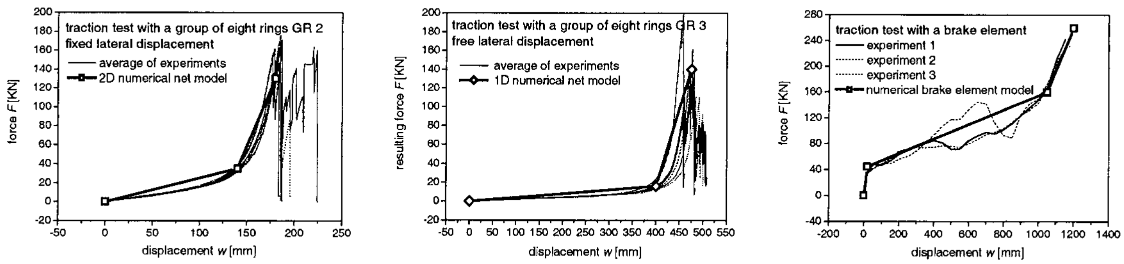


Figure 9: Material law deduced from the tensile test diagram for the 2D-, 1D-net-element and the brake-element.

The tensile test diagram and its bi-linear and tri-linear regression for the resistance force of the cable elements modeling the ring net and the brake element behavior are shown in **Fig. 9**.

5 Results

5.1 Comparison of numerical and experimental results

The motion-time-relations from the measurements in the instrumented boulder and the numerical simulation with the finite cable element model are compared in **Fig. 10**. The oscillations of the acceleration-time-relation from the numerical simulation correspond to the change of the gradient of the bi-linear material law of the 1D-net-elements. The over-simplified bi-linear model of the net with four cable elements is responsible for the overestimated braking distance of the numerical simulation.

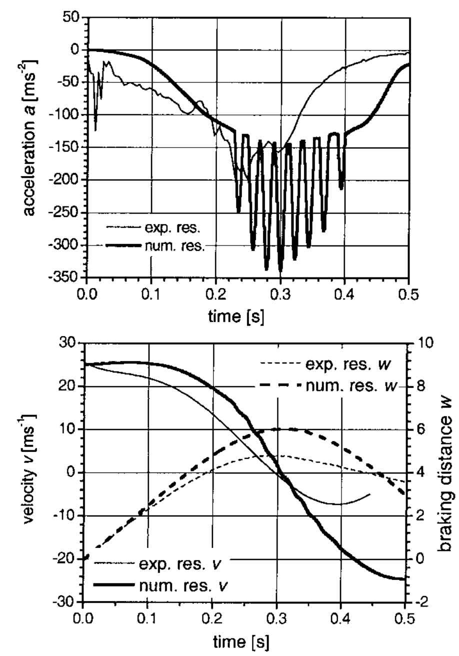


Figure 10: Comparison of experimental and numerical result of motion values of the boulder (m = 825 kg, v_0 = 25 ms^{-1}; E = 260 kJ).

In **Fig. 11** the cable forces of the numerical simulation and of the data from the traction sensors in the cables are compared. The calculated forces in the supporting cables and in the lateral anchoring cables coincide well with the measured values. The simulated results of the lower supporting cables predict the time when the brake elements engage.

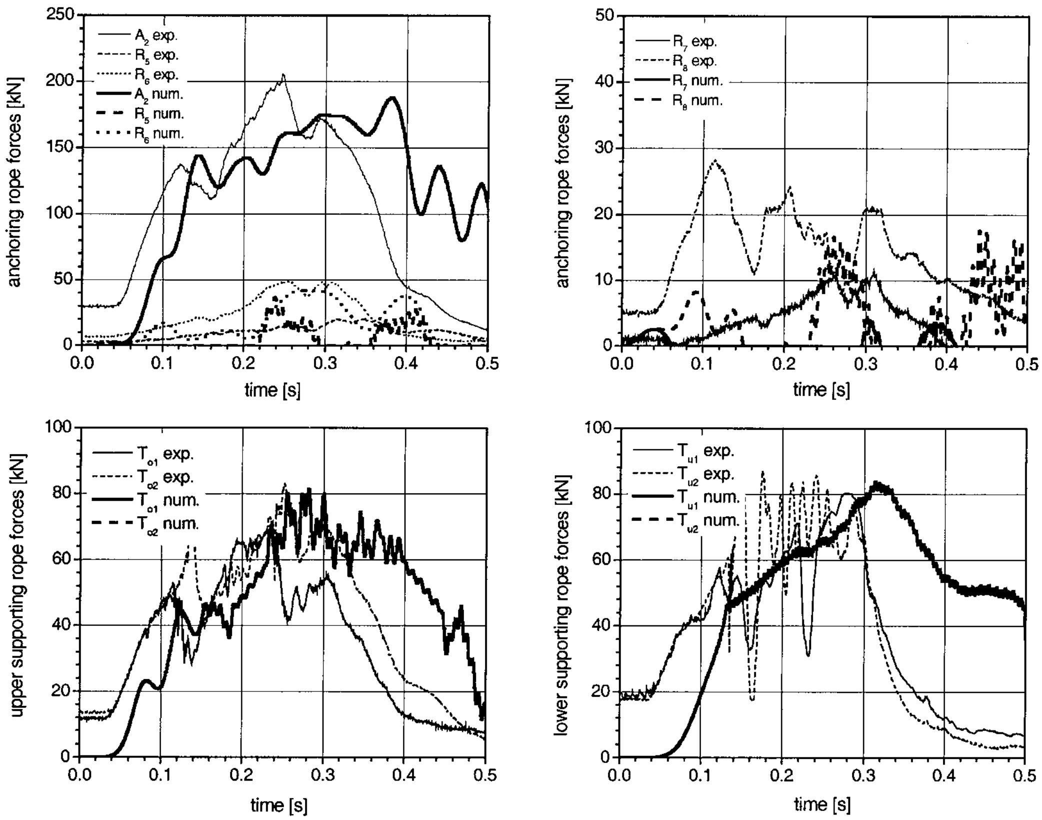


Figure 11: Comparison of experimental and numerical results of the cable force-time-relations (m = 825 kg, v_0 = 25 ms^{-1}; E = 260 kJ).

5.2 Reliability Analysis

The cable element model was integrated into a reliability analysis program based on the first order method [5, 6]. For the design resistance, nine stochastic basic variables describing the material and geometric behavior of the structure were determined. The loading action on the structure is described with the stochastic parameters of the impact velocity and the rock mass. Three different locations for the impact are distinguished: center of the net field, close to the upper supporting cable and a complete miss. The location of the impact is modeled with a log

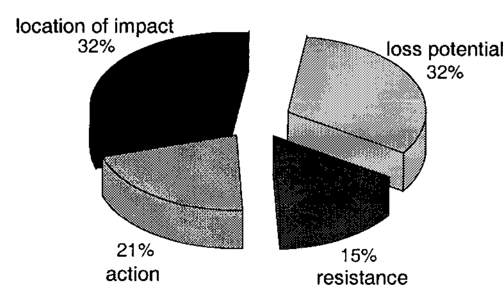


Figure 12: Influence of the resistance and action parameters as well as the reliability class.

normal distribution. The loss potential is taken into account for three different types of roads. The target reliability index β is between 2.3 and 3.7 in order to minimize the costs due to failure. The influence of the action (velocity and mass), the resistance, the loss potential (road type) and the location of impact on the reliability of the systems is illustrated in **Fig. 12**. The location of impact and the loading action have the largest influence on the system reliability.

6 Conclusions

The use of a instrumented boulder containing an acceleration measuring device provides results with a high accuracy about the dynamic response of steel-net protection systems. The combination of novel measuring techniques with a rockfall testing programme integrating experiments with single components and complete systems represents a basis for the development and validation of numerical calculation models. The simplified finite cable element model enables the simulation of the complex deformation behavior of ring net systems. The probabilistic simulation points out the influence of the predominant variables on the reliability of the protection systems.

Acknowledgements

The authors wish to acknowledge the Swiss Federal Commission for Technology and Innovation (CTI), which provided partial financial support for this work. Industrial partner is the Swiss company Fatzer AG, Profitcenter Geobrugg, CH-8590 Romanshorn with high technical engagement and financial support in this research project.

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The worldwide first official approval of rockfall protection nets

Reto Baumann [1)]

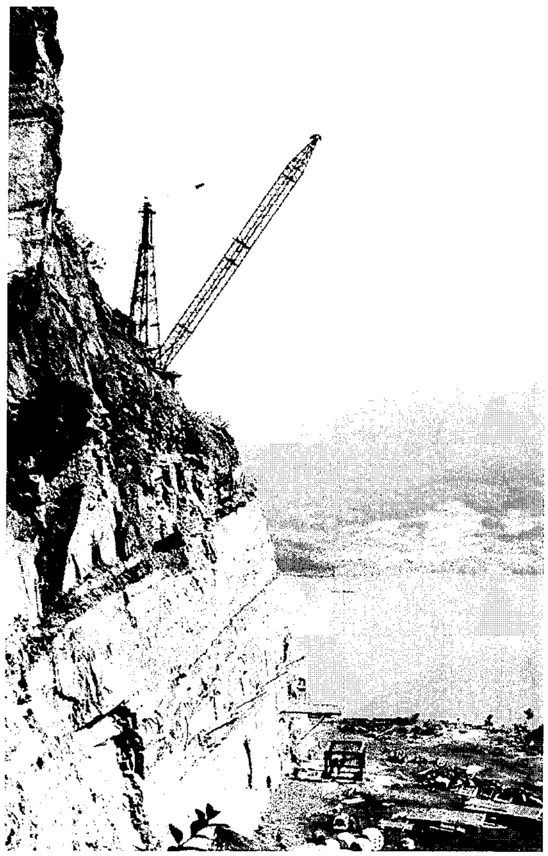

Figure 1 The Swiss Test Facility for Approval in Walenstadt

Summary

Rockfall protection structures have been evolving and being field tested in Switzerland since 1988. Today their capacity for taking up energy is a multiple of what it was then. Four manufacturers have tested their products at seven different locations. Over 350 field tests have infused new knowledge into the development of rockfall protection structures and into the testing procedure itself. High quality products have emerged out of the different test conditions, but they cannot be compared. This always leads to situations of conflict in public tenders. For this reason the Swiss Agency for Environment, Forests and Landscape (SAEFL), together with the Federal Research Institute WSL, has introduced an objective and standardized approval procedure for flexible rockfall protection nets. For this test the protection nets will be evaluated on different criteria in an energy range from 100 – 5,000 kJ.

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Switzerland has been fighting natural hazards for centuries

Switzerland is famous for being in the middle of the Alps. They have their beautiful side, such as the world famous Matterhorn. They also have a very threatening side, however, in the form of fearful natural hazards. Avalanches, floods, slides, rock avalanches and rockfalls arouse anxiety and fright in the populace. The Swiss have learned, however, to live with them and to protect themselves effectively. Evidence of this centuries long battle for survival can be seen everywhere today in the many protection structures. Avalanche structures, deflection dams, road and railway galleries stand witness, as do rockfall protection nets.

Experts call for better rockfall protection nets

The protection structures built have not always performed as well as hoped. In the past, even palisade walls and nets have often not stood up to the loads and have failed. For this reason a group of experts and practitioners had already called for the development of better rockfall protection nets back in 1974, with their functionality to be tested by an independent authority. W. Heierli presented in 1985 the first research study about protection against rockfall.

Thus it came about that the development of new and improved rockfall protection nets advanced strongly in the last ten years.

Research and industry carried out more than 350 tests

Rockfall scientists and manufacturers of protection nets were brought together by their ambitious goals and the strict quality requirements. An intensive collaboration resulted in the design of new products. It was possible to increase the capacity for absorbing energy, which is the most important criterion for the nets, from ca. 250 kJ to around 3000 kJ – a factor of 12!

Table 1 Maximum kinetic energies of rocks that were decelerated by protection nets.

Year	**1987**	**1990**	**1992**	**1995**	**1997**	**2001**
Rock mass (kg)	3 000	1 600	2 870	4 000	5 600	9 600
Max. velocity (m/s)	12	22	26	27	26.5	25
Kinetic energy (kJ)	**220**	**390**	**970**	**1 460**	**1 970**	**3 000**

Right from the beginning it was clear that no further help was to be had from calculations. The dynamic behavior of the nets cannot be calculated (yet). Tests on a 1:1 scale were required. In the period between 1988 and 2001, over 350 tests were carried out with rockfall protection nets! In the course of these tests, four manufacturers of protection nets developed new types of nets with the goal of maximizing the energy that could be taken up by the nets. Development was based mainly on realistic field tests, in which the nets were tested with moving rocks. Not only were the protection nets evolving, the testing methods also continued to improve. The experience thus gained pointed up the weaknesses in the test arrangements chosen.

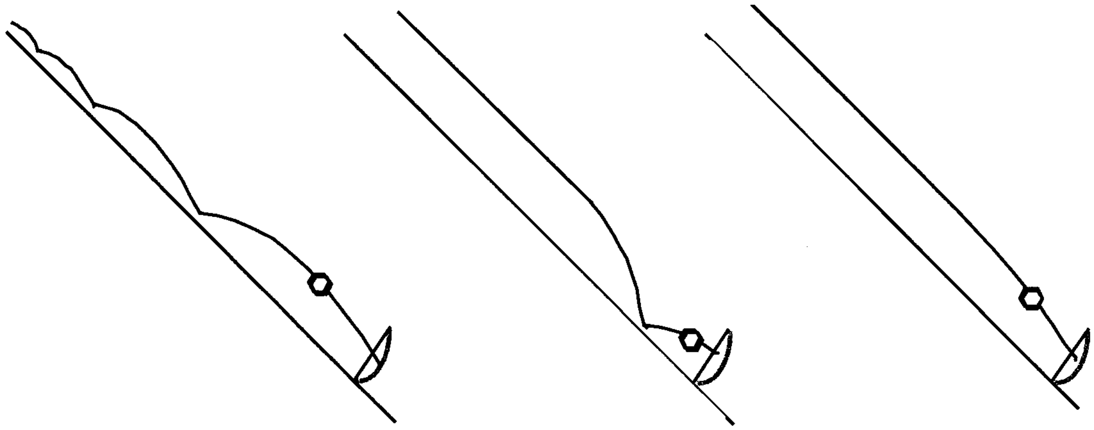


Figure 2: Schematic representation of different test arrangements

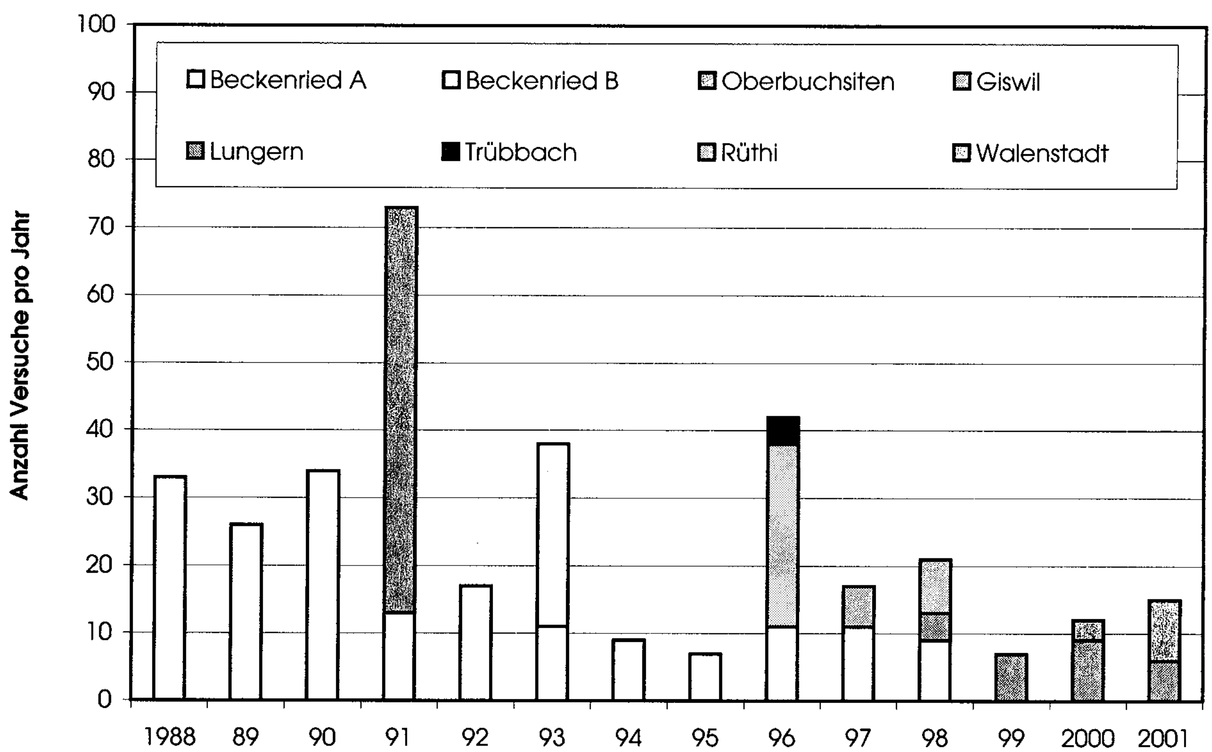
First test series
In this series the rocks were set in motion on the steeply sloped surface above the net. They subsequently slid and jumped in different directions and with differing velocities onto some point in the protection net. This test series was used especially in the early years of the tests in the Beckenried (Beckenried A) and Oberbuchsiten. The results of evaluating the velocity of the rocks gave valuable insight into the energy lost by the rocks as they made contact with the ground. It was seen that many rocks were not stopped in the protection nets but instead continued their trajectory to the left or right of them. The velocities reached ranged widely, from 6-24 m/s. The rocks landed at different places in the protection nets and with very variable velocities. Neither the kinetic energy nor the direction of incidence could be determined in advance. This situation was most unsatisfactory for both the scientists and for the developers. For this reason a better test arrangement was sought.

Second test series
For this series an inclined cable-crane setup first had to be installed. The crane equipment was not only used for transporting the rocks upwards, but equally for their downwards transport. The rocks, hanging from the rope, were dropped and then released from the rope after a certain distance was covered. The rocks struck the surface of the ground near the net and then rolled into the net. The goal of this series of tests, which were carried out in Beckenried (Beckenried B), was to increase the velocity of the rocks. This goal was not reached, however, because just as much energy was taken up in a single ground contact as by multiple contacts in test series 1. The velocities here also encompassed a relatively wide range, from 10-22 m/s. Test series 2 had the advantage, however, that the rocks always hit the middle of the protection net relative to its lateral dimension. The height and angle of incidence, however, could not be determined in advance. This situation was still unsatisfactory, and so another test arrangement was developed.

Third test series
The cable-crane setup was also used in this series to throw the rocks, aiming them into the protection nets. But this time the rocks went directly into the net, and with high velocity. This series was carried out in all test sites except for the Oberbuchsiten. It was not until the introduction of test series 3 in 1991 that the rocks could be held to a relatively narrow range of

velocities, from 25-27 m/s (Beckenried) or 29-31 m/s (Giswil, Lungern). With this test series it was possible to calculate the energy of the falling rocks in advance and to develop the protection nets to specification. At the high velocities, however, there were sometimes problems with releasing the rocks from the crane's bearing rope. This test arrangement had the further disadvantage that the large masses required for the high energies were more difficult to manage and represented a hazard for operation of the equipment. If one wanted to vary the impact of the rocks in the net, it could only be accomplished by costly modifications to the cable-crane equipment. For every newly tested and approved net, the equipment first had to be adjusted. The equipment needed a costly "ranging". The accuracy of impact was also not yet totally satisfactory.

Figure 3: Number of tests per year with protection nets



Practitioners call for approval

The purchasers of rockfall protection nets during a period of such rapid development find themselves constantly confronted with new or changed products. The different works plans do not lend themselves to comparisons. New products of unsatisfactory quality are constantly appearing on the market. Those responsible for safety are very insecure and cannot compare products in public tenders. Surveys of practitioners in 1997 and 1998 revealed an urgent need for action. Again there was a call for neutral and objective testing of the nets in Switzerland.

What are the requirements for an approval?

Separate the wheat from the chaff. The approval process must be able to separate the "wheat from the chaff". That means that it must be able to ascertain which products are not suitable for rockfall protection. These must not pass the test.

Provide relevant information. The approval must provide the information that planners and purchasers need for the design and dimensioning of the rockfall protection structure. Much information is interesting but not important for the practitioner. The information provided by the test should simplify product choice, not complicate it.

Standardize. The test procedure must be strictly standardized and reproducible at all times. The test procedure must be structured so that the test conditions are always exactly the same. Only if this constancy can be guaranteed in the test procedure will the manufacturers of protection nets be prepared to accept the results.

Model reality. As much as possible, the test should reproduce the real-life situations it is attempting to model.

Be reliable and independent. The manufacturers, planners and purchasers must be able to have confidence in the test. For this reason it must be carried out by an independent institution, and it must have an official character. Only a government authority can meet this need.

Be technically and economically feasible. The most beautiful ideas for a test are worthless if they are not technically feasible or are so expensive that they are unaffordable. Approval testing should not make the products offered on the market more expensive. That would not be in the public interest! The products for sale should be getting better, not more expensive.

The new test standard in Switzerland

Development of the approval started early in 1999 after clarifications had demonstrated the feasibility. Here again intensive collaboration was asked for. A strict but fair approval was designed with the participation of industry, research and various authorities. The test facility in Steinbruch Lochezen near Walenstadt was officially opened on May 31, 2001. This was the birth of the first worldwide and, up to now, the only official approval for rockfall protection nets!

The approval rests on two pillars. First, the new test facility permits 1:1 tests and the necessary comparative measurements. Second, the test procedure, minimum requirements and measurement methods were developed. All this is set out in the federal guideline for the approval of rockfall protection nets, issue 2001.

The following criteria, important for the purchaser, are tested:

- test energy (= main criterion)
- maximum excursion with loading as well as excursion with half loading
- impermeability to smaller rocks
- forces in tie rod
- effective height as well as effective height after loading
- adaptability to the terrain

- simplicity of construction
- ease of repair
- lifespan

Administrative procedure

Administration of the approval is mainly in the care of the SAEFL (Table 2). Manufacturers send their applications for approval to this agency, along with all necessary construction plans, detailed drawings and data on the material qualities. The agency registers the application and documents and orients the involved parties on the further course of the procedure, the rough schedule and, for the manufacturer, also the costs that will be involved. The WSL uses the documents to verify that the construction meets the basic safety requirements and clarifies how the structure can be installed in the test terrain.

Based on the documentation, the WSL verifies whether the construction meets the basic requirements for protection structures and clarifies how the structure can be installed in the test site. The WSL organizes the test, generates the measurement and observation plan and carries out the approval; it records the results and produces a comprehensive report.

The FECAR evaluates the results and determines to what extent the structure fulfills the conditions for approval. It gives the SAEFL a recommendation as to whether the test was passed.

The SAEFL writes the test certificate and forwards it to the manufacturer along with the test report.

It goes without saying that the approval is open to all manufacturers and not just intended for Swiss products!

Table 2: Administrative procedure for the testing process

Responsible / Involved parties	**Working steps**
Manufacturer; supplier	Application to SAEFL with documentation
SAEFL / SFA	Registration, confirmation of receipt, information about the costs
WSL	Examination of the documents, organization of the tests
Manufacturer; supplier	Payment of the deposit
WSL; manufacturer; delegation from FECAR	Implementation of the tests
WSL; Delegation from FECAR	Test report for the FECAR
FECAR	Overall assessment yes/no; application to SAEFL; brief report to manufacturer within 6 weeks after test
SAEFL / SFA; manufacturer, supplier	Final financial settlement with manufacturer; supplier
SAEFL / SFA	Issuing of certificate and test report

SAEFL =	Swiss Agency for the Environment, Forests and Landscape
FECAR =	Federal Expert Commission on Avalanches and Rockfall
SFA =	Swiss Forest Agency (principal division of the SAEFL)
WSL =	Federal Research Institute WSL Birmensdorf

The test facility in Walenstadt.

Experience from the first three test series led to the idea of building a test facility for testing vertical fall with test bodies. This way all of the disadvantages of the oblique fall setups could be avoided. The test site where the structures are tested now consists mainly of an almost perpendicular rock face and a level installation area. The structures are installed on the rock face at a height of 15 m; the structure's posts are fastened to a 32 m long foundation adaptation unit and anchored to the rock face by ropes on the post head.

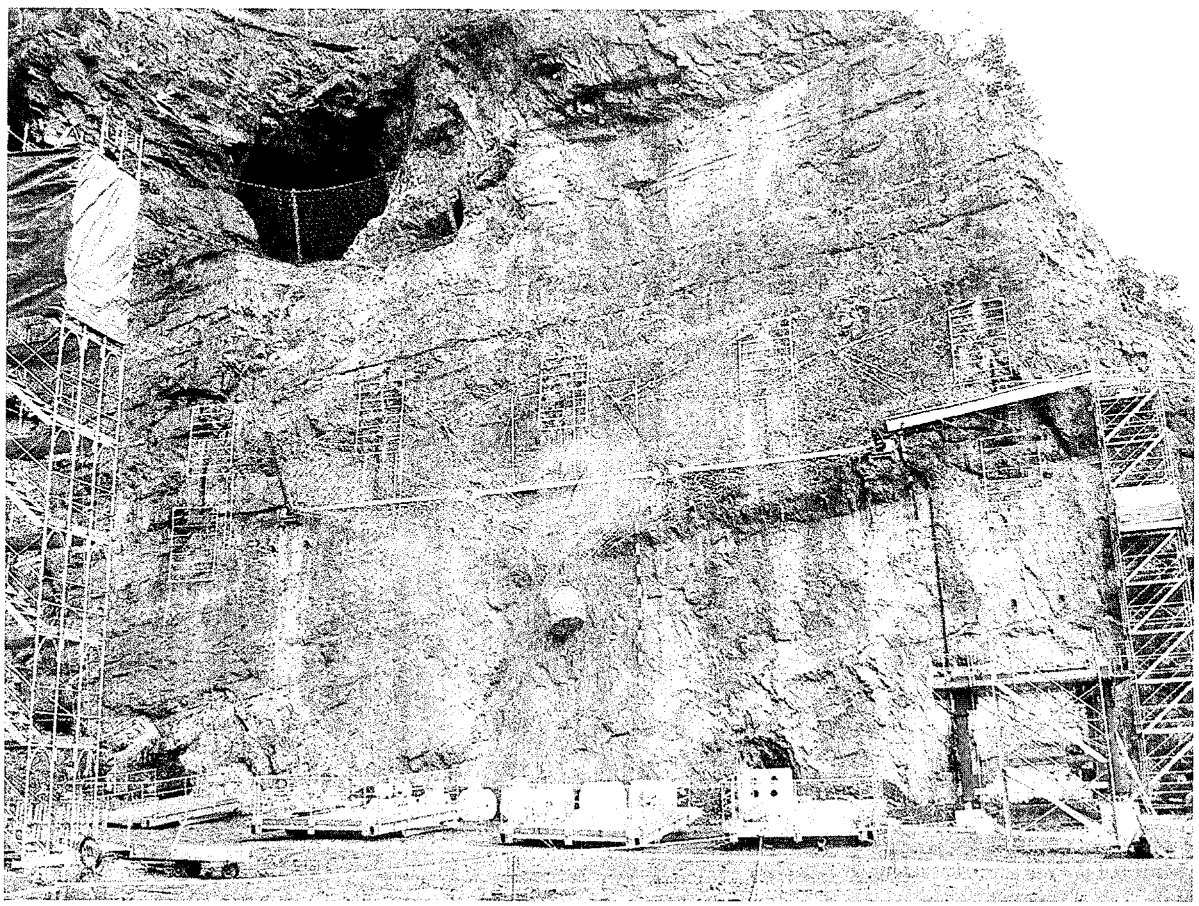


Figure 4: Frontal view of the Walenstadt test setup during a test

The posts in the structure are inclined at angle of 30° to the horizontal. The inclination of the retaining rope is approximately 40° in the side elevation (Figure 5). This arrangement will cover a large proportion of naturally occurring rock trajectories for this vertical load.

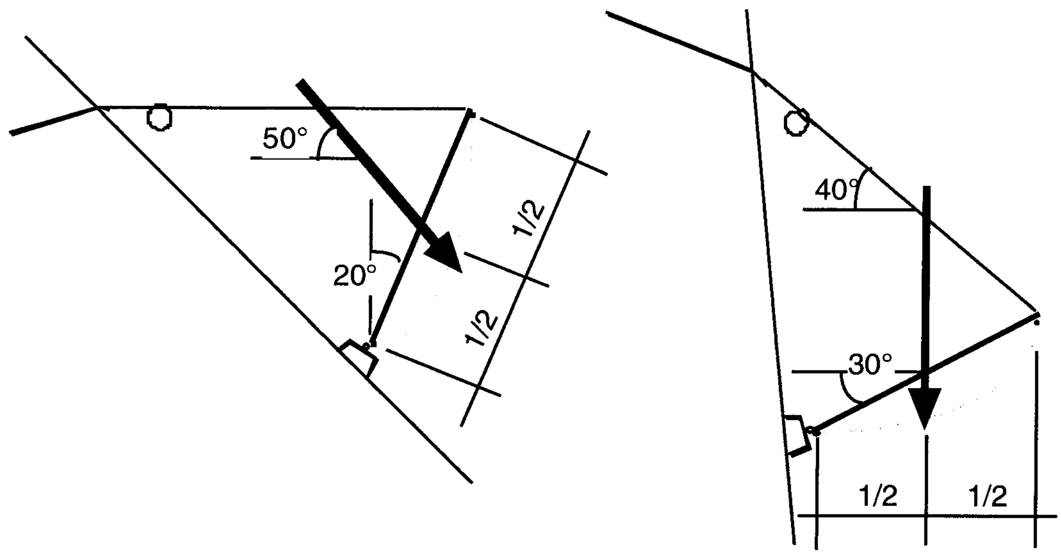
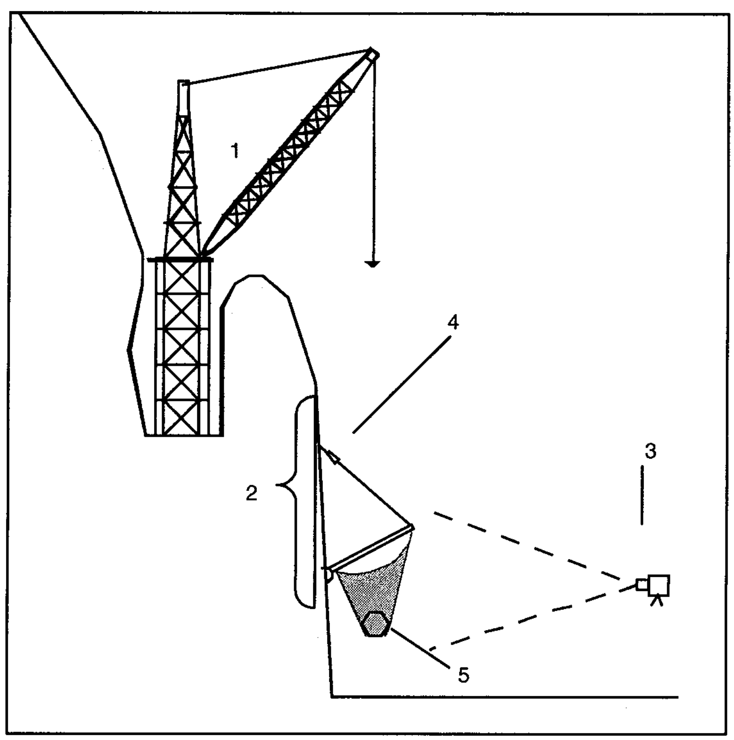


Figure 5: Test setup mainly used in recent years (left) and new arrangement at Walenstadt (right).

The works are divided lengthwise into three section widths of equal lengths. Thus for a standard structure 4 posts are installed, with 3 net sections between them. The relation of the section width to the distance between posts is left open. It is thus left up to the manufacturer of a protection structure which section width he uses in order to meet the conditions with respect to braking distance. The minimum height of the structure being tested, on the other hand, depends on the particular energy category (Table 3).

Figure 6: Arrangement and technical data for the rockfall test setup



Legend:			
1	Crane	Hoisting force	160 kN (16 tons)
		Maximum hoisting height	50 meters above ground
2	Tested nets	Maximum length (horizontal)	30 m
		Net position	15 m above ground
		Post length	3 – 7 m
3	High speed video camera	Number	2
		Position	Lateral and frontal view
		Picture	Digital technique
		Number of images per second	250
4	Tensile force sensors	Number	15
		Maximum force	500 kN (50 tons)
		Number of measurements	2000 / sec
5	Test body	Type	Steel concrete cube in ashlar shape with flattened corners and edges
		Velocity on impact	25 m/sec (=90 km/h)

Requirements for the protection nets

In principle the task of a protection structure is to stop moving rocks and blocks. In the natural environment this is mostly achieved by means of a ground contact during deceleration. This means that the protection structure only takes up part of the kinetic energy of the rock; the remainder is absorbed by the ground.

The protection structures to be tested in accordance with this guideline should be able to absorb the full kinetic energy of a falling rock. Certain additional requirements will be imposed on the maximum deformation of the structure. On one hand the braking distance should not exceed a certain mass after an event, and on the other hand the structure should still retain a certain residual useful height.

The effective surface of a structure should also remain as large as possible after an event and be impaired as far as possible only in its height, but not in its width. In recent years structures have been developed for which the effective width has been interrupted after an event. In such a case it is possible that rocks and blocks coming after can roll on unimpeded through the structure and cause damage further below.

The 4 test parts

The protection structures are divided into 9 energy categories of 100 kJ to 5,000 kJ. A structure must pass the test parts a) through d) described below in its energy category. For tests a) through c) the individual tests take place on the structure with rocks, and test d) includes a qualitative assessment of the structure and an overall evaluation of tests a) through c).

Concrete blocks in an ashlar shape, in which the corners are truncated to one third of the length of the side, are used for testing the structures. They are dropped vertically and must be decelerated by the structure. The velocity used is 25 m/s, and the mass of the blocks depends on the energy category.

a) Preliminary tests with low energies: (boundary section)

These preliminary tests serve on one hand to test the bearing and deformation capacities of the laid-on mesh and on the other hand to load individual ropes or rings in the net with smaller rocks. Rocks as listed below are dropped together by size category with a velocity of 25 m/s into a boundary section.
5 rocks 10/10 cm; total mass 12 kg
3 rocks 20/20 cm; total mass 56 kg
1 rock of 50/50 cm; mass 291 kg
For this test all of the rocks must be stopped by the structure. Deformations of the individual ropes and mesh elements are permitted. They are recorded. No repair work is allowed. The rock of 50/50 cm is omitted for structures in categories 1 and 2 (Table 3) for this test.

Table 3: The test parameters for the 50% and 100% test parts

		Test part (50%)			Test part (100%)				
Category	Post length (m)	Energy (kJ)	Mass of test body (kg)	Edge length (m)	Energy (kJ)	Mass of test body (kg)	Edge length (m)	Max. perm. stopping distance (m)	Min. net height (m)
1	1.5	50	160	0.41	100	320	0.52	4.0	0.90
2	2.0	125	400	0.56	250	800	0.70	5.0	1.20
3	3.0	250	800	0.70	500	1,600	0.88	6.0	1.80
4	3.0	375	1,200	0.80	750	2,400	1.01	7.0	1.80
5	4.0	500	1,600	0.88	1,000	3,200	1.11	8.0	2.40
6	4.0	750	2,400	1.01	1,500	4,800	1.27	9.0	2.40
7	5.0	1,000	3,200	1.11	2,000	6,400	1.40	10.0	3.00
8	6.0	1,500	4,800	1.27	3,000	9,600	1.60	12.0	3.60
9	7.0	2,500	8,000	1.51	5,000	16,000	1.90	15.0	4.20

b) Preliminary test with 50 % energy: (middle section)

This test primarily serves to test the required repair effort and the service-friendliness of a structure. It also tests the excursion at half energy. The block used for this test is dropped into the middle section with a velocity of 25 m/s. After the test the following data are recorded:

- excursion of the upper and lower bearing ropes
- position of the block in the net
- deformation of the net
- changes in the positions of the posts
- deformations of the individual brake elements
- damage to and deformations of other structural elements
- time and material required for repairs needed for full restoration of the structure for the main test.

c) Main test with 100 % energy: (middle section)

For the main test the full kinetic energy of the rock is to be taken up by the structure; the bearing capacity and the deformability are tested. The block envisaged for this energy is dropped with a velocity of 25 m/s into the repaired middle section. After the test the following data are recorded:

- excursion of the upper and lower bearing ropes
- position of the block in the net
- deformation of the net
- changes in the positions of the posts
- deformations of the individual brake elements
- damage to and deformations of other structural elements

The residual useful height of the structure can be calculated from the data for the deformations of the bearing ropes and the net. This may not be less than the value given in Table 3.
The maximum excursion of the structure with respect to the braking distance of the block is then determined from the video films. This may not exceed the value given in Table 3.

d) Assessment of the structure

During installation of the structure and tests a) through c) the structure will be evaluated for simplicity of construction. The structure should also be able to be installed as efficiently as possible in difficult terrain. If it is damaged, it should be as easy as possible to replace individual elements.

Measurement and observation methodology

For test b) and c) the tensile and compressive forces are measured and recorded at a maximum of 12 locations during deceleration of the block. Maximum forces up to 500 kN can be measured with tension load sensors, whereby the scanning frequency is normally 500 Hz. The motion of the block and individual bearing elements of the structure are filmed by video cameras from two directions (Figure 3). The effective forces acting on the rock can be calculated from the motion of the block (path/time records), and these can be compared with the measured forces and the bearing ropes.

The initial knowledge gained from the approval process

The approvals carried out thus far show that the test procedure meets the requirements. It is suited to differentiating between unsatisfactory products and those that meet the strict quality requirements in Switzerland. The information provided and the test results are objective and allow a good assessment of the protection net tested.

The measurement results obtained with the tension load sensors demonstrate that large forces can arise in the bearing ropes with the high energy nets. Peak forces of over 300 kN were measured. The anchorage must be able to take up these forces. For this reason it is especially important to pay attention to the dimensioning of the anchor and the bearing capacity of the ground when planning the anchorages. If necessary these may have to be obtained from tensile tests.

Comparison of the measured excursion at half load with that at full load shows that the difference is less than assumed earlier. The excursion does not increase linearly with increasing load.

The effort involved for an approval is considerable, despite limitation to the most important test parameters. This is also reflected in the cost, which is about $ 40,000 per construction tested.

Where can I find out more about the protection nets tested?

The SAEFL publishes the certificates on the Internet. They can be downloaded from the following address:

www.schutzwald-schweiz.ch

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53 rd Highway Geology Symposium Aug. 14 to 16, 2002

Certification of Rock fall Barriers in Europe

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Abstract: Since their first application in 1958 at Brusio, Switzerland, rock fall barriers made out of steel wire rope meshes have proved as a powerful mean for rock fall mitigation along highways all over the world. Nevertheless, no technical standard did exist either for their construction or for their technical approval so far. This situation made it difficult for designers as well as for clients to compare the different systems as for their safety and select an appropriate product for their applications. Therefore, the European Organization for Technical Approval (EOTA) is establishing a guideline for testing and certification of rock fall barriers as a first step to harmonization of rockfall mitigation within the European Community. The actual draft is presented within this paper.

The intended approach by defining certain energy dissipation levels and having the producer selecting the corresponding energy he wants his system to be tested is described. The state of the discussion as for the type of tests - drop tests or launch tests - is commented. Criteria for the assessment of the required performance of barriers under different energy levels are presented. Their practical applicability is analyzed. The minimum required number of tests as well as the desired hit areas are discussed. The pros and cons of testing not only the center of the middle panel but also the edges and the posts are presented. The practical meaning of the expected test results is analyzed from the point of view of a designer.

Introduction: In the eighties of the last century there were a lot of different rockfall barrier types, but their energy dissipation capacity was in general unknown. Only very few producers were using tests for the systematic development of their barriers, like BRUGG CABLE PRODUCTS - the later GEOBRUGG - and ISOFER, both in Switzerland (SPANG & BOLLIGER, 2001). RITCHIE's (1963) pioneering rock rolling tests intended more to establish criteria for the determination of fence heights than to analyze their energy dissipation capacity. COLORADO DEPARTMENT of HIGHWAYS (1989) tests have brought some fresh ideas into rockfall mitigation, but were not systematic enough to fill that gap. Thus CALTRANS tests at Big Sur, Ca. were the first systematic and independent field tests to compare barriers of different producers - in this case ENTREPRISE INDUSTRIELLE from France and GEOBRUGG from Switzerland (SMITH & DUFFY, 1990) and to determine their energy dissipation capacity by identical and reproducible tests.

On the other hand, the specific kinetic energy to be dissipated at a certain location, where such a structure had to be erected, could not be determined, too, no realistic computation method being available. This situation resulted in purely empirical designs with unknown safety factors (SPANG, 1988). It changed when the first reliable rockfall simulation programmes became available in the market.

State of the art of testing rockfall barriers: The actual situation is characterized by the following facts:

- A lot of rockfall barrier systems are now tested as for their energy dissipation capability, but partly by the producers themselves, by different test procedures, different energies, mass/ velocity ratios, different sizes of blocks, with or without rotation, by rock rolling, vertical drop tests, launch tests, pendulum tests, or other more exotic test procedures, targeting into the center of the panel, on the posts, on the lower border etc. etc. (HALLER & GERBER, 1998). This wide range of different test approaches does not mean more than an evident lack of comparability of their results.

- Until December 2001, when the SWISS AGENCY for the ENVIRONMENT; FORESTS and LANSCAPE published its "Guideline for the approval of rockfall protection kits" not any regulation or recommendation for the execution of these tests did exist, worldwide.

- It is well known by the scientific community that the energy dissipation of a system depends on many different factors and that the result of tests depends on the boundary conditions of the impact on the barrier. Especially it depends on:

  - The **number of panels**, a test barrier consists of. Most systems transfer a considerable amount of the impact energy to the neighboring fields resulting in a higher flexibility of the system, a bigger mass to be accelerated and a longer restitution time reducing the brake forces inside the retaining ropes and the foundation.

  - The **location of the impact** on the surface of the panel: the center of the panel has the highest flexibility and symmetry and shows considerable higher energy dissipation than the corners, the boundaries or the posts, for examples. The order of magnitude in the energy dissipation may range between 100 % in the center and 60 to 70 % in the corners. Because no systematic tests have been carried out so far, the distribution of the energy dissipation over the panel surface is not known. Thus, tests with different target areas can't be compared. Obviously, tests only in the center of a barrier will not give reliable numbers for a safe design.

  - The relationship between **barrier height and energy dissipation** capacity: The interdependence of barrier height and energy dissipation is not known. Thus, tests of barriers with different heights are not comparable.

  - The influence of **angular velocity**: In the early days of rockfall testing, rock rolling was the mostly applied method. There is no doubt it is the most realistic one, too, because it includes rotation of the rock. Most of the modern test methods are restricted to impacts with pure translation. Rotation may represent up to 30 % of the total kinetic energy of a real rockfall and the mechanism of energy transfer from the boulder to the barrier should be significantly different for the translational and for the rotational part of the kinetic energy. This was one of the most interesting lessons to be learned from the first CALTRANS tests in 1985 (SMITH and DUFFY, 1990; SPANG & BOLLIGER, 2001). Tests with pure translation may be comparable amongst themselves, but they will be of no practical use for the decision for a certain system in design.

  - The kind of **loading:** It should be trivial, that static loading of entire systems or parts of systems will not give any reliable ideas on the behavior of a barrier under dynamic impact. The reason is that the forces developing inside a system under dynamic loading depend on the brake time, which can't be predicted from theoretical considerations. Thus, these tests are without any use for design decisions.

  - **Mathematical model:** In spite of the fact that increasing scientific capacities are involved, there is no calculation method available to substitute, control or amend field tests, actually. Thus, no method does exist to calibrate results of different test methods.

- On the other hand, the actual situation is characterized by a rapid and ongoing development of rockfall simulation programmes. Therefore, it is state of the art to determine the probable kinetic energy and bounce height of a rock at the location of a rockfall barrier under design and to demand these values in the technical specifications of the tender documents (SPANG & KRAUTER, 2001). Such a requirement is useless, if there is no reliable resp. common quality standard to compare the resulting tenders as pointed out above.

European approach to standardization: As for the specific situation in Europe, many of the member states of the European Community have mountainous areas and have to cope with rockfall risks. Only few countries have their own rockfall barrier systems resp. production, however, and sell their systems to the others, or even distribute them worldwide. To make systems comparable, make the market more transparent and define equal safety standards a standardization of test procedures within the European Community obviously was urgent - not a standardization of the systems themselves. To archive this purpose the European Commission as the executive power of the member states gave a mandate to the European Organization for Technical Approval (EOTA) to establish a European Technical Approval (ETA) guideline on rockfall barriers. A system tested according to this guideline will be granted an ETA that means it is fit for the intended use.

Aim of the guideline: The guideline shall be applicable to all rockfall barriers consisting of posts, cables and nets, independent from their specific structure, whether the nets are linked to suspension cables or and directly to the posts, whether the posts are exposed to rockfall or protected by the net.

There is a vivid and lasting discussion amongst the members of the working group about the purpose of the intended tests in conjunction with the test installation. The one group favors the restriction to the pure comparability of test results as the only aim, the other, consisting mainly of public customers, designers and this author, argues for tests determining limit energy dissipation and maximum elongation as a base for design decisions and approval of tenders. This is amazingly not the same, because for comparison only, it doesn't matter which tests are executed as far as they follow the same procedure and their results being reproducible.

For design purposes it is vital to know the lowest amount of energy a system can dissipate under the most unfavorable conditions and at the most unfavorable location, should it be a hit on a lower or upper corner, assuming that the both sides show the same behavior because of their symmetry, or on a post at the foot, where the system might be the most rigid, or against the top, where the bending moment reaches its maximum. There are also systems collapsing when a retaining rope is hit. The argumentation of the first group concentrates on limitation of test cost, believing that determination of the limit energy dissipation would require a bigger number of tests and overtax small producers. By the way, they lobby for already existing test sites with ropeway installations, where the variation of targets requires the shift either of the ropeway or of the barrier, whereas a crane can easily drop a boulder at any desired point of the barrier. To this author's opinion as a designer it would be spoiled money to make tests to whatever extend, if they don't give the numbers needed for the selection of products during design and tender. This should be the first aim of any regulation and not a "l'art pour l'art" comparison.

lfd. Nr.	Test procedure	Example	Accurancy of selected hit	Variation of hit area	Variation of impact angle	Angular velocity of boulder	Variation of kinetic energy	Conformity with reality	cost of installation	(examples) for installations	Reproducibility for test results	Rating	Position
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	Rock rolling - flat slope	SMITH & DUFFY (1990)	3 (not controlled)	2 (by chance)	3 (not possible)	1	3	1	1	Shayupin, Taiwan / Big Sur, Ca.	3	17(-)	4
	- steep slope		3 (not controlled)	2 (by chance)	2 (not controlled)	1	2 (accuracy suffers with increasing path length)	1	1	Oberbuchsitten, Ch.	3	15(+)	2
2	Launch test	HALLER/ GERBER (1998)	2	3 (barrier or rope-way has to be shifted)	3 (inclination of barrier to be changed)	2 (difficult)	1	3 (with rotation)	3	Beckenried, Ch.	2	19(-)	5
3	drop test	HALLER & GERBER (1998)	1	1	3 (inclination of barrier to be changed)	2 (difficult)	1	3 (without rotation)	2	Walenstadt, Ch. Itsukaitchi, Jp.	1	14(+)	1
4	Pendulum test	COLORADO DEPT. HIGHWAYS	1	1	2 (inclination of barrier to be changed or distance between barrier and crane varied)	3 (practically not feasible)	3 (upper limit by stability of crane)	3	1	Colorado, USA	1	15(+)	3
5	Vertical sums	/	10	9	13	9	10	11	8	/	10	80	
6	Average	/	2	1,8	2,6	1,8	2	2,2	1,6	/	2	16	

Fig. 1: Pros and cons of different test installations

As shown by Fig. 1 the type of test installation has a decisive influence on the kind of tests that can be executed. According to the selected criteria and the given ratings, drop tests proved superior to the others. Drop tests are prescribed by the above-mentioned Swiss guideline, too.

Actual state of guideline: After its 7th session the working group agreed on the following propositions:

- The **test scale** shall be 1:1. No model laws do exist so far to enable tests at a reduced scale.

- The **height** of the tested barriers shall be correlated to the tested energies. This rule is reasonable, because kinetic energy and size of a block are correlated and there should be a sound relation between the size of a block and the height of a barrier, not exceeding a ratio of 1:3.

- **Width of panel:** It is known since the first tests at Oberbuchsitten in Switzerland, executed by ISOFER in 1988 (SPANG & BOLLIGER, 2001), that the flexibility and subsequently the energy dissipation capacity of a barrier increases with increasing width of the panels resp. with increasing distances between the posts. Actually, a distance of 10 m can be considered as a standard. The working group decided to let the decision on the width of the panels to the producer, who will get his certification for the tested width only. Because there is no reliable correlation between the energy dissipation capacity and the height or width of a system, the producer will be forced to order separate tests for any of the heights and widths of his barriers he wants to have certified. This will lead to the introduction of standard widths and standard heights on the market, because no producer will invest the money to have all possible combinations tested. From the point of view of this author, this is acceptable.

- The **number of panels** shall be three. This is because most of the systems transfer a considerable amount of energy to neighboring fields. Three panels - the one to be hit in the middle - is a good compromise between the consideration of this load transfer and cost of tests.

- **Angles of impact:** The trajectory of the block shall be inscribed in a vertical plane orthogonal to the straight line, connecting the base of the posts. The angle between the net plane defined by its four edges and the trajectory at the moment of the impact shall be between 70 and 90°.

- A squat regular **block** with 26 areas defines the geometry of block. The size of the block shall be not larger than one third of the height of the barrier. Tabular or other geometries are not tested.

- **Densit**y resp. material of the block is defined by reinforced standard concrete resp. by a density of 2.5 g/cm³.

- **Translational velocity** at the moment of impact shall be between 25 and 30 m/s, this range being frequently observed in nature. There is a discussion to link the velocity to the energy level, described below. The lower velocity shall be linked to the lower energy. To the opinion of this author, there is no reasonable argument to do so. If you are dealing with a natural rock slope, the level from which the blocks may start will be independent from their size. The only difference will be that the frequency of smaller blocks will mostly be much higher than that of the big ones. It would be more realistic to define one velocity only, independent from energy levels.

- **Angular velocity** is not considered. According to the experience of this author, this will lead to a serious limitation in the applicability of the test results in design. An impact without a considerable angular velocity is restricted to free falling blocks hitting the barrier without any previous surface contact. In all other cases of rolling or bouncing rolling itself or the necessarily eccentric impacts will accelerate the block around an axle and result in an angular velocity. The percentage of the rotational kinetic energy can be more than 30 % of the total kinetic energy. This is too much for being neglected. Neglecting the angular velocity might be acceptable as a first step, resulting from the difficulties of actual test installations in modeling angular velocities, as presented by fig. 1. Triggering angular acceleration is possible by rock rolling or by launch tests, if the block impacts on the ground before hitting the barrier. The low reproducibility of rock rolling tests could be improved in future by guiding the traveling block between some kind of guardrails or walls. The loss of energy during the impact of a block in the launch test is not considered as decisive in relation to the above-mentioned disadvantage of purely translational tests. The final aim has to be to test blocks in full consideration of their angular velocity.

- The guideline shall cover barriers with **limit energy dissipation capacities** from 100 to more than 2.500 kJ.

- There will be no **total energy** given. The producer has to decide on the energies he wants his system to be tested. Three different energy levels and their sequence are under discussion:

    1. **Zero Maintenance Level** (ZML) is characterized by the highest translational energy a barrier can dissipate without any plastic deformation.

    2. **Service Energy Level** (SEL) is defined as the maximum translational energy a barrier can repeatedly dissipate under the condition that the rock is stopped.

    3. **Maximum Energy Level** (MEL) is the highest translational energy a barrier can dissipate under the conditions that the rock is stopped (the system might loose its ability to stop another block with the same or an even considerably lower energy).

Independent of the energy levels the tests will start with small blocks in the order of magnitude of the primary mesh size or even below proving that the system is able to stop these, too.

One of the major questions being still under discussion is the definition of targets. An actual proposition by the delegates of Austria, France and Italy is shown by figure 2.

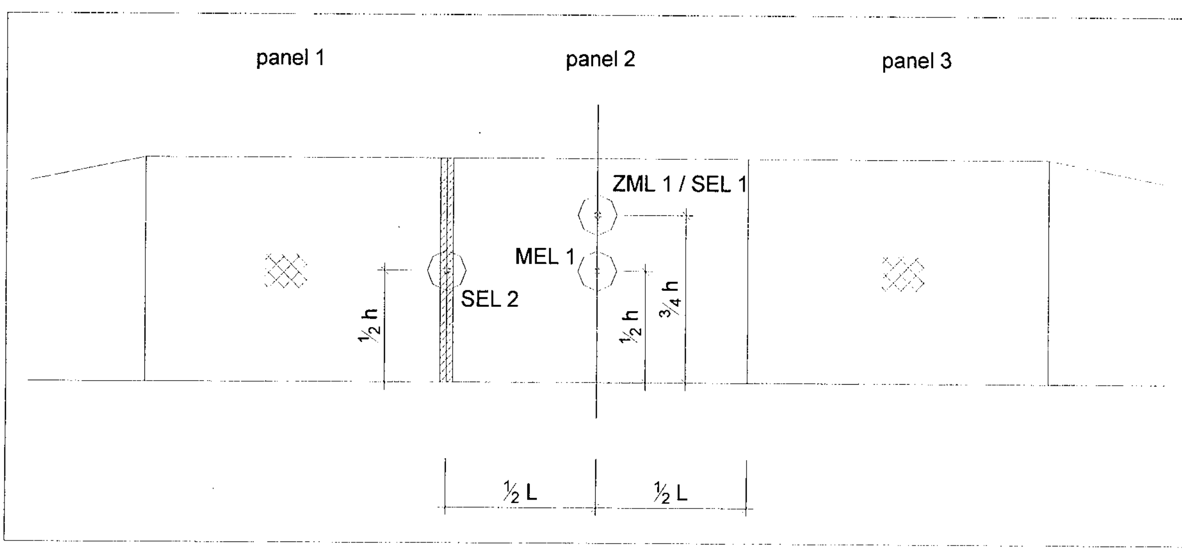

Fig. 2: Targets according to an actual proposition of the Austrian, French and Italian delegates.

The majority of the working group favors one test only of each kind with the same characteristics, i.e. the same energy under the same angle of impact and at the same point of the net. The resulting total number of tests would be four.

Assessment of the actual state: The actual proposal has some evident shortcomings.

- The **number of tests** is much too small for any statistical evidence.

- The proposed **targets** are not representative for the behavior of the system. They cannot reveal weak areas.

- The targets for the ZML and for the SEL 1 are identical whereas most of the area of the net is not tested.

- The **definitions** are qualitative. What does "no plastic deformation" mean in practice? Only small energies will be dissipated by pure elastic or better reversible behavior. Reversible behavior includes not only linear (following HOOKE's law) and non- linear elongation, but also friction due to relative displacement under normal stress. If one has ever seen an impact on a modern ring net, there is a lot of reversible displacement between the rings themselves and between the outer rows of rings and the suspension cables. In the immediate impact area, rings may become deformed permanently to squares or hexagons, depending on the number of neighbors, long before the brakes are activated. These local plastic deformations do not affect the remaining limit energy dissipation as long as no ring or cable brakes. By the way, the probability of a subsequent impact on just the same point is very low. Additionally ropes may be plastically stretched to an extent being hardly measured and not being visually detected. The definitions must therefore be based on clearly observable phenomena; "gray zones" will generate clearance for the assessment of test results and lead to disputes. The three energy levels have to be correlated to the frequency of rockfall resp. to the return period and to the resulting design considerations to give them a practical relevance.

Alternative approach: These requirements lead to the following suggestions.

- ZML represents the "daily events" with a return period up to one year. These events shall not cause any maintenance cost. The necessary removal of accumulated rock is not considered, because it does not depend on the barrier type.

- SEL represents the events with a return period of less than or equal to10 years. To accept maintenance is reasonable from economical reasons. Otherwise, the system would have to be over dimensioned. Because of that return period, being short in relation to the proposed lifetime of the structure, the barrier must be able to withstand repeated impacts with the SEL energy.

- MEL stands for events with return periods of less than or equal to 100 years. The system has to stop the block, the probability of a second one following in between the usual inspection period of less than 1 year, is very low. Thus, it is acceptable that the system looses its ability to catch subsequent blocks. For real applications the MEL might be combined with warning devices, indicating that certain important cables are broken.

Of course, it is up to the employer to decide for other return periods a design having to be based on. The suggested definitions are as follows:

1. **Zero Maintenance Level** (ZML) is characterized by the total energy a barrier can dissipate without any broken parts, gaps in the net, and activation of brake elements. Visible plastic deformation is allowed, if it is restricted to the immediate contact area between the block and the net. In case of doubt, this area is assumed to half the surface area of the block.

2. **Service Energy Level** (SEL) is defined as the total energy a barrier can dissipate in three subsequent tests at the same point of impact and without any repairs between the impacts under the condition that the rocks are stopped. Gaps are not tolerable; the remaining minimum height must be at least 2/3 of the original minimum height.

The restriction to thwo tests is necessary, because the SEL will be considerably higher than the ZML and will activate the brake elements. The brake elements have a defined maximum displacement. If this maximum is reached, the effect of the brakes is exhausted. SEL will include in practice the replacement of activated brakes. Three tests present a reasonable safety margin allowing two subsequent impacts between two inspections without creating a danger for the installation to be protected. Gaps would affect the protective effect of the barrier, as would a further reduction of height. Of course, one could discuss about a restriction to two tests and a remaining height of 50 %. To this author's opinion, it is the intention of these tests to come to safe designs and not to world records in energy dissipation. The suggested numbers follow this aim.

3. **Maximum Energy Level** (MEL) is the highest total energy a barrier can dissipate under the condition that the rock is stopped The system might have lost its ability to stop another block with the same or an even considerably lower energy. Gaps and broken parts are allowed. In practice, the system would have to be repaired or even replaced. With this tests not only the limit energy dissipation shall be determined, but the maximum elongation, too. There is a small uncertainty in the real maximum, because of the requirement the block being stopped. If the energy is bigger than that value and the net fails, the elongation might be greater.

All visible or measured changes after impact have to be described in a test report, being part of the certification.

- The proposed targets are shown in figure 3. Hits on retaining cables and posts are considered a must in spite of the facts that posts require a high accuracy of hitting and bearing the possibility of damaging the block. Because random factors shall be excluded, at least two tests are necessary to come to reliable results; three tests would be desirable. Two tests may be a compromise to be accepted from financial reasons. The more favorable test shall be neglected.

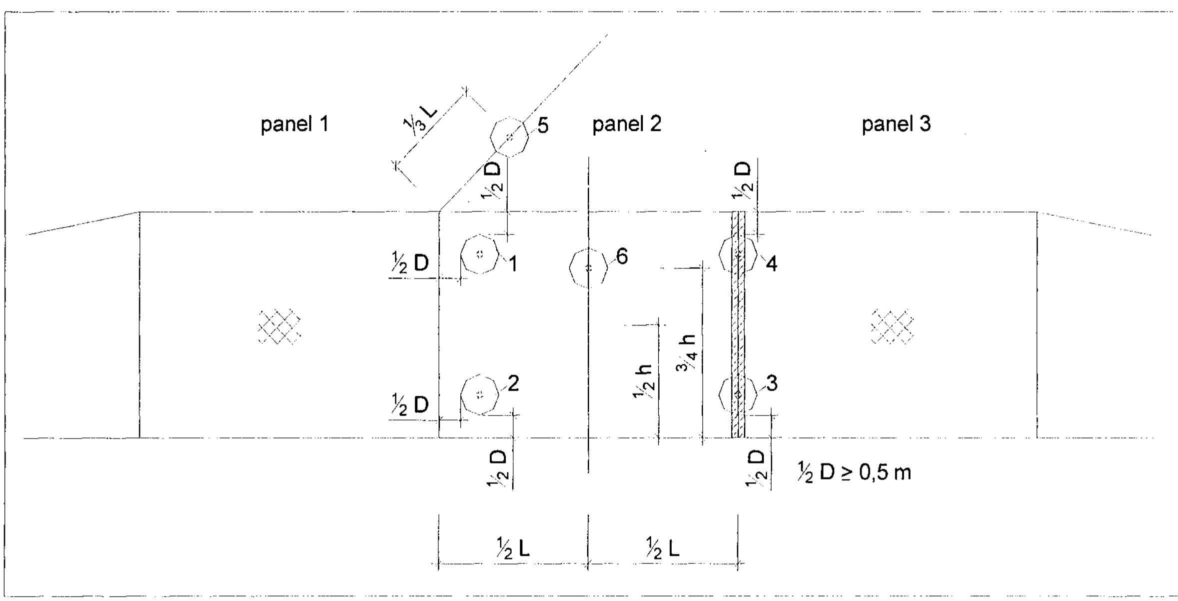

Fig. 3: Proposed targets according to this author's suggestion

- As already discussed above a comparison between different systems without getting the limit energy dissipation makes no sense. It is indispensable therefore to concentrate on the weak points of a system. The strong ones are of no practical use in design. It will not be necessary to test the weakest point of the system by the ZML, assuming that the weakest point is either independent of the applied energy or that its identification is more important for the SEL and for the MEL. This assumption reduces the number of tests without having a decisive influence on safety.

To the opinion of this author, the sequence of tests must be as follows.

1. SEL tests in the left lower and the left upper corner of the panel (or in the right corners, but anyway in the corners on the same side). Additional tests on the foot and on the top

of a post as well as in the downhill third point of a retaining rope, as far as the system allows for such hits. This are 2 x 5 = 10 tests. To reduce cost of testing the producer may determine the sequence of the above mentioned tests, for example to start with the most critical one and to interrupt the tests, if this one should fail.

2. ZML tests in the area proven as the weakest according to the previous test series. This requires two tests.

3. MEL tests in the area proven as the weakest in the SEL tests. This requires two tests. Additional two tests are necessary in the center of the barrier respectively in the area of the maximum flexibility. This requires another two tests respectively a total of 2 x 2 = 4 tests.

The total of tests is 16, if the tests series is not interrupted because a test did not fulfill the requirements.

If the system is over dimensioned in relation to the energy levels, the producer wants to have it tested, it might be difficult to identify the weakest point of the structure. In this case, the test engineer has to select the targets for ZML and MEL according to his experience with similar systems and on the base of previous tests of the producer. He has to explain the reasons for his decision as part of the certification.

In the case of a system having the posts and the retaining ropes on the downhill side of the net, SEL tests can be reduced to the above-defined corners of the middle panel (near the lower and the upper suspension rope), the zone with the highest flexibility and to the area, where a post supports the net. If the neat between two panels is apart from the posts, 2 x 4 = 8 tests are necessary, otherwise only 2 x 3 = 6. If no weak point could be identified during these tests, the test engineer shall decide on the targets for the ZML and MEL tests according to his experience and on the base of previous tests executed by the producer. He has to explain the reasons for his decision as part of the certification.

- **Measuring program:** Because the foundation conditions generally may be different from those of the test site and the forces transferred to the ground are dependent on the system a minimum approach is to measure the forces acting in the retaining ropes and in the foundation of the posts. The producer is free to order additional measurements, for example of the forces acting in the suspension ropes or / and in the posts.

- **Repair of damages** allowed: There is no repair of damages allowed during a test series with the same characteristics. Between the series it is up to the producer whether he repairs or replaces his system. If a system fails, the test has to be repeated.

- **Rock removal** after impact or accumulation allowed: The block is removed after each test. The effect of accumulation is not tested.

- **Tolerances:** Of course there is a need to accept tolerances, for example in measuring the weight of the block, its velocity and many geometrical data. These tolerances depend on the accuracy of usual measuring procedures in construction and range between 3 and 5 % of the required value.

- **Institutions** to carry out the tests and give the certifications: There will be a list established by national authorities with those institutions to certify rockfall barriers according to the above described regulations.

Conclusions

There is a need for harmonized tests not only in Europe, but also worldwide. A close international cooperation especially with our American colleges would be very desirable. The decisive points being still open are the basic tasks of the tests, the number of tests, the targets and the definition of the behavior under these different energy levels.

From a scientific point of view, there are some key points to be considered:

- The number of tests with the characteristics, i.e. the same hit area and the same energy must be at least 2, a statistically relevant number of at least 3 would be desirable.
- The most or the more favorable tests have to be neglected.
- The tests must give all data, the limit total energy dissipation capacity under different practically relevant energy levels especially, being necessary to select systems for a safe design.
- The tests must reveal the weak points of a system. Thus, all critical areas and parts have to be tested. It would be not acceptable having systems granted a ETA respectively a certification, if they were not tested under the most unfavorable conditions, for example if it is not tested that the system will not collapse under a hit on one of the retaining cables or a post.
- Economic reasons may be considered in defining the number of tests, but their influence must stay secondary according to the rule: don't compromise with safety.

Because the development of rockfall mitigation systems is rapid, the knowledge on the behavior of rockfall barriers is not developed to a general accepted state of the art, and a lot of decisive questions are still scientifically unsolved, the intended guidelines will represent a first step. An advanced version will be needed some years later, based on the experiences of the beginning tests on the Swiss test installation at Walenstadt, too. In spite of that perspective, it is urgent to finish this first approach to channel the actual development and to come to a defined safety standard, even if some shortcomings will remain.

Acknowledgements

The author is grateful to his colleges of the working group for the open and good discussions during the previous sessions and for many comments and suggestions related to the testing of rockfall barriers. He acknowledges furthermore the gentle guidance of the working group by its chairman, Mr. Baratono and the strong backup by the Mr. Henning from the German Institute of Construction, Berlin.

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REPLACEMENT OF TRADITIONAL ROCKFALL DETECTION WITH INNOVATIVE CATCHMENT AND DETECTION
A Case History at CPR's Red Rock Slide Fence

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ABSTRACT

Energized rockslide and rockfall detector fences were installed alongside Canadian Pacific Railway tracks in northern Ontario, Canada in various locations during the late 1960s and early 1970s. They are designed to trigger train control signals if individual wires on the fence are broken. One such fence was constructed at Red Rock; under a rock bluff almost 150 feet high along the shoreline of Nipigon Bay, northern Lake Superior. The fence included 2,830 feet of rockslide detector and 1,530 feet of rockfall detector.

In late February 2001 an eastbound freight train derailed at Red Rock, demolishing about 90% of the installed slide fence. In the planning process to replace the protection to the track provided by the fence, consideration was given to options other than rebuilding the slide fence in its original form. A range of options was presented to management that included sections of Brugg rockfall catchment fence, LockBlocks™, and traditional warning fence.

The solution adopted by CPR was to install 630 feet of Brugg fence, 615 feet of double LockBlock wall, to leave some sections of the rock face scaled but without additional protection, and to reduce the rock fall detector fence from 1,530 feet to 404 feet. This paper presents a case history of the derailment event, the investigation, the engineering design and the installation of the new Red Rock track protection system.

Introduction

The Trans-Canada railway line was completed by Canadian Pacific in northern Ontario in May 1855 with a "last spike" hammered in at Jackfish along the north shore of Lake Superior joining the two sections of track; one heading west from Montreal, the other heading east from Winnipeg. The rail line follows the shoreline of Lake Superior over certain sections of the territory, particularly where there would be significant elevation gains required to drive the line inland. One such area is near Red Rock, on the western shore of Nipigon Bay at the northernmost extent of Lake Superior. Rock bluffs rise to more than 1,200 foot elevation, 600 feet above the lake, with sheer faces over 150 feet high immediately adjacent to the track. At this location the tracks of Canadian National Railway run to the east of the CP tracks, closer to the shoreline of Nipigon Bay.

In order to protect the track structure, the operations of trains and thus the security of the investments of shareholders against delays and other detrimental impacts of rockfall related activity, CPR decided to install a number of energized rockfall detector fences in the 1960s and 1970s. At Red Rock, 2,830 feet of rock slide detector fence and 1,530 feet of rockfall detector fence were installed in 1968 between Mileage 66.45 and 66.98 on the Nipigon Subdivision of what was then the Algoma Division. The train derailment that occurred on the 2nd of March 2001 destroyed approximately 90% of the detector fence installation and CPR decided to investigate alternate solutions to protect their investment in the aftermath of the derailment and to improve the protection against rockfall events.

This paper describes the Red Rock location geologically and geographically, reviews the derailment event, discusses the follow up track engineering and rock engineering required to rehabilitate the site, and comments on the construction of the new integrated Red Rock track protection system.

Geology and geography

CPR's mainline track in this area was driven through various parts of the Precambrian Canadian Shield. In the very northern extent of Lake Superior, there is a dominant sequence of diabase sills (1109 Ma) overlying flat-bedded sedimentary sequences (Sibley Group) of the Mesoproterozoic with low-level metamorphism in the lowermost greenschist and zeolite facies. The bluffs above the track are made up of thickly bedded meta-sedimentary rocks, mostly iron-rich silty sandstones, overlain by the thickly flow-banded diabase Nipigon sills[1]. The overlying sills have produced erosion-resistant rock masses that present as flat-topped hills with pronounced sheer sidewalls. The underlying metamorphosed sedimentary rock masses are locally preferentially eroded, leaving overhangs of the more durable volcanics, but not in all cases. The rock masses are predominantly massive, with limited natural jointing.

The north shore of Lake Superior has a number of very large bays that extend northwards from the main body of the lake. They have been formed by the flooding of glacially eroded valleys with northeast to southwest drainage patterns. The western margin of

[1] Ontario Geological Survey 1991. Bedrock geology of Ontario, west-central sheet; Ontario Geological Survey, Map 2542, scale 1:1,000,000

Nipigon Bay runs approximately north south over almost a mile underneath the bluffs of sedimentary and mafic igneous rocks, as Nipigon Bay pinches into the Nipigon River, see Photo 1.

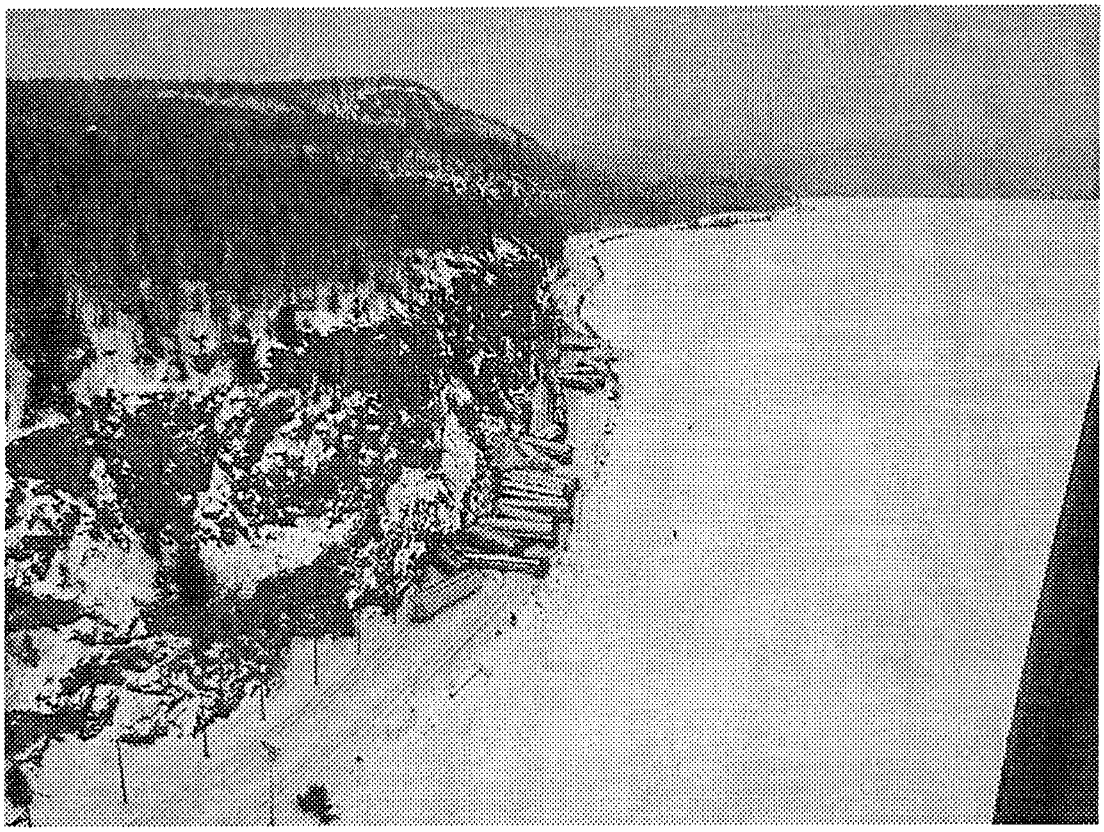

Photo 1. 2nd March 2001. Ice and snow cover Nipigon Bay and the Red Rock bluffs looking north. The CPR track is to the left, and the CNR to the right. In the distance the track curves around towards Nipigon at the mouth of the Nipigon River. The remains of the detector fence can be seen at the bottom left of the photograph.

Recent history – 2nd March 2001 derailment

The rock masses making up the Red Rock bluffs have been responsible for a limited number of rockfall events over the years, but enough for the railway company to decide to install a warning fence system in the late 1960s. The energized fence concept is designed to allow the railway signalling system to alert a train engineer that at least one of the fence wires has been broken. This breaks the flow of current and causes the signal to display a red light. The rail traffic coordinator can permit the train to proceed through the "block" at reduced speed such that the train can be safely brought to a halt within the sight distance ahead. Thus the train crew is not delayed unduly if the wires have been broken by strong winds, trees, ice or other benign cause, but can stop in time to avoid a train strike or derailment if there is a large rock on the track.

On the 2nd of May 2001, an eastbound grain and mixed goods train had a green signal and was proceeding through Robford and past Red Rock on its way towards Nipigon. As it reached Mile 66.8 Nipigon Subdivision the third unit was reported to have "dropped a wheel" and as the derailed train ground to a halt, 16 grain cars "concertinaed" into the rock bluff, see photo 2.

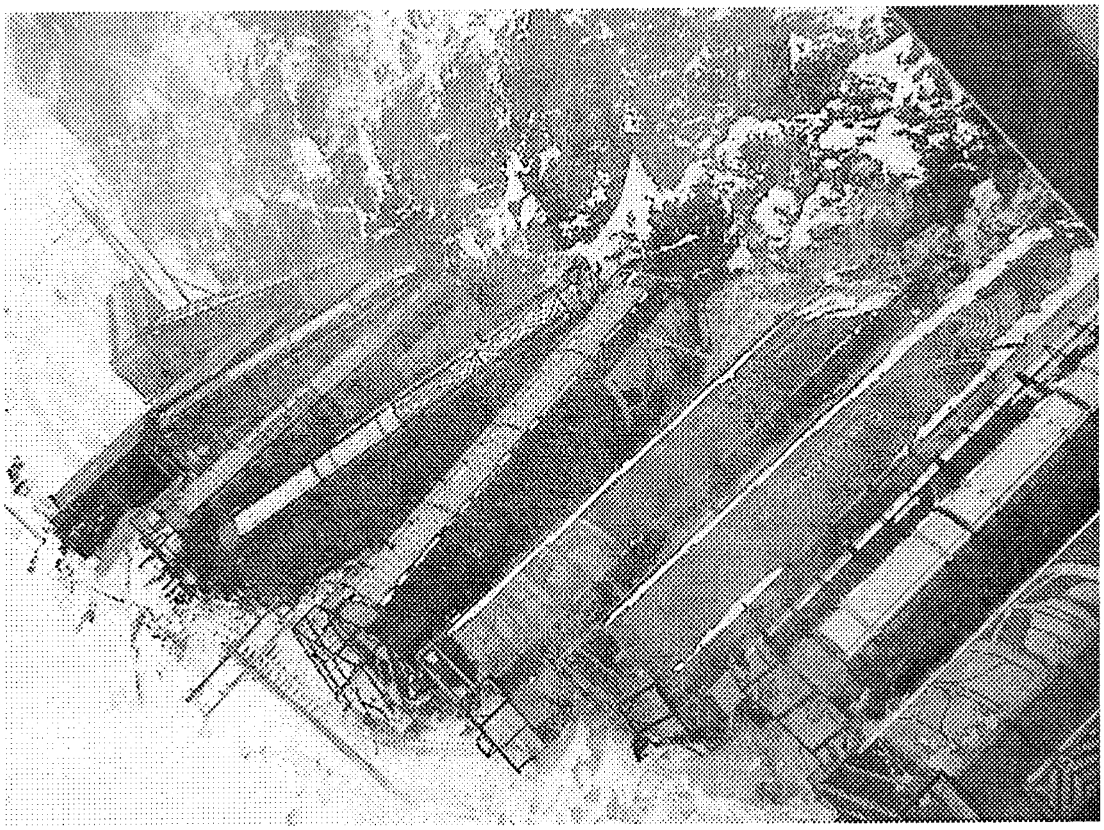

Photo 2. Aerial view of the grain cars derailed into the bluff at Red Rock. Note damage to CN track near standing pole and destruction of warning fence.

In doing so, both main lines of Canadian Pacific and Canadian National were taken out of service, as well as the detector fence. The first order of business was to remove the damaged rolling stock, rebuild the damaged track and ensure that the rock face was safe for the resumed running of trains without the detector fence in operation. CPR employed a 24-hour a day watchman to patrol the site until the new replacement system was operational. The point of derailment was identified as being approximately Mile 66.8. It was also noted that the lead units on the train that derailed were not affected by the derailing mechanism, although one wheel of the third unit had dropped. It was reported that the slide fence triggered within a number of seconds of the train passing by the slide fence, although there is no evidence to indicate that the fence was triggered by falling rock – it is quite likely to have been triggered by the derailing train. Most of the slide fence had been removed during the clean-up operation and it was now possible to make a detailed survey of the slope configuration without slide fence wires impeding the process.

Site Inspection

The authors visited the site on 20th March 2001 in order to review the rock engineering aspects of the project and to recommend an appropriate course of action. At the time of the site visit the derailment had been cleaned up and track panels had replaced the torn-up track. The principal purpose of the rock engineering review of the slope was to determine the suitability of installing energy absorbing catchment nets (Brugg fences particularly) at the site to replace the existing rock detector fences installed in the late 1960s in order to provide appropriate protection against rockfalls. The signal maintainer for the area was interviewed on site and he reported that there was only one location where he recalled rockfall events to have occurred, although there were many locations where ice was responsible for triggers.

To determine the design requirements of rockfall catchment nets, it was necessary to take a series of slope measurements in order to estimate the energy generated during rockfall events and to evaluate the rockfall potential – and thus hazard – associated with the steep rock bluff at the site. At each 1/100th mile station from 66.90 to 66.62 (as established using a measuring wheel on the south rail from the Mile 67 board) the following measurements were obtained:

- Distance from inside edge of north rail to rock face – equivalent to catchment width, or ditch width. In some locations the rock face was overhung, and estimates of the amount of overhang were made.

- Slope distance and slope angle from the outside of the south rail at an instrument height (HI) of 5.9 feet to the crests of the various slope segments visible above the track. The rock bluff is made up of a thick meta-sedimentary layer and two sub-horizontal lava flows, each 30–50 feet thick. The different layers lead to intermediate benches at different heights up the bluff and attempts were made to identify and measure the locations of benches, where appropriate and possible.

- Brief comments regarding slope attributes were also recorded for each station.

At five (5) locations along the track, detailed cross sections were taken to assist in developing rockfall simulation models that could be used to confirm the adequacy of proposed catchment control devices. These sections were taken at Miles 66.84, 66.81, 66.79, 66,71 and 66.68. Further sections to the east were not considered necessary since the crest of the bluff moves away from the track to the east of this location creating a wider natural catchment area. Measurements were taken as follows:

- The offset from the toe of the slope was measured – the sections were all taken from outside the east rail of the CNR track close to the crest of the small slope to Nipigon Bay, 45 to 62 feet from the toe of the rock bluff. The distance from the toe of the slope to the gauge of the CPR north rail was also measured.

- The slope angle and slope distance to three to six points on the face were measured using a laser range finder and a clinometer.

Rockfall zones

The rock mass in this area is made up of a series of sub-horizontal rock sequences, as described above. There is a lower, meta-sedimentary sequence overlain by two major igneous intrusive events that account for the 150-feet vertical section making up the rock bluff to the west of the track.

The lower 1/3 of the bluff, the oldest, makes up the lower slope adjacent to the track between 30 and 50 feet high. It is characterized by moderate weathering, blocky to slabby structure, dark reddish grey to orangey red colour, fine grain size, medium strength, and is a slightly metamorphosed silty sandstone. This portion of the rock mass is susceptible to weathering and ravelling from differential erosion. The block size of the loosened rock created is small to medium, generally in the 6" to one-foot size range. To date there has been no evidence of a collapse of overhanging material, but this is cause for some concern in the long-term.

The lower sill is characterized by slight weathering, blocky to columnar structure, greyish red colour, fine grain size, it is a strong, igneous intrusive, diabase. This igneous flow is made up of various bands of different character material that respond differentially to the weathering influences at the site. It appears from aerial inspection that there have been a number of rock fall events from this part of the geological section that have deposited loose rock on an intermediate bench above the upper portion of the lower flow. This sill appears to be the source of the majority of the rock fall material in recent years. In fact, the signal maintainer reported that there is only one location along this stretch of slide fence that generates rockfall triggers and that is at Mile 66.79, approximately 50 feet above the track. All other triggers of the fence are due to ice, wind, trees, etc.

The upper sill is characterized as fresh to slightly weathered, massive to columnar, grey to dark reddish grey, fine grained, strong to very strong, igneous intrusive, diabase. This upper igneous flow appears to be tight with little evidence of local rockfall potential. There are, however, some very large columns of rock that are detached from the main bluff and standing vertically with little restraint. These were noted during the aerial inspection and are scheduled for ground truthing when it is possible to get onto the slope during the construction season with scalers. Although this upper flow accounts for the surface of the bluff up to 150 feet above the track, it is not seen as a major threat to the safe operation of the railway.

The track length below the rock bluff was divided into eight (8) zones, each with reasonably distinct characteristics based mainly on slope geometry. The criteria used to differentiate zones included: available ditch width, overall slope height and slope angle relationship, presence or absence of intermediate benches and 'launch' features, block size and shape of potentially unstable material, and weathering potential.

Using the slope configurations measured at the time of the rock engineering site visit, a number of 'test sections' were drawn up and simulated rockfall events were carried out. Results of rockfall simulation tests (using the software package Rocfall; see Figure 1, for example) showed that various rockfall events from the bench around 55 ft above the track released 42 to 90 kJ of kinetic energy. Once this work had been completed a cost analysis was carried out to compare the overall costs associated with different rock engineering solutions.

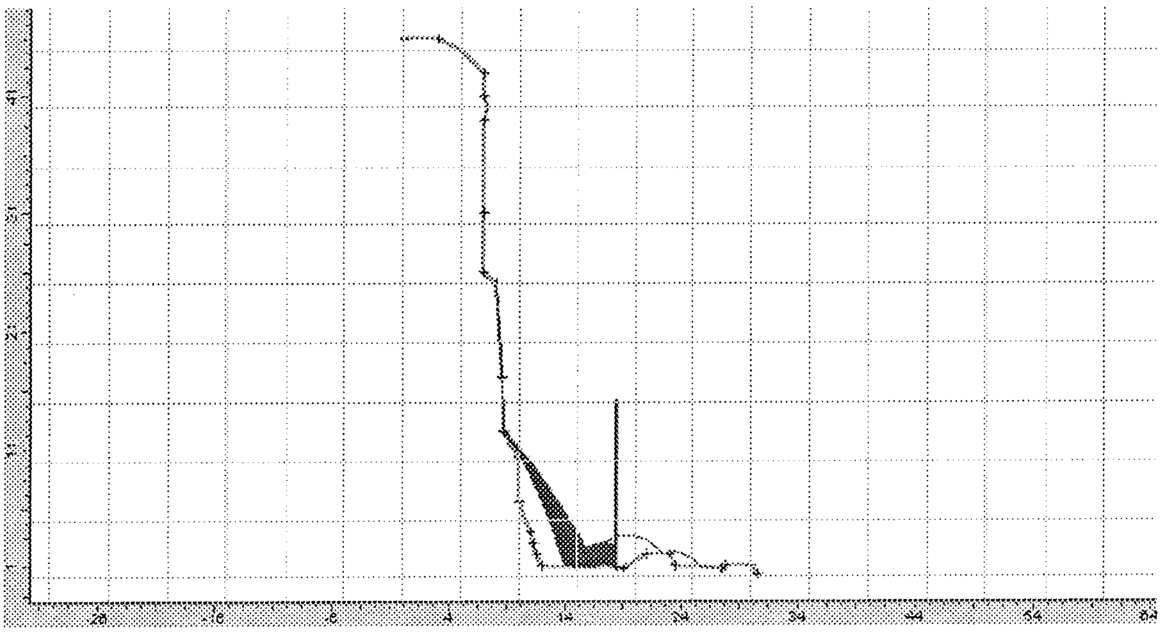

Figure 1. Rocfall simulation a Mile 66.71 Nipigon Subdivision. 1000 rocks falling from the upper bench using slope geometry from field survey and showing Brugg fence.

Final Design

The final design selected for protection of the track and safe train operations at the site called for the following integrated catchment and detection system. The catchment design incorporated a 110kJ (40 ft. ton) ring net, or Rocco, system designed by Geobrugg where rockfall energies were considered significant and a LockBlock wall where lower energies would be expected. A further site visit was undertaken in May 2001 to refine and finalize the boundaries between the different components of the system.

All elements were designed to be installed a minimum distance of 10 feet from the nearest rail.

The various location zones were based on the initial filed studies, with some modifications to allow for operational constraints, fence panel sizes and the like.

Location	Length	Description
Zone 1	125 feet	2 high LockBlock™ wall – 49 blocks
Zone 2a	250 feet	10' high Brugg fence – 10 panels 10'x20', 2 panels 10'x25'

Location	Length	Description
Zone 2b	264 feet	CPR rockfall and rock slide detector fence
Zone 3	380 feet	10' high Brugg fence – 19 panels 10'x20', 20 posts 11'
Zone 4	140 feet	CPR rockfall and rock slide detector fence
Zone 5a	120 feet	Rock face, hand scaling required
Zone 5b	280 feet	Rock face, no protection required
Zone 6	370 feet	Rock face, no protection required
Zone 7	490 feet	2 high LockBlock™ wall – 195 blocks, 4-wire slide fence
Zone 8	280 feet	4-wire CPR rock slide detector fence

Construction

Initial hand scaling and some rock bolting work was carried out by one contractor in late July 2001, followed by the installation of the various catchment and detection elements in August through October 2001 by another contractor.

The construction work followed in a sequence that became established for the project. The sequence was repeated for each zone although the final effect was quite different from zone to zone.

Site Preparation – the area was cleaned out of all debris to the base of the rock face, this included rock, overburden and railway materials left over from the derailment. Fibre optic cables had also been installed adjacent to the track throughout the site; these had to be carefully located to ensure that no damage took place. There were also a few strings of rail that were to be removed before construction work was started.

Foundation Preparation – the area was marked for either posts (Brugg fence and CPR detector fence areas) or for the footprint of the LockBlock walls, and trimmed to line and grade as required. Templates were used to mark the outline of the Brugg fence posts, which required 8-foot long vertical bolts to secure them. These were drilled using hand-held plugger drills, and were grouted into place.

Erection – the Brugg fence posts or the CPR detector fence posts were then erected, see Photo 3, and the LockBlocks installed, Photo 4. Once the posts were stabilized with guy anchors and maintained in their vertical position, the Brugg fence panels were hung, Photo 5, the detector fence wires were strung and once the LockBlocks were placed an upper, 4-wire fence was installed in Zone 8.

Electrical Connection – the final component of the installation was to string power to the CPR detector fences, through conduits where required across zones where the fence was not active, and also through the Brugg fences. They were designed to allow the signal wires to detect abnormal movement of the Brugg fence, so that if a significant rockfall event were to take place that exceeded the energy absorption capacity of the net, the signals would be tripped exactly the same way that would occur for a regular detector fence.

Photo 3. Brugg fence posts and guy wires, Zone 3

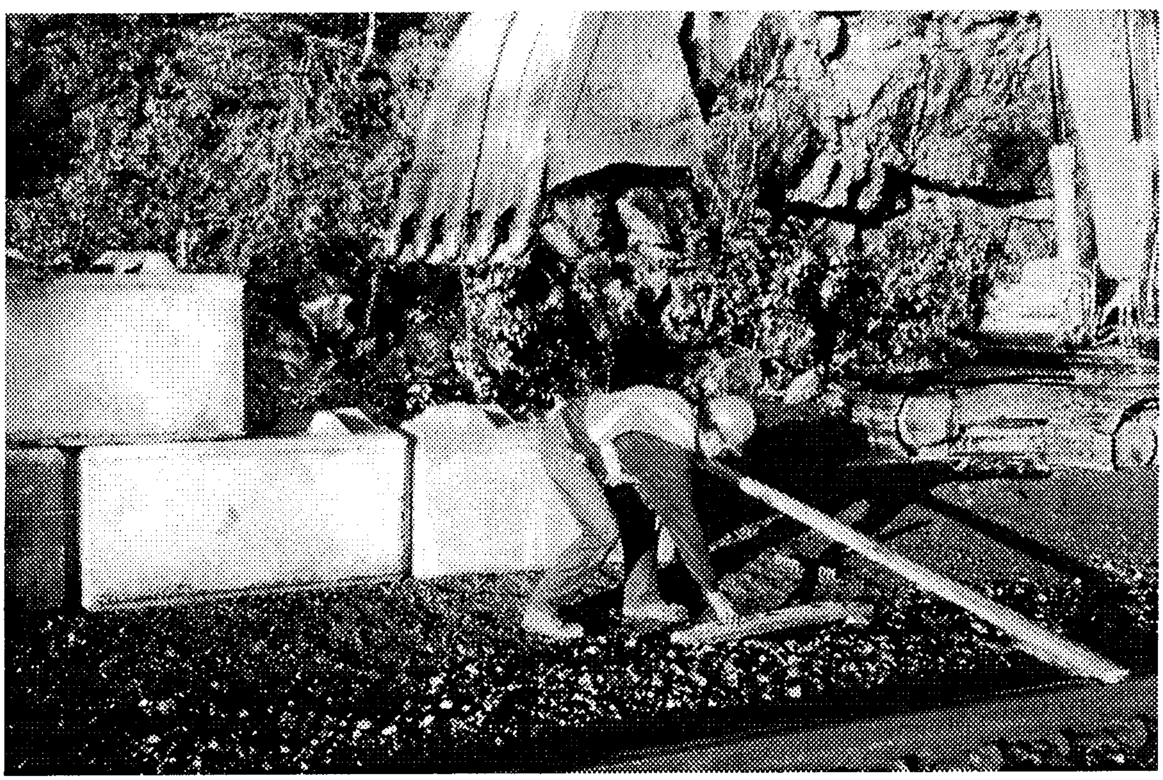

Photo 4. 2 block high LockBlock wall under construction at Zone 7

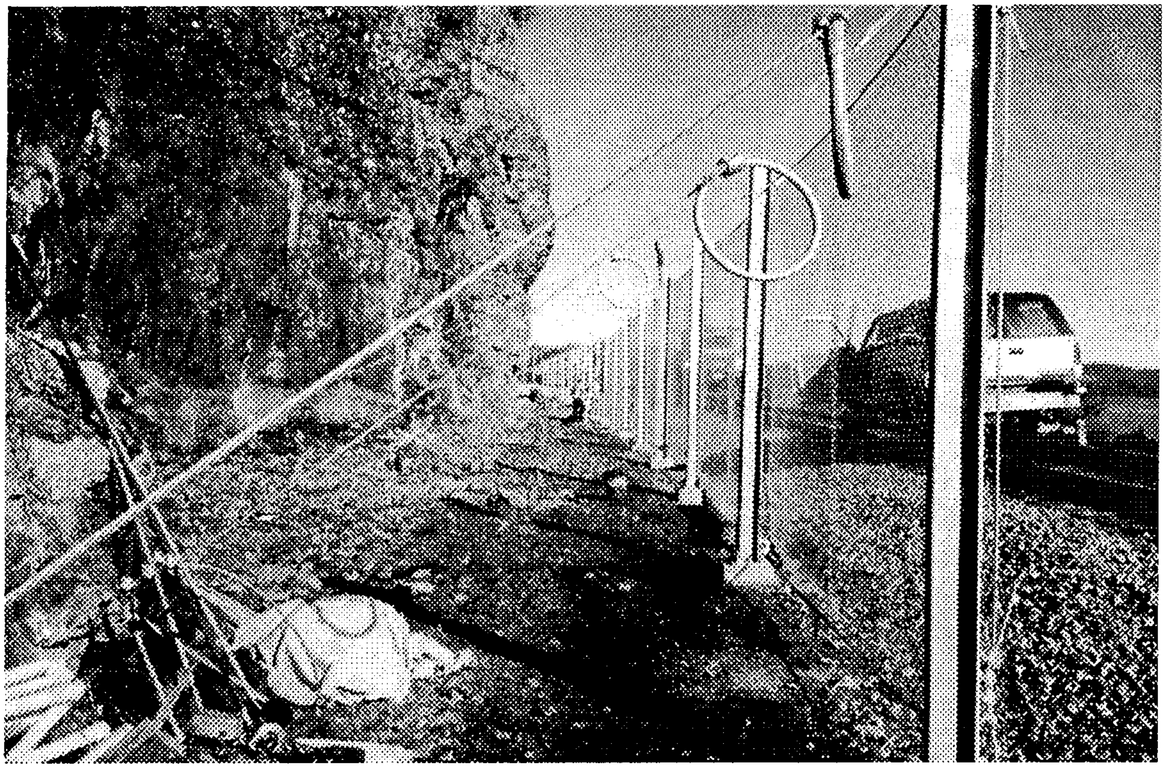

Photo 5. Brugg fence with net panels hung, preparing for final electrical connection, 11 September 2001.

Clean Up – Prior to demobilization, the installing contractor removed all remnant debris from the site and reinstated as much of the railway infrastructure as possible.

Costs

The following table, Table 1, gives the revised cost estimate for the innovative solution of catchment and detection components. The original estimate to replace the detector fence system was $392,832.00. The final solution was completed on time and within budget.

Conclusions

Following the derailment at the Red Rock bluff on 2nd March 2001 CPR elected to carry out an engineering study to evaluate the track protection required along the length of the 100-150 foot high rock face. Careful assessment of the rock mass conditions in different zones along the bluff allowed a variety of elements to be installed that would cater directly to the style of rockfall hazard identified. The final solution replaced 2,830 feet of rock slide detector fence and 1,530 feet of rockfall detector fence that had been installed in 1968 with 630 feet of Brugg fence, 615 feet of double LockBlock wall and 404 feet of rock slide and rockfall detector fence. Construction started with scaling and rock bolting in August 2001 followed by fence and LockBlock wall construction in September through October 2001. The final system was officially put into operation at 1330 hrs on 6th November 2001.

Mile 66.8 Nipigon Sub. - Slide Fence Replacement Revised Cost Estimate 20/06/2001					
Zone	**Location**	**Length (ft)**	**Description**	**Cost per ft. (Installed)**	**Total**
1	M 66.9 - M 66.876	126	5ft (2 high) lock block wall	80.00	$10,080.00
2a	M66.867- M 66.82	248	10ft.high Brugg rock catchment net	225.12	$55,829.76
2b	M 66.82 - M 66.77	264	Standard CPR Overhead 20 wire rock fall fence with 12 wire slide fence	200.00	$52,800.00
3	M 66.77 - M 66.70	380	10ft.high Brugg rock catchment net	234.13	$88,969.40
4	M 66.70 - M 66.675	140	Standard CPR Overhead 20 wire rock fall fence with 12 wire slide fence	200.00	$28,000.00
5a	M 66.675 - M 66.665	120	Scale only	50.00	$6,000.00
5b	M 66.665 - M 66.62	280	No fence required	0.00	$0.00
6	M 66.62 - M 66.55	370	No fence required	0.00	$0.00
7	M 66.55 - M 66.458	490	5ft (2 high) lock block wall with 4 wire slide fence	95.00	$46,550.00
8	M 66.458- M 66.405	280	Re-install existing 4 wire slide fence	20.00	$5,600.00
					$288,229.16
			Plus 10% contingency		$28,822.92
			GRAND TOTAL		***$317,052.08***

Table 1. Revised costs estimates prior to construction of the fence system.

Cost savings of over $75,000.00 were realized compared to re-installing the original detector fences and the final installation will be less demanding on the maintenance budget by reducing spurious fence triggers. The outcome is a cost effective solution with improved track safety.

Acknowledgements

The authors wish to acknowledge the support of Canadian Pacific Railway in the preparation of this paper, and their long-standing commitment to rock engineering across their North American system.

LAND SLIDE AND ROCK FALL HAZARD MITIGATION IN A MULTIPLE INTRUSIVE PRECAMBRIAN DIABASE SERIES ON US 60 IN SALT RIVER CANYON, ARIZONA

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ABSTRACT

US 60 was constructed in the 1930's through a series of diabase dikes and sills that outcrop in the Salt River Canyon, in Central Arizona. Spheroidal weathering in the diabase has reduced much of the rock mass in the road cuts to core block boulders in a weakly cemented, sandy matrix. In time periods ranging from 3 to 10 years, the matrix ravels from the cut faces due to wetting and drying, freeze/thaw and weathering, which de-couples large blocks, producing rockfall and landslide hazards, that require on-going maintenance. Cut slopes for the highway range in height up to 200 feet.

At one location on the east side of the Canyon, there have been a series of rockfalls that have blocked the highway. Attempts have been made in the past to mitigate the problem by scaling the slope and, later, by covering the slope with mesh. These efforts temporarily controlled hazardous rockfall from this slope. However, during the winter of 2000, a landslide destroyed the slope mesh and exposed a larger decomposed diabase slope. After renewed rockfalls in early 2001, the Arizona Department of Transportation (ADOT) decided to implement more substantial measures to mitigate the problem.

Preliminary designs were prepared for several options, each of which was evaluated on the basis of cost, construction safety, impact on traffic, reduction of risk of future rockfalls, and environmental impacts. The selected design involves realignment of a portion of the highway, using conventional, cantilever retaining walls, creation of an elevated rockfall catchment area with an MSE retaining wall, and erection of a GeoBrugg rockfall containment fence at the top of the retaining wall.

INTRODUCTION

Approximately 125 east of Phoenix, Arizona, US 60 passes through the rugged Salt River Canyon between the communities of Globe and Show Low (refer to the Arizona Project Location map – Figure 1). At Milepost 295 a series of rockfalls, and landslides have blocked the highway. The Arizona Department of Transportation (ADOT) has implemented rockfall mitigation measures including slope scaling and wire meshing at the site over the past several years. During the evening hours of November 7th 2000 a large mass of rock talus, colluvium and weathered bedrock detached from the region directly above the highway in the vicinity of previously placed wire mesh (refer to Plate 1). The failure was apparently of a translational nature with the movement surface being shallow and sub-parallel to the colluvium/bedrock contact. As this mass moved it dislodged the wire mesh, supporting cables, and anchors, transporting it en-mass down slope to the highway. Removal of the failed material was complicated by the presence of the mesh and cables (see Plate 2).

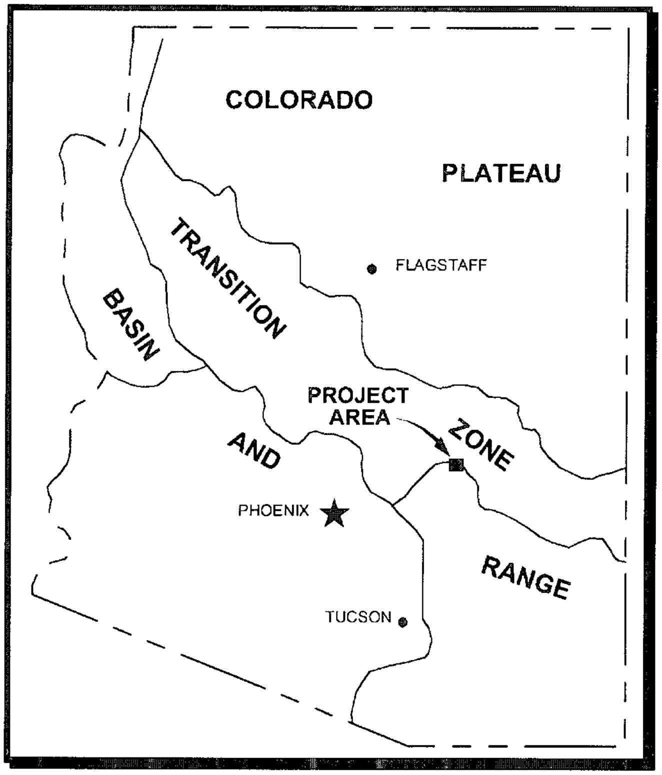

FIGURE 1. Arizona Project Location

PLATE 1. Draped Mesh on Slope Prior to November 2000 Failure

PLATE 2. Failed Rock and Mesh in Highway Ditch

In response to the failure, ADOT immediately initiated an emergency rockfall-scaling project and retained Golder Associates to commence a study of alternative stabilization and rockfall mitigation measures for the site. In February 2001 and November 2001 additional scaling was carried, followed by construction of an elevated rockfall catchment area with an MSE retaining wall and a GeoBrugg rockfall containment fence. This work required minor realignment of the highway and was completed in April 2002.

Project History

This section of highway was constructed in 1934. Publications indicate that work was initially advanced through the rugged country with, "sweat, muscle, steel and dynamite"(Rath 1934). Approximately 120,000 pounds of high explosives are reported to have been consumed in this section of the highway. After access was gained, heavy earth moving equipment was employed to remove rock and build the highway embankments. In the project area, the original as-built plans display, an 8-foot outside and a 16-ft inside lane, at a 6% grade, with an especially detailed widened shoulder, cut into the toe of the rock slope.

In the time period from 1940 through the early 1960s very little rockfall documentation, has been discovered in the highway records, for this section of US 60. However, there are indications of rockfall and landslide events in the state photographic archives and maintenance workers recollections of car-sized boulders on the road from time to time. Routine storm and rockfall patrols undoubtedly removed countless amounts of material that was never completely documented. Shride, (1967) was the first to document the accelerated deterioration of rock slopes in diabase materials in the vicinity.

In the period from 1993 through the present, the site has received the attention of 5 separate rockfall mitigation projects. The focus of most of these efforts had been to remove potentially unstable rock by scaling. However, each of these efforts led to additional slope raveling and rockfall. In 1998-1999 a wire mesh drape was placed on the slope to decelerate and redirect rock fall. This improvement failed during a high intensity rainstorm on November 7^{th}, 2000 when the fabric became overloaded with debris from a landslide originating behind the fabric de-bonding anchors that were holding support cables in place. (It should be noted that although the net was dislodged it still contained a significant portion of the mass movement allowing maintenance workers to reopen the road to traffic in a short period of time.) The latest efforts to control rockfall consist of removing additional material and creating an elevated rock fall containment system at the toe of the slope with independent access for rock removal. To date over 2.3 million dollars have been spent to mitigate the rock fall hazard at this site.

GEOLOGY

Regional and Local Geology

The Salt River is the physiographic and political boundary between the San Carlos Indian Reservation and the White River Apache Reservation and US 60 crosses from the south side of the river to the north side of the river approximately 1 mile west of the rockfall site. Upstream of the project area, the Salt River consists of "a drainage area of 864 square miles, and delivers approximately 380,000 acre feet of water per year" to Roosevelt Lake. The gradient of the Salt River varies from between 50 ft/mile (in the upper Canyon) to less than 10 ft/ mile (in the lower Canyon)" (Davis and Others 1981)

The Salt River Canyon is located approximately 30 miles north of Globe, in Gila County, Arizona. This regional hydrographic feature extends approximately 50 miles in a northeast to southwest direction in central Arizona's Transition Zone, between the confluence of the White and Black Rivers to the western shores of Roosevelt Lake. US 60 traverses the canyon with a series of maximum grades and hairpin turns through extensive outcrops of Upper Proterozoic clastic and igneous lithologies (refer to the Geology of Salt River Canyon map – Figure 2). When first conceptualized, it was noted that in order to traverse the canyon, (a horizontal distance of five miles), eleven miles of highway and approximately 3,000 feet in elevation change, had to be overcome (1400ft. down, 1500 ft., up) (Jones, 1931).

The rock units in the project area have been assigned to the Apache Group, which consists of five distinct formations. They are the Pioneer Shale, Dripping Springs Quartzite, Mescal Limestone (with overlying basalt flows), Troy Quartzite, and Multiple Diabase Intrusive Sills. "The sills are remarkably persistent. Some, only a foot thick, extend more than a mile and several, a few hundred feet thick, extend at least 20 miles" (Shride, 1967).

The sedimentary formations exhibit numerous unconformable contacts between members of differing depositional environment, degree of consolidation, and shear strength. The great age of the materials has also exposed much of Apache Group to repetitive cycles of burial and exhumation, weathering cycles and tectonic stresses. Karstic features have also been identified in the Mescal Limestone.

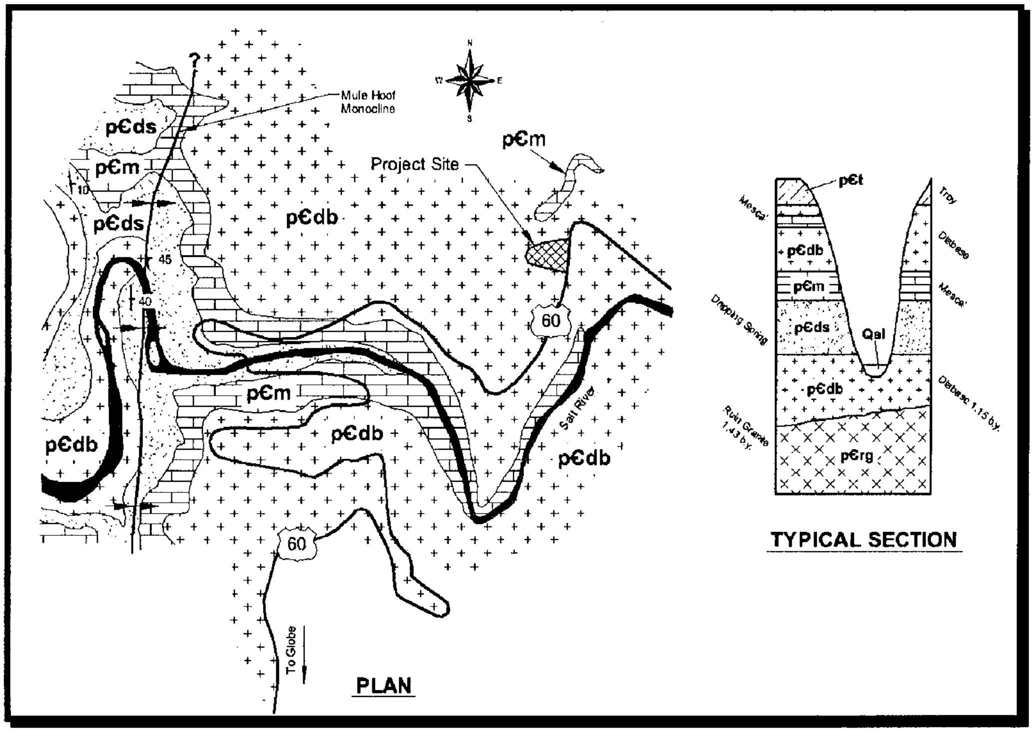

FIGURE 2. Geology of Salt River Canyon

Diabase has selectively exploited the weakest zones and dilated parts of the Apache Group with extensive sills, which make up a majority of the road cuts in the Salt River Canyon. .

Within the area of study, the Mescal Limestone is the host of a pronounced multiple sill which has been described by Fouts (1974).

> A large multiple sill is exposed in the walls of the Salt River Canyon near Highway 77 (now US 60). The total exposed thickness of diabase of is approximately 920 feet. The diabase overlies the Mescal limestone and in turn is unconformably overlain by Devonian Martin Limestone. Thus an unknown amount of diabase has been removed by erosion and the original thickness of the complex is unknown. The upper 80 feet of the complex is deeply weathered, apparently resulting from surficial exposure prior to deposition of the Martin Formation. A short distance west of the highway the diabase complex consists of at least two separate sills separated by a thin sliver of Mescal Limestone that appears to be little disturbed. The limestone pinches out toward the east so that the two sills are in contact, forming a thick multiple sill. This contact is marked by a zone of fine-grained diabase, presumably representing a chilled zone, and suggests that the lower sill crystallized before the upper sill was emplaced. The lower

sill is approximately 670 feet thick and the remaining part of the upper sill is about 250 feet thick.

Shride (1967) has given the classical petrographic description of Central Arizona diabase. A few excerpts are included below. The diabase is a fine to coarse grained, medium to dark-gray holocrystaline rock, of ophitic to sub ophitic texture. It weathers light olive green to moderate yellowish brown and can be distinguished at a distance by the characteristic greenish gray or olive gray hue of barren concave slopes. Polkilitic grains of pyroxene characteristically protrude on weathered surfaces, which therefore appear knobby or warty. Spheroidal weathering to rounded boulders is common. The rock disintegrates to a light green or yellowish brown granular soil that commonly includes abundant kernels that also represent ophitic aggregates.

Geologic Conditions that Contributed to the Rockfalls and Landslides

Weathered diabase is classically exposed on the lower half of the road cut at the site. The spheroidal to warty textures and the deeply weathered condition, are probably the result of numerous exhumations and reburial over the past 1.4 million years, although relic jointing and blocky texture are apparent. The material easily yields to moderate hammer blows and the elements. Generally, the rock quality of this lower portion of the sill is no more durable than a residual soil. Shortly after being exposed to the elements the more durable portions of the lower sill succumb to liesegang weathering and produces "nests" of core blocks diabase material having a unit weight of approximately 190 lbs per cubic foot.

Cliff-like erosion of resistant diabase makes up much of the upper half of the sill. This material characteristically exhibits persistent, planar, vertical and transverse joints. The condition of the upper sill varies somewhat with the height of the cut. There are zones of resistant fractured rock cliffs and ledges (resembling basalt flows). There are also rock materials, which appear to have deteriorated to saprolitized clay. In other places portions of the lower sill appear to intercalate with the more durable rock material. Generally, the weathering is so developed that disturbing material some ten feet away can dislodge the resistant materials. The orientations of the joints in the resilient diabase are also adversely oriented towards the highway. Consequently the slope can produce all manners of rock failure, including block sliding, wedge, and toppling.

The top of the upper half of the sill is mantled with a clayey to sandy boulder colluvium that is deposited on a bench formed between the top of the sill and the bottom of the overlying sill. Angular rock material from overlying slopes accumulates in the form of talus on the upper and side slopes adjacent to the resistant diabase. These materials are deposited at their angle of repose and only a slight disturbance from wildlife, rainfall or freeze/thaw could send a boulder hurtling down the 200-foot slope face of the landslide area.

Modes of Failure

The lower part of the sill dislodges core blocks and differentially erodes much quicker than the upper part of the sill. The upper portion of the sill is then undermined from the bottom and susceptible to large block movement. Development of saporatic clay seams in the adversely oriented joints reduces the cohesion, friction angle and joint roughness between interlocking blocks. During high intensity rainstorms the joint spaces become conduits for moisture that further reduce the strength characteristics of the clay seams. Consequently, as the blocks loosen the overburden soils are disturbed. With enough material is disturbed and the moisture content rises from rainfall, tension cracks form and the top of the slope fails as rockfall or a landslide.

Additionally, the steeply developed colluvium and talus deposits concentrate moisture in the upper slope soils and a temporary piezometric surface forms near the soil rock interface. As these soils become saturated they fail as debris flows, removing the overburden, some talus and portions of the upper sill block. The slope then resembles an inclined gullied surface that is experiencing head ward erosion.

The recent failure of a rockfall mesh appears to have occurred when large, angular blocks from the top of the sill were entangled in the mesh. These blocks did not drop to the catchment ditch at the toe of the slope but were caught midway down. Subsequently, additional material dislodged and was deposited on top of the large blocks during a torrential rainstorm. At this time, the mesh was probably well strained. The rainstorm saturated the upper slope. The resulting mass movement de-bonded the mesh anchors and over 200 yards of rock materials dropped to the toe of the slope. It should be noted that the mesh retained a great deal of the debris and allowed the highway to be open to traffic in a reasonable amount of time. However, removing the rock material with the entwining mesh turned out to be a very tedious and costly operation.

OPTIONS FOR MITIGATION

Figure 3 shows a cross section through the slide area following the rockfall in November 2000. At that time, there was a head scarp up to 20 feet high, which consisted of colluvium and highly sheared diabase. There were several loose blocks of rock resting on the slide surface. At about mid-level in the slope, and slightly below this, there were two competent knobs of rock protruding a few feet out of the slope, as shown in Figure 3. These competent knobs have the potential to act as launch ramps, which would throw blocks of rock out onto the traveling surface. Colorado Rockfall Simulation Program (CRSP) computer runs indicated that most rocks that start at the top of the slope would hit these knobs and land on the traveling surface and destroy the Jersey barrier or the far side guardrail.

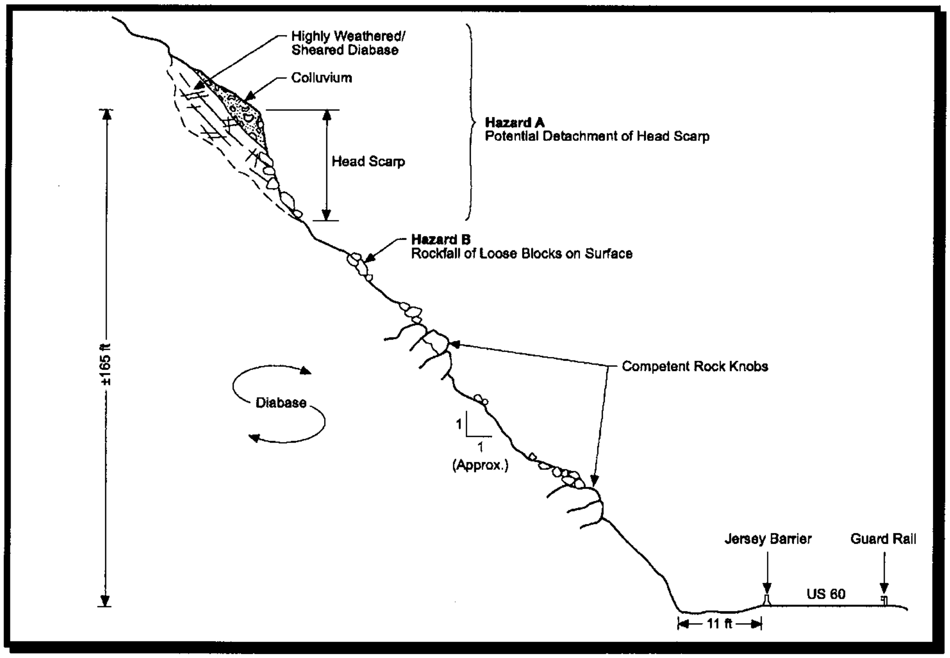

FIGURE 3. Cross Section of Slide Area

With the loose colluvium and sheared diabase in the head scarp and the loose blocks of rock on the slope surface, it was considered unsafe to proceed with construction at roadway level. Consequently, it was necessary to scale off much of the loose material before starting construction. This scaling was done in two phases, in February 2001 and in November 2001 by a team of high-scalers working from ropes attached at the top of the slope. The scaling proceeded using half-hour traffic stoppages during normal daylight working hours. The scaling significantly improved the immediate safety for construction at the toe of the slide area. However, this could only be considered as a temporary measure as further raveling can be expected due to weathering, freeze-thaw, wetting and drying.

Several options were discussed and evaluated so that a preferred solution could be selected on the basis of technical acceptability, minimum environmental and aesthetic impact, minimum interference with traffic, and acceptable costs. Preliminary designs were developed for the four most promising options, which are described below:

Option 1 - Road Realignment: The most obvious option was to realign the road by moving it further away from the base of the rock slope to provide a wider catch ditch with a rockfall catch fence. A cross-section of this option is shown in Figure 4. Shifting the alignment would require the construction of the long MSE wall on the outer, or down slope side, of the existing roadway. This option was rejected because the MSE wall did not fit in with the aesthetics of the Canyon, because of disruptions to traffic, and because it was difficult to satisfy stability criteria by imposing additional load at the top of the existing slope at the shoulder of the road. In addition, the rockfall catch fence would have to have a very high capacity to withstand the energy of the rockfalls.

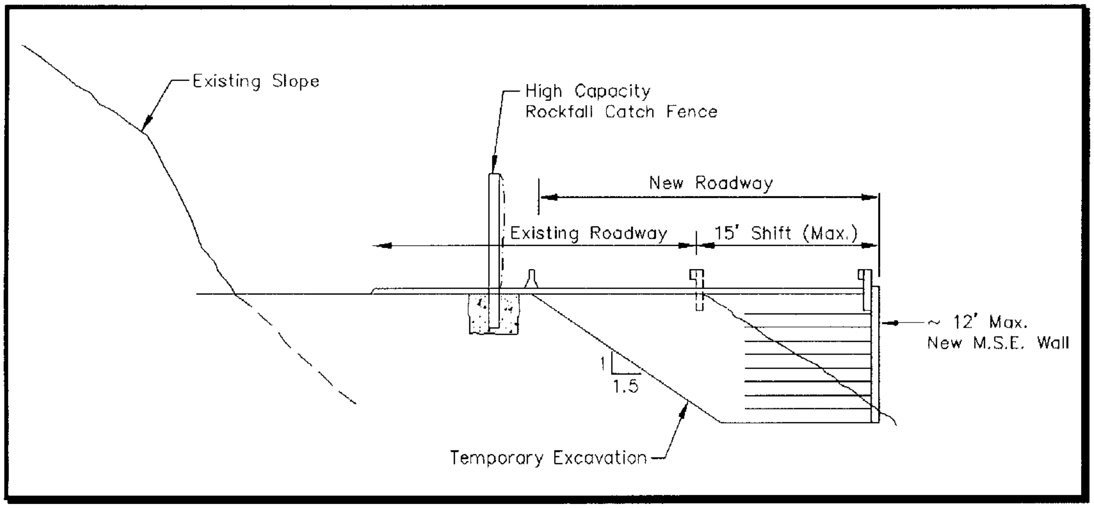

FIGURE 4. Mitigation Option 1 – Road Realignment

Option 2- Draped Cable Mesh and Rockfall Catch Fence: The CRSP analyses indicated that it would be possible to reduce the bounce height and energy of falling rocks so they could be stopped by a draped cable mesh near the bottom of the slope, and a rockfall catch fence beside the highway. There are few projects where a draped cable mesh has been installed and there is no accepted criterion for the design of these installations. However, it was considered that a satisfactory design could be developed by strategically locating draped cable mesh in the main chutes of the slope. This option is shown in Figure 5. It was discarded because of aesthetic objections to the draped cable mesh and the limited number of case histories of this type of installation.

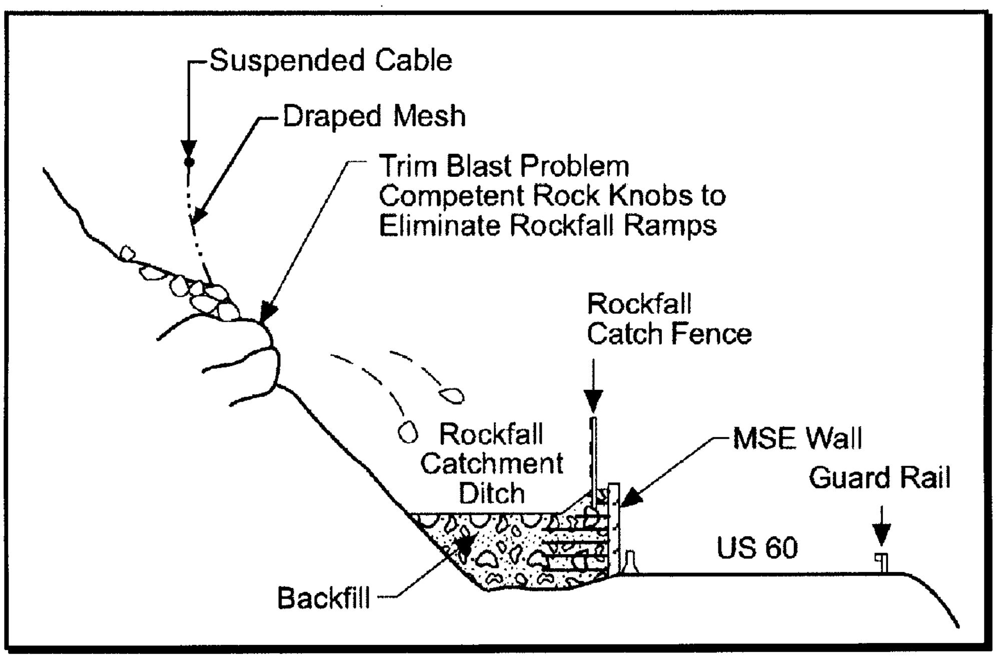

FIGURE 5. Mitigation Options 2 & 3 Draped Mesh, MSE Wall & Rockfall Fence

Option 3- MSE Wall and Rockfall Catch Fence: The CRSP analyses indicated that 99 percent of the rockfalls could be arrested by constructing an MSE wall, an elevated catch ditch and a rockfall catch fence at the base of the slope. Several options are available for the height of the wall, the size of the catch ditch, and capacity of the rockfall catch fence. A typical layout of this option is also shown in Figure 5. This option has several attractive elements: the MSE wall can be of limited height (and hence will have low visible impact), there is little disruption to traffic and the costs of this option are moderate.

Option 4 – Rock Shed: An unusual, but potentially attractive option, evaluated construction of a rockfall shed over the roadway, as shown in Figure 6. This option would consist of pre-cast beams spanning the highway with the bridge decking and granular fill above the beams. The beams would be supported on an MSE wall on the rock slope side and columns on the downhill side. No lighting or ventilation would be required, as the columns could be sufficiently far apart to avoid obstructing air flow and light. While this option was attractive technically, its cost was somewhat higher than other options. There were also some objections to the aesthetics of the rock shed in the Salt River Canyon.

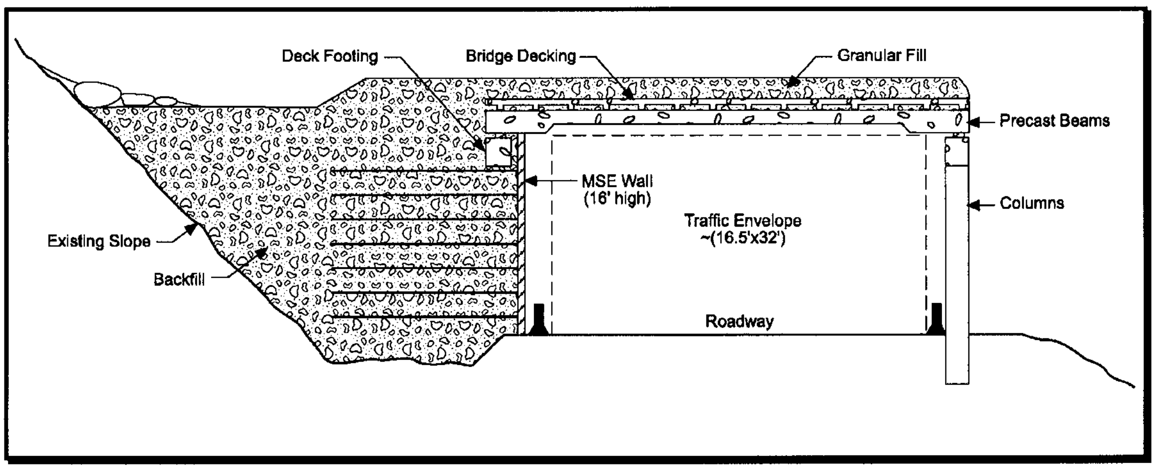

FIGURE 6. Mitigation Option 4 – Rock Shed

Combinations of several of these four options were also studied and, on the basis of this evaluation, a combination of Option 1 and Option 3 was selected as illustrated in Figure 7. Details of the selected design are given in the following section.

SELECTED DESIGN AND CONSTRUCTION

The results of CRSP analyses indicated that the combination of a 12 ft. wide catch ditch on top of a 15 ft. high MSE wall and a 240 ft. long 10 ft. high GeoBrugg rockfall catch fence with 350 ft. tons capacity would contain 99 % of the rockfalls from the slope. This included rock particles up to 6.5-ft. average dimension. The design had to incorporate provisions for maintenance equipment access and had to be aesthetically compatible with the Canyon surroundings. The footprint of the MSE wall required slight realignment of the highway so two short 6 ft. high cast-in-place (CIP) retaining walls were required on the downslope side of the highway. Equipment access was facilitated using earth ramps at both ends of the MSE wall and the retaining wire ropes on the rockfall catch fence were attached to drilled rock anchors using removable shackle connections. To minimize loading on the face of the MSE wall from impact loads transmitted to the bases of the columns supporting the catch fence, bottom-retaining wire ropes were also used. The MSE wall was faced with textured shotcrete to meet aesthetic criteria.

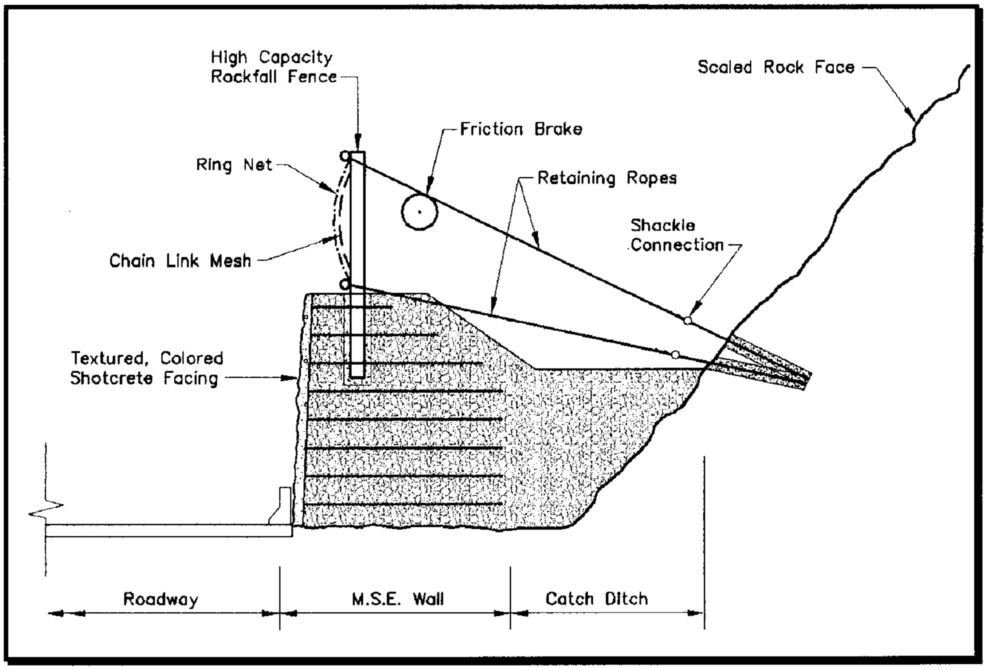

FIGURE 7. Selected As-Built Design

ADOT issued a competitive tender for the project and awarded the work to Stronghold Contracting, Inc. of Phoenix, Arizona. Following completion of final scaling construction of the downslope CIP walls was undertaken first to allow for rerouting of traffic away from the base of the slope. SSL of Scotts Valley, California supplied the selected MSE wall system. Brugg Cable Products of Lake Oswego, Oregon supplied the rockfall catch fence.

All work on the project was completed in late April 2002.

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Acknowledgements:

Thanks to; Dr. George Davis, University Of Arizona, as well as The Arizona Geological Society for the use of " Figure 10 b, Geologic Map of the Mule Hoof Bend Area, AZ Geol. Digest Vol. XII," as the base map for this report.

CORRIDOR H SECTION 16 FROM ELKINS TO KERENS RANDOLPH COUNTY, WEST VIRGINIA LANDSLIDES INVESTIGATION & CORRECTIVE MEASURES

James C. Fisher
West Virginia DOT

Abstract

In the 1960's the Appalachian Regional Commission proposed a system of 26 highway corridors that would foster and promote economic and social development throughout the Appalachian Region.

In West Virginia the last Corridor to be constructed is Corridor H. A completed 40-mile section extends from Weston to Elkins. The remaining 100-mile section would extend from Elkins to the Virginia State Line. This section of the road crosses some of the most rugged and environmentally sensitive areas of the state.

The focus of this paper is a 5.5-mile section of Corridor H that extends from Elkins to Kerens and is scheduled to be completed in the spring of 2002. As the construction and excavation progressed on this project, exposure of the rock units came in full view. The exposed bedrock is of Devonian age and consisted of the Harrell Shale. The formation consists of thinly bedded layers of gray black silty shales that are interbedded with lenses of sandstone. The strike of the units is to the northeast and is dipping from 22 to 29 degrees to the southeast.

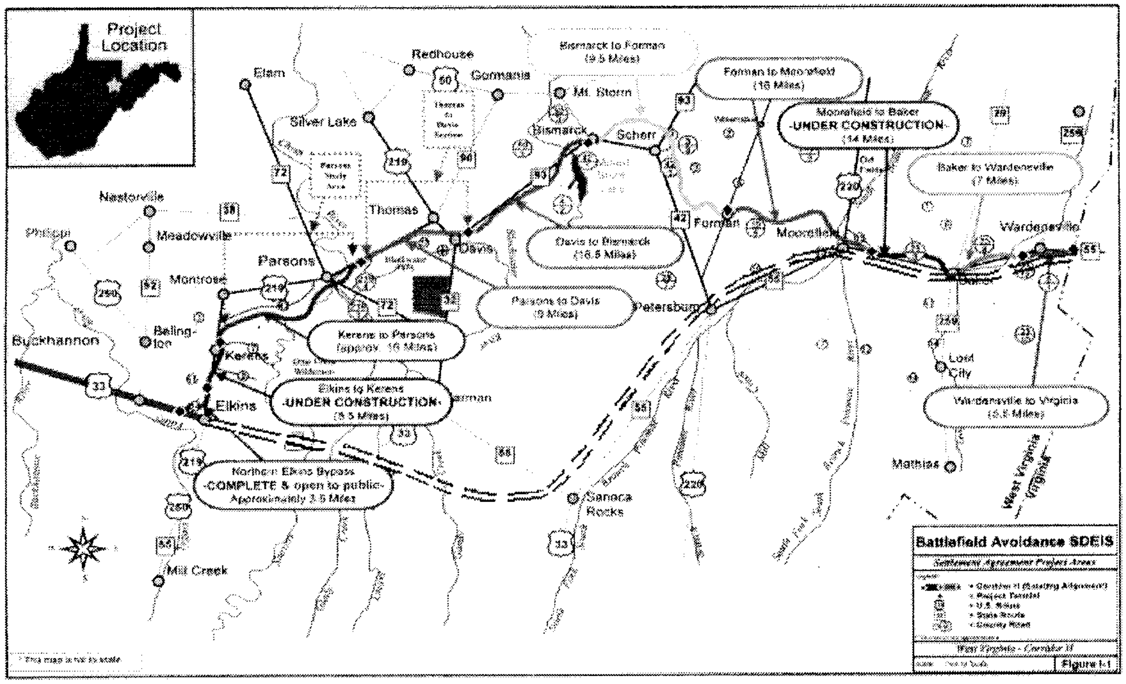
Five major landslides occurred during the excavation of the road cuts. All of the landslides occurred on the western side of the road. The rock units on these cuts all dip toward the roadway. As each landslide developed corrective designs were explored and

ultimately initiated. The landslides greatly increased excavation quantities on the project, which also increased accompanying cost overruns. The paper will review the preliminary field investigation, landslide occurrences and corrective measures initiated.

Project History

The U.S. Congress passed the Appalachian Regional Development Act of 1965. The act created the Appalachian Regional Commission as well as authorizing the creation of the Appalachian Development Highway System. The Highway system was intended in conjunction with the Interstate system to grant access to the Appalachian States and promote economic development. Included in this act was a vision to construct a series of four lane highways through the area that were designated as Corridors, with a single letter designation.

Corridor H in West Virginia is the only corridor in West Virginia that has not been completed. The original scope of the project was to connect to Interstate 79 in Weston and run east to west and intersect with I-81 in Virginia. Construction progressed on the Corridor in the 1970's and 1980's between Weston and Buckhannon. A 6-mile section was constructed during the 1970's just to the east of Elkins. Due to envirnomental issues and funding, no further construction was completed on the section between Elkins and Buckhannon during the 1980's. No action was taken on the section between Elkins and the Virginia State Line. The portion of the Corridor between Elkins and Buckhannon was completed during the 1990's. In 1990 the project between Elkins and the Virginia State Line was resurrected and a fresh approach to all routes was initiated All routes were reevaluated, and a draft environmental impact statement was prepared. In 1996 after several years of doing alignment studies and environmental impacts statements a revised route was settled on and a Record of Decision was issued on the preferred alignment. Following the record of Decision, Corridor H Alternatives sued to stop construction. After years of legal battles, an arbitration agreement was reached in February 2000, and the project was given the green light to begin construction.



Corridor H Route from Elkins to the Virginia State Line

Ten Sections agreed upon by WVDOT and Corridor H Alternatives

Dashed red line was the original southern route established during the 70"s

Elkins to Kerins Section

This section consists of a 5.5-mile section of four-lane highway consisting of nine bridges and five box culverts. New cuts and fills are required. Excavation quantities will exceed amounts needed to construct the fills, requiring disposal of the excess material in waste sites. The new road roughly parallels US route 219.

Geology

Most of the rocks in West Virginia are sedimentary deposits during the Paleozoic Era. Movement of the earth's tectonic plates, which caused episodes of mountain building which with subsequent erosion and production of sediment, has shaped the geologic landscape of the state.

The first of these mountain building episodes to effect West Virginia was the Taconic Orogeny which formed mountains to the east of the state. These mountains were a source of sediment during the Ordovician, Silurian and early Devonian periods. Highlands to the northeast formed in the Acadian Orogeny were the source of clastic sediments in the Middle and Late Devonian. These sediments formed the bedrock exposed within the project limits.

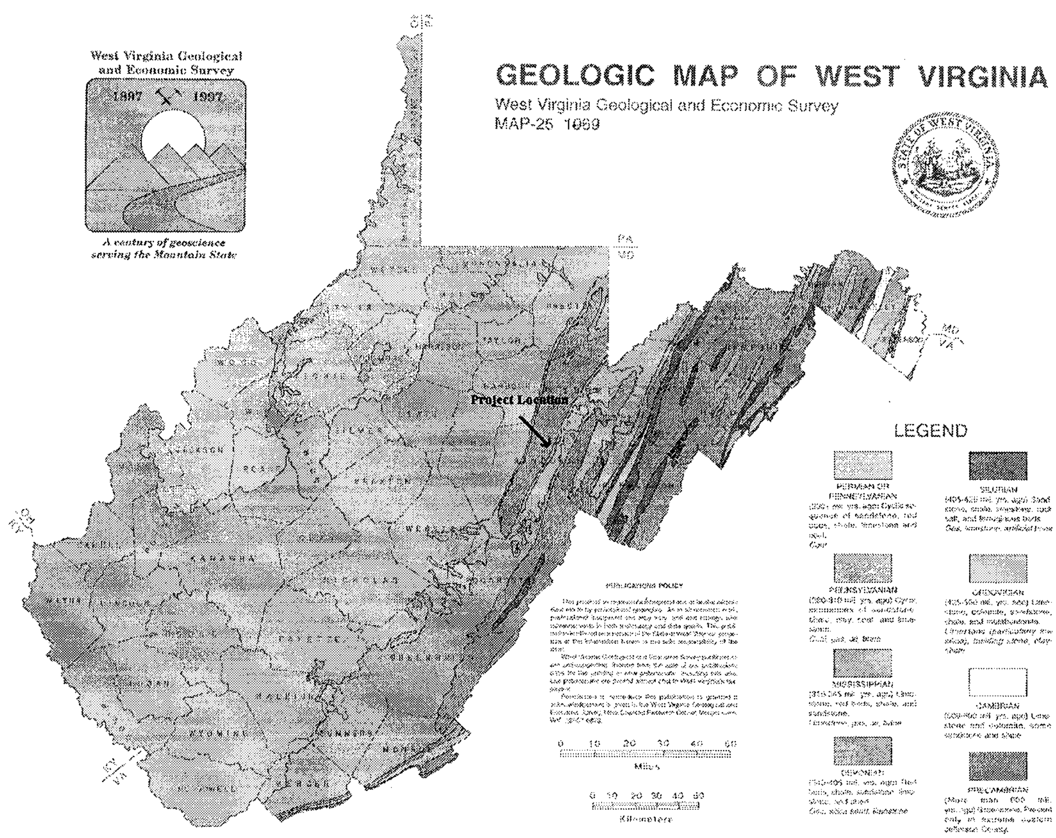
The sea regressed toward the west at the end of the Devonian, and continental red beds were deposited over most of the state. Once again the sea covered the state in the Middle Mississippian. During this time the Greenbrier group composed mostly of limestone was deposited. This formation caps the mountains adjacent to the project.

The sea retreated again near the end of the Mississippian, and during the Pennsylvanian West Virginia was low lying and swampy. During this time sediments of nonmarine sandstones shales and coal were deposited.

During the Permian Period. Uplift of mountains began as a result of the Alleghenian Orogeny, which was the prominent event for the formation of the Appalachian Mountains. Folding and thrust faulting occurred in the eastern part of the state, which is the corresponding location to the subject project. After the uplift of the Permian the area was being eroded through the Triassic.

In the late Jurassic another uplift occurred followed by erosion through the Cretaceous. Yet another uplift occurred in the late Cretaceous or early Tertiary followed by erosion. In late tertiary another uplift occurred which formed the fourth range of Mountains, which is presently being eroded.

The forces that caused thrust and uplift were able to break the rocks in the project area. This was due partly on account of proximity, and partly because the protective mantle of the Pennsylvanian did not extend greatly southeastward, and the thrusts were able to act on the area with great force. Further toward the west, increasing distance and the increasing protective Pennsylvanian mantle were able to reduce the effect of the great uplifts.



Geotechnical Investigation and Design

H.C. Nutting Company performed the subsurface/geotechnical investigation under contract with De Leuw Cather and Company (the Consulting Engineer). H.C. Nutting's scope of services included; reconnaissance of the proposed alignment, evaluation of the existing cut and fill slopes; logging of outcrops; a study of the published geologic data; test boring inspection during drilling operations; preparation of the final boring logs for laboratory evaluation and development of geotechnical recommendations for embankment construction and cut slope design. Based on H.C. Nuttings investigation, recommendations were made for constructing the embankments and cut slopes and approved by the Department Review of exposed outcrops within the project limits revealed both horizontal and folded units of siltstone shale and sandstone.

The bedrock in the rockslide area was of Devonian age and a member of the Harrell Shale formation. The Strike and dip of the exposed folded bedrock was variable with strikes occurring from the southwest to the northeast and dips of the units being very variable between 20-80 degrees.

The roadway test boring program consisted of 355 test borings to investigate the proposed alignment. Borings within the area of the slope failure consists of thinly bedded layers of gray black silty shales that are interbedded with lenses of sandstone. Core recovery was very good with values typically recovered above 80 %. However, RQD values were very poor with a range of values between 5-50 %. Slikenslides were noted on several of the borings.

As stated in the Geotechnical report "Slope Design of the main bedrock characteristics considered in the design of cut slopes was the bedding plane orientation and bedding thickness of the rock to be excavated. Steeper bedding planes characterized the first 60% of the alignment and thus the cut slope design was more conservative for this section of the project." Backslope cuts were designed to be constructed on a slope ratio of 1:1. This was judged to be adequate to accommodate the dipping bedrock. .

Construction

During the initial stages of construction, residual soils were being removed from the top of the cuts and the bedrock was excavated to template, when project personnel noticed tension cracks appearing in the excavated cut slopes. The central engineering office was contacted to access the situation and recommend any necessary corrective measures.

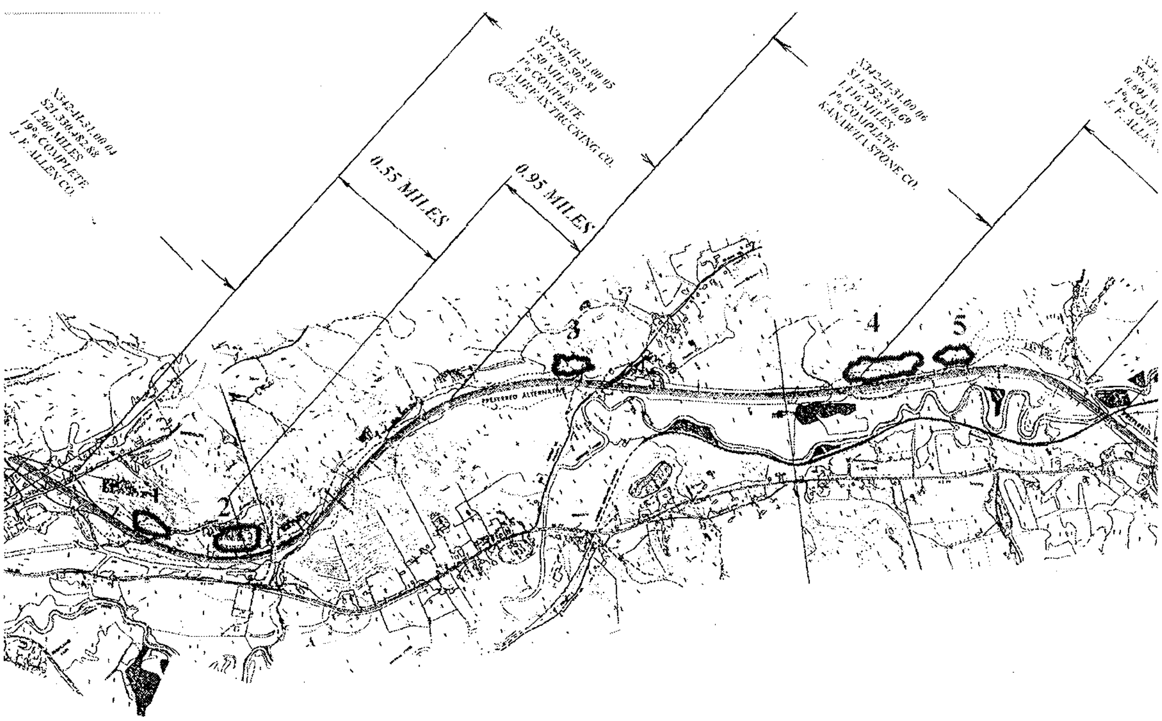
Upon inspection it was apparent the original design could not be constructed as designed. Landslide failures were becoming sequential, as each slope excavation began, so did the landslide movement. Five different areas of excavation eventually experienced landslide movement.

Bedrock was exposed on all of the failure surfaces. Slickenslides were prominent on each exposed rock face. Dip measurements were made on the exposed rock faces and found to be between 22-29 degrees. The bedrock was dipping toward the proposed location of the new road. A corrective designed was prepared to reshape the slopes from the original design ratio 1H: 1V to a ratio of 2H: 1V.

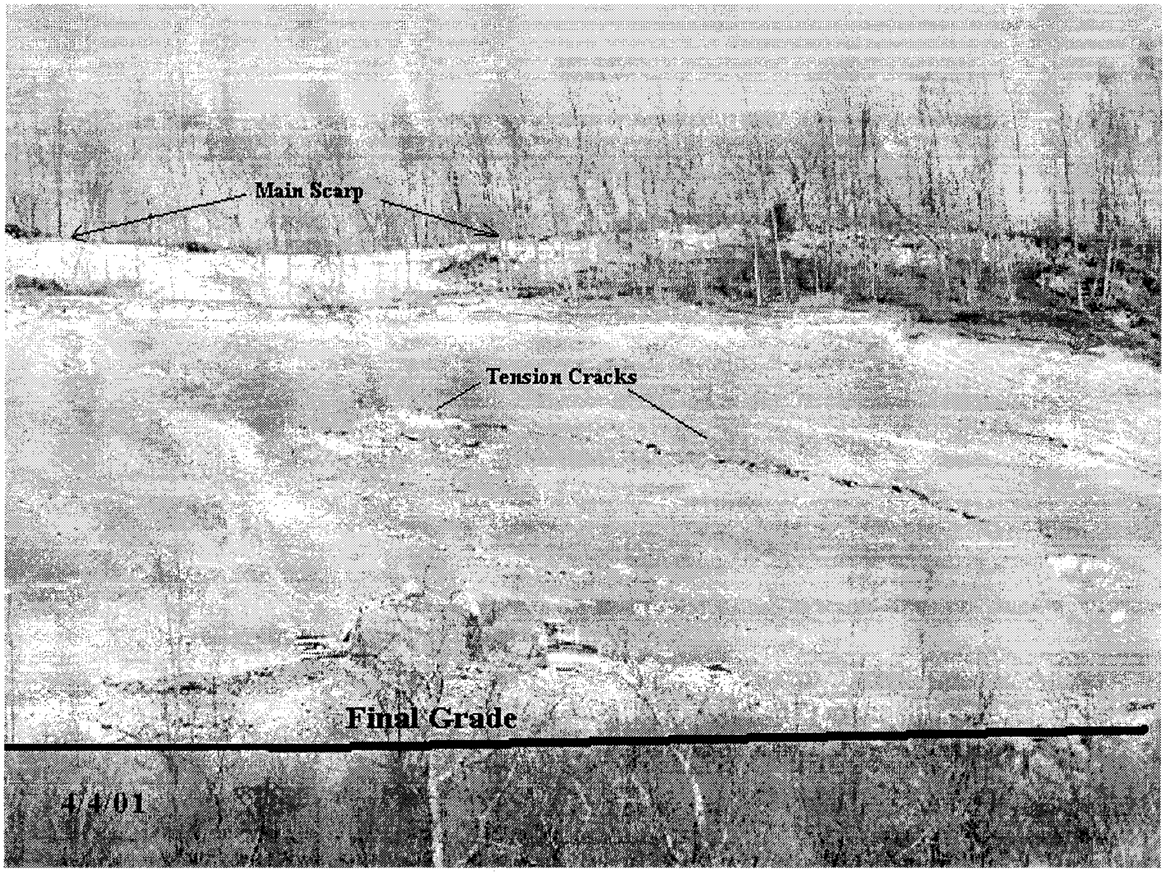
Excavation proceeded on the redesigned slope. Success was not achieved with the new design, as excavation proceeded the slopes continued to fail. Another approach to stabilize the slopes was necessary or an unknown amount of excavation and flattening of the slope was going to be required to achieve a stabilized state. Ultimately it was decided to construct a buttress at the toe of the slopes.

The buttress design was intended to stop further movement of the bedrock, and if further movement occurred a bench constructed 50 ft. above the final road grade would catch the sliding material. The slope ratio from the ditchline to the top of the bench was constructed on a slope ratio of 1.5H: 1V. Excavated material from the cuts was used to construct all of the buttresses. The material was reshaped and compacted to meet the design requirements.

Four of the landslide areas were stabilized with the buttress design. (No. 2,3,4,5) One slope (No. 1) was stabilized by excavating the slope using the original corrective design of cutting the slope on a ratio of 2H: 1V



Plan View Location of The Landslides



Site 4 Shortly After Major Movement Event

Summary

The original design anticipated excavating the folded and dipping bedrock. An attempt to address this situation was made by flattening the back slopes. However inadequate this design proved no one anticipated the magnitude of the failures, or the accompanying excavation overruns. Excavation quantities due to the landslide movement were 1,318,215 cubic yards (1,007,848 cubic meters). Only after the excavation began exposing the bedrock, did the unstable nature of the slopes become apparent. The limits of core drilling information were exposed; that being the orientation of the road in relation to dip direction of the bedrock; the variability of the dip angle; the orientation of the joint patterns that would be encountered. All of which would have been of great service during the original design.

One factor that is rarely mentioned in dealing with landslides is the psychological reaction to the failure. Zaruba and Mencl (1969) describe their view of this reaction as follows: "Landslides are of serious concern to the persons involved; the intensity of the natural phenomenon creates a depressive atmosphere and anxiety arises to whether the collapsing slope can be stabilized at all. The question of guilt as to whether or not the failure could have been prevented also arises" There is little doubt when viewing these landslide all of these feeling were expressed. If we are to learn from our mistakes then let the record reflect that RQD and the relationship of the bedding and roadway grade need further defining. The landslides greatly increased excavation quantities. "Feeling of anger, helplessness and despair creep into the individual or colllectively creep into the human phychic. The event must be viewed philosophically as a temporary set back that should become a learning experience that is recorded and used to prevent a similar situation in the future."

Acknowledgements

The author would like to thank Mr. Mike O'Neil and Mr. Scott Ratliff for their input regarding this project. Without their assistance this paper could not have been completed.

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EVALUATION OF A PEAT FILTRATION SYSTEM FOR TREATING IUGHWAY RUNOFF IN A KARST SETTING

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The deleterious character of highway runoff, especially following long periods without precipitation, has been well documented in literature. It transports hydrocarbons, heavy metals, and other contaminants from highways, contributing to the pollution of surface water and groundwater. Groundwater is particularly vulnerable in karst areas where highway runoff is transferred quickly into subsurface conduit networks through open sinkholes and/or sinking streams. The adverse impact of highway runoff on karst aquifers is being addressed by P.E. LaMoreaux & Associates, Inc. in a research project supported by the Federal Highway Administration (FHWA) and Departments of Transportation (DOTS) in fifteen states. A major goal of this study is the development of a practical peat filtration system for removal of highway runoff contaminants prior to being transported into karst aquifers.

The study site is located at the I-40/I-640 interchange in eastern Knoxville, Tennessee. Approximately 86,000 vehicles pass through the interchange each day. After runoff drained into a "new" sinkhole (repaired in 1988), the water passed through an "old" sinkhole, two small karst windows and was discharged at a spring. The characteristics of the highway runoff and its impact on the spring were investigated during a sampling event in 1996. The initial design of the peat filtration system involved direct installation inside the "new" sinkhole, but this design was later changed to a peat barrier system on the surface after the sinkhole was filled. In this new design, runoff from approximately 60 acres flows through two rip-rap barriers, through gaps between concrete barriers, into a shallow detention basin, and then into a sorption/filtration wall consisting of gravel and peat. A by-pass channel allows only the first-flush of runoff to be treated. The outflow channel of the filter system directs water into the "old" sinkhole, which was permitted as a Class V Injection Well. Recent field tests indicate that the system can significantly decrease the concentrations of analyzed constituents including PAHs (polyaromatic hydrocarbons), Copper, and Zinc. Long-term monitoring is required to determine the true effectiveness of the designed filtration system.

EXCESSIVE DIFFERENTIAL SETTLEMENT IN THE NORTHERN NEVADA RAILROAD UNDERCROSSING STRUCTURE AT U.S. HIGHWAY 50 NEAR ELY, NEVADA

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The redesign of US Highway 50 over the Northern Nevada Railroad involved replacement of the 42-year-old bridge structure over the Northern Nevada Railroad. This site is located west of Ely, Nevada in a steep sided canyon, with an intermittent creek flowing west to east. The construction of a new undercrossing (railroad tunnel) began in the Spring of 2000 and was completed in the Fall of 2000. The structure was constructed as a three-sided concrete box, supported on spread footings. It was built along the railroad alignment, beneath the existing four span highway bridge. The new highway alignment was to cross over the tunnel on embankment fill.

Construction on the project resumed in the Spring of 2001, and at that time, several problems were evident. The interior of the tunnel exhibited signs of moisture along cracks in the roof towards the west end. Subsequently, the embankment fill and moisture barrier was removed from the top of the structure on that end. Following inspection, severe longitudinal cracking was observed along the south side. Upon further inspection, the structure was found to have settled approximately 14 inches at the northernmost corner.

A forensic study was ordered, and mitigation procedures began to be explored. Several factors have become apparent regarding these problems; among them are: lack of adequate preliminary geotechnical investigation, failure to utilize existing structure plans and site information, and lack of communication between team members. Mitigation efforts, consisting of ground modification utilizing pressure grouting, are ongoing at this time.

Construction-Phase Design Changes in Response to a Compound Rockslide, Preacher Canyon Section of State Route 260, Near Payson, Arizona

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Abstract
The Preacher Canyon Section of State Route 260 is an ADOT-owned highway located approximately ten miles east of Payson, Arizona in a bedrock terrain. The two-lane highway was reconstructed to a four-lane divided highway on undisturbed forest land, requiring 1.8 million cubic yards of rock excavation. A compound rockslide in granite occurred during construction of a 170-foot deep roadway cut for the project. The compound slide consisted of an initial steep wedge-type failure which then prompted planar movement toward the wedge failure along an adjacent fault, ultimately resulting in a unique low-angle wedge failure in the weakened rock mass. The slide included head crack development, slow creep-type movement extending to within 25 feet of the toe, and internal cracking. Initially, remedial design of the original ¾H:1V slope necessitated flattening the slope to 1-¼H:1V, de-energizing a critical 345-kV transmission line serving the Phoenix area, removing and relocating an existing steel transmission line tower, temporarily supporting the line on wood poles, and stabilizing the ground at the base of the poles with rock anchors, as the failure projected to above the temporary poles. The design also required construction of a rock-fall containment system to prevent rocks rolling from the flattened slope onto the roadway. To retain the visual characteristics of the surrounding area, a deepened cut ditch was installed in lieu of a rock-fall barrier. Continued creep movement of the slide necessitated a final adjustment to the design to remove more head mass and additional slope recontouring to mitigate further movement and reduce required future maintenance.

Introduction
The Preacher Canyon Section of State Route (SR) 260 is a 3.1-mile section of highway (MP 260 to 263.1) within mountainous bedrock terrain of the Tonto National Forest. SR 260 serves as a major access route to recreational areas within the Tonto and Sitgreaves National Forests, as a link between the Phoenix metro area and numerous small towns and communities northeast of Payson, and as a connector route to I-40. A Design Concept Report (DCR) and Environmental Impact Statement for the 50-mile long SR 260 corridor extending from Payson to Heber, Arizona were completed in 1997 and 1999, respectively. The DCR identified the Preacher Canyon Section as one of six projects along the existing route requiring realignment to an alternative corridor that bypassed the existing two-lane roadway to upgrade roadway geometry, improve operational characteristics and enhance highway user safety. The final design of the section was completed in April 2000 and construction of the new four-lane, divided highway was completed during 2000-2002. Key project features included new EB and WB bridges (600 to 850 feet in length) across Preacher Canyon, EB and WB single-span wildlife crossing bridges to

accommodate elk herds, and significant roadway cut and fill slopes, with maximum excavation depths reaching 170 feet and embankment heights exceeding 50 feet.

During excavation of a 170-foot deep roadway cut slope just east of the Preacher Canyon bridges, a compound rockslide in granite occurred in a portion of the ¾H:1V cut on the south side of the EB roadway (Figure 1). The crest of this cut was located below a pair of steel lattice towers supporting live Arizona Public Service (APS) 345-kV transmission lines that cross the roadway alignment. Movement of the slide occurred initially as a steep wedge-type failure followed by a shallower wedge along a fault and resulted in head cracks through the foundation of one of the towers.

Investigation of the rockslide was completed and included identification and analysis of the failure mode and remedial design of the cut slope. The objectives of this paper are to present descriptions of the regional and site geology; roadway design development; details and analyses of the slide failure mode; and descriptions of the slope failure, remedial designs and construction, including stabilization and relocation of the lattice towers, analysis and design of a rock-fall containment system.

Regional Geology

The Preacher Canyon Section lies in the Transition Zone Physiographic Province (TZP), which represents a transition between the Basin and Range Province to the southwest and the Colorado Plateau Province (CPP) to the northeast. The TZP is characterized by short block-faulted mountain ranges separated by small intervening basins. The geology of this area is dominated by bedrock conditions but varies significantly between the two provinces. The TZP is dominated by igneous, metamorphic and deformed sedimentary and volcanic rocks mostly of Precambrian age, and the CPP is dominated by a younger sequence of relatively flat-lying sedimentary rocks primarily of Paleozoic age. The primary geologic units exposed in vicinity of the site include Precambrian granite with lesser rhyolite, greenstone, and aplite and latite dikes. Precambrian Matazal Quartzite, localized Paleozoic sedimentary rocks and local Tertiary to Quaternary sedimentary deposits occur in the general area towards the northeast.

Site Geology

The site area is underlain by bedrock composed essentially of Precambrian granite, predominantly with a medium-grained and equigranular texture. Locally, the granite has a coarse-grained or slightly porphyritic texture. The rock is typically pinkish brown, although a number of color variations exist, including light gray and light to medium brown. Light brown aplite dikes locally cut the granite in the general area, have a fine-grained texture and are very irregular in shape. Exposures of aplite dikes are abundant in cut slopes on the north side of SR 260, but generally are absent from south side cuts in the failure area. Very localized dark green latite dikes also cut the granite in the area. Latite exposures have a fine-grained texture and typically occur as thin elongated zones commonly along faults; the latite dikes are locally present in the south cut, but typically absent in the north cut.

Exposures of granite typically are decomposed to highly weathered in the upper section of the cuts, varying with depth from highly to moderately weathered with some slightly weathered zones. The granite is moderately soft in more weathered zones to hard in less weathered zones. The aplite dikes are less weathered and harder than the granite. The latite exposures typically are moderately soft and highly weathered.

The rock mass of the granite within the south cut is moderately to highly fractured, with very close to moderately close joint spacings (typically from about 0.2 to 3.0 ft). Three primary joint sets occur within the project area: Set 1 strikes at about N45E and dips northwest; Set 2 strikes at about N60W and dips to the northeast; and Set 3 strikes about N50E and dips to the southeast. Also, a subordinate joint set with a north-south strike and a near-vertical dip is present. Several minor shear zones or faults are also present within the project area. A larger shear zone bisects a road cut on the north side of the highway just west of Preacher Canyon, strikes approximately east-west and dips steeply to the north. A few faults are exposed in both the north and south cuts of the failure area. The exposed faults, except for the large shear zone, vary considerably in orientation across the site and represent edges or boundaries of numerous small rotational fault blocks internal to the overall rockmass. As a result of the internal faulting and rotation, the fractures comprising the rockmass fabric are moderately warped, curved and undulatory over relatively short distances. Faults similarly are curved and warped and usually short features. Thus, considerable variations of the primary fracture sets occur across the site and specific fracture/fault orientations control slope performance. Descriptions of specific features associated with the slope failure are presented in the following sections.

The fracture surfaces generally are stained to partially filled with reddish-brown clayey-silt and commonly are slightly rough and wavy with step-like asperities up to 1 inch high. Occasional smooth planar fractures also occur within the slope. Most fracture surfaces generally are discontinuous and extend only 5 to 15 ft, and fractures commonly terminate at the intersection with another set. However, occasional fractures extend for more than 40 ft in height, but are commonly slightly curved.

Roadway Design Development

The two-lane highway was reconstructed on undisturbed forestland to a four-lane divided roadway with independent horizontal and vertical alignments. Each roadway consists of two 12-ft traveled lanes with 10-ft outside and 4-ft inside shoulders. The new roadway traversed the mountainous bedrock terrain through a natural saddle to minimize both disturbance to forest land and volume of excavation. The saddle was located approximately 280 ft north of the existing APS 345 kV transmission towers. With these constraints in mind, the roadway was designed with the maximum profile grade (6%) and minimum median width (43-ft) permitted by ADOT design guidelines. The $20.4 million construction project included approximately 1,830,000 cy of excavation and 1,470,000 cy of embankment. Approximately 340,000 cy of the 1,830,000 cy was contained within the 800-ft long compound rockslide area.

Along the 800-ft segment of EB roadway, the horizontal alignment is curvilinear with a 2-degree horizontal curve (radius of about 2,864 ft) and is super-elevated at a rate of 4.8%, sloped upward toward the outside and positioned to fit within the established right-of-way. A 30-ft wide cut ditch including a 6H:1V foreslope was designed on the outside shoulder to intercept potential rock-fall from entering the roadway.

Description of Failure, Remedial Design & Construction

The south cut slope from EB Stations 518 to 526 initially was excavated at ¾H:1V. During excavation, small wedge failures occurred at the east end of the cut near the original slope crest. Inspection of the failure areas revealed variably shaped wedge failures, a variable degree of weathering in the slope face and considerable undulation of the fractures comprising the rockmass. Additional discontinuity data was collected across the remaining slope and the face was monitored to evaluate potential for additional failures, particularly below the existing western transmission tower at EB Station 521+50. Measurements of fractures/faults were collected as sets in real space across the slope face, since with the warped orientations, only sets that truly intersect and occur together in space could form wedges. The data indicated a relatively steep wedge directly below the tower. Excavation of the next lift exposed the toe of a steep wedge terminated on a fault, and movement at the toe initiated a slope failure. Granite exposed in the slope at the wedge toe was very closely fractured and broken, likely the result of previous shearing along the fault forming the wedge bottom.

Figure 2 depicts the stereographic solution for the steep wedge in the south cut. Five discontinuities are plotted on the figure; however, Sets 2, 3 and 5 represent undulations of the same set. The plunge in the line of the two intersecting planes represents the wedges varying from 40 to 44 degrees and each intersection falls within the critical zone (area between the ¾H:1V slope plane and a 30-degree friction angle). Movement at the toe caused sliding of the mass and tension cracks to form above the crest of the ¾H:1V slope and between the legs of the western transmission tower. Slow creep-type movement ensued, with separation at the head scarp crack exceeding one foot, and less vertical displacement. Movement within the steep wedge subsequently triggered movement along the adjacent fault (termed the west fault), producing a compound wedge failure at a shallower orientation than the first wedge. The west fault strikes nearly perpendicular to the slope and variably dips to the east. The shallower wedge failure is bounded on the west by the fault and on the east by a primary fracture set with numerous curved splays discontinuous in the upslope direction. Figure 3 presents a stereonet plot of the fault and primary fracture set, and includes two orientations for the west fault, as the fault surface warps to a flatter orientation toward the toe of the failure. The fault dips at about 46 degrees (from horizontal), flattening to 39 degrees at the toe. As shown on the plot, the line of intersection of the two planes (west fault and primary fracture set) defining the wedge plunges to the northeast at 24 to 32 degrees. Previous measurements indicated an even flatter plunge of about 18 to 24 degrees. Based on an assumed friction angle (shear strength) of 30 degrees, movement of the wedge should only occur at the steepest plunge angle and not at the toe. However, compound movement of the slide, first along the steep wedge and then down the fault, likely produced a reduced (~residual strength) friction angle and continued movement at the flatter angle. The west fault is well defined, but the east boundary fracture is poorly defined. Further, most of the movement was associated with the west fault, and sliding appeared to be on a relatively flat surface.

Slope Flattening

Figure 4 is a photograph of the ¾H:1V slope face with faults, extent of the failure zone and the relative direction of sliding of the mass indicated. Response to occurrence of the slope failure was to redesign the slope to 1¼H:1V to remove the mass of the steep failure and some of the mass above the fault. The entire mass above the fault was not removed, due to the fault orientation (nearly perpendicular to slope face). Also, lines of intersection of the fault with other primary fractures plunged at angles very close to the 1¼H:1V slope, or less than the presumed 30-degree friction angle for fracture/fault surfaces. A rough outline of the compound failure zone and the new cut slope is shown on Figure 5. The entirety of the new slope was not constructed at 1¼H:1V, but rather varied from 1H:1V to 1¼H:1V. The upper two-thirds of the slope were close to 1¼H:1V, with some areas approaching 1H:1V, and the lower one-third was predominantly 1H:1V. Additional movement during reconstruction of the slide created cracks at the crest and small slumps in the upper slope.

Tower Relocation

The existing western tower, located just above the crest of the original slope, was temporarily relocated (by APS) to just behind the crest of the new 1¼H:1V slope. The temporary tower was constructed of wood poles until a new steel tower could be sited, designed and constructed by APS after the peak summer power demand period had passed. Due to the limited length of available wood poles, the temporary poles were located only 45 to 50 ft upslope of the slope crest and 50 ft above the head scarp of the failure.

Rock Anchor Installation

As the entirety of the failure zone was not removed from the 1¼H:1V slope, rock anchors were installed between the slope crest, the temporary western tower and the adjacent eastern permanent steel tower to ensure stability of the ground supporting the towers. Excavation below the temporary tower was halted until the rock anchors were installed and tensioned. Three rows of 8 rock anchors (epoxy-coated Grade 150 threadbar anchored in cement grout, tensioned to 75 kips and locked-off) extending to depths of 50 to 55 ft and inclined toward the towers at 35 to 50 degrees from the horizontal were installed. Five anchors at each tower site initially were grouted only in the bond length and proof-tested to 133 percent of the design (lock-off) load. Anchor depths were staggered to avoid formation of a weakened plane at the anchor tips. Subsequent to tensioning, the anchor head assemblies were painted with epoxy paint and buried.

Additional Excavation

After installation of rock anchors, additional excavation proceeded to remove more of the disturbed mass and further relieve the slide head. The flattened upper half of the slope yielded a slightly bowl-shaped surface with a nominal 35-degree slope face (less steep than 1¼H:1V). After excavation of the bowl, visible movement in the slope was not evident for a couple of months and the slope appeared stable. Following the approximate two-month period, renewed movement occurred in the slope. Cracks up to one foot wide opened at the crest and movement occurred along the west fault extending down into the 1H:1V lower slope. The toe of the wedge failure propagated approximately 25 ft above the toe of the slope.

The creep rate of the slide was very slow; approximate rates of movement within the flattened slope (measured at the toe) are as follows:

Date	Cumulative Offset Across East Fracture (inches)	Rate of Movement Since Previous Measurement (inches/day)	Cumulative Offset Across West Fault (inches)	Rate of Movement Since Previous Measurement (inches/day)
8-16-01	Set Reference Marks	N/A	N/A	N/A
8-19-01	0	0	0	0
8-31-01	0	0	0.25	0.02
9-19-01	0.38	0.02	1.44	0.06
10-7-01	0.44	0.003	1.75	0.003
10-16-01	0.50	0.007	1.75	0

Offset measurements across the west fault at the slope crest were made for the period of 9-19-01 to 10-16-01; total offset was 0.88 inches (rate of 0.03 inches/day) for the period. Movement at the slide head exceeded that at the toe (as indicated above) for the same period. Cracks up to 8 inches wide and 40 inches deep exposed in the steeper, lower section of the slope (just below the base of the bowl) also indicated movement of toe of the slide mass independent from the head. It appeared that initially the toe moved away from the head, creating the mid-slope cracks and separation, followed by movement of the head mass, closing the cracks. This creep-type movement is a product of the mass sliding on a relatively flat failure surface.

Final Stabilization

Upon installation of the new west tower and removal of the temporary tower in October 2001, additional stabilization of the slope proceeded with eastbound traffic confined to the inside lane of the eastbound roadway. Based on the geometry of the slope, the projected intersection of the controlling east fracture and the west fault, and the measured rate of movement, further stabilization of the slope was required to minimize future maintenance needs. This was accomplished through excavation and removal of primarily the upper portion of the slide, with slight flattening in the lower slope, to unload the head of the slide, and reduce the risk of future movement. The sides of the new excavation surface were designed at approximately 1H:1V in a V-shaped configuration to grade to drain primarily toward the roadway and below the west fault. The western side of the excavation was designed to parallel the west fault, but advanced below the fault to effectively remove the slide mass above the fault and the weakened zone along the fault. The eastern side approximately corresponds to the eastern edge of the bowl area, with the crest no closer than 75 ft to the east tower. Excavation slopes were warped and rounded to blend in with the current slope face at an angle flatter than 1H:1V. During final stabilization and construction, removal of a wider zone in the upper section of the slope was performed to facilitate equipment access to the notch; this resulted in more of an L-shaped surface. The rock anchors were de-tensioned, cut and removed as excavation progressed; however, the rock anchors installed in front of the permanent steel (east) tower remain in-place. Figure 6 depicts the remedial excavation to further remove the head mass of the fault. Figures 7 and 8 present cross sections of the repair, and Figure 9 is a view of the final slope repair.

Rock-fall Containment

Initially, rock-fall containment characteristics of alternative ditch widths and foreslopes were analyzed to establish the ditch configuration which would best minimize rocks from rolling onto the roadway from the 1¼H:1V slope. Simulated rock-fall trajectories for 12 geometric cases were completed using the Colorado Rock-fall Similation Program (CRSP) Version 3.0 . Conservative conditions determined from previous runs were used in the simulations and included 1.5-ft diameter rocks on a relatively smooth slope. The model was completed for 30-, 40- and 50-ft wide ditches, 50- to 200-ft high slopes, and 3H:1V and 4H:1V foreslopes.

As a result of the modeling, fifteen of 100 rocks from the 200-ft high slope with the 50-ft wide 3H:1V ditch reached the roadway with a 4.4-ft maximum bounce height. For lesser height slopes, progressively fewer rocks reached the road. However, as a significant portion of the constructed slope height is over 150 ft, the analyzed geometry still possessed a high risk to road users without rock fall hazard mitigation. Additional analysis using the CRSP model produced a tentative recommendation of a 1¼H:1V cut slope and provision of a 4-ft high rock-fall barrier.

Several alternatives were considered from a highway geometric point of view, and due to the position of the EB and WB roadway alignments, the only viable solution was to provide rock-fall mitigation in the area between the edge of the roadway and the cut slope. The project team met and discussed various alternatives for rock-fall hazard mitigation. Initially, a conclusion as to the best-fit solution, considering engineering design, cost of treatment and visual aesthetic impacts, was not reached. However, it was generally agreed that a temporary treatment was required to protect road users as the roadway was open to traffic. A deepened cut ditch was provided with geometric features that contained rock-fall having a 4-ft bounce height.

Four alternative rock-fall mitigation barrier systems were analyzed (shown in Figure 10) and presented to the team, including:

1) Low-impact containment fence directly behind new guardrail - *(estimated construction cost of $95,000)*
2) 32-inch to 42-inch high concrete barrier with a 12-inch or 18-inch high chain link fence on top - *(estimated construction cost $95,000)*
3) 20-ft high retaining wall with a 4-ft high low-impact rock-fall containment fence on top of the retaining wall – *(estimated construction cost $610,000)*
4) Modified cut ditch directly behind new guard rail – *(estimated construction cost $25,000)*

Rock-fall Mitigation Barrier System (4) was recommended by the design team and selected by ADOT due to minimal visual impact, provision of easy access for snow removal from the roadway, and lowest cost. Dumped rip-rap dikes were also added to provide mitigation against high drainage flows along the 6% longitudinal grade of the ditch.

Conclusions & Acknowledgements

The compound failure consisting of multiple direction of movement of the slide mass was controlled by a few select fractures and one fault in a warped and block-faulted rock mass. Predictability of the failure was difficult due to variability of discontinuity orientations across the slope in the warped rock mass and a few specific controlling features. Adjustments in design to stabilize the slope and accommodate a revised slope angle which promotes more rock-falls than the original steeper slope were required as construction progressed. Visual impacts and consideration for maintenance of transmission towers were considered in the adjusted designs as the project progressed. Upon completion of final stabilization and rock-fall containment system work, both lanes of eastbound traffic were restored. Monitoring of the slope continued and no additional slope creep or cracking has been identified to date. As a result of early failure detection and prompt remedial design, SR 260 highway users travel more safely.

The authors gratefully acknowledge the assistance of the project team members, including Myron Robison, Bill Pearson, and Jim Wieler of ADOT Payson Construction District, John Lawson, Doug Alexander and Bill Hurguy of ADOT Materials Group - Geotechnical Design Section, and Terry Brennan and Robert Ingram of the Tonto National Forest.

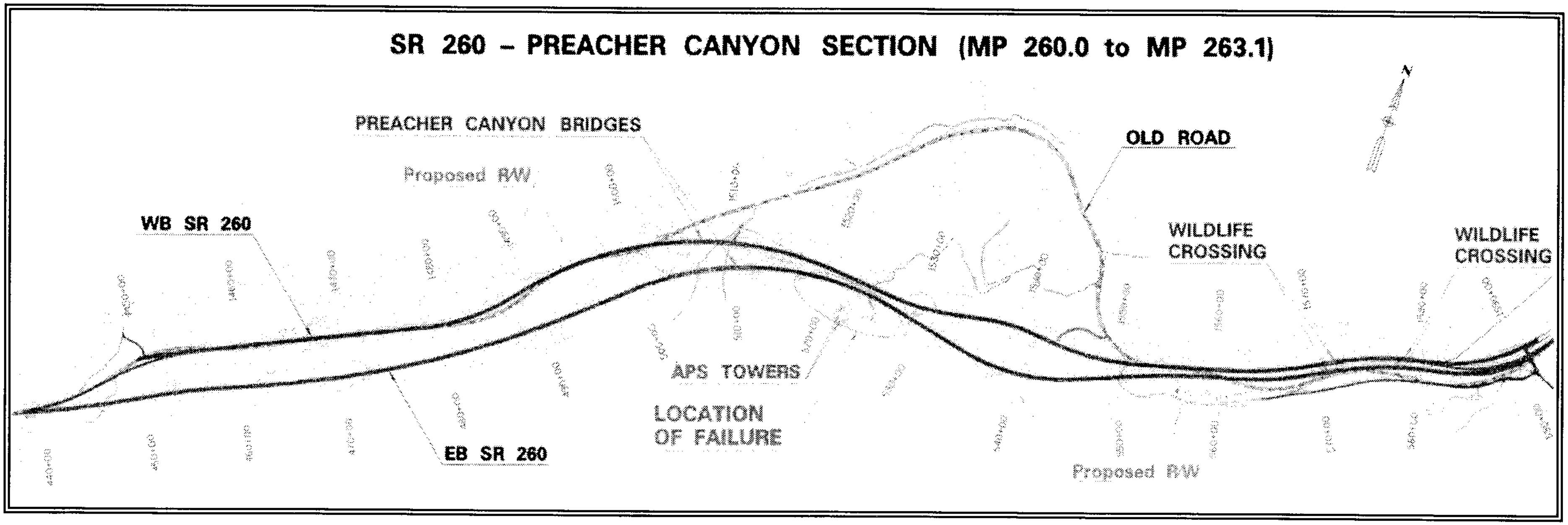

SR 260 – PREACHER CANYON SECTION (MP 260.0 to MP 263.1)
PREACHER CANYON BRIDGES
Proposed R/W
WB SR 260
OLD ROAD
WILDLIFE CROSSING
WILDLIFE CROSSING
APS TOWERS
LOCATION OF FAILURE
EB SR 260
Proposed R/W

FIGURE 1

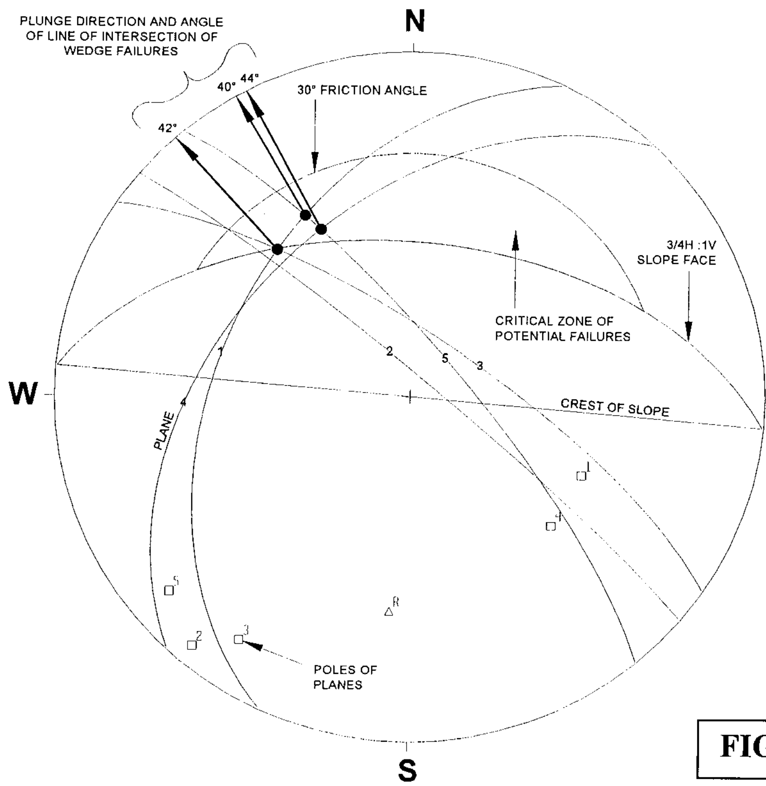


FIGURE 2

STEREOGRAPHIC SOLUTION FOR STEEP WEDGE FAILURES IN ORIGINAL 3/4H : 1V SLOPE

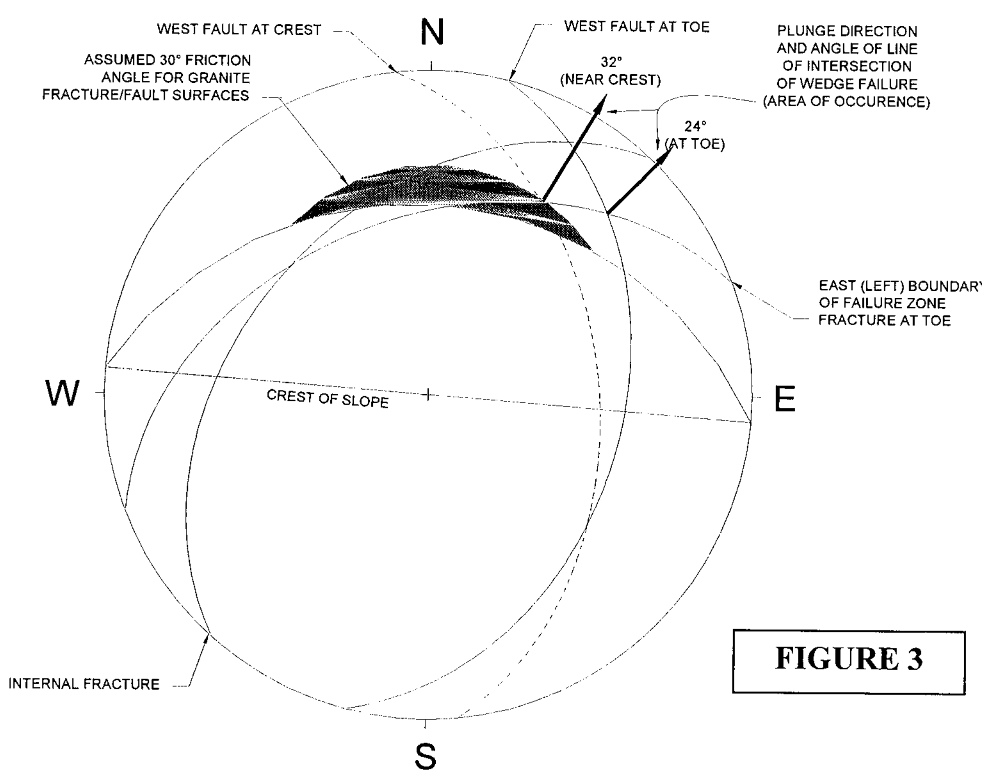


FIGURE 3

NTS

STEREOGRAPHIC SOLUTION OF KEY FRACTURES CONTROLLING SHALLOW WEDGE FAILURE

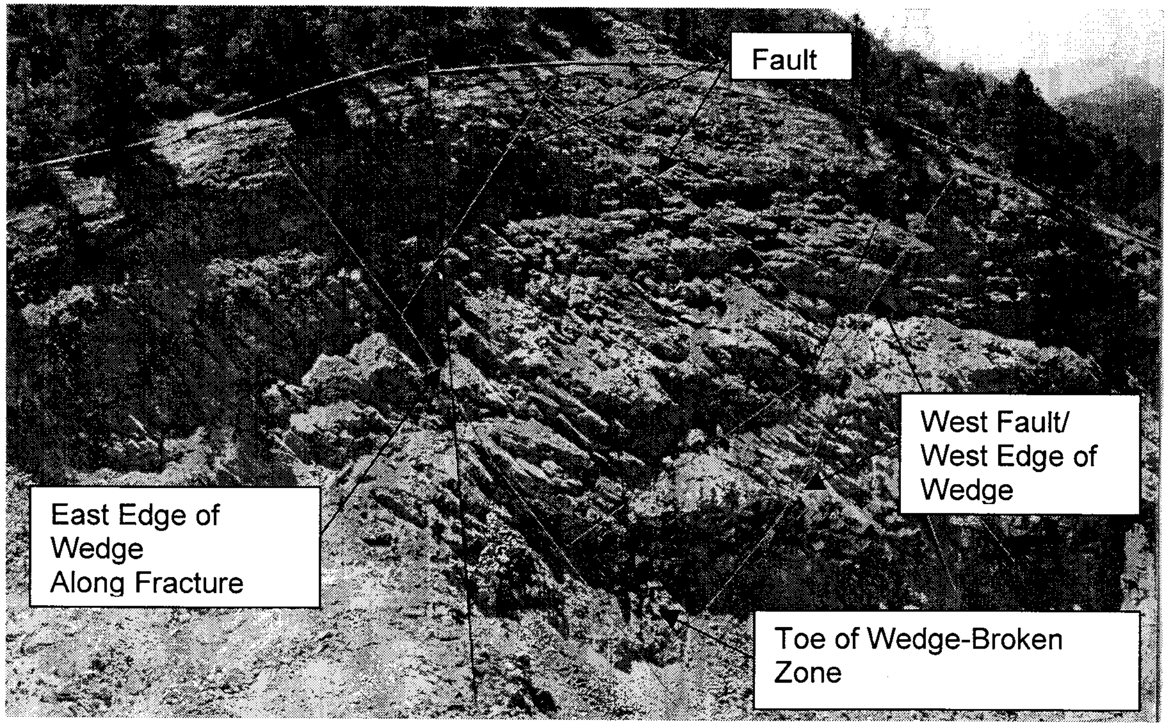


FIGURE 4. Wedge failure in 3/4H:1V slope. Red arrows show direction of movement.

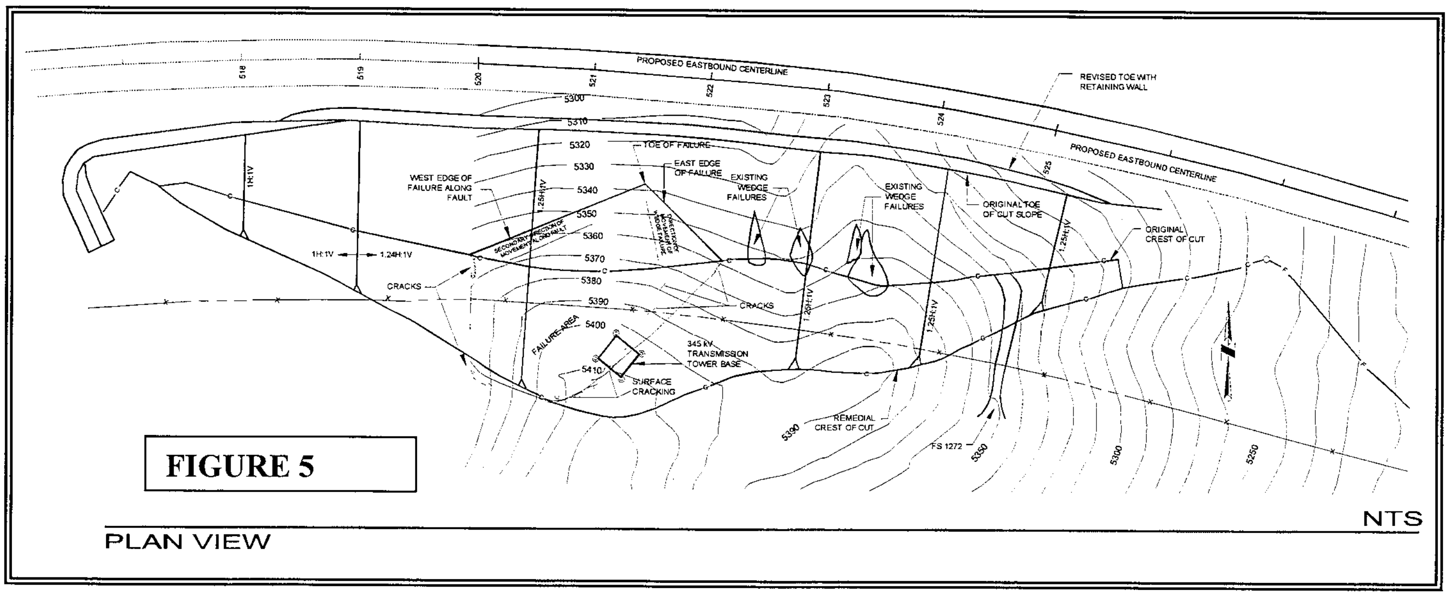
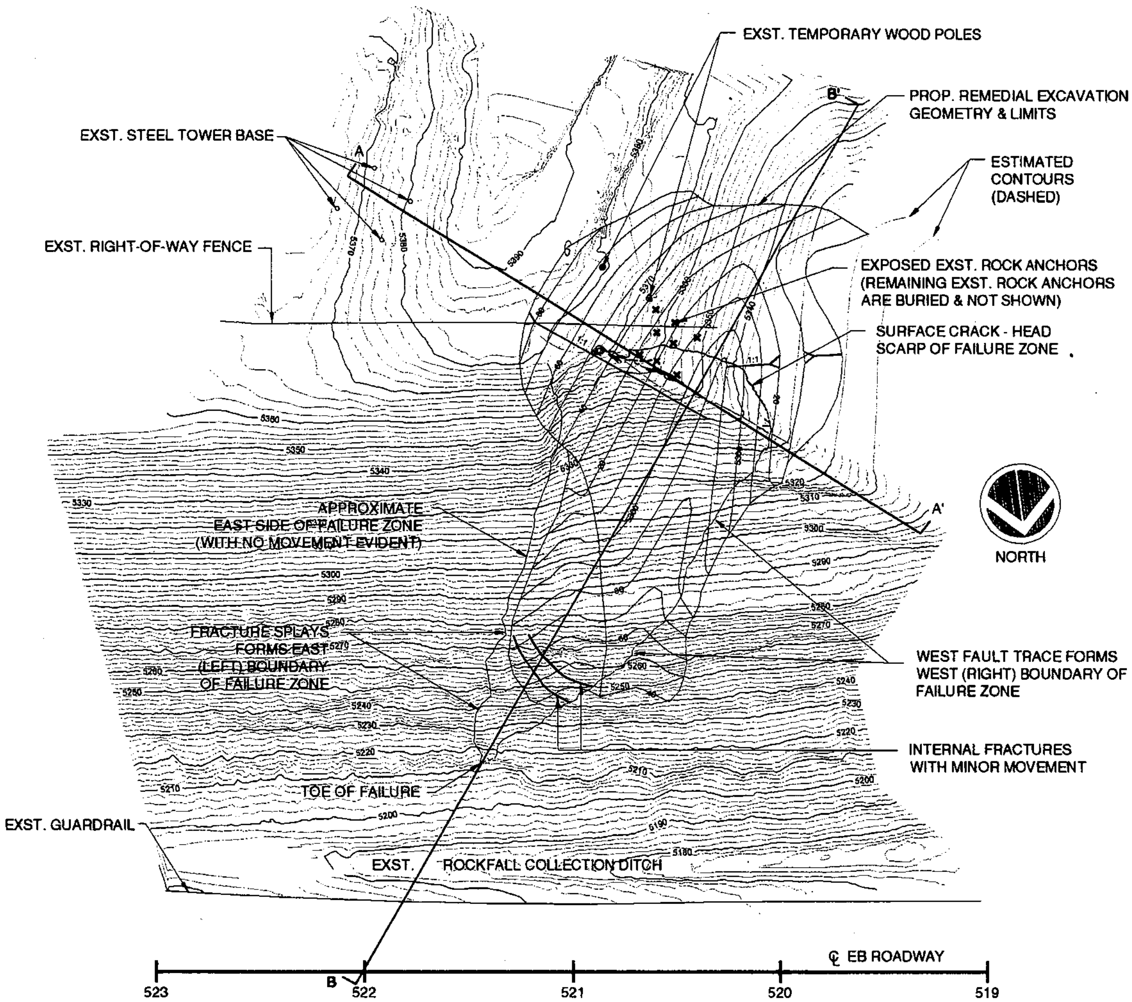


FIGURE 5



PLAN VIEW OF SLOPE - SHOWING EXISTING TOPOGRAPHY & FAILURE ZONE

FIGURE 6

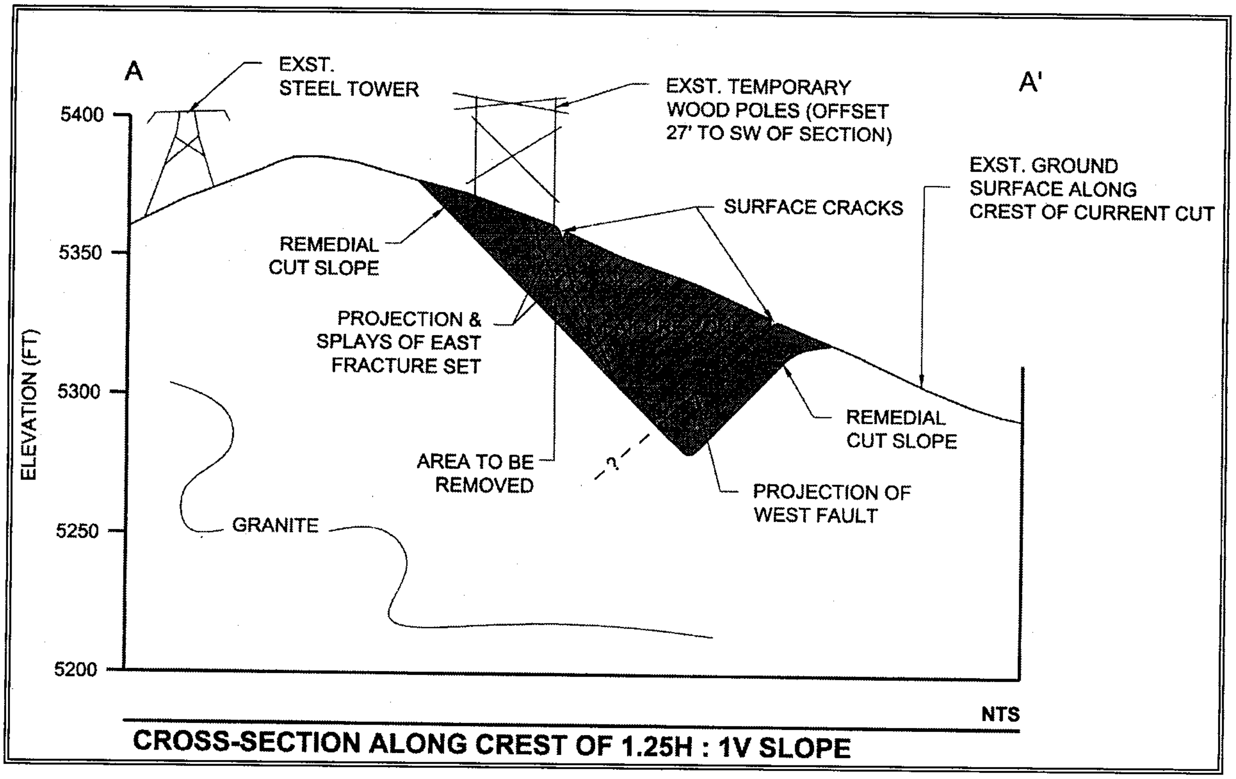


FIGURE 7

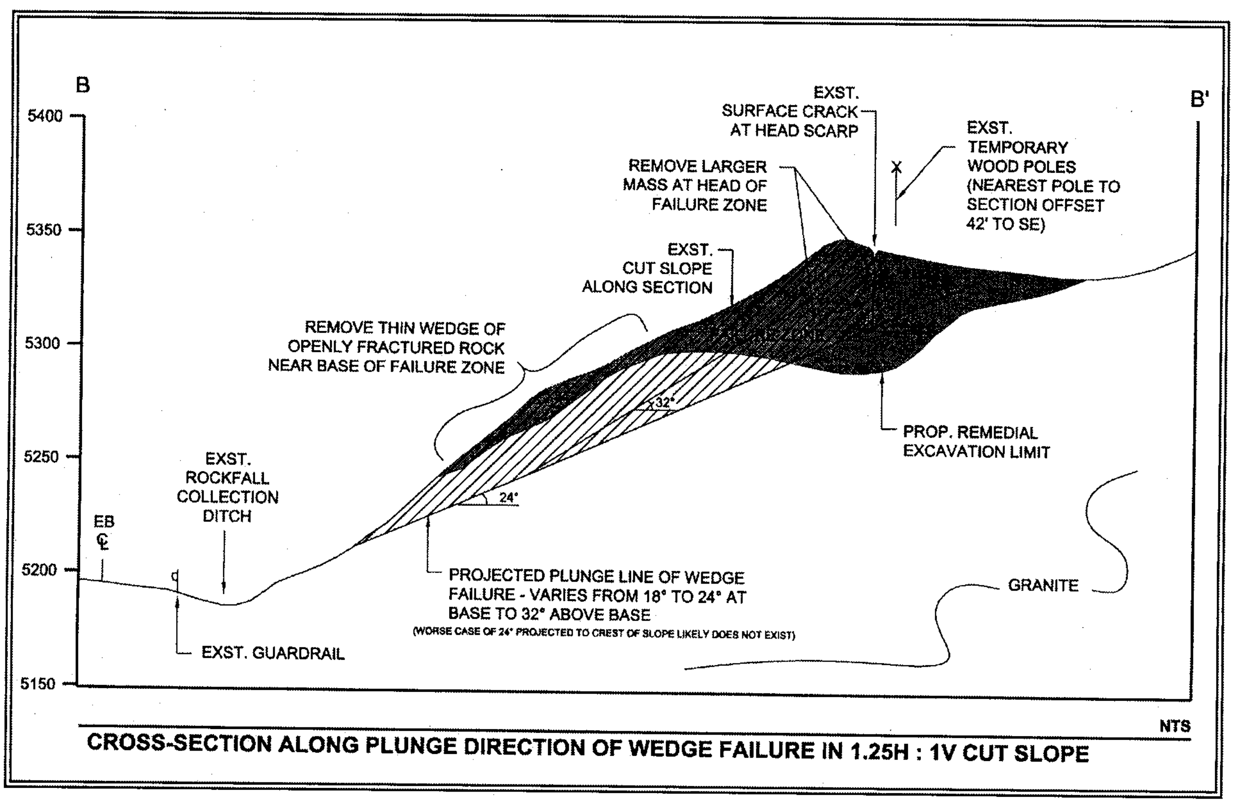


FIGURE 8

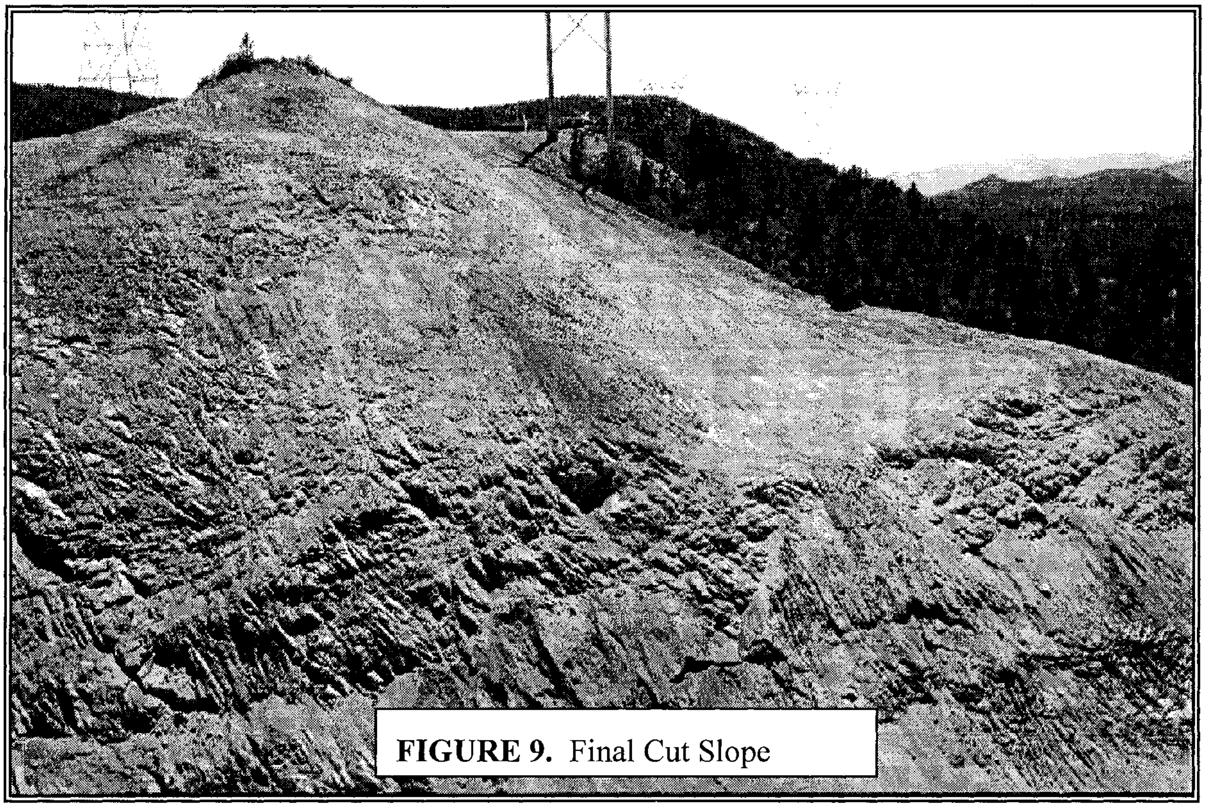


FIGURE 9. Final Cut Slope

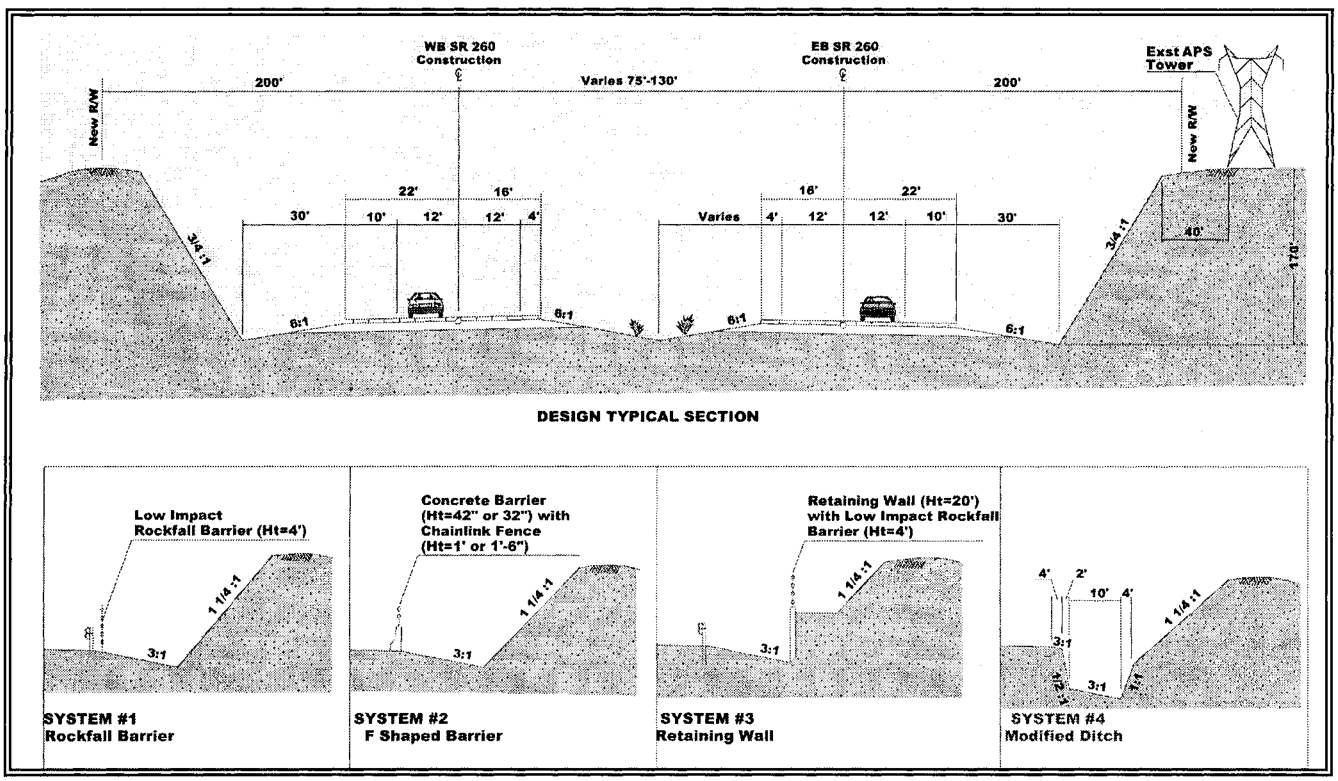


FIGURE 10

Emergency Rock Slope Remediation at "Deadman's Curve" in Pinkham's Grant, New Hampshire

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ABSTRACT

During the spring of 1999, rock fall from a large rock cut landed on N.H. Route 16 in Pinkham's Grant, New Hampshire. The site is located at the base of Mt. Washington in the White Mountain National Forest. The 110 foot high rock slope is carved into the valley wall along the inside of a sharp curve in the road. On the opposite side of the road there is a steep rock fill embankment slope, which drops 200 feet in elevation to the Ellis River. Rock fall has consisted of blocks ranging from 1 to 6 feet in diameter. Further instability in the rock slope posed an imminent hazard to motorists, prompting the New Hampshire Department of Transportation to temporarily close one lane of the road and erect jersey barriers to keep falling rock from hitting vehicles.

An emergency project was undertaken for remediation of the rock slope. The work included closing a section of N.H. Route. 16 for approximately 2 months, the construction of a 0.8 mile long temporary detour road, removal of 122,000 cubic yards of rock, construction of a rock fall catchment area with energy absorbing stone and the installation of prestressed rock anchors. Work started on August 10, 1999 with a deadline of October 1, 1999 to complete the rock removal and to open N.H. Route 16.

The challenges were formidable which included limiting impact to the U.S. Forest Service Land, keeping rock out of the Ellis River, not disturbing environmentally sensitive areas and completing the work in a very short time period without disrupting traffic.

INTRODUCTION

During the first week of April 1999 several rock fall events occurred at a rock cut along N.H. Route 16 in Pinkham Notch. The site is located approximately 11.0 miles north of Bartlett, N.H. in the White Mountain National Forest just below Tuckerman's Ravine at the base of Mt.Washington (Figure 1).

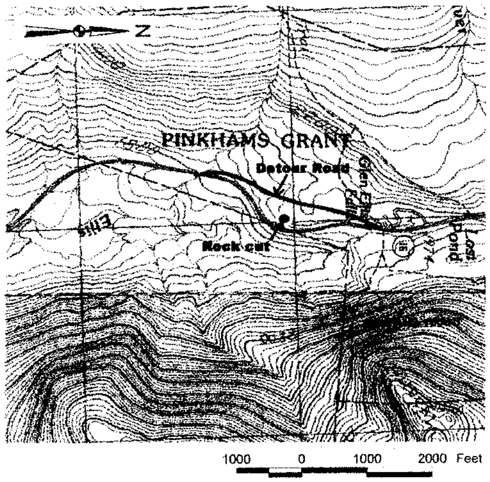

Figure 1. Location map.

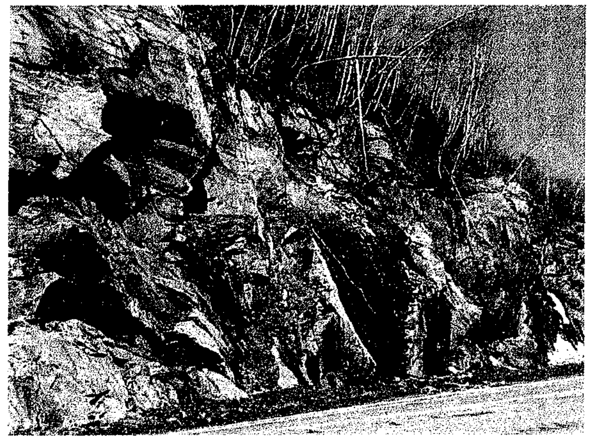

Figure 2. Existing unstable rock slope.

The rock fall occurred over several days during a very active period of freeze-thaw and consisted of fragments ranging in size from 1 to 6 feet in diameter. Most of the falling rock and debris landed in the roadway. An immediate field inspection of the rock slope noted unstable overhangs, numerous detached blocks and slabs, and highly weathered and fractured sections (Figure 2). Many of the potentially unstable sections appeared to be in imminent danger of falling. The hazardous conditions prompted the New Hampshire Department of Transportation (NHDOT) to close the lane adjacent to the rock cut and to temporarily erect portable concrete jersey barriers to protect vehicles. An evaluation of the site was completed and remedial measures were developed to restore full use of the roadway.

GEOLOGY AND SITE CONDITIONS

The bedrock consists of metasedimentary and metavolcanics of the Littleton, Madrid and Small Falls Formations. The exposed bedrock is weakly foliated schist with intrusions of pegmatite on the south end and transitions into coarse, grained granite on the north end. Large clusters of quartz crystals occur in cavities and pockets along quartz veins. The rock slope is extremely ragged with numerous large overhanging blocks and slabs. Many of the larger slabs and unstable blocks dip toward the road and are located on the southern end of the rock cut. There are weathered pockets and seams scattered throughout the rock cut and several long continuous fault planes with rock gouge infilling. There is continuous seepage of water from many of the open joints and fractures. Heavy ice build-up on the rock face is common during the winter months. Roots from small trees and bushes have grown into many of the open cracks along the crest and on the rock face. The ground immediately above the highest portion of the rock slope rises to a narrow hump and then drops down in elevation behind the rock cut to where the terrain levels off for a distance.

The overlying soil above the rock slope is a glacial till deposit with numerous boulders. There are sections of the soil slope above the rock face that have sloughed 3 to 4 feet and have open scars with exposed earth and unstable boulders. Much of the soil slope is wet with signs of long-term creep and old localized failure scars.

HISTORY

N.H. Route 16 hugs a steep valley wall, which runs through rugged terrain at the base of Mt. Washington, the highest peak in the northeastern United States. The rock slope is located on the inside of a sharp curve on a hill with poor site distance. The existing rock slope reaches a maximum of 110 feet in height and is approximately 1000 feet in length. The existing roadway alignment and rock slope were constructed during the early 1960's. The rock is immediately adjacent to the edge of the road with no ditch along the toe of the cut (Figure 3). On the east side of the road (outside edge of curve) there is a steep rock fill embankment slope that drops 200 feet in elevation to the Ellis River. The rock fill embankment slope ranges between 1.25H:1V to 1H:1V. There is a paved shoulder and parking area on the outside of the curve, which has settled an estimated 2 to 3 feet since construction. The existing cable guard-rail was leaning down slope. The high steep embankment fill was most likely constructed with large pieces of blasted rock from the original rock cut and was probably not adequately chinked with smaller rock fragments, resulting in numerous voids.

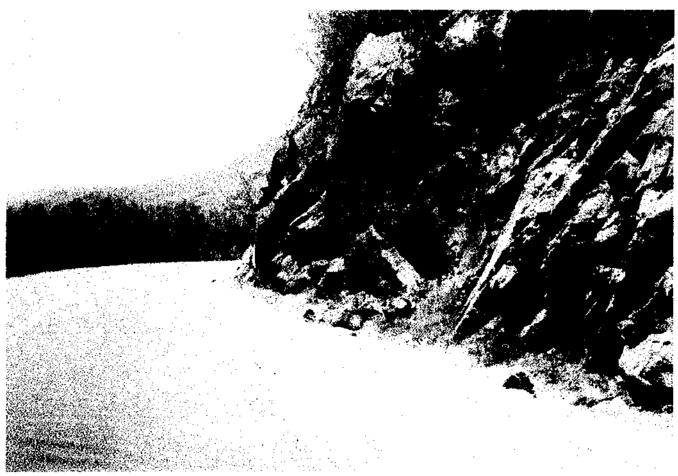

Figure 3. Existing narrow ditch on inside of curve.

In recent years this rock cut has been an active site for rock fall. Falling rock fragments have ranged from 1 to 6 feet in diameter. The overall condition of this rock cut has been deteriorating at an accelerated rate over the past three years.

DATA COLLECTION AND ANALYSIS

The structural features (joints, foliation, faults, etc.) in the rock were mapped and were graphically presented with sterographic projection methods. The patterns of these features and the resulting potential stability problems were evaluated. The primary structural features strike north-northwest and dip west to southwest, and strike northeast and dip southwest. The structural features exposed in the rock slope consisted of the following:

- Short discontinuous joints that dip toward the road

- Long continuous joints that are oriented diagonal to the road
- A fault surface oriented diagonal to the road
- Joints that dip into the rock slope away from the road

Short discontinuous joints dipping toward the road may result in localized rock fall, but are not likely to cause a large-scale slope failure. In many instances, joints that dip toward the road combine with joints dipping into the rock slope, causing dangerous overhangs. Combinations of intersecting structural features form numerous potentially unstable blocks, wedges and slabs throughout the rock cut. In addition, some of the joints have soil and weathered rock infilling, which significantly decreases the strength along rock surfaces. The presence of abundant groundwater in the rock slope reduces the shear strength of the potential failure surfaces, increases the forces that induce sliding, and accelerates weathering. The freezing of the groundwater causes frost pry and blocks the drainage paths which leads to increased water pressure within the slope. In summary, the most common instabilities at this site are the failure of isolated blocks and slabs, and differential weathering.

The investigation identified several factors that contributed to the instability of the rock slope at this site.

- Freeze-thaw action
- Extensive seepage of groundwater from both the rock slope and the overlying soil
- Tree roots extending into cracks and prying loose rock fragments
- Differential weathering
- Adverse orientation of structural features in the rock
- Uncontrolled blasting, which resulted in an irregular rock slope with a shattered rock face

It became evident that any remedial work at this site could not be completed within the required time frame without closing this section of N.H. Route 16. This road is a major commercial truck route and the only road for tourist and local commuters from the Conway/Bartlett area to Gorham. The only existing detour would be to re-route traffic for an additional 30 to 40 miles through Crawford Notch further to the northwest. This would not only cause considerable delay, but would impose regional economic burdens and would drastically increase traffic on several other routes.

A portion of the abandoned roadbed for the old highway through Pinkham Notch, last used in 1930, runs further to the west and at a higher elevation than the existing road. This abandoned alignment begins in the vicinity of the Glen Ellis Falls parking area (0.3 of a mile north of the rock slope), runs in a southerly direction behind the existing rock cut, continues south on a steep downgrade (14% grade) and then joins the existing N.H. Route 16 near a stream crossing. Exposed bedrock, numerous boulders and generally wet conditions were encountered along the abandoned alignment. The old highway is approximately the width of a single travel lane. Small trees and bushes were so thick in some areas that it was difficult to follow the roadbed of abandoned road.

After conducting a title search on the abandoned alignment, the Department discovered that ownership of the old highway was never transferred to the U.S. Forest Service. Although a technicality, this helped in obtaining permission to use the abandoned alignment as a temporary detour during construction. Since existing plans and survey were not available, a Global Positioning System (GPS) was utilized to map the location of the abandoned alignment. A laser profiler was used to accurately measure the height of the existing cut and to profile the rock slope in critical areas. This information was valuable in the initial planning and design phase, and was critical in meeting the deadlines for this emergency project.

SELECTION OF REMEDIATION MEASURES

The U.S. Forest Service, environmental agencies, local towns and the NHDOT together formulated the criteria for selecting an appropriate stabilization method(s) for this rock slope. The selected remediation measures and method of construction had to meet the following requirements:

- The solution(s) must provide long-term safety for the traveling public
- Only minor traffic delays with no additional travel time are acceptable
- Permanent roadway alignment changes for N.H. Route 16 are not allowed
- The existing N.H. Route 16 roadway must reopen for traffic no later than the first week of October 1999
- Rock and debris from the construction activities are not allowed in the Ellis River
- Adversely impacting the U.S. Forest Service Land is unacceptable

Several alternative stabilization methods were considered.

1. Extensive scaling to remove unstable rock was an option, but it would only be a temporary and partial solution. Many of the larger unstable blocks could not be removed with this method, making it marginally effective. This type of operation would be dangerous, due to the difficult access, the height of the rock slope and the risk to the traffic below.

2. Measures to control and contain rock fall were investigated. Intercepting and directing rock fall with wire mesh netting, cable mesh netting or a ring net draped over the slope would not be practical. The maximum potential size of individual falling rocks was estimated to be 15 feet in diameter. Wire mesh or cable mesh nettings are not able to handle rocks of this size and there is inadequate room for rock to accumulate at the toe of slope. The embankment slope on the other side of the road is both high and steep. This prevents the road from being shifted further away from the unstable rock slope so a rock fall catchment area can be created.

3. Rock anchors or rock bolts could be used to support some individual blocks and reinforce portions of the rock mass. Many of the blocks and slabs at this site have become detached and are not stable enough to drill. Rock anchors or bolts alone would not be able to secure all the potentially unstable rock. Some areas of the rock slope cannot be reinforced due to the active "ice jacking" and rapid differential weathering.

4. Protection measures such as barriers and rock fall fences constructed along the toe of the rock slope were not considered feasible. Room at the toe of the rock slope is inadequate and the maximum size of the falling rocks is too large. Rock fall modeling at this site predicted a high percentage of falling rocks bouncing over the top of a barrier or fence.

5. The feasibility of constructing a rock shed over the highway was studied. The cost of construction for this protective measure would be significant. It would require the installation of an anchorage system into unstable rock on the upslope side and the founding of deep columns on sound rock on the down slope side. The columns would have to be drilled through a rock fill of unknown depth. The roof of the rock shed would be inclined so the rocks would roll across the shed with minimum impact. The falling rock would likely be redirected down the steep embankment slope and into the Ellis River. This was not acceptable.

The selected remediation was to cut the rock slope further back from the road and to provide a catchment area for any future rock fall. It was recommended that the southern half (highest section of the rock cut) of the rock slope be excavated in two lifts, so the hump at the top of the slope could be eliminated. The upper lift was to be excavated on a 1H:8V presplit slope, resulting in a rock face of approximately 20 feet in height (Figure 4). A 50-foot set back of the upper lift would eliminate the narrow hump at the top and would provide a stable bench for drilling the lower lift. It was recommended that the lower lift be presplit on a 1H:8V slope and extend for the remaining height of the rock slope (70 to 80 feet). The NHDOT Standard Specification requires rock cuts to be excavated in 35-foot high lifts with a lateral offset of 18 to 24 inches

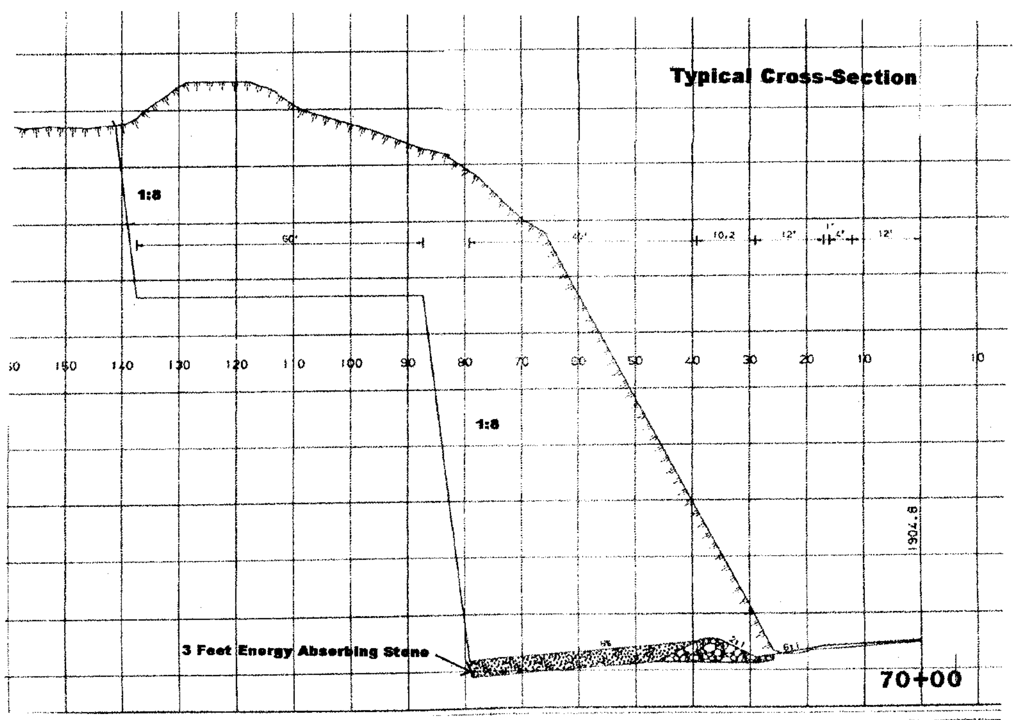

Figure 4. Typical cross-section of proposed rock slope remediation.

between successive lifts. Due to the very difficult access, previous experience with similar sites and the need to limit the impact at this location, it was decided that the presplit holes for the bottom section of the rock slope would be drilled as a single lift. State of the art drilling equipment was used to control drill wander. To improve the chances of obtaining a stable uniform cut slope, drill hole deviation measuring devices were required. It was recommended that the northern half of the rock slope be excavated in one lift on a 1H:8V presplit slope. To provide an adequate rock fall catchment area, the toe of the rock slope would be setback a minimum of 30 feet from the roadway. It was also recommended that a 3-foot thick blanket of energy absorbing stone (one inch minus) be placed in the ditch at the toe of the rock slope to help contain future rock fall. Unloaded relief holes between the loaded presplit holes were required to promote shearing along the desired cut slope and to minimize back break along the crest of the rock slope.

It was recommended that prestressed rock anchors be provided in the contract as a contingency item for securing unstable rock and for providing long-term stabilization for potential areas of instability. The decision to install the rock anchors would be made during construction by the Department's Engineering Geologist. Rock anchor locations and depths would be determined after a final inspection of the cut slope.

Figure 5. Detour road.

Figure 6. Temporary detour road with Jersey Barriers

The Department recommended that a portion of the abandoned highway be utilized as a temporary detour road during construction. The contractor was able to utilize the detour for access to the top of the rock cut and to use the closed portion of N.H. Route 16 as a staging area for the project. This resulted in minimal disruption to the construction work and allowed traffic to be maintained with only minor delays and no increase in traveling distance.

EMERGENCY PROJECT

The first phase of the project was the construction of a temporary 0.8 of a mile long detour road, which followed a portion of the abandoned highway (Figure 5). To minimize the impact to the area, the width of the temporary road was restricted to two 12-foot lanes with concrete jersey barriers placed along both sides of the road (Figure 6). In addition, the profile of the detour was held close to the existing ground, all trees and bushes were flush cut, grubbing was eliminated and the topsoil was left in place. A heavy geotextile fabric was placed over the existing ground along the entire length of the detour (Figure 7). The roadway fill and base course materials were kept to a minimum. The detour was removed in the spring of 2000 and vegetation was replanted.

Figure 7. Geotextile on detour road.

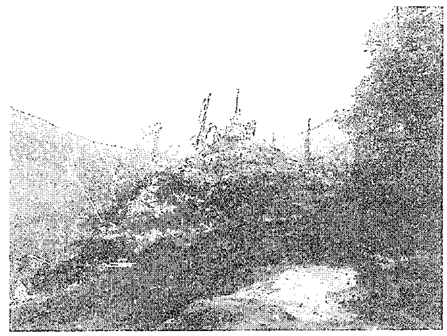

Figure 8. Drilling the hump.

The second phase of the project involved the excavation of the hump (Figure 8) in the upper section of the rock slope. This was accomplished prior to the blasting, excavating and crushing of the remainder of the rock cut. The excavation and crushing activities were conducted simultaneously on both ends of the rock cut (Figure 9). On several occasions the blasting operation had to be slowed so the crushing activities could catch up. The Contractor averaged two blasts per day with individual shots ranging in size from 2,000 to 12,000 cubic yards of rock (Figures 10 & 11). The entire height of the rock cut was presplit in a single operation. To help control blasts and to limit the amount of rock going over the steep embankment slope, the production blasts were shot in multiple lifts. This method of excavation was successful in preventing fly rock from reaching the environmentally sensitive areas along the Ellis River. To improve drilling accuracy and to minimize drill wander (deviation), guide bits and accessories were required for all drilling. The blaster was required to measure the deviation of all drill holes, 50 feet or greater in length, prior to loading explosives. A borehole

Figure 9. Crushing Operation.

Figure 10. Loading Blast Holes

deviation system was used to measure the orientation of the drill holes. The blaster was required to use the borehole measurement data in his blast design and in determining the placement and the amount of explosives to be used in the blast holes. A drill hole deviation of less than 2% for depths of up to 80 feet was specified in the contract. The blaster was able to maintain a 0.5 to 0.6 foot deviation for all drill holes.

The disposal of excavated rock was not allowed within the boundaries of the White Mountain National Forest. Therefore, all excavated rock and debris had to be hauled and disposed of offsite. Hoe-ram hammers were used to downsize the larger rock fragments, which remained after each blast. To limit the void space within the loaded trucks and to reduce the risk of damage to the haul trucks from oversized boulders, the Contractor crushed nearly all the excavated rock to minus 6-inch size. Crushing the excavated rock with two portable crushers located on the project, also reduced the total number of truckloads and minimized damage to the surrounding roadway system. All the excavated rock was fed through two track-mounted portable crushers capable of handling material in the 30-inch range. To avoid heavy traffic, hauling began everyday at 3 AM with most of the rock being stockpiled at sites 15 to 30 miles away. The excavation phase started on August 10th, 1999 with a deadline of October 1st, 1999. The Contractor utilized more than 30 haul trucks to move approximately 122,000 cubic yards of rock.

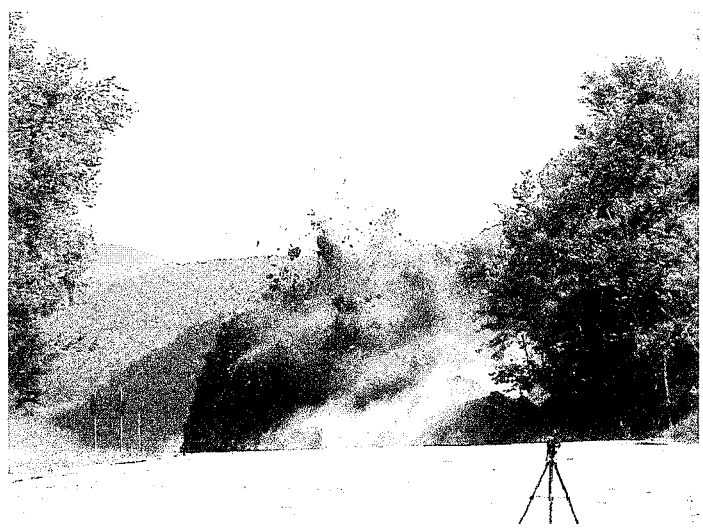

Figure 11. Blasting.

In October 1999, the rock removal operation was completed and the existing N.H. Route 16 was opened for traffic. Patching of holes and temporary pavement overlay was placed on the roadway to repair damage from the construction work. Prior to winter, the fill, base courses and geotextile fabric were removed from the detour road. During the spring of 2000, the old pavement on N.H. Route 16 was removed, new base course materials were placed and permanent pavement was installed along the entire length of the project area. Drainage work, landscaping, final restoration to the detour road and installation of the rock anchors were completed in the spring of 2000.

An inspection of the rock slope was conducted during the early spring of 2000. Several large slabs on the south end of the rock cut and a highly fractured fault zone with large loose blocks in the middle of the rock cut were identified (Figure 12). It was decided that the

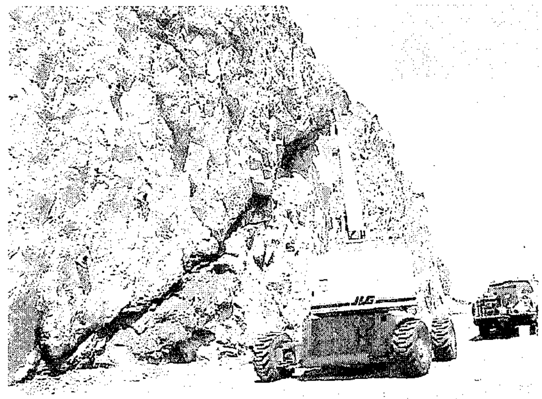

Figure 12. Fractured fault zone.

installation of prestressed rock anchors at critical points would help secure the rock slope and prevent localized slides. The location and angle of installation for each rock anchor was based on the size of the blocks and the orientation of the potential failure surface. The angle (both horizontal and vertical) at which the anchor holes could be drilled relative to the existing rock face was limited by the design and configuration of the drilling platform. Twelve 40-foot long, fully grouted, prestressed rock anchors were installed to secure several potentially unstable sections of the rock slope. The rock anchors were 1 3/8 inch diameter, epoxy coated, grade 150 steel and continuously threaded bars. The anchors were designed with a 15 foot long bonded zone (anchor zone) and a 25 foot long unbounded zone (free stressing zone). The components for a typical rock anchor are shown in figure 13. The contractor's drill rig worked from a pre-fabricated steel platform, which was suspended from a 180-ton crane.

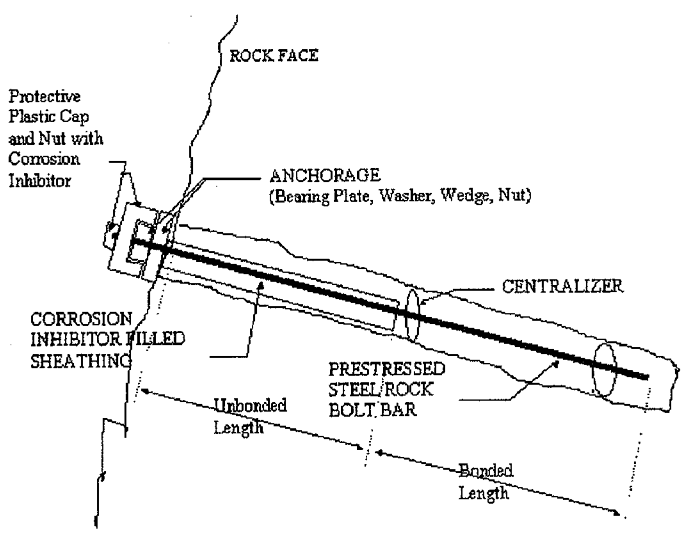

Figure 13. Schematic of prestressed rock anchor.

Due to the irregular rock face and the angle at which the bolts were installed, a concrete pad had to be constructed for each rock anchor bearing plate assembly (Figure 14). Proof tests were conducted on each installed rock anchor (Figure 15). Locating sound rock for each rock anchor, determining the orientation of the anchors in relation to the rock face and installing rock anchors at heights of up to 70 feet above ditch level were challenging (Figures 16 & 17). Construction of the concrete pads for the bearing plate assembly and establishing a fixed control point for measuring the elongation of the anchors during the testing were extremely difficult tasks due to the continually changing and severe weather conditions at the site. Each rock anchor had a design load of 125 kips and was incrementally stressed during the proof test with a 110-ton hydraulic jack. A creep test was conducted to make sure there was no significant loss of load over time and a lift-off test was conducted to verify the load transferred to the anchor. The rock anchors were installed in accordance with criteria established by the Post-Tensioning Institute.

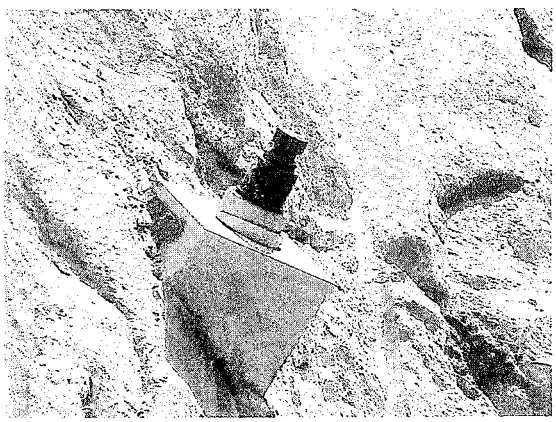

Figure 14. Anchor with concrete pad.

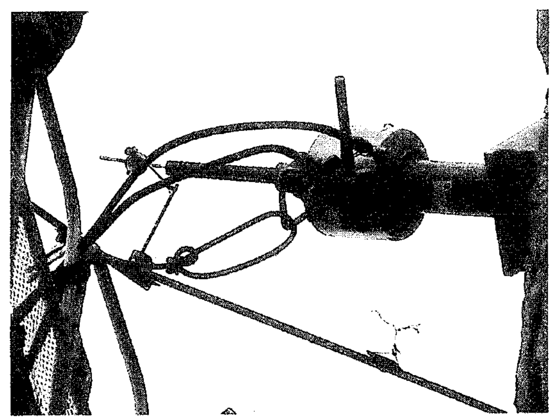

Figure 15. Proof testing of rock anchor

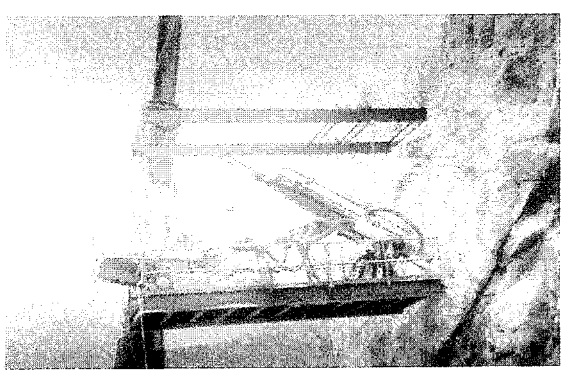

Figure 16. Drilling rock anchor holes.

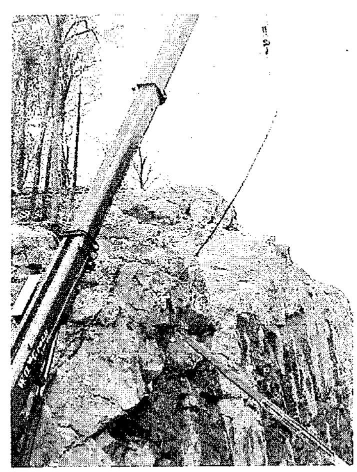

Figure 17. Installing anchors.

SUMMARY

The project was completed on time with an estimated 122,000 cubic yards of rock blasted, hauled and crushed in less than two months. N.H. Route 16 was reopened for traffic during the first week of October 1999. The temporary detour road to include pavement, base course materials and geotextile was removed prior to the winter of 1999. The installation of the rock anchors, landscaping and final paving were completed in the spring of 2000.

Since the completion of the remediation work, the rock slope has experienced two winter seasons without major problems. There have been numerous small rock fall events (1 to 3 foot in diameter) over the past two years, all easily contained within the catchment area (Figure 18). During the spring of 2000, several localized areas of settlement (2 to 3 feet across) appeared in the stone covered ditch at the toe of the rock slope (Figure 19). This settlement is the result of voids in the underlying blasted rock. The local Maintenance District repaired the voids.

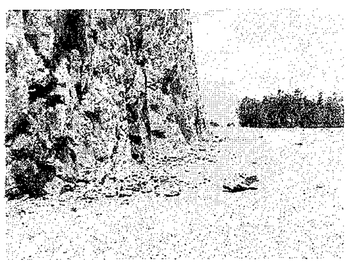

Figure 18. Rock fall in ditch

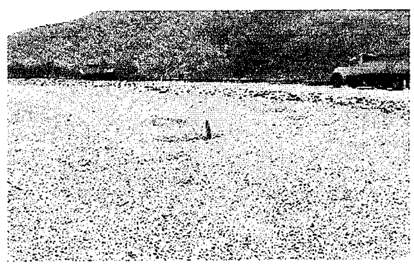

Figure 19. Voids in ditch

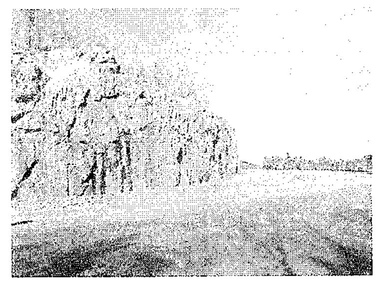

Figure 20. Completed project

This project could not have been successfully completed within the limited time period without the closure of N.H. Route 16 and the construction of the temporary detour road. The total cost for the project was approximately 3.5 million dollars. Construction was completed without a negative impact to the local economy, a disruption to traffic (commercial, tourist, local) and without an adverse impact to the surrounding environment (Figure 20).

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Post-Tensioning Institute, 1996, Recommendations For Prestressed Rock and Soil Anchors, Post-Tension Institute, Phoenix, Arizona.

Trojak, L., 2000, Blasted Mt. Washington Rock Recycled, New England Construction, Associated Construction Publications, Norcross, GA, 22 p.

ABSTRACT
Ruby Creek Rock Slope Stabilization

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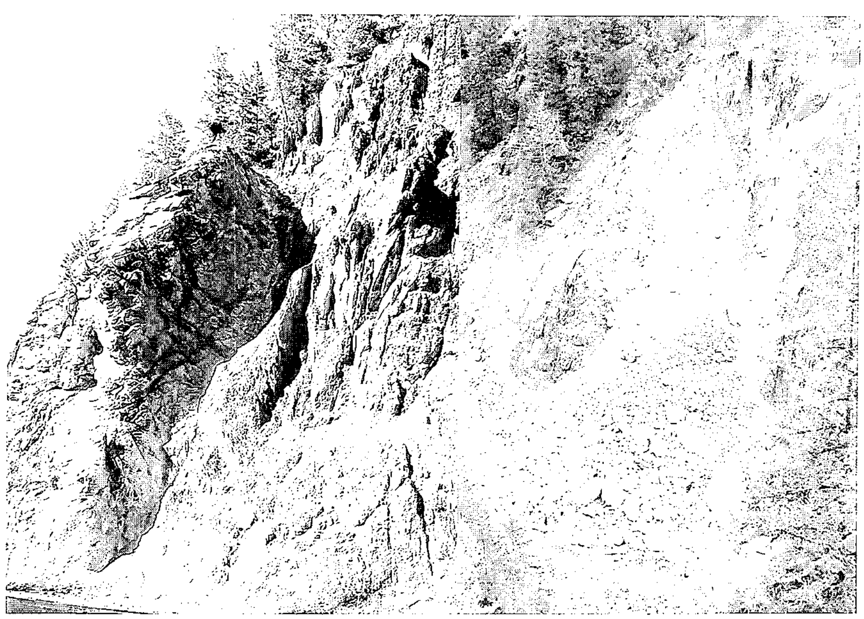
Tom J. Byers – Golder Associates Inc.
Steve M. Lowell – Washington State Department of Transportation

Located on State Route 97 near Blewett Pass in the Cascade Mountains of Washington State, the Ruby Creek site consists of a 200-foot high rock cut originally constructed in 1959. The complex geologic setting for the project site includes slates or argillites, tectonically emplaced greenstone blocks and basalt. As part of its Unstable Slope Management Program, WSDOT had rated this site as a high priority based upon the hazard to the highway and the potential economic impacts if a slope failure was to occur. The significant hazards included a 5000-yd^3 mass of rock susceptible to a planar mode of failure, an 11,500-yd^3 rock mass susceptible to a wedge mode of failure and numerous small-scale features contributing to rock fall.

Site investigations included aerial reconnaissance, detailed surveying of the location of major fault structures, joint population mapping including multiple traverses down the slope face on ropes, shear strength testing of fault gouge material and testing for intact rock strength. Stereographic analyses and computerized stability analyses led to the conclusions that the planar mass should be removed by controlled blasting and that the wedge mass should be stabilized through reinforcement. Design criteria and specifications were provided for blasting and scaling, high capacity rock bolts, low capacity rock bolts, untensioned dowels and shear pins. Shotcrete and extensive application of cable netting were recommended to control ongoing rock fall.



Greenstone Block Before and After Blasting

The blasting portion of the project was subject to stringent timing constraints to accommodate summer holiday traffic and to safeguard biological resources in the area. These constraints required the consideration of multiple blasting strategies to remove the 5000-yd^3 planar block. The adopted plan consisted of a single shot drilled from the top of the block. The results of the blast provided valuable lessons and specific data on the issues of access for drilling, hole deviations in 160-foot long angled blast holes, muck pile run out, flyrock trajectories, blast cleanup methodology, and traffic interruption. The scaling, bolting and cable net installations also yielded practical information on the challenges of converting design intention to construction reality.

INTRODUCTION

For the Washington State Department of Transportation (WSDOT), the Ruby Creek project represented a major hazard reduction effort as part of its Unstable Slope Management Program. The project was designed and partially constructed in 2001 and completed in the spring of 2002. The stabilization program had to be compatible with heavy traffic volumes, fishery and wildlife habitat constraints, engineering risk reduction and costs. The elements of this project are presented under the main topics of *planning, investigation, engineering design, construction, and hindsight.*

PLANNING PHASE

Site History

The current alignment of State Route 97 is a realignment of the Old Blewett Pass Highway. SR 97 is a heavily traveled north south corridor connecting Interstate 90 with State Route 2 to the north. Remnants of the Old Blewett Pass Highway are located on the opposite side (southwest-side) of Peshastin Creek and may be seen from the project site. Construction of the existing alignment including the Ruby Creek rock cut occurred around 1959 and was likely completed in order to reduce grades and curves inherent in the Old Blewett Pass Highway alignment.

Unstable Slope Management System

WSDOT has developed an integrated management system to inventory and prioritize unstable slopes along the state highway network. This system has eleven categories for rating the hazard represented by a given slope. With a maximum score or 81 points per category, the maximum point rating for a given slope is 891. The Ruby Creek slope was rated according this system in 1993, and received a total rating of 453 points.

Historically, rock fall problems along the 200-foot high cut have been relatively minor, normally handled with routine maintenance in the form of regular ditch cleaning. The largest single event recalled by WSDOT maintenance personnel involved an estimated tens of yards of debris. However, in addition to the rock fall hazard, the potential for large volume, structurally controlled failures was recognized in the slope rating assessment. The magnitude of these potential failures was such that the highway could potentially be closed for weeks resulting in severe economic impacts. Coupled with the high traffic counts, the resultant risk yielded a favorable cost benefit ratio and justified a major slope stabilization program.

INVESTIGATION PHASE

Ruby Creek Geologic Setting

As shown in Figure 1, the project site is located within a zone of complex geologic and structural conditions known as the Ingalls Tectonic Complex. This complex is a wide belt wrapped around the south end of the Mount Stuart batholith as described by Tabor and others (1987). The most abundant rocks within the Ingalls Tectonic Complex include serpentinite, serpentinized peridotite, and metaperidotite and include sandstone argillite, radiolarian chert, pillow basalt, diabase, and gabbro. All of the rocks have been tectonically mixed in what is termed a melange complex. The age of the tectonic mixing has been interpreted by Tabor and others (1987) as Late Jurassic or Early Cretaceous age (~150 million years before present). This terrain is interpreted to have been thrust over older geologic terrain prior to the intrusion of the Mount Stuart batholith in Late Cretaceous time.

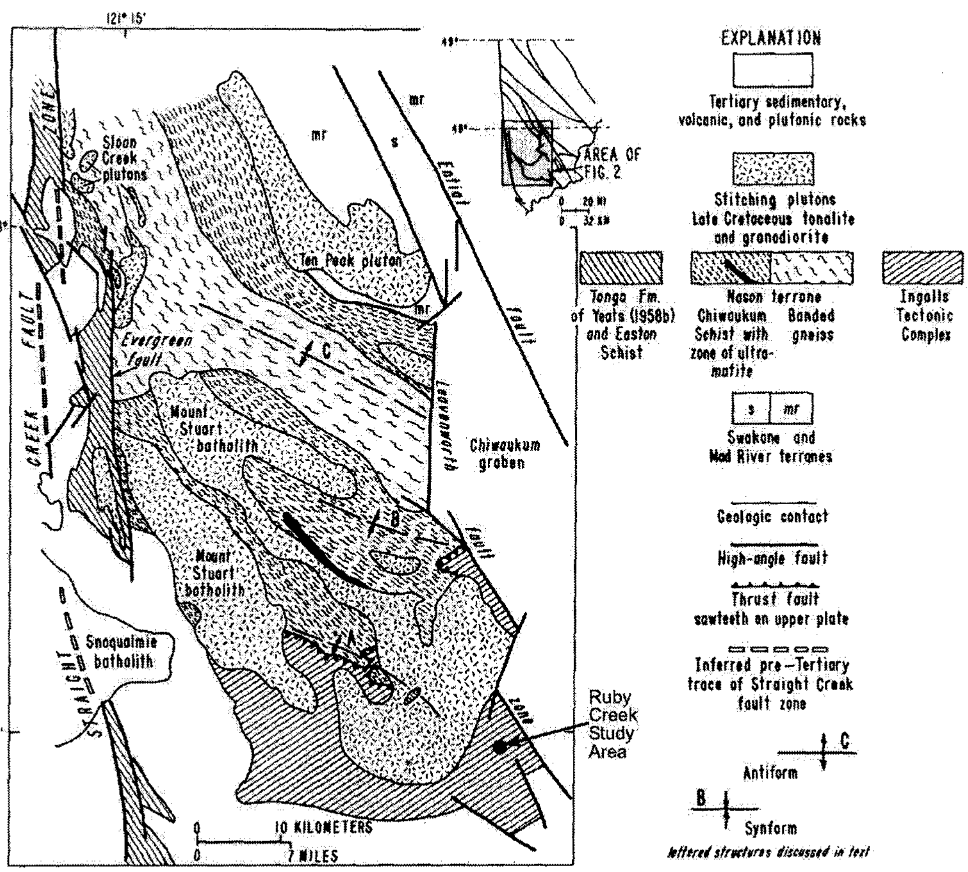


Figure 1 Regional Geologic Setting *(after Tabor & Others 1987)*

The Leavenworth Fault, a major regional structural feature, is located approximately 2 miles northeast of the site. This northwest striking fault forms the southwestern boundary of the Chiwaukum graben, a major structural feature of central Washington State. The Entiat Fault forms the northeast side of the graben. The Leavenworth Fault juxtaposes lithologies of the Ingalls Tectonic complex on the southwest side against conglomerate and sandstone units of the Middle to Upper Eocene age Chumstick Formation on the northeast side of the Leavenworth Fault.

The rock types located within the project area include slates or argillites of the Peshastin Formation as described by Alexander (1956), tectonically emplaced Greenstone Blocks, and basalt. Most of the metasedimentary and metavolcanic rocks in the Peshastin Creek area are medium grade and partially granoblastic bearing green hornblende and biotitie (Tabor et al.. 1982). Due to the melange nature of the tectonic complex, all rock types are bounded by faults or tectonic shear zones.

Seismic Setting

The project site is located within an area of low historic seismicity. Earthquakes that do occur within eastern Washington are generally confined to shallow (less than 15 miles) crustal events. One of the largest historic earthquakes to have occurred in the Pacific Northwest occurred in 1872 in the North Cascades. This earthquake had an estimated magnitude of 7.4 and was followed by many aftershocks and was probably at a depth of 10 miles or less within the continental crust (University of Washington PNSN). The epicentral area is generally interpreted to be located north of Lake Chelan (approximately 60 to 70 miles from the project site).

Based on the Seismic Hazards Map for the Conterminous United States (USGS, 1996), the project site falls within 0.1 g acceleration contour for 10% exceedance in 50 years.

Site Characterization Activities

The project site was subdivided into five structural domains on the basis of geologic structure, rock type, and preliminary conclusions regarding rock fall / slope stability mitigation (Figure 2). Structural mapping was performed along 11 traverses at the base of the slopes and from rappelling rope traverses down the slope face. An experienced scaler was employed to safety scale the rope traverses in advance of the engineering geologists. A total of 300 structural measurements were recorded in conjunction with the mapping activities within the 700-foot length of the stabilization project.

Index strength testing included 60 tests with a Schmidt impact hammer and 42 tests with a point load apparatus (Broch and Franklin, 1972). Direct shear testing of fault gouge was also performed.

DESIGN PHASE

Engineering Analyses

An appreciation for the potential hazards associated with the existing cut could be gained by simple inspection, anecdotal information about historic performance, and the WSDOT slope hazard assessment. In the design phase of this project, stability analyses were

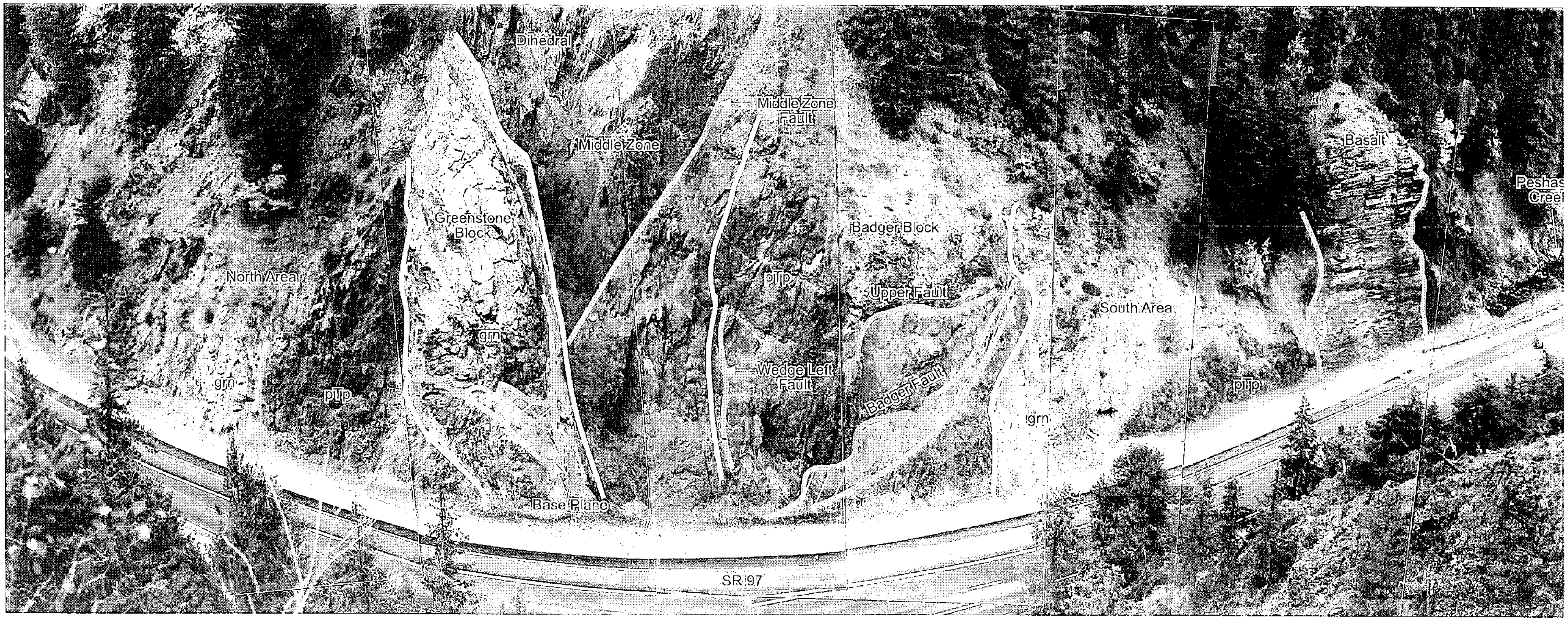


Figure 2 Engineering Geology Mosaic

completed to provide a more quantitative assessment of the potential hazards, and to develop stabilization designs as appropriate.

The greenstone and slate at the Ruby Creek site are generally brittle, fractured rocks. The stability of slopes in such materials is usually controlled by rock fabric, in the form of either systematic sets of discontinuities, or "unique," randomly oriented features. The structural mapping completed during the field investigation was intended to characterize the rock fabric in terms of orientation, persistence, spacing, surface properties, and infillings.

Exceptions to the characterization of the greenstone and slate as brittle, fractured rock occurred in crushed zones along fault contacts between these lithologies, and local zones of intense fracturing within the slate. These discrete zones with low rock mass strength were not themselves adversely oriented with respect to the cut, and instability within them was therefore limited to raveling.

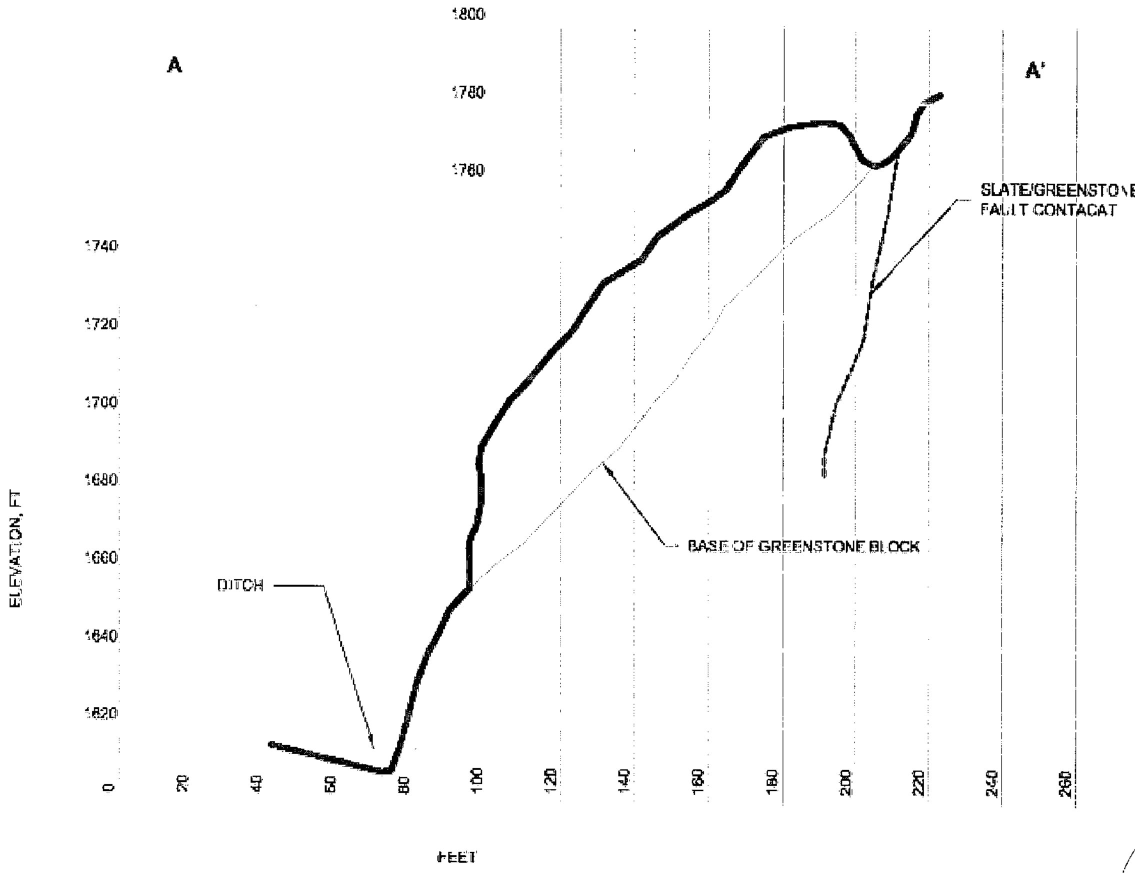
With the potential for significant failures related to rock fabric, stability analyses focused on structurally controlled failure modes. This was a two-part process, involving an initial analysis of the structural mapping data to identify kinematically possible failure modes and their relative impact on highway safety and operation; followed by limit equilibrium stability analyses of specific failure modes where appropriate.

Kinematic Analyses

Kinematic failure modes involve the movement of intact blocks along one or more discontinuities, typical examples include planar, wedge, and toppling failures. The potential for these types of failures was evaluated stereographically as shown in Figures 4 and 6, which present the data for the Greenstone Block and Badger Block domains, respectively. With this type of plot, the great circle representation of the cut and the cone that represents discontinuity friction define "envelopes", and structures with orientations that fall within these envelopes have the potential to effect specific types of kinematic failures.

These kinematic analyses, considered in the context of the WSDOT hazard rating, maintenance history, and field observations, suggested that the most significant hazards within each structural domain were:

- Greenstone Block -- ***Large-scale planar failure*** along the base plane; (Figures 3 and 4)
- Badger Block -- ***Large-scale wedge failure*** along Badger and Wedge-Left Faults, along with significant rock fall hazard; (Figure 5 and 6)
- Middle Zone – Significant rock fall hazard;
- South End – Minor rock fall hazard
- North End – Minor rockfall hazard



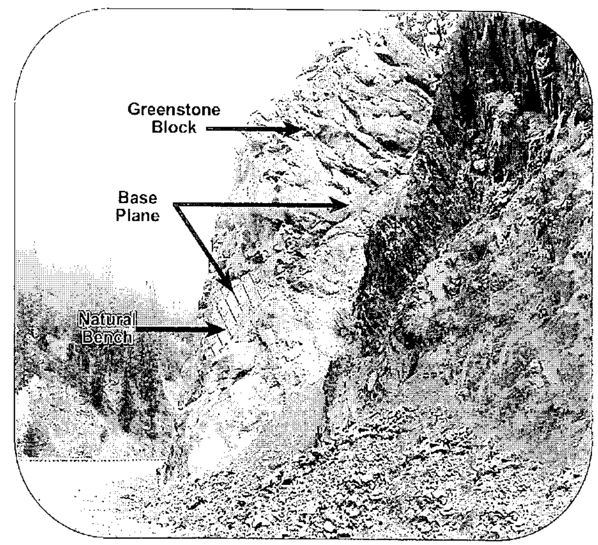


Figure 3 Greenstone Block Cross Section

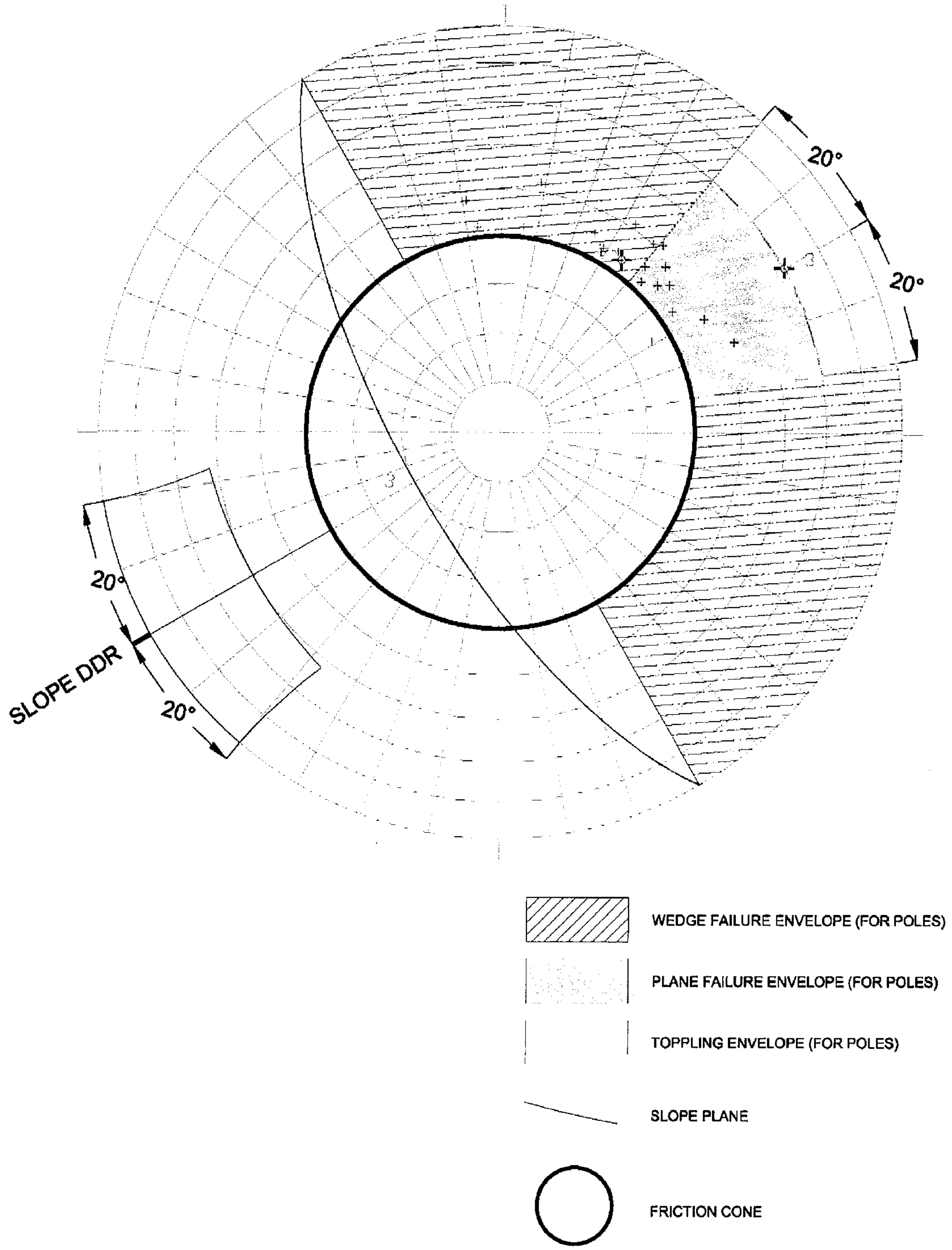
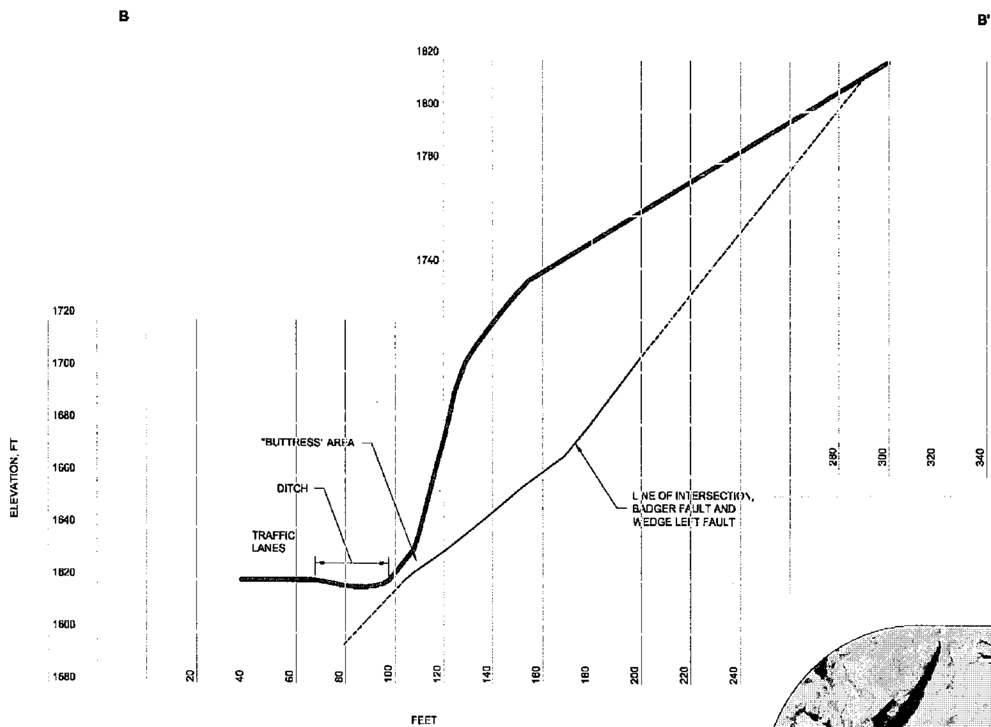


Figure 4 Greenstone Block Kinematic Analysis



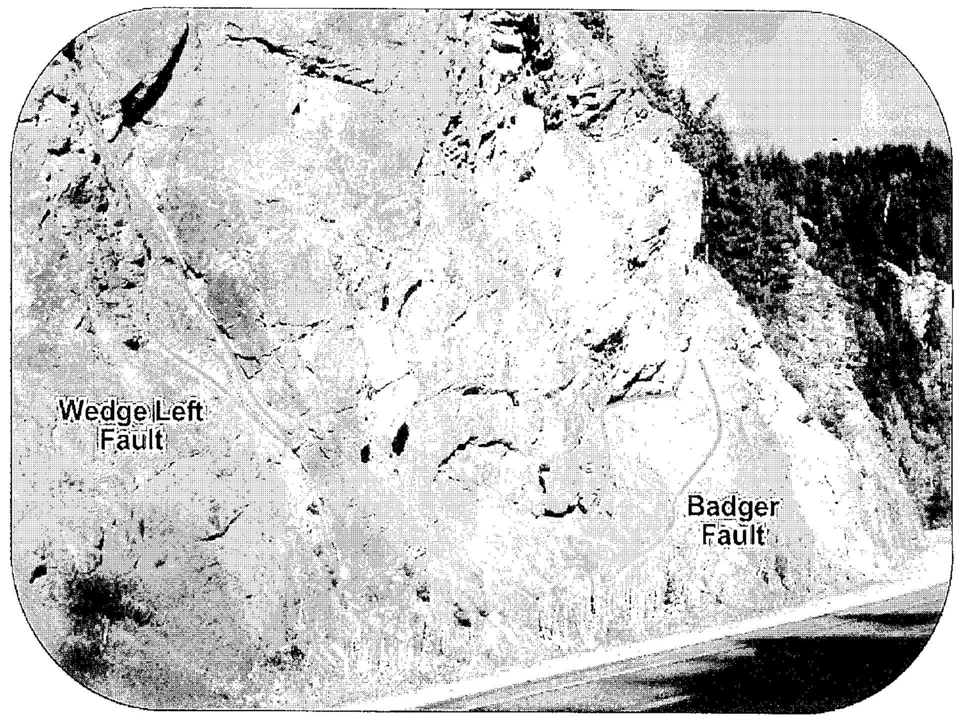


Figure 5 Badger Block Cross Section

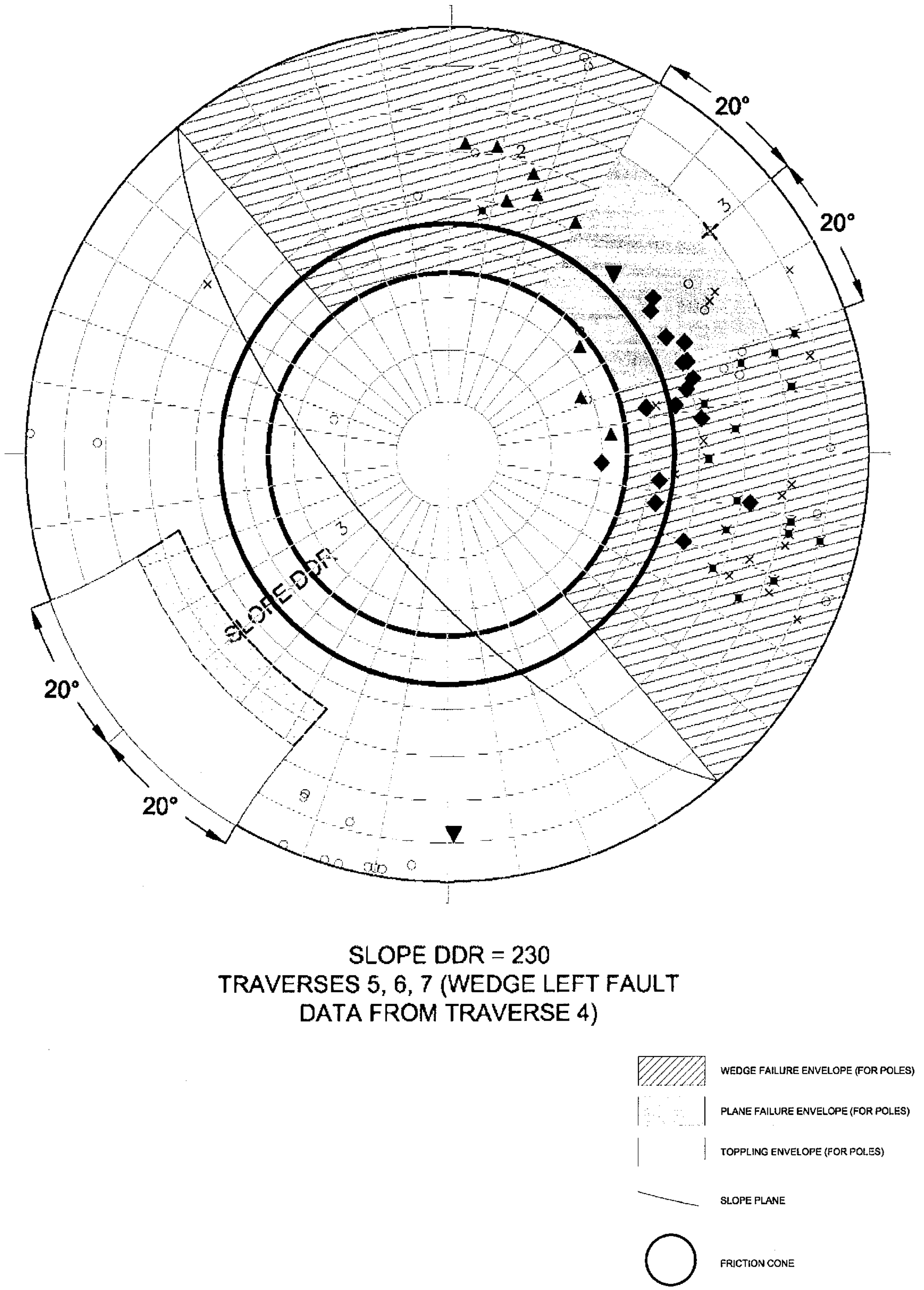


Figure 6 Badger Block Kinematic Analysis

The rock fall hazard for the overall slope was mitigated with a combination of cable net slope protection, untensioned steel bars (dowels), and tensioned low capacity steel bars (rock bolts). The potential failure mechanisms for the Greenstone Block and the Badger Block required additional design consideration.

Limit Equilibrium Analyses

It was the consensus opinion of rock slope experts that examined the cut at the start of the project that the possibility of a large-scale failure involving the Greenstone Block should be mitigated by removing the potential failure mass above the base plane by blasting. This was considered practical in part because of the observed continuity of the 43° dipping base plane, and the fact that the potential failure mass was clearly detached from the rest of the cut. Thus, it was considered likely that removal of the Greenstone Block would not have a significant impact on the stability of the rest of the cut, and that the limits of the material to be removed could be predicted with sufficient confidence.

Because of the steep natural slope above the cut, uncertainties with respect to the limits of the wedge behind the cut and possible interaction with the rock mass in the Middle Zone, stabilization by reinforcement was considered to be the preferred approach to mitigating the failure potential of the Badger Block wedge. With the Greenstone Block addressed at least conceptually, and hazards in other domains that were minor by comparison, more detailed analyses focused on the stability of the Badger Block wedge, and the design of reinforcement measures for this wedge.

Rigid Block Stability Analysis of Badger Block

The computer program SWEDGE (Rocscience, 2000) was used to model wedge stability. This program uses the methods described by Wyllie and Norrish (1996) and treats the wedge as a three-dimensional, rigid block, bounded by five discrete planes, each with a defined orientation. Three of these planes are discontinuities in the rock, namely the two structures that form the wedge, and a tension crack at the back of the wedge. The other two planes are the slope face, and the slope behind the crest. Shear strength parameters are assigned to the two planes that form the wedge. The orientation of the line-of-intersecton (LOI) of these two wedge-forming planes is critical to the viability of the wedge failure.

The wedge solution includes the assumption that the wedge daylights in the slope. As shown in Figure 5, the Badger Block wedge does not actually daylight in the slope, but dips under the road; as such, a "buttress" of slate exists between the LOI and the face. The influence of this buttress on stability of the wedge was incorporated into the stability model by applying an external, horizontal force on the wedge, in the direction 050°, i.e. opposite of the trend of the LOI. The magnitude of this force was estimated at 320 tons, based on a characterization of rock mass shear strength of the buttress (Hoek and Brown, 1988; Hoek and Karzulovic, 2000). This characterization was in the form of a shear strength envelope, which was applied over a buttress area at road level of 50 square feet.

Compared to the 25,500-ton total weight of the wedge, the intact rock buttress constituted a relatively small stabilizing force.

The Badger Block wedge was modeled by the planes listed below, along with a slope height of 100 feet:

Plane	Dip, °	Dip Direction, °	c, psf	ϕ, °
Badger Fault	45	250	Varied	34
Wedge Left Fault	52	193	250	38
Slope Face	70	230	N/A	N/A
Upper Slope	32	230	N/A	N/A
Tension Crack	70	230	N/A	N/A

Stability analyses were completed for the following purposes:

- Develop a "design" value for cohesion along the Badger Fault, given a ϕ of 34° from laboratory direct shear testing, and shear strength along the Wedge Left Fault defined by a nominal cohesion of 250 psf and a friction angle of 38° characteristic of a rough surface in competent rock, without a significant thickness of infilling; and
- Determine support force requirements for the Badger Block wedge to provide adequate Factor of Safety (FOS) under design seismic loading and a conservative groundwater assumption.

The analyses that concerned the design cohesion value for the Badger Fault were essentially "back-analyses" of the existing, stable wedge. While there are uncertainties in all parameters for which "design" values are given in the above table, it can be argued that the most significant uncertainties in this problem revolve around the Factor of Safety (FOS), historic groundwater conditions, and historic earthquake loading. A series of sensitivity analyses were completed, in which the FOS was determined for various values of cohesion along the Badger Fault, groundwater conditions, and pseudostatic loads. Ultimately, engineering judgment was involved in selecting a design cohesion value for the Badger Fault from these analyses. A value of 1000 psf was selected, based on a past "worst case" groundwater condition of the wedge forming structures 50% filled with water, and a FOS for this condition on the order of 1.1.

Given this cohesion value, and other design values incorporated into the wedge model, analyses focused on support force requirements for the wedge that would result in a FOS of 1.25 for a groundwater condition represented by the wedge forming structures 50% filled with water; and a FOS of 1.1 under seismic loading represented by a pseudostatic coefficient of 0.12, but with drained groundwater conditions. The analyses indicated that a total stabilizing force on the wedge of about 2,500 tons was necessary to meet the FOS requirements.

This force was divided into components of 1,400 tons to be provided by a line of shear pins installed along the trace of the Badger Fault; and 1,100 tons to be provided by 15 high capacity anchors. This combination was selected because of the ease of installation of shear pins along the exposed Badger Fault plane near road level and to limit the length of the anchors by locating them near the LOI of the faults at the toe of the slope (see Figures 5 and 7). The shear pins consisted of Grade 75, #20 (2 ½ inch diameter) deformed steel bars as shown in Figure 8. The high capacity anchors were specified as Grade 75, #14 (1 ¾ inch diameter) deformed steel bars with a design load of 147 kips. Required bolt lengths ranged from 35 to 55 feet. All reinforcing steel was specified as Class 1 for corrosion resistance in accordance with PTI, 1996.

Greenstone Blast Design

The blast design incorporated multiple project constraints including:

- Blasting only permitted during the month of August to be in compliance with wildlife and fishery concerns.
- Safety of the traveling public.
- Safety of the construction personnel.
- Minimization of the time of highway closure.
- Elimination / minimization of rock entering Peshastin Creek.
- Limited access by overland to the top of the Greenstone Block.

Due to these constraints, WSDOT retained an independent blasting consultant to prepare alternative blast plans for the Greenstone Block and to review the plan submitted by the contractor. The primary issue was that of safety which related to the staging of the blasts. Two approaches were contemplated; multiple small blasts versus a single large blast. Concerns for the progressive destabilization or perhaps an unanticipated premature failure of the Greenstone Block led to the adoption of the single blast approach. The next issue became the drilling approach to best distribute the explosives in the block given the access limitations presented by the site. Consequently four plans were developed for the consideration of the contractor (Figure 9):

Alternative:

1. Angled down holes drilled from the crest parallel to the long axis of the block
2. Angled up holes drilled from the toe parallel to the long axis of the block
3. Vertical holes drilled from a crane
4. Angled holes normal to the base plane drilled from a crane

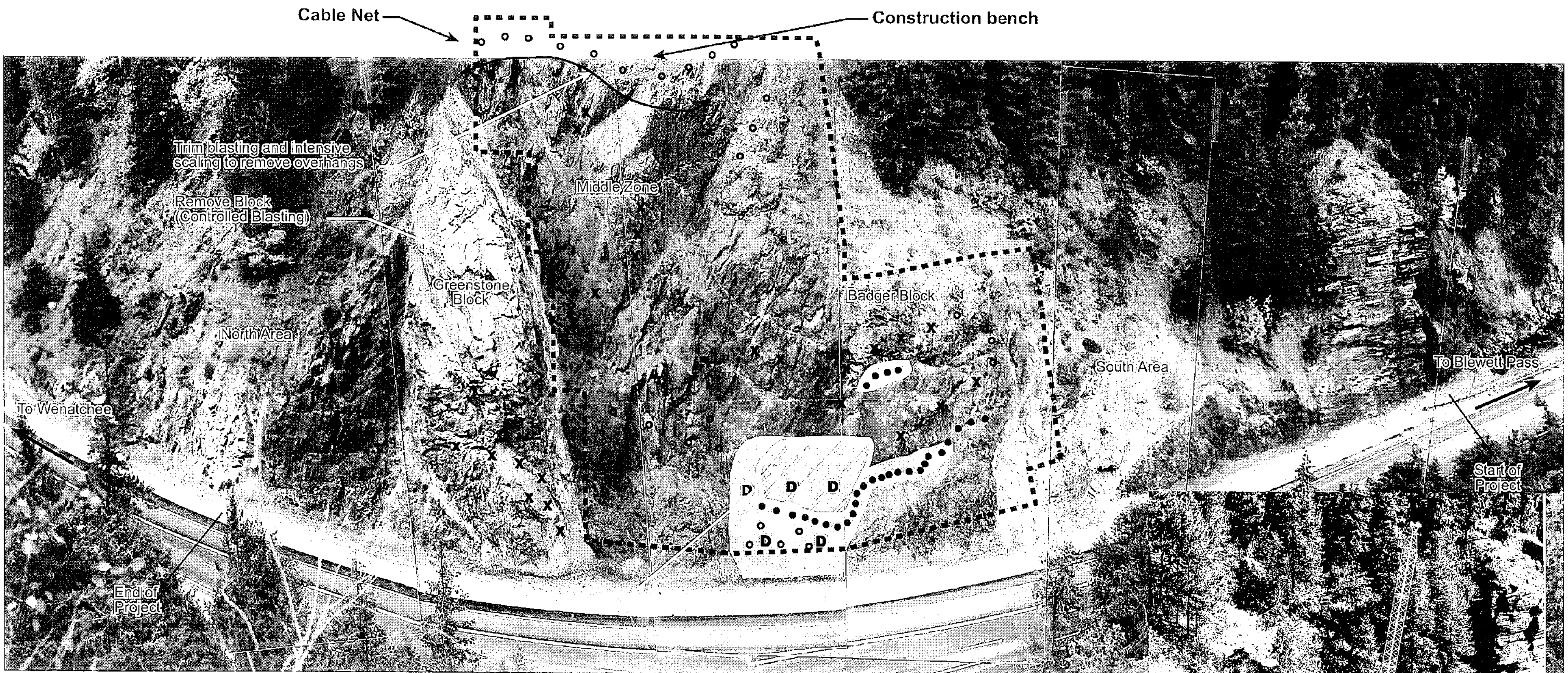


Figure 7 Rock Slope Mitigation

○ Rock dowel
● Shear pin
x Low capacity rock bolt
D

Inset Photo – 250 ton crane, 300-foot boom

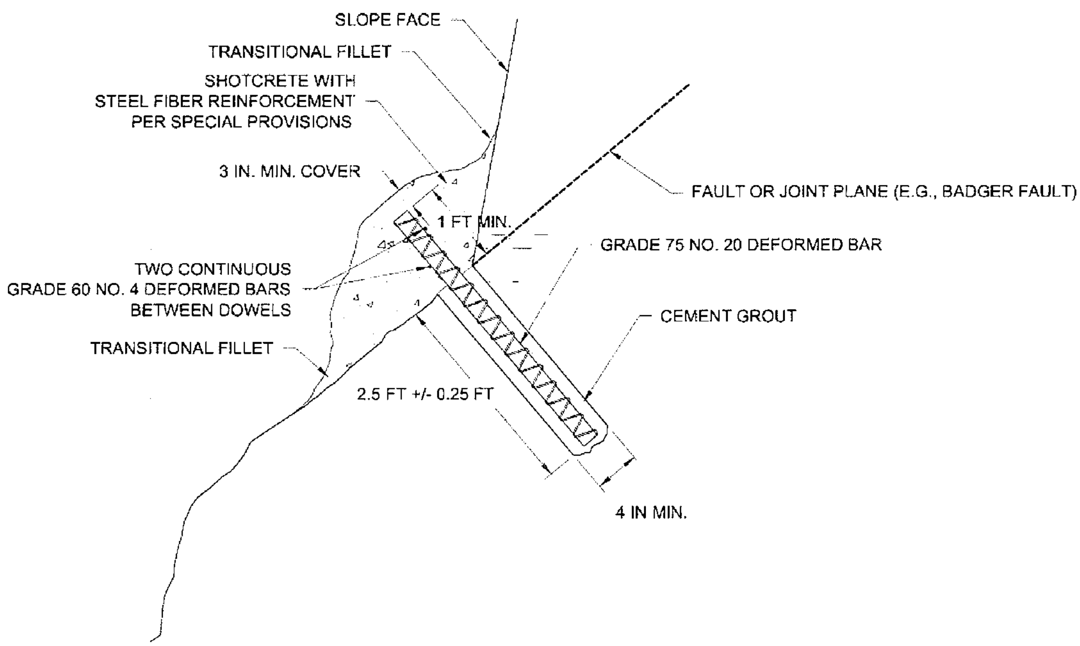


Figure 8 Shear Pin Detail

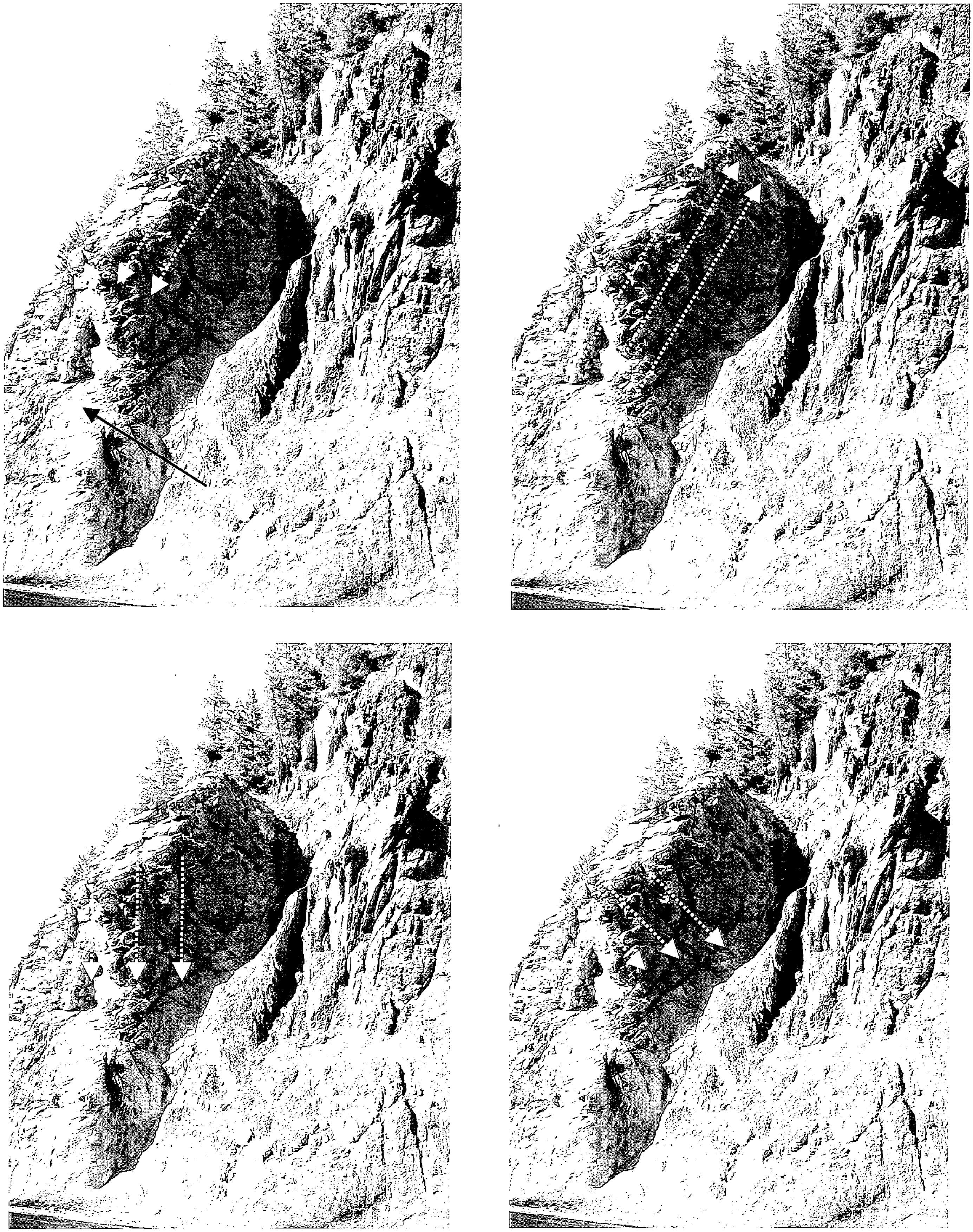


Figure 9 Pre Construction Drilling Alternatives for GreenStone Block Blast Holes
(*Conceptual only – actual plans included multiple rows and holes/row*

All the plans contemplated a heavier than normal explosive loading to minimize rock blocks "hanging up" on the slope. For the first two alternatives it was proposed to use a delay pattern that would cause the broken rock to move toward the trough on the south boundary of the Greenstone block and thereby minimize rock throw toward the creek. The preferred alternative at this stage was to drill angled holes from the toe.

CONSTRUCTION PHASE

Bidding Process

Considerable discussion took place on the bidding approach for the drilling and blasting of the 5000 cy Greenstone Block. The conventional approach would be on a unit volume basis with an incentive/penalty clause to encourage timely execution and cleanup of the blast. The less conventional approach that was adopted was to perform this phase of the work under force account. The rationale was that there were too many external constraints to hold the contractor accountable to a schedule. Consequently the owner took responsibility to direct all aspects of the drilling, blasting and muck removal operations. In the final analysis, this arrangement worked out very well as one lane of traffic was open within two hours and two lanes within four hours.

Unit Rates

The 2001 unit rates (material and installation) for the major stabilization elements were as follows:

High capacity bar anchors (147 kip):	$75 ft
Low capacity bar anchors (53 kip):	$75 ft
Untensioned dowels (Grade 75, #8 bars):	$60 ft
Shear pins (Grade 75, #20 bars):	$60 ft
Shotcrete (4000 psi, steel fiber reinforced):	$1,000 cy
Cable net (8in x 8in):	$9.00 sf
Scaling:	$300 crew hr
Horizontal drains:	$25 ft

The total project cost based on actual quantities was approximately $1.34 million.

Blasting

Of the four conceptual approaches to drilling the Greenstone Block, the contractor proposed a modified version of Alternative 1 in which angled holes were drilled in a radial pattern from the top of the block. The intent of the radial pattern was to lift the rock mass normal to the base plane and to shear the rock between the blast holes. Due to concerns for misfires, the contractor elected not to incorporate delays between adjacent blast holes in an attempt to displace the broken rock laterally.

The access issue for drilling was overcome by using hand-held sinker pneumatic drills to excavate a temporary 14 x 18-foot drilling bench at the top of the Greenstone Block. Due to excessive overbreak in the highly fractured and weathered slate, a concrete pad was required to complete the drilling bench. From this vantage, nine, 5-inch diameter blast holes ranging in length from 47 to 161 feet in length were drilled in a radial fan pattern using a down hole hammer drill. Upon completion of the first hole a deviation survey was performed using a tilt/temperature compensated compass magnetometer manufactured by Advanced Orientation Systems. This instrument detected a deviation of nine feet horizontally and 4 feet vertically between the actual and planned hole bottoms. In part this deviation was thought to result from the contractor's drilling practices including inaccurate collar setup and excessive drilling speed to meet the schedule dictated by the fish and wildlife window. With changes to these practices modest improvement to the alignment of three subsequent holes was measured, typically five to seven feet from planned bottom. Given the irregularity of the Greenstone Block surface, these alignment deviations meant that the burden distance on the blast holes was less uniform than planned.

The contractor's functional blast design called for a powder factor of 0.90 +/- 0.05 pounds of explosive per cubic yard of rock. Based on the hole deviation surveys and the newly acquired block topography, the loading was adjusted to 4765 pounds for the 5500 cubic yard block (*volume revised during construction*) for a powder factor of 0.87 lb/cy. ANFO was selected as the explosive with the loading and stemming varied in accordance with the burden on each hole. This was particularly important near the convergence of the radial pattern at the drilling bench. Both open hole and bagged ANFO was employed dependent on burden distances.

As shown in the photographs in the abstract and in Figures 10 and 11, the blast was a technical success with the displaced rock largely coming to rest as a muck pile along the toe of the slope. Hand scaling and local rock bolting was adequate to dress the basal plane of the Greenstone Block. Due to the non-uniform explosive distribution resulting from the radial hole pattern adopted in the drilling, large blocks were present in the muck pile (Figure 10). Fortunately these blocks could be moved to temporary storage locations in the ditch for subsequent secondary blasting. The highway was open to single-lane traffic within about two hours of the blast and normal traffic resumed within four hours. This schedule was better than the most optimistic of estimates prior to the blast.

Regrettably a small amount of rock entered Peshastin Creek and had to be removed to restore the habitat. Given the overall project constraints, particularly the safety issues related to the blasting, this was probably an unavoidable incident.

Slope Mitigation

The mitigation elements included hand scaling to remove loose rock, reinforcement with bolts, dowels, shear pins and shotcrete, and rock fall protection with cable nets. All these elements were implemented largely as planned but with the inevitable adjustments

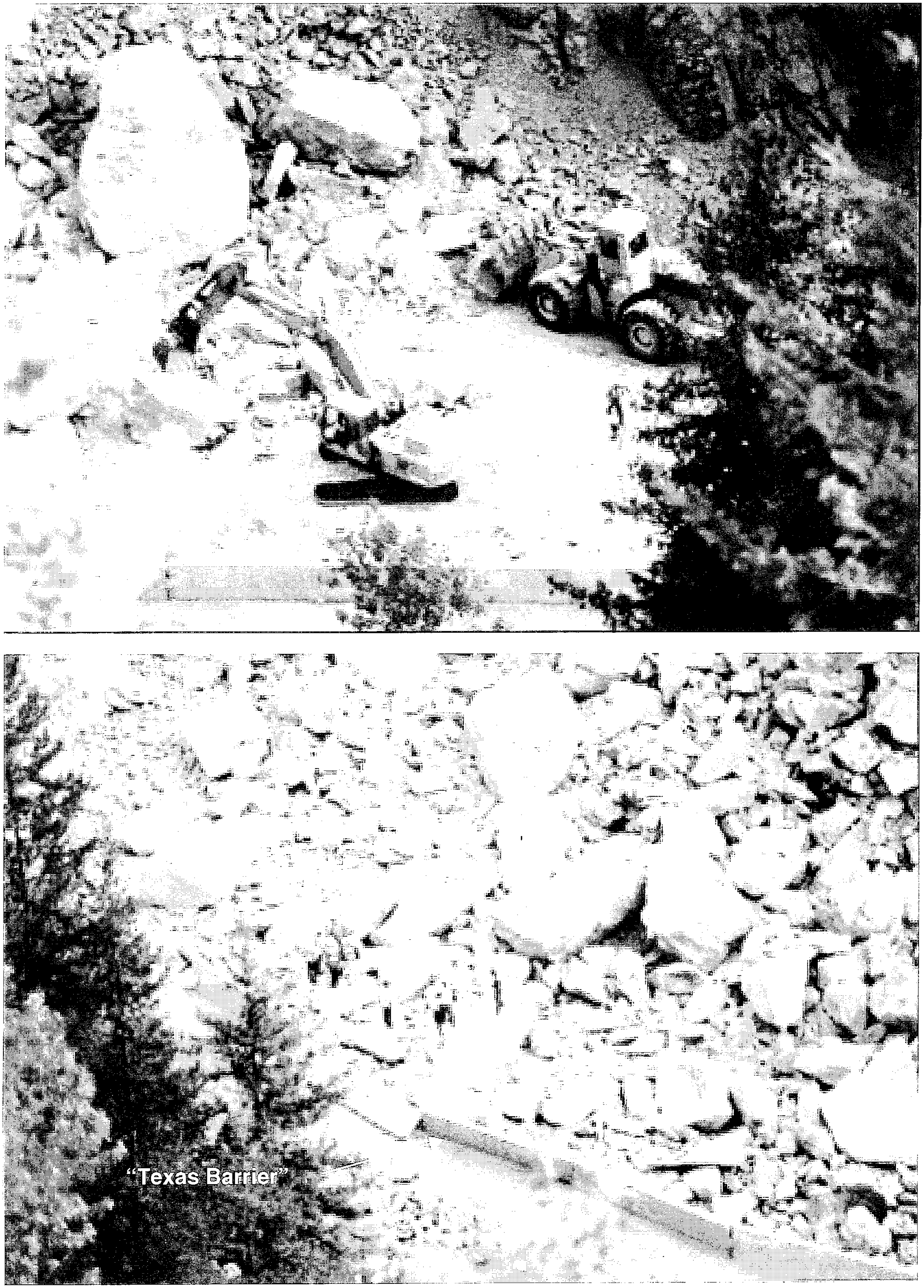


Figure 10 Greenstone Block Post Blast

required during construction. Due to the heavily fractured nature of the slate, the scaling time was greater than planned. On the upper north slope above the Greenstone Block, scaling proved to be impractical and the cable net was extended to the north to protect this area. Shotcrete proved to be an effective method to smooth out the irregular rock surface prior to the drilling and installation of high capacity rock anchors (Figure 11). A number of horizontal drains that were dry when installed in October, were making water when the project construction resumed in April. Figure 11 shows the project site at completion.

HINDSIGHT PHASE

A side benefit of a challenging slope stabilization project is the opportunity to summarize a list of ***lessons learned*** that hopefully will be remembered and applied to the next project. The important lessons from the Ruby Creek project are:

- The quantity of cable net was underestimated due to the roughness of the slope. ***Lesson*** *– For rough slopes the slope distances should be increased by up to 20 percent in each dimension to calculate the area coverage of the cable net.*
- During the blast design phase, much consideration was given to the feasibility of various plans under the constraints of difficult access. In the final analysis, the successful contractor used an access and drilling method not contemplated in the design phase. Although consideration of constructability is necessary during design, an undue emphasis can bias the engineering aspects and objectives. ***Lesson*** *- Let the contractors bring their expertise and innovation to the project in the bidding process.*
- While trying to estimate the highway closure time during the design phase, the issue of the muck pile profile and swell factor were key unknowns. ***Lesson -*** *After the blast the muck pile exhibited an angle of about 25° initiating from the toe of the base plane.*
- For the Greenstone blast, flyrock travel distance was documented at 670 feet horizontal at a location 160 feet above the center of the blast. This obviously has implications for safe viewing vantage points. ***Lesson*** *- Never let bravado mask common sense when estimating the distance that flyrock might travel from a blast.*
- Because WSDOT directed the blasting and cleanup effort, the selection of appropriate equipment was a key issue. ***Lesson –*** *With the large block sizes present in the muck pile, track-mounted excavators proved much more effective than rubber-tired loaders to move the blocks to temporary storage locations beyond the traveled roadway.*
- All the high slope rock drilling for dowels and bolts was staged from a crane. Due to problems with approval of submittals, the drilling advanced through tens of holes without the completion of any grouting or tensioning. This led to a rock fall that struck the skip with sufficient force to move the 250-ton crane on its outriggers. ***Lesson*** *- Rock slope stabilization activities staged from a crane are*

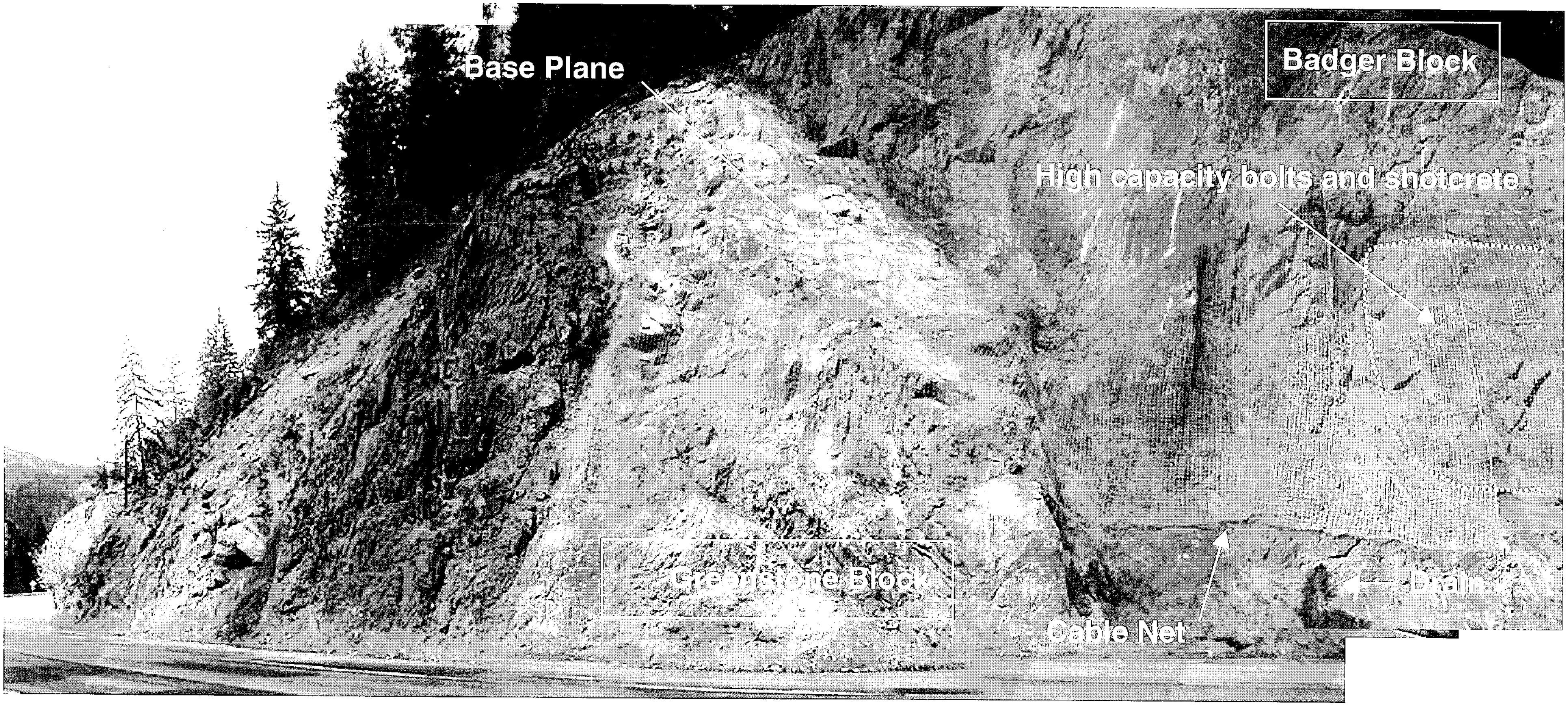


Figure 11 Ruby Creek Project at Completion

particularly exposed to rock fall and therefore scaling, dowels, bolts and shotcrete should be progressively installed working from the top down.

- The bid units included a grout loss item to equitably share the risk between the owner and the contractor for an unquantifiable item. Payment for excessive grout loss (in excess of 200 percent of hole volume) was on a force account basis. This resulted in the crane costs, equipment and personnel being included in the payment for excessive grout. At the bidding stage contractors will have various equipment spreads for the project that can result in very different costs being applied to an item such as grout loss under force account. ***Lesson*** *– A better approach is to have a bid item for the excess grout loss on a volumetric basis so that the owner is protected from unreasonable cost allocations.*

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The Stabilization of Red Top Landslide at American Canyon Solano County, California

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ABSTRACT

During 1995/96 a massive ancient landslide activated initiating the slow downslope displacement of a 4,000-foot long segment of Interstate 80, which links the San Francisco Bay Area with the City of Sacramento. Located within the Coast Ranges of central California, the landslide is within a larger inactive ancient landslide. Large landslides are on both sides of American Canyon, leaving no realignment alternative. The activated landslide is 1,341 m wide, 793 m long, up to 70 m deep and encompasses 200 acres of gentle to moderately sloping terrain. The slide is primarily in the sandstones, shales and mudstones of the Eocene age Markley Formation. Through a subsurface exploration program, it was determined that high artesian pressure existed at the slide plane, that artesian pressure increased in proportion to rainfall, and that increased artesian pressure caused landslide movement.

During late 1997, heavy El Niño rains were forecast and, in response, ten pumping wells were constructed across the slide, within the freeway right-of-way, in an attempt to control slide movement. In addition, monitoring wells with multiple piezometers were installed. Analysis of slide movement rate, piezometric head, well pump rates and rainfall data indicate that the existing pumping wells can control slide movement only when rainfall is normal or less. Rather than expanding the well field, the strategy is to excavate a 93.5 m, 6.5 m diameter vertical shaft with three levels of radiating horizontal drains to reduce the piezometric head driving the landslide. Construction of the first of two shafts began in November 2001.

INTRODUCTION

The Red Top Landslide at American Canyon is located in Solano County, California, about 3.75 km (2.3 miles) west of the Route 80/680 interchange (Cordelia Junction). Route 80, an 8-lane split profile freeway, traverses the width of the landslide in an east/west direction for a distance of 1.3 km (0.8 mile).

The active portion of the landslide is 1,341 m (4,400 feet) wide, up to 70 m (230 feet) deep, ranges between 152 m and 793 m (500 and 2,600 feet) in length and extends from the head scarp on the north, 1,300 feet to the toe of the landslide at American Canyon Creek. The landslide mass consists of two contiguous slide masses: a relatively small slide on the west [(427 m by 152 m) 1,400 feet wide by 500 feet long] that has been causing the westbound lanes of Route 80 to slowly and intermittently subside ever since the freeway was constructed in 1969. The second is a much larger easterly slide [(915 m by 793 m) 3,000 feet wide by 2,600 feet long] that reactivated in 1994. This report will deal only with stabilizing the larger easterly slide.

The large easterly landslide began to reactivate in late 1994, and by mid-1996. It was determined that the approximately 200-acre active landslide is within a much larger ancient landslide that extends nearly to the top of the mountain to the north and American Canyon Creek to the south.

Topographically, the surface of the reactivated easterly slide has a fairly gentle slope with an elevation difference between the head scarp and toe of 152 m (500 feet) over a distance of 823 m (2,700 feet). Most of the terrain slopes at approximately 8 degrees, except near the head scarp of the slide where it is steeper, and at the toe where American Canyon Creek has carved a steep-sided gully 12 m (40 feet) deep into the terrain.

The slope on the opposite or south side of American Canyon Creek is also underlain by a large landslide complex, which does not impact the highway facilities. Field evidence and geologic mapping suggest that the two opposing landslide masses likely converged during the geologic past to pinch off American Canyon Creek, followed by the creek cutting a gully down through the slide. It is possible that this process has occurred a number of times in the past and could occur again in the future.

A head scarp up to 1.5 m (5 feet) in height is present along the northeasterly two-thirds of the slide, while the eastern edge of the slide has up to 1.5 m (5 feet) of left-lateral horizontal displacement with associated pressure ridges. The western edge of the eastern slide shows no evidence of surface rupture, but there are areas of settlement that generally indicate its limits. The toe area of the slide is not well defined, but is generally located along American Canyon Creek.

Translational movement of this glacier-size landslide mass has displaced the highway to the south, towards American Canyon Creek up to 1.5 m (5 feet), disrupting and damaging the pavement, median barrier and drainage facilities. Much of this movement occurred between February and July 1998 during the 1997/98 El Niño year, which had 1,041 mm (41 inches) of rainfall, nearly twice the normal rainfall for this area of California. The slide is continuing to creep at a very slow rate.

INITIAL STABILIZATION MEASURES

Ongoing geotechnical studies indicated that the primary force driving the slide is an unusually high artesian water table. Because El Niño rains were forecast for 1997/98, geotechnical studies were accelerated and an emergency Director's Order was initiated during late spring of 1997. Eight large diameter, 610 mm (24-inch), pumping wells ranging from 43 m to 61 m (140 to 200 feet) in depth were constructed on a slide within State right-of-way. The purpose of the wells was to lower the piezometeric head along the slide plane and thereby slow the movement of the slide. Pumped wells were selected because gravity drainage through horizontal drains is not possible.

The performance of the pumping wells was monitored by an array of 109 piezometers installed at various levels in 37 monitoring wells, plus survey monitoring of the ground surface. All eight wells were in operation by February 1998. The total pumping rate for the 8 wells averaged 265 m^3 (70,000 gallons) per day with surges up to 484 m^3 (128,000 gallons) per day.

Considerable slide movement occurred between mid-February and July 1998. During March 1998 two of the wells located in the most easterly, active portion of the slide were sheared by movement along the slide plane. It was later concluded that the slide movement and resultant pavement distress would have been even greater if the wells had not been in place.

During the summer and fall of 1998, four new wells were constructed within State right-of-way. Two of the wells replaced the wells that had sheared off during the prior winter, and two new 91 m (300-foot) deep wells were installed in the deepest portion of the slide mass, bringing the total number of operating wells to 10.

Rainfall for 1998/99 was 533 mm (21 inches), which is slightly above the normal 508 mm (20 inches). The 10 wells averaged a total pumping rate of 430 m^3 (30,000 gallons) per day, which is less than half of the previous El Niño year's average pumping rate. The ground surface surveys indicate that the slide mass did not move during the 1998/99 year [limit of accuracy is approximately 0.009 m (0.03 foot)]. Visual observations revealed no movement of the slide, and the piezometric head in the monitoring wells remained low compared to 1997/98.

Using pumping wells to lower the piezometric head along the slide plane appears to be a qualified success. The east slide is now considered stable during years of normal rainfall.

PHYSICAL SETTING

The Pacific Ocean to the west dominates the climate of the region. Average temperatures range from 47 degrees Fahrenheit in the winter to 74 degrees Fahrenheit in the summer (Solano Soil Survey, 1977). These small temperature changes are typical of the region. Climatic conditions are predominantly Mediterranean with warm to hot, dry summers and cool, relatively wet winters.

Rainfall for this portion of Solano County ranges from 356 mm to 1,016 mm (14 to 40 inches) (Solano Soil Survey, 1977). Local rainfall, in the area of the project, is approximately 508 mm to 635 mm (20 to 25 inches). About 80 percent of the annual rainfall occurs between November and April (Solano Soil Survey, 1977). Winds are locally controlled by the northeast/southwest trending mountains and valleys around the project location, but are regionally from the north in the winter months and from the west during the summer months (Solano Soil Survey, 1977).

American Canyon is a northeast-southwest trending valley that is likely fault controlled. The project is located at the eastern end of American Canyon on the northern flank of the canyon. The American Canyon drainage basin rises in elevation from near sea level at Suisun Bay on the east to an elevation of near 152 m (500 feet) above sea level at the ridge of the Sulfur Springs Mountains on the west (Cordelia Quadrangle, 1980).

The main drainage of American Canyon is the American Canyon Creek. Water collected in the upper watershed combines from a number of small tributaries to form American Canyon Creek. The water flows south until it crosses under Route 80 in the vicinity of Lynch Road Undercrossing, where it turns east and subparallels Route 80. Then American Canyon Creek exits the mouth of American Canyon and continues east until it drains into Suisun Slough and eventually into Suisun Bay.

GEOLOGIC CONDITIONS

Regional Geology

The area is within the northeastern Diablo Range, part of the Coast Range Geomorphic Province of Central California. The province is characterized by a series of northwesterly trending ridges, faults, and intermountain valleys formed by compressional tectonic forces. The site is located 1.6 km (1 mile) west of the boundary between the Great Valley Geomorphic Province and the Coast Range Geomorphic Province.

The project area is underlain by sedimentary rocks and landslide deposits composed of members of the Eocene Kreyenhagen Formation (Manson, 1988). These include the Markley sandstone at the surface and, at depth, the Nortonville shale and Domengine sandstone (Bezore, Wagner, and Sowers, 1998). The project location is mapped as lying between two regional folds, an anticlinal structure to the east and a synclinal structure to the west. The active Green Valley fault trends north-northwest less than 1.6 km (1 mile) east of the project site. Also, there is an inactive fault structure that is mapped parallel to the trend of American Canyon and then turns north near the Lynch Road Undercrossing (Bezore, Wagner, and Sowers, 1998). The relative movement on this fault appears to be up on the southern block in relation to the northern block. This fault zone is obscured at the surface throughout the project area due to it being covered by active landsliding.

Site Geology

Principal rocks of the slide area are members of the Eocene Kreyenhagen Formation (Manson, 1988). The dominant member exposed within the project limits is the Markley sandstone. It is a massive, grayish to yellowish brown, medium to coarse grained, micaceous, feldspathic sandstone. It is locally cross-bedded and pebbly, indurated but not usually cemented. The Nortonville Shale member underlies the Markley sandstone. The shale is indurated also and is characterized as being dark chocolate brown in color, nonmicacous, and is often calcareous. Some calcareous sand (calcarenite) lenses exist within the unit and are seen as marker beds. Where exposed, the Nortonville weathers to a dark brown soil. The Nortonville crops out in a topographically low area adjacent to the landslide area east and west of the project location.

The Markley Formation is identified as a natural gas-bearing unit (California Oil and Gas, 1982). Based on this, it is anticipated that the site is classified as **gassy** for both construction and maintenance operations.

Seismicity

Eight major active faults are located within 80 km (50 miles) of the project site (Brown, 1970). The faults all trend in a northwesterly direction and are designated as Earthquake Studies Zones. These are the Green Valley, Cordelia, West Napa, Concord, Rodgers Creek, Hayward, Calaveras, and the San Andreas fault zones. It is expected during the design life of this project that the site will experience ground movement due to a seismic event on one of the following fault zones with an estimated Maximum Peak Bedrock Acceleration of

0.59 g. The expected effect of an earthquake of this magnitude on the well system would be to activate the landslide and damage or destroy the wells.

EXPLORATION

From 1996 to date, 79 exploratory borings have been drilled throughout the project area. A majority of the holes were drilled using 8-inch hollow stem augers with a dry core sampling system. Once the holes were completed, either groundwater or slope monitoring instrumentation was installed into the boreholes.

Two pilot borings (P-78 and P-79) were drilled at the center of the proposed shaft location from August 24 to October 5, 2000 using a Mobile B-61 drill rig. P-78 was drilled using a conventional NX core system (47.6-mm diameter), and soil and rock samples were continuously collected. The boring was completed to a depth of 96 m (315 feet). Several intervals were enlarged by rotary drilling to accommodate institute pressure meter testing. From those intervals, 15-17 m and 21-23 m (50-57 and 70-75 feet), no core was recovered. Overall core recovery was very good, averaging 89%. P-79 was located 2.2 m south of P-78 and was completed to a depth of 24.5 m (80 feet). The purpose of P-79 was to get additional site-specific soils information and to perform additional Standard Penetration Test to develop N values for the soil not well characterized in P-78. The Log of Test Borings for these two borings is located in Appendix A.

Subsurface Soil and Rock Conditions

Soil and core samples recovered from the borehole indicate that there are three major lithologic breaks. The first unit is an engineered fill placed during the original construction of Route 80 and is approximately 5 m (15 feet) thick. From 5 m to 21 m (15 feet to 68 feet), the material consists of lean clayey silty sand soil unit. At 21 m (68 feet), there is a gradational transition from soil to rock material. The rock unit is identified as the Markley Member of the Eocene Kreyenhagen Formation as described above.

Geophysical Studies

Caltrans Geophysical unit conducted a detailed down-hole geophysical survey of the pilot hole once it was completed. Different geophysical tools used in the survey consisted of caliper, resistivity, natural gamma, spontaneous potential, induction, sonic, shear-wave velocity, and acoustic televiewer. Results of the survey indicated that no major lithologic changes occurred once below the soil unit, and that bedding and fracture orientations could be identified and changes in their attitudes at given intervals were probably related to slide activity. These results compare very favorably with other information and were critical in the development of the shaft design.

A large-scale surface streaming potential geophysical survey of the slide surface was conducted by Norcal Geophysics, Inc., of Petaluma, California, for the purpose of determining where groundwater may be located below the surface and its rate and direction of flow. The survey consisted of establishing a grid over the entire project area and taking surface spontaneous potential readings at each point within the grid. After analyzing the data and conducting field testing to verify results, it was determined that the study was inconclusive.

In addition to the surface geophysics, Norcal also conducted down-hole geophysical surveys of many of the exploratory borings for the purpose of stratagraphic correlation. The geophysical tools used in their surveys included caliper, resistivity, spontaneous potential and natural gamma. The results indicated that correlation between two adjacent holes could be made, but not throughout the entire project area. It appears that, due to the extensive episodic slide movement over long periods of time, there are entire sections of rock that are missing through the center of the slide.

Instrumentation

There are a total of 62 exploratory borings drilled to date throughout the project area. In these 62 borings, 117 multi-level piezometers and 23 inclinometers have been installed. Of the 23 inclinometers, 11 are still active and 12 are inactive. In addition to the monitoring installations, 13 dewatering wells were drilled and constructed. Of the 13 wells, 10 are actively pumping, two have been lost due to slide movement and one was drilled for testing purposes only.

In addition, there are 25 surface monuments throughout the project area that are surveyed periodically to detect surface movement.

Rock Mass Classification

Several rock mass classifications were used to build a picture of the composition and characteristics of the rock mass to provide initial estimates of support requirements and to provide estimates of the strength and deformation properties of the rock mass. The rock mass is divided into a number of structural regions and each region is classified separately. The structural regions were identified by changes in rock type or structural features (bedding plane and joint attitudes) noted in the drill core and/or the geophysical logs. It is recommended that at least two methods be used at any site during the early stages of the project (Hoek, 2000), which was done on this project.

The Rock Mass Rating (RMR) system was one of the classification used for this project and was developed by Bieniawski in 1976. The classification system incorporates the following six parameters to classify the rock mass: uniaxial compressive strength, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, orientation of discontinuities, and groundwater conditions.

The Rock Tunnel Quality Index (Q) was the second classification used for this project and was developed by Barton in 1974. This classification system incorporates the following six parameters to classify a rock mass: RQD, jointing, joint roughness, joint alteration, joint water, and a stress reduction factor.

Based on our field investigation, down-hole geophysical survey, and rock mass classification, it has been determined that, in general, the in-place rock hardness ranged from very soft to moderately soft; and the rock is described as very poor to poor rock.

GROUNDWATER

Based on borings and monitoring wells, it was determined that hydrostatic groundwater levels were high and that a confined aquifer with an artesian head was present at a depth in the general vicinity of the slide plane throughout the slide area. This high artesian head is believed to be the major force driving the landslide. Perched water tables have been encountered at various depths during the drilling operation and some appear to be isolated and not interconnected.

Water Levels and Artesian Head

Frequently, an artesian head was encountered during drilling. Typically, the boring was relatively dry until reaching a moderate depth; then, over a short period of time, water would rise in the boring to a height of 15 m to 23 m (50 to 75 feet) or more. Later when conducting pump tests, the presence of an artesian system was confirmed. Some piezometers had almost no drawdown response during pump tests, while others at both deeper and shallower levels, especially those deeper zones closer to the slide plane, indicated tens of feet or more drawdown, which is indicative of confined aquifers.

Pumping Wells and Monitoring Wells

During the early spring of 1997, heavy El Niño rainfall was forecast for the coming year. The ongoing slide investigation was accelerated by initiating a Director's Order to install eight 610-mm (24-inch) diameter dewatering wells in an attempt to minimize the anticipated increased rate of slide movement. In addition to a large number of monitoring wells, additional inclinometers were also installed. A total of eight dewatering wells were installed (W-1 through W-8) within State R/W along the north side of the highway. All eight wells were on-line by late January 1998.

Pump Tests

Pump tests were conducted at each of the original eight wells (W-1 through W-8) to provide data to calculate the transmissivity and estimate the productivity of each well. Most of the pump tests were conducted for a period of 48 hours, although a 2.5-hour test was used on W-2, and a 7-hour and an 8-day test was utilized for well W-1. Water levels were monitored at each piezometer within proximity of the pumping well. Pump tests were not performed on replacement wells W-2R and W-3R nor were they performed on new wells W-9 and W-10, due to potential conflict with ongoing pumping operations.

The transmissivity of each well was determined from recovery tests. Transmissivity values ranged from a high of 23.5 m^2/day (354 ft^2/day) for well W-6 to a low of 1 m^2/day (11.0 ft^2/day) for well W-8, with most in the 2.3 to 3m^2/day (25 to 36 ft^2/day) range. Comparing transmissivity for each well with the pumping rate, it is noted that there is only a general correlation between transmissivity and pump rate.

During June 2000, another well (W-11) was drilled on the south side of the highway to investigate the feasibility of expanding a well field into that area. The location for this well was determined, in part, by the results of Streaming Potential – geophysical surveys (Kirker, 1999) that indicated that groundwater movement was more likely to be occurring in that area. The pump test on well W-11 indicated that the transmissibility of the formation was very low; and, therefore, it was not considered worthwhile to extend the power lines to develop a permanent well. Transmissivity for well W-11 was 0.5m^2/day (5 ft^2/day), which is the lowest of any of the 8 wells tested. Water levels in test well W-11 are being monitored manually.

Piezometer Data

Monitoring wells consist of vibrating wire piezometers installed at multi-levels within each boring. From two to five piezometers are installed within each monitoring well. Each piezometer is installed near the bottom of a sand zone that is confined and separated from other sand zones by bentonite plugs of various thicknesses. An attempt was made to place the piezometers in zones where sandstone or other potential aquifers are present and the bentonite plugs in areas of shale or mudstone. Boring logs and resistivity logs were used to select zones with potential aquifers, as discussed previously.

There are 39 monitoring wells at the Red Top Slide with a total of 117 piezometers that are monitored daily (plus others monitored by hand methods). The piezometer data is stored on a data logger and later collected and processed; the elevation of the piezometric head and the date are inserted into a spreadsheet program. The elevation of the piezometric head for each piezometer is then plotted graphically vs. time, and both the data and graph are stored on a CD. A file is available for each piezometer containing the numerical data plus a graphical plot of the piezometric head for the entire 3-year period of time since inception.

There is a wide range in the amount of movement in the hydrostatic head between each piezometer within a monitoring well. The piezometric head in some piezometers barely moved during the past 3 years, while in others it has lowered only a few meters, and in others has lowered many tens of meters. The deeper piezometers, located closer to the slide plane, generally show the most lowering of the hydrostatic head.

Special attention was given to analyzing one piezometer within each monitoring well that is believed to best represent the hydrostatic conditions present near the slide plane. Drawdown varied considerably from 0.3 to 0.6 m (1 to 2 feet) to as much as 34 m (112 feet) depth at a distance of approximately 50 m (164 feet) from the nearest pumping well. The average drawdown is approximately 15 m (49 feet) at 50 m (164 feet) distance.

The effect of pumping over time indicated that drawdown of the water table was occurring in the vicinity of the wells, all across the slide, but piezometric head levels still remain high to the northeast.

RAINFALL

Precipitation in the region is highly seasonal with approximately 90 percent of the annual rainfall occurring during the 6-month period November through April. Mean annual precipitation for the area is 508 mm (20 inches) in the vicinity of Red Top Slide (Rantz, 1971).

Cumulative rainfall at Fairfield, CA, for 1997/98 rainfall year (an El Niño event) was reported at 1,041 mm (41 inches), which was extremely high. The following year, 1998/99 was below average rainfall at 445 mm (17.5 inches), followed by a year of near average rainfall in 1999/2000. The 2000/01 year rainfall was below.

Looking at mean annual rainfall over a longer time period suggests that while rainfall is the dominant factor causing slide movement, it is probably not the only factor that initiated slide movement. Note that 1981/82 and 1982/83 both had higher rainfall amounts but did not initiate slide movement. Since this is an ancient slide that only recently reactivated, it is reasonable to conclude that continued stress and strain over a long period of time probably resulted in the shear strength along the slide plane lowering to its residual level. The combination of the shear strength along the slide plane approaching the low residual shear strength and the high piezometric head was evidently sufficient to initiate and promote continued sliding. It is possible that erosion at the toe of slide along American Canyon Creek may also have been a factor in reducing slide stability, though this is neither obvious nor certain.

Small amounts of localized movement still occur between February and April during years when rainfall is average or above average.

SLIDE MOVEMENT

Slope Inclinometer Data

A total of 21 slope inclinometers were installed on or in the proximity of the east slide between September 1996 and December 1999.

The slide plane surface defined by the slope inclinometer data is that of a deep channel leading southeast toward American Canyon Creek, with gentle to moderate side slopes and a steep head scarp. The elevation of the slide plane is substantially lower than that of American Canyon Creek over much of the slide. The maximum detected depth of the slide plane is 55.47 m at SI-27, located in the lower easterly portion of the slide. This slide plane depth corresponds to an elevation of 26.82 m above sea level. Other borings encountered the slide plane at up to 75 m depth due to topographic conditions at the slide.

Slide Movement Analysis

All of the available slide monitoring data, plus observations, were used to reconstruct the estimated rate of slide movement beginning February 1995, when the slide is believed to have activated, to the present time. Ten wells have pumped for a period of 3+ years. It seems clear from the reconstructed rate of slide movement since slide activation in 1995 that the combination of groundwater pumping and average or less-than-average rainfall have drastically reduced the rate of slide movement since all 10 wells were placed on-line in late 1998.

Taking a closer look at the rate of slide movement following November 1998 when all 10 wells went on-line, clearly, slide movement is primarily confined to the area north of the line of pumping wells. Reviewing data of the cumulative horizontal slide movement from November 1998 to February 2001 for two monuments, MP08 located north of the line of wells, and MP17 located south of the line of wells. Monument MP08 shows 52 mm of movement over a period of 27 months, while MP17 shows only a few millimeters of movement during the same time period. This differing rate of slide movement between the area north and south of the wells seems typical.

Since MP17 is located directly downslope from MP08, it seems anomalous that the upper portion of the slide is moving faster than the area downslope, which is barely moving.

A reasonable hypothesis to account for the difference in the rate of slide movement between the upper and mid-slide area is that by lowering the piezometric head, the wells are creating a stable wedge within the slide.

This more stable wedge of material appears to be acting as a buttress that helps to support the upper portion of the slide. Eventually, of course, either the upper slide must slow down or the buttress will fail and allow movement to translate downslope.

Correlation between Rainfall, Piezometric Head, Pump Rate & Slide Movement

The main factors causing reactivation and continued slide movement are:

1. The amount of rainfall that occurs during a rainfall year (July to June).

2. The level of the groundwater reservoir within the slide, left over from the previous.

3. The piezometric head along the base of the slide resulting from groundwater recharge causes an upward or buoyant force on the slide mass. In addition, seepage pressures increase as the groundwater moves downslope toward American Canyon Creek.

Without rainfall there would be no side movement since, physically, the slide is already in a stable configuration.

In attempting to investigate the mechanics of slide movement, it is necessary to analyze and compare the following interrelated data:

1. Rainfall amounts and the time distribution pattern;

2. Groundwater levels and piezometric head, as measured by piezometers and monitoring wells;

3. Pumping rate of wells, which are an indicator of the amount of available groundwater;

4. Ground movement surveys on the slide and slope indicator monitoring data.

Analysis of this data is ongoing; but, from the data, we see a significant rainfall event occurs and is then detected later as an increased pumping rate, following a lag time of 7-9 days, which increases as the rainfall season progresses.

Stability Analysis and Slide Stabilization Alternatives

A number of alternatives were evaluated to stabilize the eastern slide. First, a back calculation was conducted to estimate a residual friction angle of the slide plane. We also back calculated strength parameters of cohesion and friction angle to check the sensitivity by varying the strength parameters. Based on the magnitude of displacement that has occurred along the slide plane, we concluded that a residual friction angle (cohesion intercept = 0) would be an appropriate representation of strength estimates along the slide plane.

The first alternative analyzed was unloading the head of the slide by removing material acting as the driving force on the slide. By removing the top 18 m (60 feet) from the top of the active slide, there was an increase in slide stability 5% (factor of safety = 1.05). However, removal of support from the larger inactive slide to the north was a concern.

A second alternative was analyzed which consisted of placing a 12-m (40-foot) high earth buttress at the toe of the slide by filling in American Canyon Creek. This alternative increased the stability of the slide by 8%

(factor of safety = 1.08), but creates potential severe impacts to the riparian environment and the existing stream system.

In comparison, dewatering the slide and lowering the piezometric head by 15 m (50 feet) over the entire slide would produce an estimated 16% (factor of safety = 1.16) increase in the factor of safety for stability, which is considered sufficient given the site conditions.

Based on the stability analysis, dewatering of the slide mass is the logical solution to attempt to control slide movement.

Analysis of the dewatering option using seismic loading indicates that in the event of a Maximum Credible Earthquake on the Green Valley fault system that there will likely be movement or deformation of the landslide mass. It is estimated that there would be as much as 2.4 m (8 feet) of movement of the landslide.

CONCLUSION

The wells appear to have stabilized the slide during normal rainfall years, but the question is will the existing pumping wells provide adequate stability during another high rainfall El Niño year? Survey data suggests that slide creep occurs when rainfall exceeds normal annual rainfall. It is concluded that an as yet undetermined number of additional wells would be required in locations outside of the highway right-of-way to lower the piezometric head over a larger area of the slide mass to provide adequate stability during high rainfall years. Based on our experience with the existing pumps, it is anticipated that the long-term maintenance of a larger system will require full-time maintenance by one or more maintenance staff. In addition, the well system will also fail if the slide moves more than 0.15 to 0.3 m (0.5 to 1 feet). Another more favorable option, in lieu of pumping wells, would be to use one or more drainage shafts, also located outside of the highway right-of-way. These drainage shafts would be large diameter reinforced concrete lined shafts excavated to a depth below the slide plane with several levels of horizontal drains radiating horizontally out of the walls of the shaft. The drained water will have to be pumped from the shaft.

The drainage shaft system was selected for Red Top Slide and will consist of a single 93.5 m deep (307 feet), 6.5 m (21 feet) diameter shaft with radiating horizontal drains located at three different levels extending an average length of 152 m (500 feet) from within the shaft. This will create a multilevel gravity drain system with a diameter of roughly 305 m (1,000 feet) and will be patterned after a similar successful drainage shaft system used by the Oregon Department of Transportation to stabilize the Arizona Inn Landslide on Route 1 along the Oregon Coastline. The only difference will be the necessity to have a pump installed below the slide plane at Red Top. Due to the depth of the slide plane, it is infeasible to use a gravity outfall. It is anticipated that the drainage shaft system will be relatively easy to maintain and will tolerate small to moderate slide movement and continue to function as a drainage shaft. However, if several feet of movement were to occur along the slide plane, it is anticipated that the shaft liner will crack and that it will need to be repaired.

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Rock Slope Stabilization for the Boulevard of the Allies Reconstruction Pittsburgh, Pennsylvania

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ABSTRACT

The Boulevard of the Allies is a major four-lane roadway located in the City of Pittsburgh, Pennsylvania. The roadway was initially constructed in the 1920's at the edge of a near vertical, 115-foot high rock slope locally known as the Duquesne Bluff. In addition to normal traffic flow, the Boulevard of the Allies currently serves as a major detour route for an ongoing bridge and tunnel reconstruction project. In anticipation of the increased service of the roadway, the Pennsylvania Department of Transportation (PENNDOT) completed an in-depth rehabilitation prior to the bridge and tunnel closure. This effort included the widening of the roadway and reconstruction of four bridge structures along its one-mile length. A primary component of this rehabilitation consisted of the identification of adverse geotechnical conditions and design of corrective measures to stabilize the underlying rock slope, protect the traveling public from future geologic hazards, and to facilitate the proposed construction activities. These concerns included; the presence of several adversely oriented discontinuity sets leading to potential rock block instability, differential weathering resulting in potentially unstable rock masses, and the mass wasting of the rock slope causing the loss of bearing support at structure locations. To remedy these conditions, construction activities consisted of; extensive use of rockbolts to secure rock blocks along adversely oriented discontinuities, reconstruction of cantilevered roadway sections, installation of pin piles through existing structures to improve the bearing resistance, and the construction of slope netting as well as a rock-fall retention fence.

INTRODUCTION

Expansion of the City of Pittsburgh during the early part of the 20th century forced urban sprawl to boundaries imposed by the city's three rivers (Figures 1 and 2). At the southern limit of the city, in the Uptown District, the Monongahela River created a near vertical rock face, locally known as the Duquesne Bluff, through many years of erosion down cutting. During the 1920's, in an effort to improve travel between the city and adjacent communities, the Boulevard of the Allies was constructed at the crest of this 115-foot high bluff. The presence of four major educational institutions and several hospitals adjacent to the roadway and in neighboring communities contribute to the vehicular traffic of approximate 35,500 motorists daily. The roadway proved functional until increased demand forced the Pennsylvania Department of Highways, now the Pennsylvania Department of Transportation (PENNDOT) to widen the roadway in the 1950's. The presence of Duquesne University, Mercy Hospital and established businesses adjacent to the inbound, westward, lanes of the Boulevard of the Allies

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forced the designers to cantilever beyond the rock slope at several locations. Concurrent with the Boulevard of the Allies widening activities in the 1950's, the Penn Lincoln Parkway (Interstate Route 376) was under construction at the base of the Duquesne Bluff. The Penn Lincoln Parkway, bounded by the Duquesne Bluff and the Monongahela River, was constructed on alluvial and colluvial depositions as well as fill material. The Penn Lincoln Parkway currently facilitates the travel of approximately 70,000 vehicles daily.

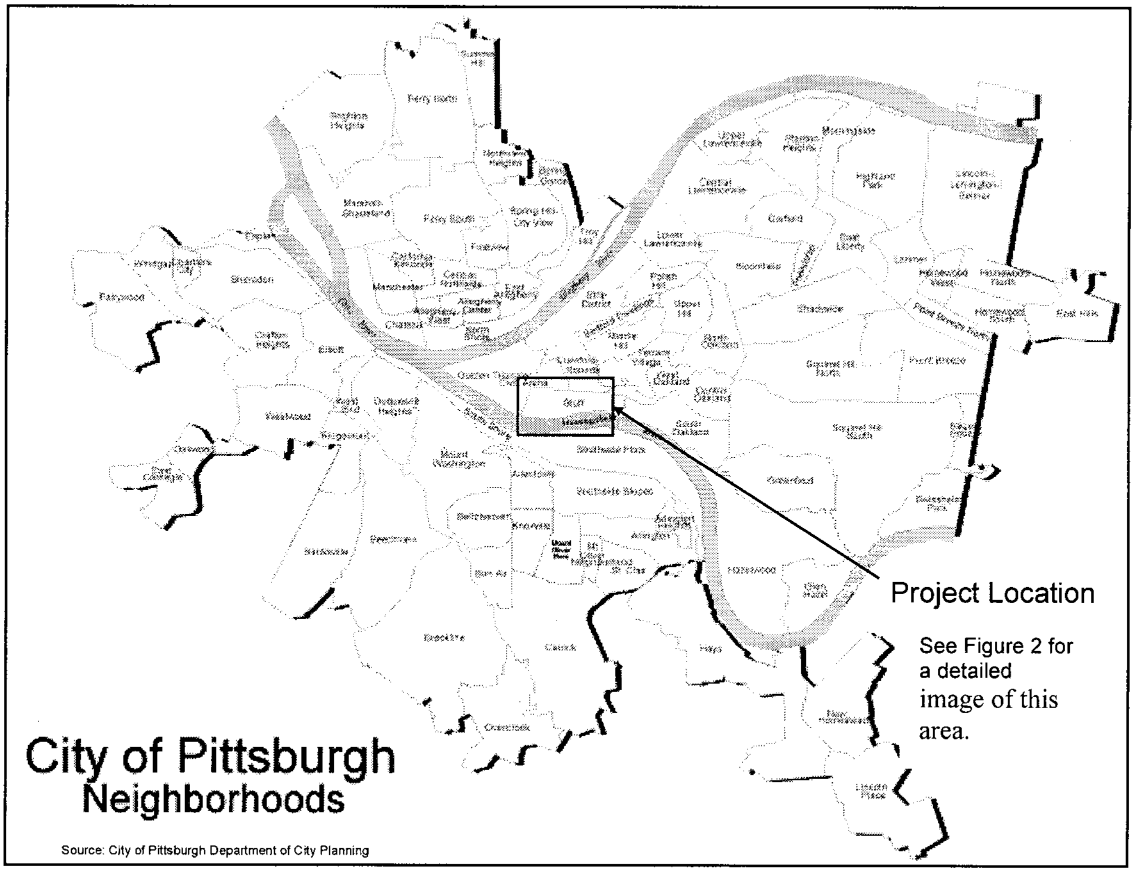


FIGURE 1

Slope failures in the late 1970's prompted PENNDOT to initiate slope remedial studies. Investigation of remedial alternatives began in 1980 with work performed to identify and recommend remedial alternatives for rock slope stabilization and the protection of traffic on the Penn Lincoln Parkway. As the result of these evaluations several concepts for stabilization were identified and included: 1) trimming of overhanging or potentially unstable rock masses; 2) construction of a reinforced concrete slope protection to protect weaker rocks from weathering and to provide stabilization of overhangs above; and 3) constructing a drop zone to contain rock debris from the upper slope. The recommendations were constructed in the mid 1980's. These construction techniques have proved to be effective; however, the relentless effects of weathering have resulted in undercuts of the concrete buttressing and several roadway support structures.

In anticipation of the closure of the Fort Pitt Bridge and Tunnel, which serve as a primary access to the central business district, PENNDOT desired to improve the condition of the Boulevard of the Allies for use as a detour route. The primary objective of the rehabilitation was to widen the travel lanes by reducing the width of the parapet on the outbound, eastward lanes along the bluff. Therefore, the outermost edge of the parapet would be maintained, but lane loads would be shifted toward the slope. Due to the new loading conditions, several structures required evaluation and reconstruction to meet current design standards. The two prominent structure types consisted of either concrete piers notched and embedded into the rock face or counterweight cantilevered concrete slabs bearing on support walls. The foundations of these structures were the focus of the geotechnical investigation to ensure that proper bearing support could be provided and that future instabilities and loss of the rock slope would not occur.

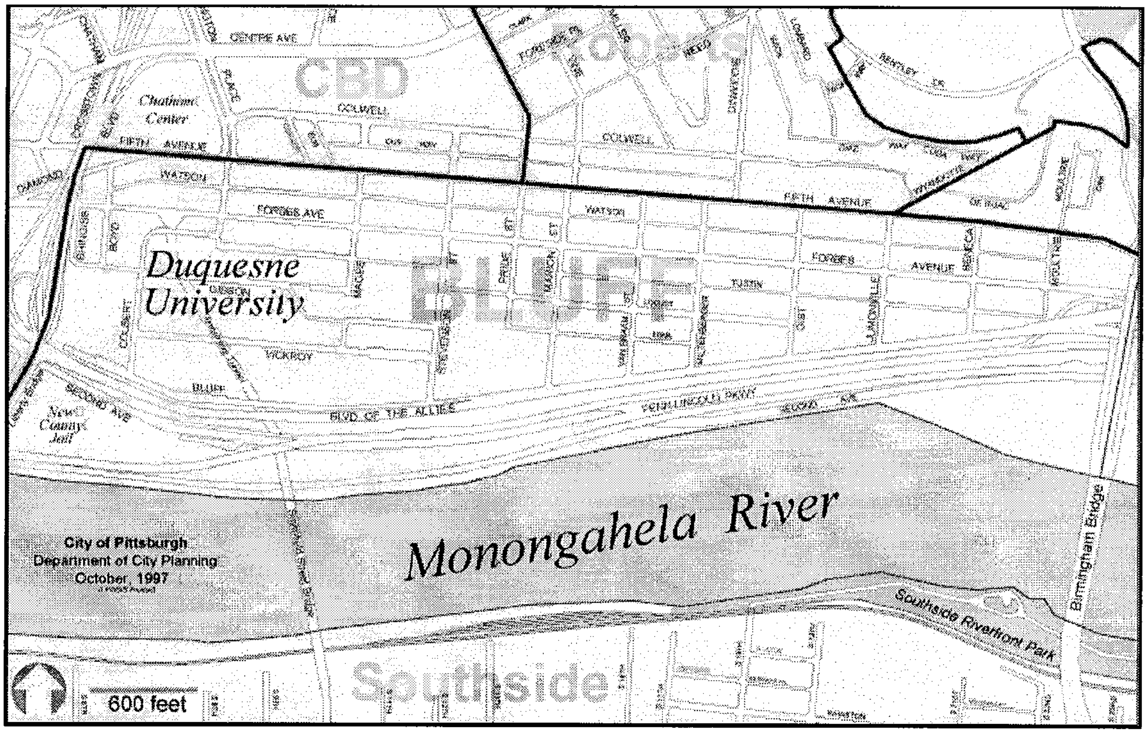


FIGURE 2

PHYSIOGRAPHIC AND GEOLOGIC SETTING

The Duquesne Bluff is located within the Appalachian Plateaus Province in the Appalachian Highlands. The Appalachian Plateaus consists of gently folded, relatively flat lying rock units dipping regionally to the southwest at a rate of approximately 1 foot per 100 feet (M.E. Johnson, 1928.) The topography within the region consists of steep hillsides and deep river and stream valleys with magnitudes of vertical relief typically ranging between 200 and 400 feet. Specifically, the 115-foot maximum vertical relief of the Duquesne Bluff was formed as the result of long term erosion processes of the Monongahela River and the accompanying valley wall stress relief (H.F. Ferguson, 1967, H.F. Ferguson and J.V. Hamel, 1981).

Stratigraphically, the Duquesne Bluff reveals exposures of rock units within the Conemaugh Group of the Pennsylvanian system with geologic units in the lower Casselman and upper Glenshaw Formations. The main lithologic types are shale, claystone, marine and freshwater limestone, sandstone, siltstone, and coal. The strata composing the bluff are characteristic of the late Pennsylvanian deltaic depositional environment (Figure 3). Definitive contacts can be observed between the stratigraphic units exposed on the bluff with the apparent dip being from east to west. The stratigraphic units consist of; the Morgantown sandstone, the Wellersburg claystone, the Birmingham shale, the Duquesne claystone, the Ames limestone, and the Pittsburgh redbed (Figure 4).

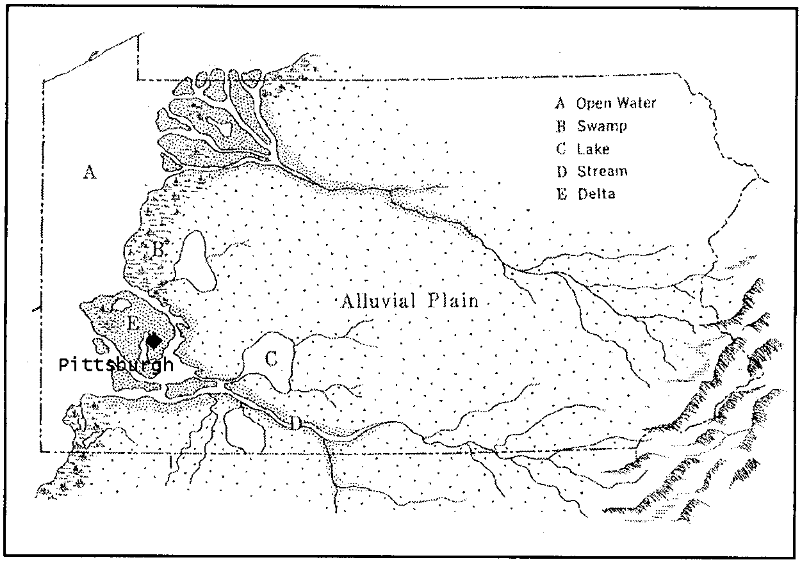


FIGURE 3

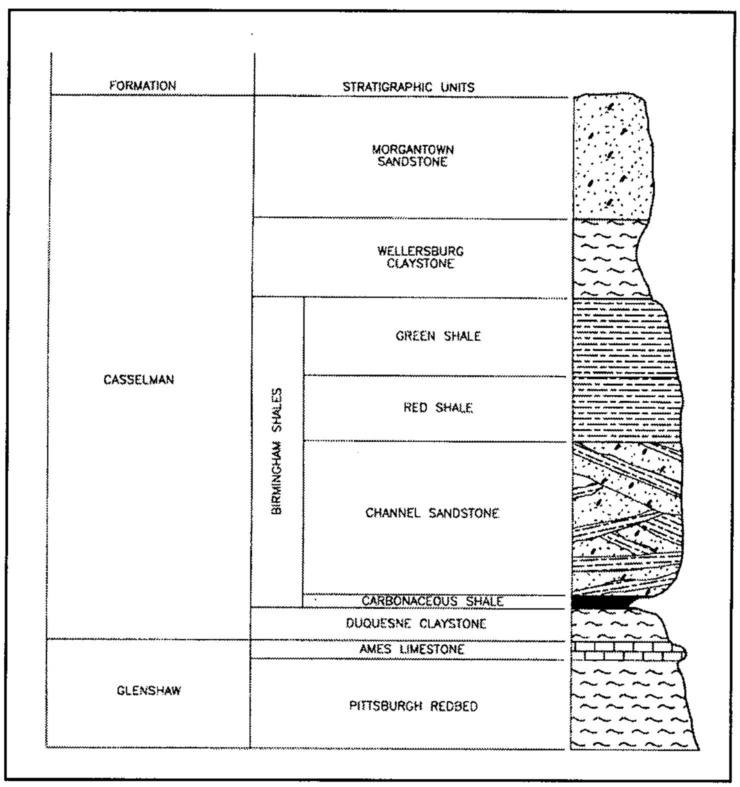


FIGURE 4

In addition to the alternating sequences of durable and less durable rock units along the slope, jointing is readily observed on the bluff face and has been instrumental in the development of current slope conditions and instabilities. Many of the joints exposed on the slope are observed to be closed; however, several joints are open as much as 6 inches. Primary jointing types, including tectonic and valley stress relief, have resulted in the formation of rock wedges and the potential for rock fall conditions. Tectonic joints, formed by the lateral compressive deformation of the earth's crust, are found to be systematically perpendicular and intersect the slope face at angles ranging between 30 and 60 degrees. Stress relief joints are also present along the slope face and have formed by the relief of stresses through valley down cutting by the Monongahela River (H.F. Ferguson, 1967, H.F. Ferguson and J.V. Hamel, 1981). These joints are curvilinear but generally parallel to the slope face and have been measured at plus or minus 12 degrees of due east. Jointing conditions, as described, have promoted slope instabilities leading to wedge and toppling failures. Geomorphic processes including root pry and frost wedging have initiated many of these failures.

SITE INVESTIGATION AND DATA COLLECTION

Site Investigation. A site investigation consisting of 39 geotechnical borings, extensive laboratory testing and significant field reconnaissances was conducted to characterize the slope and determine potential instability based on the variability of the strata and discontinuity orientations. A "snooper truck" (Figure 5) was used to access the slope and record joint spacing and orientation, measure the depth of undercuts at structure locations, confirm structure dimension, assess the condition of concrete structures and to photograph slope and structure features for archive purposes. This was the first application of its kind for the snooper truck contractor who typically uses the equipment for under bridge inspections. This equipment proved to be an innovative method to access the slope which would have otherwise required repelling or other more dangerous methods.

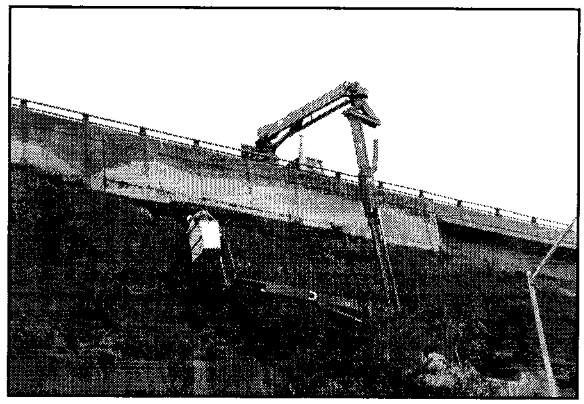


FIGURE 5

Interpretation of Data. The discontinuity data was evaluated using the Spheristat 2.2® computer program developed by Pangaea Scientific. The data set consisted of 180 discontinuity orientations taken from the slope face. Five principal joint sets were identified as follows: (1) N78°E, (2) N78°W, (3) N41°E, (4) N59°W, and (5) N6°E (Figure 6). Dip directions were recorded to be 82°S, 80°S, 81°SE, 79°SW, and 85°E, respectively. Joint sets 1 and 2 are interpreted to be valley stress relief joints with the strike directions occurring in adjacent quadrants confirming the undulating, curvilinear, surface observed in the field. The valley stress relief joints, measured to be sub-parallel to the slope face, were recorded to be spaced approximately 2 to 5 feet apart (Figure 7). Joint sets 3, 4, and 5 are tectonic. The bedding orientation identified in this region is measured to strike N56°W and dip 5°S. The strike of the slope surface was, on average, N80°E, with an approximate dip of 80°S. All readings were adjusted for a declination of 7.5° based on the City of Pittsburgh mapping.

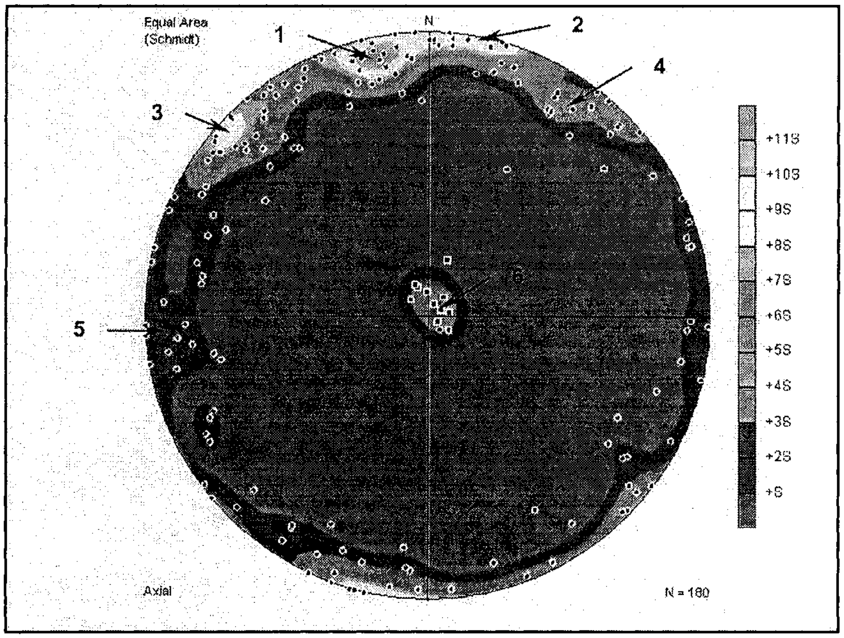


FIGURE 6

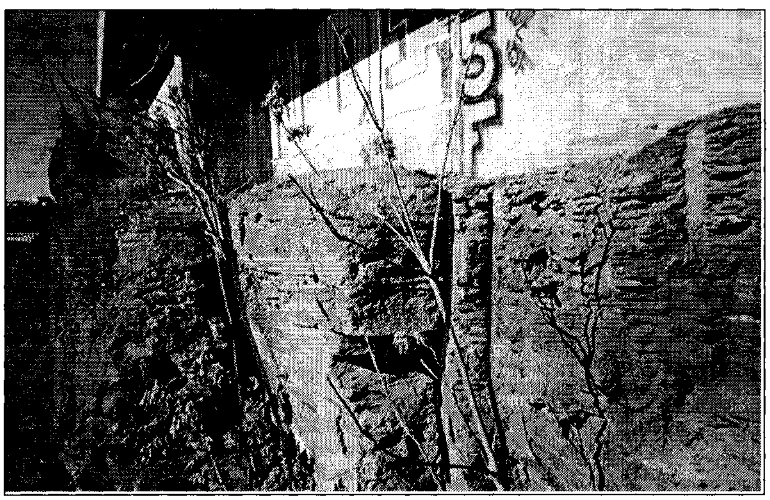


FIGURE 7

Oblique Aerial Survey. Due to the nearly vertical nature of the slope, conventional survey would have been impractical, if at all possible. The designers used an innovative and state of the art concept of oblique aerial digital imagery. The slope was photographed with a Precision Aerial Survey System (P.A.S.S.) Bell 47 Helicopter and a Zeiss RMK TOP15 precision aerial camera. These images were used by Gannett Fleming CADD operators to develop an Intergraph MicroStation topographic file that was the basis for generating cross sections at discrete locations along the slope. These cross sections were critical in determining slope geometry for rock fall modeling, stability analysis and constructability determination. Additionally, a digital photogrammetric mosaic showing existing slope and structural features as well as proposed treatments was prepared. These images were provided in the bid documents and were used by the contractor during construction.

GEOTECHNICAL CONSTRUCTION AND SLOPE STABILIZATION

Current observations of the support buttressing constructed during the 1980's rehabilitation initiative indicate that the designs have been effective. However, aggressive slope trimming operations conducted during that project may have led to weathering and mass wasting of the less durable rock units and the undercutting of the concrete buttressing and of roadway structures at various locations. Although localized removal of large rock masses may have been warranted given the precarious rock overhangs, aggressive overall slope trimming may have promoted the conditions currently observed on the slope (J.W. Kovacs and W.R. Adams, 1997).

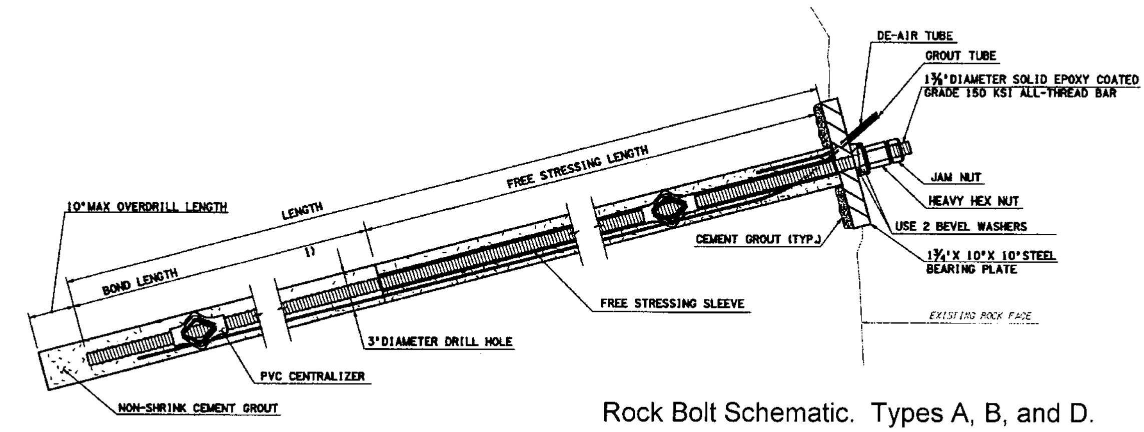
Current studies indicate the need for rock fall protection for travel lanes of the Boulevard of the Allies and the underlying Penn Lincoln Parkway and additional slope stabilization and structure support to address the proposed loading conditions. These measures consist of the installation of rock bolts, pin piles and the construction of rock slope mesh and a rock fall retention system.

Rock Bolts. Rock bolts were used for two applications; 1) to stabilize rock blocks underlying the roadway structures, and, 2) to stabilize existing concrete buttresses that have been undercut by loss of underlying rock. Four rock bolt designs were developed for these applications. These designs are summarized in Table 1.

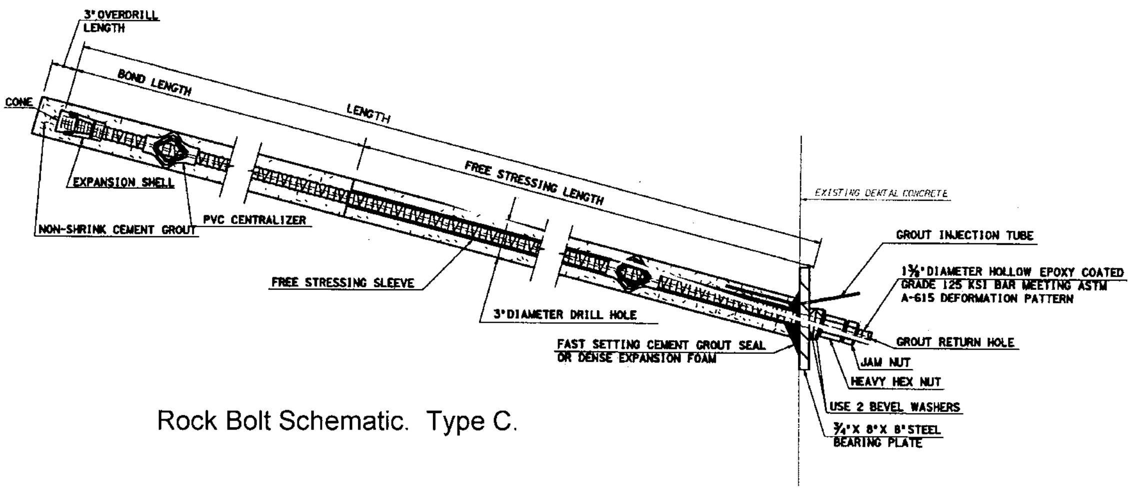
Designation	Purpose	Length	Bond Length	Design Load	Declination
Type A Grouted	Secure Morgantown sandstone at pier structure	28 feet	15 feet	135 kips	15°
Type B Grouted	Secure Morgantown sandstone at pier structure	18 feet	15 feet	135 kips	15°
Type C Spin Anchor Grouted	Secure existing undercut concrete buttresses	16 feet	5 feet	45 kips	15° (inclined)
Type D Grouted	Secure Birmingham shale below cantilever structure	30 feet	15 feet	100 kips	20°

TABLE 1

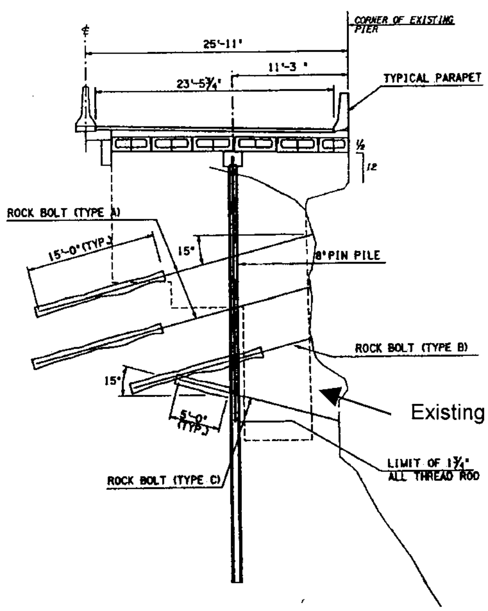
Rock bolts for slope stabilization consisted of 1-3/8 inch all thread bars grouted in a minimum 3-inch diameter drill hole. The as-built drill hole diameter was 4.5-inches. The existing concrete buttresses were secured with 1-3/8 inch hollow all thread bars with spin lock anchorage. The purpose of the spin lock design was to secure the bolt in the 15° inclined position to anchor the existing concrete buttress on the slope and to facilitate grouting operations. At these locations, grout was injected through a grout tube penetrating the bearing plate and returning through the hollow core of the all thread bar. Declined rock bolts were grouted through a grout tube extending to the bottom of the drill hole with return through a de-air tube penetrating the bearing plate (Figure 8). All rock bolts were tested to 125% of the design capacity and locked off at the design load.



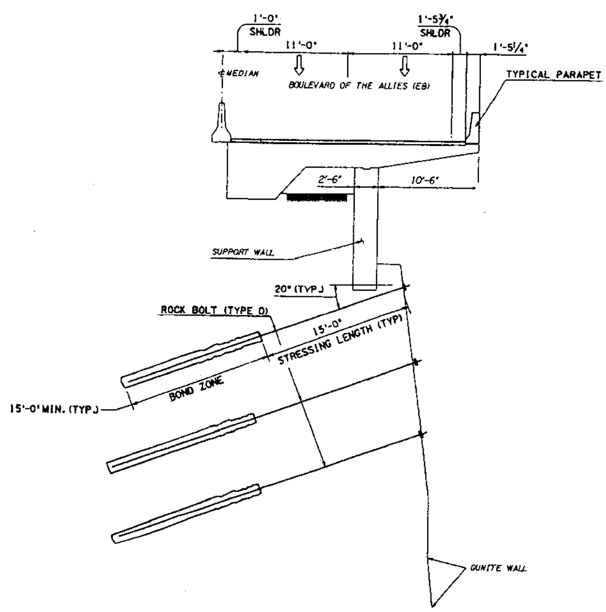
Rock Bolt Schematic. Types A, B, and D.



Rock Bolt Schematic. Type C.



Cross Section View of Rock Bolt and Pin Pile Layout



Cross Section View of Rock Bolt Layout at Cantilever Structure

FIGURE 8

The urban setting of this project presented unique construction challenges involving equipment mobility, temporary loading of existing structures and the requirement for suspended drill rig platforms for the installation of rock bolts (Figures 9 and 10). Existing structures were evaluated to ensure that the crane loads could be supported.

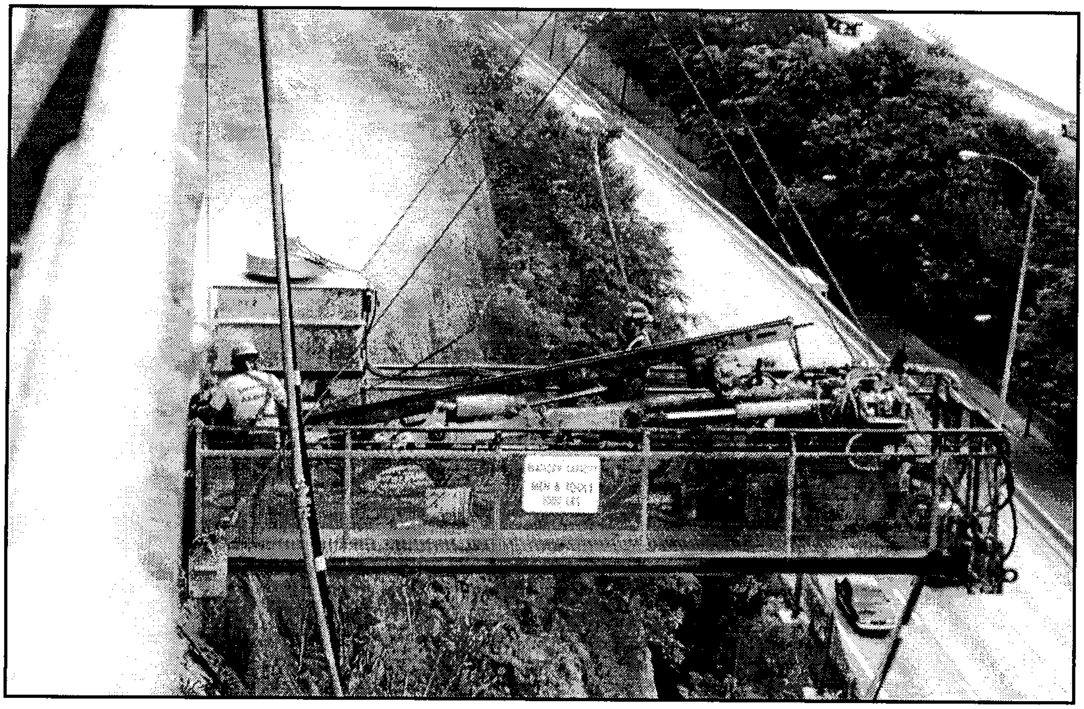


FIGURE 9

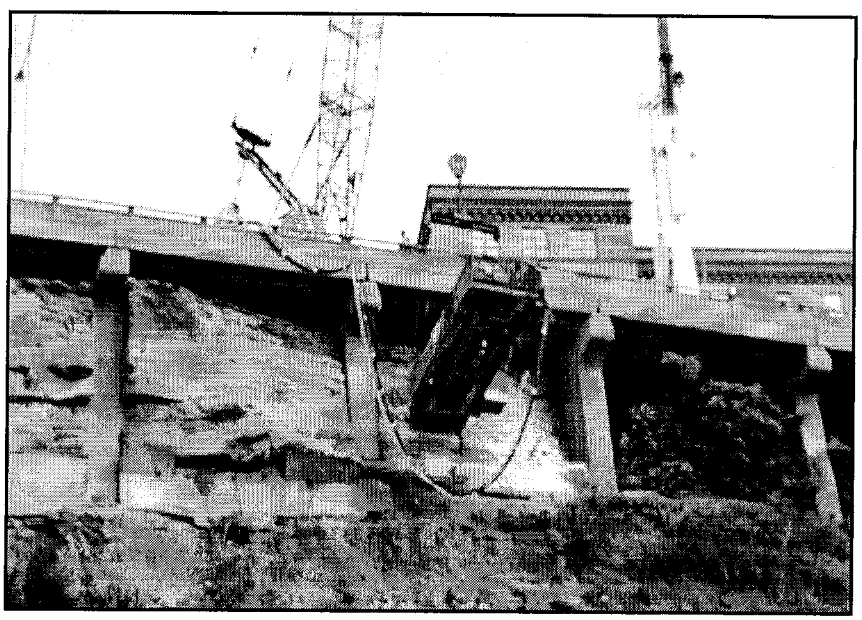


FIGURE 10

Pin Piles. Pin piles, also referred to as micro-piles, were used to construct a new traffic sign structure and to provide additional bearing support for existing structures (Figure 8). Due to limited space to construct spread footings or caissons at the sign structure location, pin piles were a viable alternative. Analysis concluded that these piles would be subject to both compression and tension and were designed accordingly.

Pin piles were chosen as a method to increase the bearing capacity of the existing concrete piers notched into the Morgantown sandstone. Although current conditions do not suggest instability, the designers chose to install additional foundation support as a preventative measure to account for long term mass wasting and loss of weaker rock units underlying the structure. One pile was installed at each pier location.

The pipe pile design consisted of a Grade A53, 35 ksi yield strength, Schedule 80, 8-inch diameter (nominal) pipe. A design capacity of 138 kips was specified for the piles at the pier locations and 142 kips was the design load of the sign structure piles. The piles were installed with a percussion down the hole drill bit mounted on a Cassegrande M9 drill rig. The piles were load tested in accordance with ASTM D3689. Due to the limited site access and required reaction piles in order to conduct a compression test, the test was performed in tension only and provided a conservative result since end bearing does not influence the results. A cyclical test was also performed to model the pile behavior under the anticipated sign structure wind loads.

Rock Fall Retention System. Analysis was performed using the Colorado Rockfall Simulation Program (CRSP) based on anticipated rock sizes and the slope geometry to predict rockfall trajectory and energy. It was determined that the fence must be constructed 10 feet high and capable of withstanding a 90-foot-ton impact load to account for statistically probable rockfall. The as-designed system consisted of drilled shafts for the foundation elements. However, the contractor proposed pre-cast concrete blocks that rested against the back face of the barrier. The system was evaluated and determined to offer an equivalent level of service as the originally designed scenario. Figure 11 shows the as-designed and as-built cases.

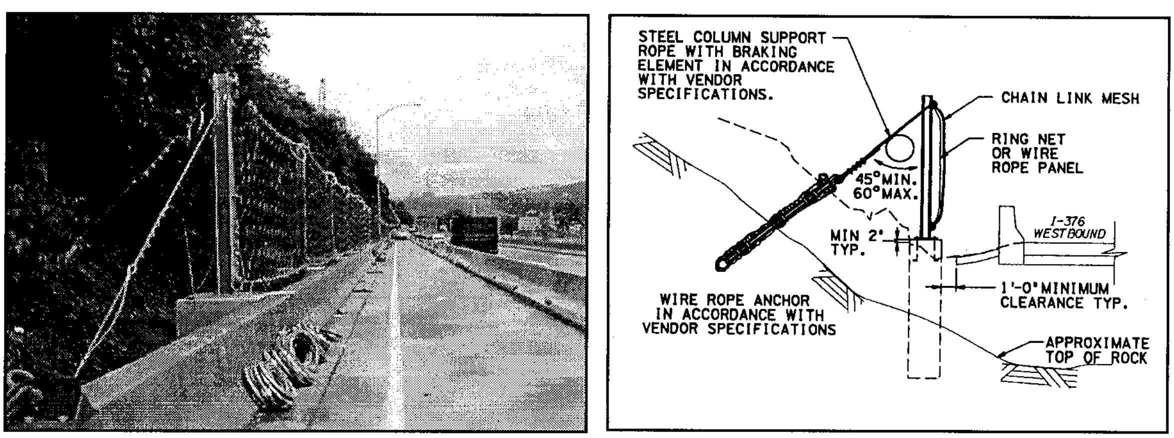


FIGURE 11

Rock Slope Mesh. Visual observation and geologic mapping indicated the presence of adverse jointing and the potential for rockfall on a portion of the slope that is exposed above the Boulevard of the Allies. The anticipated cause of this rockfall is exposure to the elements, the mechanics of freeze/thaw, and from root pry of existing vegetation.

The rockfall at the site was considered typical of a slope with these jointing features and geometry. Rockfall is likely to enter the traveled lanes of the Boulevard of the Allies without remedial action and wire mesh slope netting was identified as an appropriate solution for protection of traffic. The proposed netting consisted of PVC coated wire mesh anchored at the top of the slope by epoxy resin anchors spaced at 12-foot centers. The PVC was color matched to the slope. The mesh was left open at the bottom such that the falling rocks can be collected in a drop zone and removed as necessary. Figure 12 shows the slope prior to placement of the wire mesh treatment.

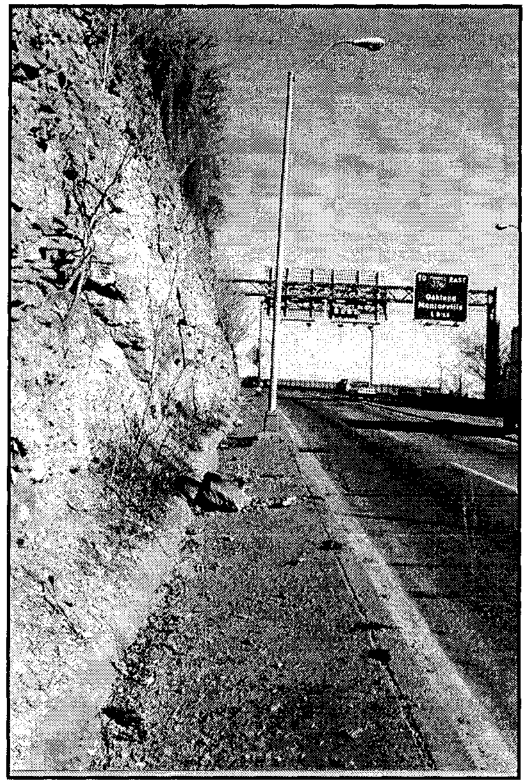


FIGURE 12

CONCLUDING REMARKS

Geologic conditions of the Duquesne Bluff consist of alternating layers of durable and non durable rock units that are prone to differential weathering and the formation of rock overhangs. In addition, valley stress relief joints formed parallel to the bluff face and systematically perpendicular tectonic joints have resulted in unstable rock wedges. Through detailed site investigation and kinematic analysis of the slope, designers were able to predict failure mechanisms and develop loads for use in designing stabilization systems.

Proposed reconstruction of structures along the Boulevard of the Allies changed the loading conditions imposed on the underlying rock slope. In order to improve the rock slope conditions and to provide adequate foundation support, rock bolts were implemented to stabilize previously constructed concrete buttressing and to secure rock blocks formed by adversely oriented discontinuities. Pin piles were installed to provide increased bearing support to structures subject to undercutting by loss of non-durable underlying rock strata. A rock fall retention system and slope mesh were also installed as a preventative measure to protect motorists from potential future rock fall debris.

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GEOLOGY AND GIS FOR PLANNING: AN EXAMPLE FROM MONTEREY COUNTY, CALIFORNIA

ABSTRACT

A comprehensive digital database on Monterey County geology was prepared for the Monterey County 21st Century General Plan Update. The database includes geologic units, faults, landslide and liquefaction susceptibility, and mineral resources. These data are in a geographic information system (GIS) format that provides a unified, seamless coverage for the seventy-nine 1:24,000 scale quadrangles encompassing Monterey County.

Landslides are probably the most costly geologic hazard in Monterey County. Highly susceptible parts of the county to landsliding include areas underlain by the Franciscan Complex (Big Sur coast) and areas underlain by the Monterey Formation (Carmel Valley). Coastal erosion of sand dunes along the Monterey Bay and of marine terrace deposits along the Big Sur coast present a significant constraint to development in these areas.

Other geologic hazards in Monterey County include fault rupture, strong shaking, and liquefaction. Fault rupture hazards are highest along the San Andreas fault and high for the San Gregorio, Reliz/Rinconada, and the Monterey Bay/Tularcitos fault zones. Severe damage from strong shaking and liquefaction occurred during several earthquakes, most notably in the 1906 San Francisco and the 1989 Loma Prieta earthquakes. Liquefaction is most likely in saturated and loose, geologically young, granular deposits such as in the Moss Landing and the Salinas Valley areas.

The digital databases can be plotted as paper maps or viewed interactively on a computer in the office or field. The GIS format allows users to create their own specialized maps such as delineating the distribution of geologic hazards along highway and lifeline alignments.

INTRODUCTION

Monterey County lies within the California Coast Ranges physiographic province astride the broad boundary between the Pacific and North American tectonic plates. The relative motion between these two plates—which is still occurring at a rate of about 2 inches per year— has produced most of the geologic structure of the county. Its most evident manifestations are the active San Andreas and San Gregorio faults. The uplift that formed the Coast Ranges was much more rapid in Monterey County than in other parts of California. The dramatic cliffs of the Big Sur coast and steep slopes of the Santa Lucia Range are products of this rapid uplift.

Geologic forces such as mountain building and mass wasting are long-term processes that shape the landscape. Earthquakes and landslides are two familiar types of geologic events associated with these long-term processes. When these geologic events pose danger to property or safety, they become hazards. Common examples of geologic hazards present in Monterey

County include ground rupture along faults, strong shaking, liquefaction, landslides, and erosion. In land-use planning, the term "hazard" implies that there is a negative effect associated with these naturally occurring geologic events. From a planning point of view, these "hazards" are potential constraints on the intended use of the land. By analyzing these constraints, the risks can be assessed and usually mitigated to an acceptable level.

The County of Monterey regulates land-use based on policies and ordinances derived from the county's official geologic hazard maps. The previous geologic hazard maps were prepared in 1975 as part of the Monterey County General Plan—which is revised about every 20 years. However, the 1975 General Plans maps did not include large parts of the county such as the Big Sur coast and southern Monterey County. In addition, recent mapping by the U.S. Geological Survey shows features such as previously unmapped active faults and liquefaction-prone sediments. The General Plan is presently (2002) being updated and takes advantage of modern technology such as GIS software. This General Plan Update (GPU) provided an excellent opportunity to update the county's geologic maps, create derivative hazard maps, and develop planning policies that accurately reflect the current state of knowledge. This paper presents a summary of several geologic hazards analyzed for the GPU. For a comprehensive discussion of Monterey County geologic hazards, see the report by Rosenberg (2001).

PROCEDURES

Although digitally produced geologic maps are extremely useful, they are deceptively difficult to produce. Preparing a geologic hazard map from a GIS digital database involves several issues—each with their own attendant problems—such as capturing data from paper maps and converting them to digital format, compilation from various vintage and scale geologic maps, and recasting geologic units into units which reflect engineering behavior. Organizations considering producing regional geologic databases should be aware of some of these critical issues, which are discussed in the following paragraphs.

Capturing Data

Approximately 80 geologic maps were used to create the Monterey County geologic database. The maps ranged from old unpublished paper maps to recent GIS databases. The various maps used in the compilation were converted to digital format using a Vidar large-format monochrome drum scanner at a resolution of 300 dots per inch. The scanned images were saved as uncompressed TIFF raster image files, and then were georeferenced, projected, and autovectorized using ESRI ARC/INFO software. The vectorized geologic contacts and fault lines were converted into ARC/INFO coverages and then into ArcView shapefiles. Finally, the geology was compiled onto 1:24,000 scale USGS digital raster graphic maps (the digital equivalent of USGS 7.5-minute topographic quadrangle maps) and the shapefiles were clipped to the Monterey County political boundary.

Although three 7.5-minute geologic map quadrangles were available in ARC/INFO format, they had to be downloaded from the Internet and converted into the same format as the Monterey County geologic map. This involved importing the ARC/INFO coverages into shapefiles and changing the projection of the imported data to match the existing data.

Compilation

The purpose of this task was to create a seamless geologic digital database of Monterey County. The philosophy behind this compilation was to combine "the best of the best" sources into one database, for which 82 maps and reports on Monterey County were used. It is important to note that much of the geologic mapping of Monterey County was done between 1930 and 1950, focusing on oil and mineral resources. To evaluate geologic hazards, maps emphasizing the distribution of earthquake faults and unstable slopes are necessary. The older maps are useful for geology, but usually do not have geologic hazard information.

The sources used for the compilation were originally mapped at scales ranging from 1:2,400 to 1:125,000, with the majority at 1:62,500 scale. The compilation at 1:62,500 scale requires simplification in some areas and combining of some geologic units. Also, minor adjustment of contacts across the quadrangle boundaries was locally necessary. Figure 1 shows an example of a scanned map and its conversion into a digital database.

One problem encountered in creating the seamless geologic map was that geology did not always match from quadrangle to quadrangle. Part of this discrepancy is the differences in mapping styles between the geologists that mapped them. Other inconsistencies included different terminology in naming geologic units and faults, and drafting errors. Although the source map linework was accurately digitized, mapping errors by the original authors still exist.

A unique issue associated with digital databases is the concept of map accuracy and precision. For example, a line shown as well-located on a published USGS geological map typically is accurate to within 50 feet at 1:24,000 scale. If that map is digitized as part of a geological database, enlarging the image (either digitally or by photographically) does not increase the accuracy of the line and can cause misinterpretation of the data. Users should be aware that at local scales (more detailed than 1:62,500) the geology may be different than depicted in the database due to the regional focus of this compilation.

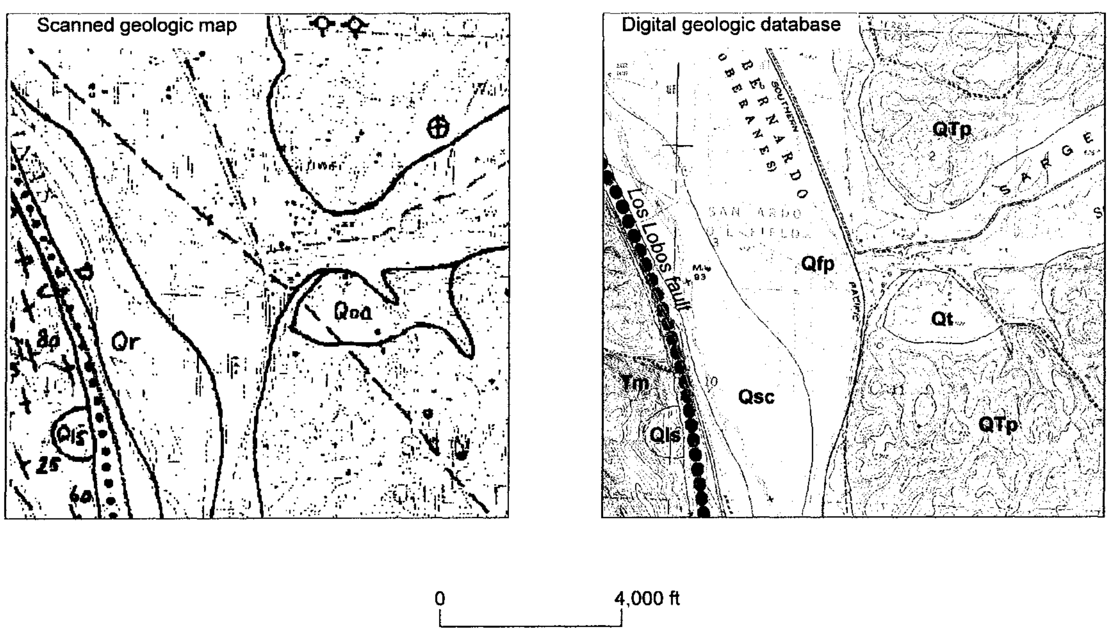

Figure 1. Conversion of scanned map into digital geologic database.

GEOLOGIC DATA MODEL

For GIS-based geologic and seismic hazard evaluation, the foundation is the geologic data model. In this study, the model is a digital compilation of stratigraphic formations and faults in Monterey County. T.W. Dibblee, Jr. mapped most of the geology in Monterey County for the U.S. Geological Survey in the 1960s and 1970s. Other major contributors of mapping include J.C. Tinsley, III, W.R. Dupré, V.M. Seiders, D.C. Ross, J.C. Clark, and C.A. Hall.

As pointed out by Wagner (2001), preparing a stratigraphic framework and map explanation is one of the first, and most important, steps in compiling a regional geologic map. In Monterey County, the San Andreas fault and the Sur/Nacimiento fault are major structural boundaries, and as a result, there are three different stratigraphic columns. To provide a logical framework for the many geologic units in Monterey County, the author of most of the geologic maps used in the compilation, T.W. Dibblee, Jr,. prepared a countywide correlation of map units, with emphasis on the Tertiary and Cretaceous age sedimentary rocks. Cretaceous plutonic and metamorphic rocks follow the nomenclature of Ross (1972, 1976). Quaternary deposits were classified based on the work of Tinsley and Dupré. Due to time and funding constraints, detailed structural data such as folds, bedding attitudes, and depositional contacts were not included in this database.

ArcView was used with the GeoEditor extension (Walker and Black, 2000; http://www.geomaps.geo.ukans.edu) as a tool to classify geologic units and faults into a consistent format. Other fields such as liquefaction susceptibility (LIQ_SUSC), estimated strength (STRENGTH), and the 1997 Uniform Building Code site response class (UBC) were added to the geology data dictionary to facilitate preparing derivative geologic hazard maps. An example of the dictionary is shown in table 1.

Table 1. Geology data dictionary

Detailed name	Det. label	Lithology	Era	Period	Epoch	Source	Liq. susc.	Strength	UBC
Monterey Formation	Tm	Shale	Cenozoic	Tertiary	Middle-Early(?) Miocene	Dibblee (1971)	Low	C	C
Franciscan complex	KJf	Metavolcanic	Mesozoic	Jurassic-Cretaceous	--	Seiders (1989)	Low	A	B
Alluvial fan deposits of Chualar	Qch	Alluvial fan	Cenozoic	Quaternary	Late Pleistocene	Dupré and Tinsley (1980)	Low	B	C
Landslide deposits	Qls	Hillslope debris	Cenozoic	Quaternary	Holocene-Pleistocene	Dibblee (1971)	Low	C	D

Metadata

All too commonly, GIS datasets lack descriptive information about how the data were obtained, modified, and classified. To remedy this shortcoming, metadata ("data about data"), which describe the content, quality, condition, and other characteristics of data, were prepared for each data set using ESRI ArcCatalog 8.1 software. The metadata files are in both text format and "Frequently Anticipated Questions" hypertext format. These metadata comply with the Federal Geographic Data Committee Content Standard for Digital Geospatial Metadata (FGDC-STD-001-1998).

GEOLOGIC HAZARDS

Fault Rupture

Fault rupture is a seismic hazard that affects structures sited above an active fault. The hazard from fault rupture is the movement of the ground surface along a fault during an earthquake. Typically, this movement takes place during the short time of an earthquake, but can also occur slowly over many years in a process known as creep. The only known creeping fault in Monterey County is the part of the San Andreas between San Juan Bautista and Parkfield. Most structures and underground utilities cannot accommodate the surface displacements of several inches to several feet commonly associated with fault rupture or creep.

In California, an active fault is legally defined by the State Mining and Geology Board as one that has *"had surface displacement within Holocene time (about the last 11,000 years)"*. The term "potentially active" fault was used by the CDMG until 1988 to define faults that had evidence of Quaternary (about 1.6 million years) surface displacement (Hart and Bryant, 1999). Current usage in seismology is to specify the time of latest fault movement such as "Holocene active" or "Quaternary active".

In this study, the relative activity of faults in Monterey County was assessed using the approach of Jennings (1994). in which faults are classified based on the time of their most recent movement. The State of California has only zoned the San Andreas fault within Monterey County as active; however, other studies document Holocene activity on local faults. These include the offshore parts of the Navy and Chupines faults (McCulloch and Greene, 1989); the Berwick Canyon (Vaughan and others, 1991); the Tularcitos, Sylvan thrust, Hatton Canyon (Rosenberg and Clark, 1994); the Garrapata/Palo Colorado and Rocky Creek fault segments of the southern San Gregorio fault zone (Clark and Rosenberg, 1999); the San Gregorio fault offshore of Point Sur (Eittreim and others, 1998), and part of the Vergeles fault (Coppersmith, 1979). These other Holocene faults were delineated as County fault zones, which require trenching to determine the fault location and setbacks from the fault trace.

The faults are stored in the database based on the location accuracy (type of fault) and the age of most recent known faulting (recency) as shown in table 2. This database structure allows the user to analyze and plot maps based on fault activity.

Table 2. Fault data dictionary

Name	Type	Source	Recency
San Andreas	fault-certain	CDMG (2000)	Historic
Rinconada	fault-inferred	Hart (1985)	Late Quaternary
Sur fault zone	fault-concealed	Hall (1991)	Quaternary undifferentiated

Ground Shaking

Strong ground shaking is the seismic hazard most likely to affect Monterey County. The county is subject to very strong (0.3–0.6*g*) to severe (greater than 0.6*g*) shaking from three main Holocene active faults: the San Andreas, San Gregorio, and Reliz/Rinconada (Petersen and others, 1999). The severity of ground shaking depends on several variables such as earthquake magnitude, epicenter distance, local geology, thickness and seismic wave-propagation properties of unconsolidated materials, ground water conditions, and topographic setting.

Ground shaking hazards are most severe in areas near the San Andreas fault and in the unconsolidated alluvial areas of the county such as the Salinas and Carmel Valleys. In addition, there are local areas which will likely experience amplified ground shaking and greater damage due to their geologic and topographic characteristics such as soft soils, basin geometry, and depth to ground water. However, the degree and location of amplification are currently difficult to quantify because of the lack of established methods of characterization.

The 1997 Uniform Building Code (UBC) addresses ground response by classifying the soil and rock into six soil profile types based on shear-wave velocity: S_A (hard rock), S_B (rock), S_C (very dense soil and soft rock), S_D (stiff soil), S_E (soft soil), and S_F (soils requiring site-specific evaluation). The geologic units in the database were reclassified by their estimated shear-wave velocity into UBC soil profile types using procedures developed by Wills and Silva (1998) and Wills and others (2000). These UBC classes were included in the geologic database for planning purposes.

Liquefaction

Liquefaction is a process in which sediments below the water table temporarily lose strength during an earthquake and behave as a viscous liquid rather than a solid. Liquefaction does not occur at random, but is restricted to certain geologic and hydrologic environments, primarily recently deposited sand and silt in areas with high ground water levels. Areas in Monterey County most susceptible to liquefaction include the Salinas River and floodplain, the Moss Landing and Elkhorn Slough areas, the Carmel River and floodplain, the San Antonio and Lockwood Valleys, and the Peachtree and Cholame Valleys.

Detailed liquefaction susceptibility studies published by the U.S. Geological Survey for the Monterey Bay region (Dupré and Tinsley, 1980; Dupré, 1990; Pike and others, 1994) provide a framework to evaluate the relative liquefaction susceptibility of geologic materials in Monterey County. For the area south of Salinas, the USGS methodology was followed with the assumptions that the age, near-surface stratigraphy, and physical and engineering properties of the Quaternary sediments were similar to their mapped counterparts in the northern part of Monterey County.

The type and distribution of geologic materials were mainly determined from geologic and soil maps. Each geologic unit in the database was assigned a high, moderate, or low liquefaction susceptibility based on age, lithologic type, depositional environment, and correlation with similar units with engineering properties that were well defined by field and laboratory testing. Because of the regional focus of this study, there was no comprehensive integration of engineering borings and water well logs to refine the assumed geologic conditions. Areas that liquefied in the 1906 San Francisco or 1989 Loma Prieta earthquake as mapped by Youd and Hoose (1978) and Tinsley and others (1998) were overlaid on the liquefaction map to confirm the zonation.

Current systematic shallow ground water level measurements do not exist for most of Monterey County. In this study, records of the highest historical ground water levels were used in evaluating the liquefaction susceptibility. The best available information on historical high water levels are the reports by Lapham and Heileman (1901) and Hamlin (1904). Lapham and Heileman (1901, plate 77) show that depth to water is less than 10 feet along the Salinas River floodplain from Elkhorn Slough southward to Chualar, as well as in several now-dry unnamed lakes. Hamlin (1904) tabulated water levels for 270 wells in the Salinas Valley from the mouth of the river southward to King City. Of these wells, most along the river had water depths of about 10 feet and the wells on the alluvial fans had water levels of 20 feet or deeper, with a median depth to water of 15 feet. Hamlin did not specify how many of these water levels represent artesian (confined) aquifers or shallow perched aquifers. However, Carpenter and Cosby (1929, p. 76) noted that in irrigation wells along the Salinas River, *"... the level of water in the wells apparently corresponds to the elevation of the water in the near-by channel of the Salinas River, indicating a close relationship between this stream and the underground water supply."* Based on these observations, areas of Holocene alluvium were assumed to be saturated because irrigation and releases from the San Antonio and Nacimiento Reservoirs into the Salinas River keep water levels shallow along the Salinas River year-round.

In order for liquefaction to occur, susceptible deposits need a large enough level of ground motion during an earthquake. Although the statewide CDMG probabilistic seismic hazard maps allow reasonable estimates of ground motions, the specific engineering characteristics of the sediments on a countywide scale are not known. Therefore, it is difficult to predict whether a deposit will liquefy at a certain shaking level. However, because the peak ground acceleration with a 10 percent probability of being exceeded in 50 years is 0.35*g* or greater for the alluvial basins in Monterey County, it is likely that ground shaking levels that are strong enough to cause liquefaction are present in all the alluvial basins in Monterey County (Petersen and others, 1999). Detailed analysis of geotechnical borings would constrain which sediments were most susceptible to liquefaction at a specific shaking level. As recommended in CDMG Special Publication 118, the following areas are considered liquefaction hazard zones (CDMG, 1999):

1. Areas known to have experienced liquefaction during historic earthquakes.
2. Areas of encompassed fills containing liquefaction susceptible material that are saturated, nearly saturated, or may be expected to become saturated.
3. Areas where sufficient existing geotechnical data and analyses indicate that the soils are potentially liquefiable.

4. Areas containing young (less than 15,000 years) soils where there are limited or no geotechnical data.

The main criteria used in this study were items one and four. Estimated ages of geologic materials are included in the GIS database. Fill is not mapped on a countywide scale, so item two was not used. The scope of this study did not allow a complete review of the thousands of geotechnical boring logs contained in the Monterey County Planning Department library, so item 3 was used only locally. A more detailed study would include detailed analyses of the boring logs in the county files and in the incorporated Salinas Valley cities. An example of the liquefaction zonation is shown in figure 2. Note that figure 2 has a 1-mile-wide buffer zone on each side of U.S. Highway 101 to illustrate the use of GIS for highway and lifeline planning.

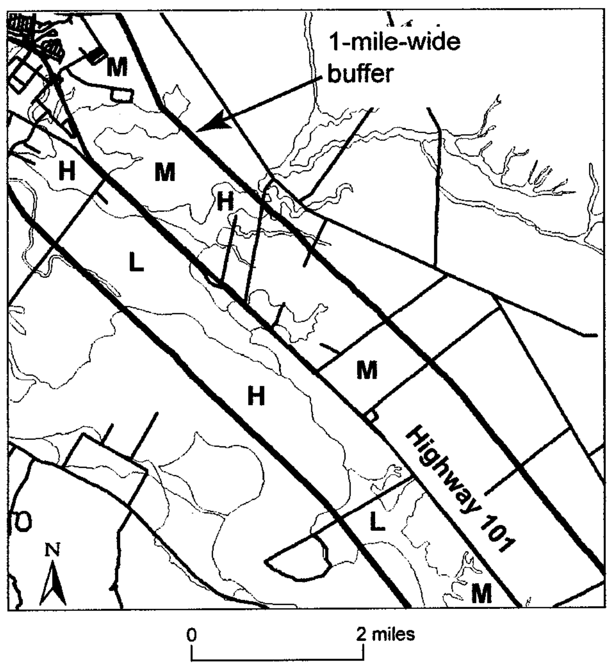

Figure 2. Liquefaction zonation along Highway 101 corridor, south of Salinas.

Landsliding

Landslides are common in Monterey County due to the combination of the rapidly uplifting mountains, locally fractured and weak rocks, and intense rainfall along the coast. Some of these landslides formed millions of years ago and are presently stable, whereas others are much younger and unstable. In general, existing geologic maps greatly underrepresent the distribution of landslides are in Monterey County because of their focus on bedrock geology.

The landslide database rates the relative earthquake-induced landslide susceptibility of Monterey County. Although rainfall-induced landslides are more common than those caused by earthquakes, the correlation between rainfall thresholds needed to initiate landslides in the various parts of the county is unknown, making it difficult to realistically model rainfall-induced landslides. Therefore, a model that uses fairly well characterized variables such as relative rock strength and slope inclination is more appropriate. The model for earthquake-induced landsliding developed by Wieczorek and others (1985) was used in this study with several assumptions.

The geologic strength data and slope inclinations were evaluated using a simplified version of the methodology developed by Wieczorek and others (1985) for their analysis of earthquake-induced landsliding in San Mateo County, California. Wieczorek and others (1985) used the Newmark method to estimate the amount of ground displacement of a landslide caused by an earthquake. Because of the lack of engineering data such as cohesion and shear strength for most of the rock units in Monterey County and the lack of near-field strong motion records (with the exception of the sparsely settled Parkfield district) it was not possible to calibrate the San Mateo County methodology for Monterey County. However, because many of the same geologic units and faults are present in both San Mateo and Monterey Counties, these simplifying assumptions are appropriate.

The model by Wieczorek and others (1985) divides rock units into three categories: category A (crystalline rocks and well-cemented sandstones), category B (unconsolidated and weakly cemented sandstones, and category C (shales and clays). In Monterey County, category A consists of granitic and metamorphic basement and hard Cretaceous sedimentary rocks; category B consists of Tertiary sandstones and Pleistocene alluvial fan deposits; and category C includes most of the Franciscan Complex, serpentinite, the Monterey Formation, the Pancho Rico and Paso Robles Formations, and unconsolidated Quaternary deposits. The ratings are subjective and based on similar classifications by Wieczorek and others (1985), descriptions of similar units by Ellen and Wentworth (1995), and professional judgment.

The landslide susceptibility database is based on analysis of geologic and topographic data. Geologic units in the geology database were reclassified into three categories based on their assumed strength characteristics. The vector strength polygons were then converted into a raster grid with a 200-foot cell size. Digital terrain data was from the U.S. Geological Survey National Elevation Dataset (NED), which has a 30-m resolution for Monterey County. Slope inclinations were derived from the NED files using ArcView Spatial Analyst software. The resultant values of geologic strength grid and the slope inclination were calculated for each cell and reclassified into the "high, moderate, and low" categories. Finally, the raster map was converted into a vector map. As a result of the raster-to-vector conversion, there were many areas of less than a few acres, giving the map a patchy appearance. To simplify the map, the data boundaries were smoothed using four passes of the ArcView "boundary clean" and "region group" tools. This modification slightly sacrifices accuracy for an easier to use map and is not a significant limitation for the countywide scale of mapping.

ACKNOWLEDGEMENTS

This project was made possible by the generous contributions of the veteran geologists that mapped Monterey County. I am grateful to Thomas W. Dibblee, Jr. for reviewing his monumental volume of work and developing a master correlation of the Tertiary and Cretaceous rocks. I also owe a large debt to Richard R. Thorup, who probably knows more about Monterey County geology than anyone else, for sharing his vast knowledge with me. John C. Tinsley, III (USGS), Carl Wentworth (USGS), Chris Wills (CGS), John D. Duffy (Caltrans), Steven Reneau (Los Alamos National Laboratories), Alisa Klaus (D&M Consulting Engineers, Inc.), and Lloyd

Oshiro shared unpublished geologic mapping. Annette Chaplin, Jared Ikeda, Doug Garrison, Rob Johnson, Gale Foss, and Ed Almanza (County of Monterey) deserve special thanks for facilitating this project and being helpful team members.

This project was funded by the County of Monterey, Environmental Resource Policy Department. This project utilizes previous research partly supported by the U.S. Geological Survey (USGS), Department of the Interior, under USGS award numbers 1434-94-G-2443 to Lewis I. Rosenberg and Joseph C. Clark and 1434-HQ-98-GR-0007 to J.C. Clark and L.I. Rosenberg. The California Geological Survey provided support to Rosenberg and Clark for fieldwork in the Spreckels quadrangle during 1998. The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Government or the State of California.

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GEOLOGY AND LANDSLIDE MAPPING ALONG HIGHWAY CORRIDORS IN CALIFORNIA: FACTORS INFLUENCING LANDSLIDE POTENTIAL

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ABSTRACT

Geologists with the California Geological Survey have mapped landslides along four highway corridors: Highway 50 in the Sierra Nevada, Highway 101 in the northern Coast Ranges, Highway 1 along the Big Sur Coast and Highway 60 across the San Timoteo Badlands of Riverside County. Additional studies are underway in the northern Coast Ranges and in the Transverse Ranges (Figure 1). For each highway corridor, we prepare a geologic map and a landslide inventory map, then prepare a map showing those landslides most likely to affect the highway and descriptions of the types of movement that could be expected. Bedrock geology is complied from previous maps and modified, commonly with Quaternary geology and landslides added. The landslide map classifies each landslide by type, recency of activity, and confidence of interpretation. Our mapping shows that the number, activity, and type of landslides is related to the underlying geology, climate and slope steepness.

Along Highway 50, igneous and metamorphic rocks have been weakened by deformation and deeply weathered. Active landslides occur along or near boundaries between different rock types and where partings and shears in bedrock parallel the slope, causing out of slope movement. In the Highway 101 corridor, Franciscan complex bedrock includes "broken formation", hard blocks of sandstone separated by weak beds, and shear zones and "melange", weak sheared clay and shale. Broken formation has large, deep masses of rock that slide on narrow zones of weakness. Melange typically fails as earthflows. The short Highway 60 corridor crosses very steep hills of poorly consolidated sands and gravels. Few large slides occur in these materials, but major storms in 1938, 1969 and 1998, each triggered hundreds of small debris flows on the slopes near the highway.

On The Big Sur coast, uplift of the Santa Lucia Mountains and continuing wave erosion has formed precipitous slopes in many units. South of Hurricane Point, wave erosion of weak rocks undermines harder but faulted and fractured rocks on a steep slope. In Big Sur, deeply weathered metamorphic rocks of the Sur complex are susceptible to debris flows when heavy rains follow a wildfire. The bedrock at Julia Pfeiffer Burns State Park is relatively landslide-resistant, but failed in response to the extraordinary rainfall of 1983. At Lucia, the rocks are so weak that the wave erosion at the base of the slopes and typical rainfall is sufficient to cause sliding. South of Pacific Valley, weak seams of serpentinite cut the weak bedrock, leading to a concentration of large, active landslides.

GEOLOGIC SETTING OF HIGHWAY LANDSLIDES

California is divided into 11 geomorphic provinces with distinctive geologic characteristics, including bedrock types, amount and type of folding or faulting, as well as landforms (Jenkins 1938, Wagner, 1997). One aspect that is common in nine of the 11 provinces is landsliding. The mapping of landslide hazards by the California Geological Survey for Caltrans began in response to the 1997 Mill Creek Landslide, which closed Highway 50 across the Sierra Nevada for 27 days. Following mapping of the Highway 50 corridor, CGS has mapped highway corridors in the Northern Coast Ranges, Peninsular ranges and Transverse Ranges. The geologic composition and deformation of each of these areas leads to differing styles and severity of landslide hazards as discussed below.

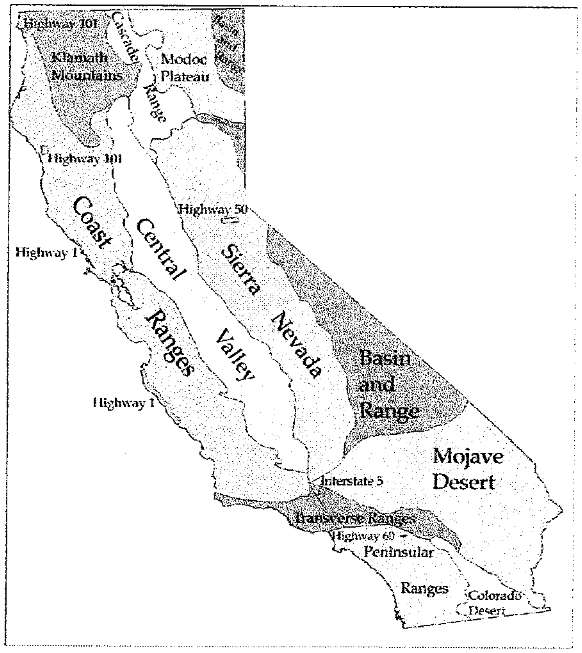

Figure 1. Geomorphic provinces of California showing the location of highway corridor landslide studies that have been completed or are underway.

The Sierra Nevada is a tilted fault block nearly 400 miles long. Its east face is a high, rugged multiple scarp, contrasting with the gentle western slope (about 2°) that disappears under sediments of the Great Valley. Deep river canyons are cut into the western slope. Their upper courses, especially in massive granites of the higher Sierra, are modified by glacial sculpturing, forming such scenic features as Yosemite Valley. The metamorphic bedrock (still partly capped by Tertiary volcanics), contains gold-bearing quartz veins of the Mother Lode system; a north-south structural trend is predominant in the western flank and northern end of the Sierra.

Highway 50 crosses the Sierra by following the canyon of the south fork of the American River. The corridor that we studied includes the metamorphic rocks of the Sierra foothills and the granitic rocks of the High Sierra. Landslide hazards are related both the deeply weathered mafic metmorphic rocks, and to previously unmapped areas of granitic grus that could be the source of debris flows.

The Coast Ranges are mountain ranges and valleys that trend northwest, subparallel to the San Andreas Fault. The province terminates on the east where strata dip beneath alluvium of the Central Valley; on the west at the Pacific Ocean with mountains rising sharply from the coast, and on the south at the Transverse Ranges. The Coast Ranges are composed of thick late Mesozoic and Cenozoic sedimentary strata. The San Andreas fault extends more than 600 miles through the Coast Ranges and to the south, from Pt. Arena to the Gulf of California. The Salinian block to the west of the San Andreas has a granitic core.

South of Cape Mendocino, the province is characterized by northwest-trending mountain ranges and valleys bounded by right lateral strike slip faults. North of Cape Mendocino, active subduction of the Juan De Fuca plate beneath the North American plate leads to active compressional tectonics and uplift. Partly because of this change in tectonics, the northern coast ranges do not have the broad northwest-trending valleys that make natural transportation corridors in the southern Coast Ranges. Highway 101 north of Eureka follows the coast and the valleys of several minor streams but is forced to climb or traverse numerous steep slopes.

Rocks of the northern Coast Ranges are typically sedimentary rocks of Cretaceous through Tertiary ages. The most widespread unit is the Franciscan Complex, composed of graywacke sandstone, highly sheared shale and several other rock types including serpentine, greenstone (metamorphosed volcanic rocks), and chert. All of the rock types tend to be weak, sheared rocks or overlying unconsolidated deposits. Along Highway 101 in Del Norte County and Highway 1 in Sonoma County, wave erosion at the base of sea cliffs has maintained steep slopes in these weak rocks. Along Highway 101 in Mendocino County, incision by the Eel River results in similarly steep slopes.

Along the Big Sur coast, the tectonics are dominated by strike-slip movement on the San Andreas and San Gregorio, Sur- Nacimiento and Hosgri systems. Nevertheless, this area is noted for dramatically high, steep slopes, which rise from sea level to over 3000 feet within less than three miles. The Big Sur corridor along Highway 1 has a richly varied geologic composition, which has lead to an abundance of a wide variety of landslides.

Much of the Highway 1, Big Sur corridor is underlain by sheared shale and graywacke of the Franciscan Complex, as described above. The northern part of the Highway 1, Big Sur corridor is within the Salinian Block, a block of distinctive rocks is bounded by the San Andreas fault on the east and the Sur-Nacimiento faults on the west. In contrast to the areas underlain by the Franciscan complex, large areas of the Salinian block are underlain by granitic and high-grade metamorphic rocks. One of the more extensive areas of granitic rocks is the northern Big Sur coast. Metamorphic rocks of the Sur complex and overlying Cretaceous through Miocene sedimentary rocks underlie the remainder of the Salinian block along the Big Sur coast.

The Transverse Ranges are a complex series of mountain ranges and valleys distinguished by an anomalous dominant east-west trend, contrasting to the NW-SE direction of the Coast Ranges and Peninsular Ranges. The Cenozoic sedimentary section is one of the thickest in the world. The western limit of the province is the island group of San Miguel, Santa Rosa, and Santa Cruz. The eastern limit, within the Mojave Desert, includes the San Bernardino Mountains on the east side of the San Andreas Fault.

The Peninsular Ranges are series of ranges separated by longitudinal valleys, trending NW-SE, subparallel to faults branching from the San Andreas Fault. The trend of topography is similar to the Coast Ranges, but the geology is more like the Sierra Nevada, with granitic rock intruding the older metamorphic rocks. The province is generally characterized by northwest-southeast oriented mountain ranges and valleys bounded by major right lateral strike slip fault zones; the San Andreas on the east and the San Jacinto and Elsinore through the center of the province.

Highway 60 crosses the northern part of the Peninsular Ranges geomorphic province through a an area known as the San Timoteo badlands. This area is underlain by the upper Pliocene San Timoteo Formation, a thick sequence of weakly consolidated, nonmarine, siltstone, sandstone, and conglomerate.

Geologic mapping

For each corridor, we prepare a geologic map from compilation of existing geologic maps with additional interpretation of aerial photographs and field mapping. The available geologic maps commonly have major differences in the identification and location of geologic units, and very few show any detail in the Quaternary deposits or landslides, which are important for showing the materials on which the highway was constructed and its stability. To prepare a complete geologic map of the area, which includes both Quaternary and landslide deposits, we digitize many of the existing maps, splice the digital files together, and add our own interpretations and observations to fill in those areas that lacked complete geologic maps or where existing maps differed. Differences in mapping and nomenclature are generally resolved by using the most detailed source of mapping. Field mapping is commonly needed to resolve the differences between the sources of mapping, add detail of Quaternary units, and improve the accuracy of contacts between rock units. The resulting map is usually sufficiently detailed to present at

1:12,000 scale, although the size of the Highway 1, Big Sur corridor and limited time available resulted in a 1:24,000 scale map.

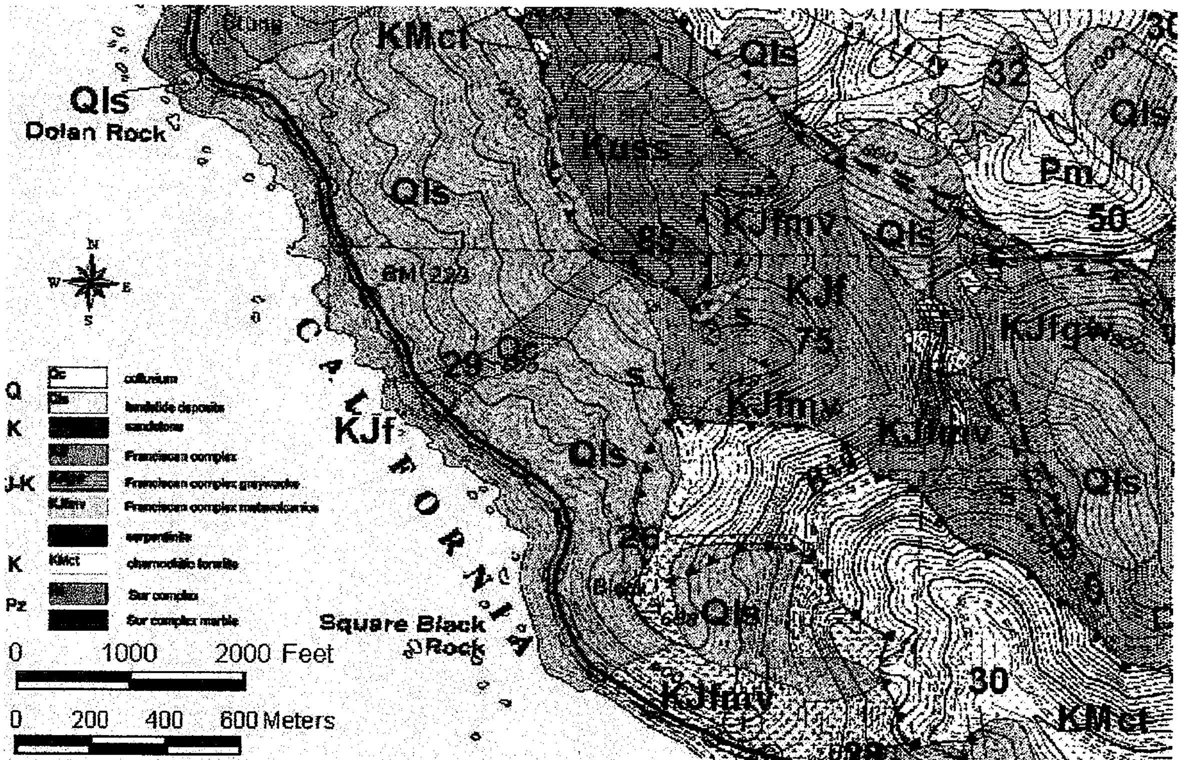

Figure 2, geologic map of part of the highway 1 corridor along the Big Sur coast

Landslide mapping

For each highway corridor, we have recognized, classified, and mapped landslides based on their morphology. Landslides displace parts of the earth's surface in distinctive ways, and the resulting landforms can show the extent and characteristics of the landslide. Recognition of these landforms (scarps, troughs, benches, and other subtle topographic features) allows the geologist to recognize, map and classify most landslides. Landslides were recognized by their topographic expression, as interpreted from topographic maps and aerial photographs, and seen in the field. Landslides shown on previous maps and in reports prepared by or for Caltrans, were checked on aerial photos and in the field, if possible. The boundaries of landslides from previous work were revised and additional landslides were added based on geomorphic interpretation for this investigation.

For each landslide we have attempted to record the characteristics of the slide, generally following the recommendations of Wieczorek (1984). Portrayal of landslides on the map includes a pattern, which designates the type of slide (materials and type of movement). The color of the slide area signifies its level of activity, and the thickness of the outline signifies the confidence of our interpretation as described below.

Types of landslides

Each landslide is classified according to the materials involved and the movement type, as deduced from the associated landforms. A two-part designation is given to each slide, based on the system of Cruden and Varnes (1996). Materials are called either rock or soil, and soil is

subdivided into fine-grained (earth) and coarse-grained (debris). This system was designed to allow a series of names that completely describes the materials and processes involved in a landslide. We have simplified the system slightly to use it in preparing an inventory map of an area. We use the terms and definitions of Cruden and Varnes (1996), but have attempted to simplify the designations by listing only the primary classification of a given landslide. For example, our diagram of a rock slide (see below), is a rotational rock slide-flow in which the upper part of the slide has moved by sliding, but the lower part has disaggregated and is flowing. On our map this type of slide is shown simply as a rock slide. Using the Cruden and Varnes system to classify rock versus soil is also complicated by the various vague and overlapping meanings of those terms in common usage. In California, many geologic formations are not hard or indurated rock and it is possible to find all gradations between weak, soil-like, and hard rocks. Our general system is to call material "rock" if it has a geologic formation name and the original geologic structure can be discerned.
Applying the system of Cruden and Varnes (1996), with the criteria described above, we generally describe six types of landslides as shown in figure 3.

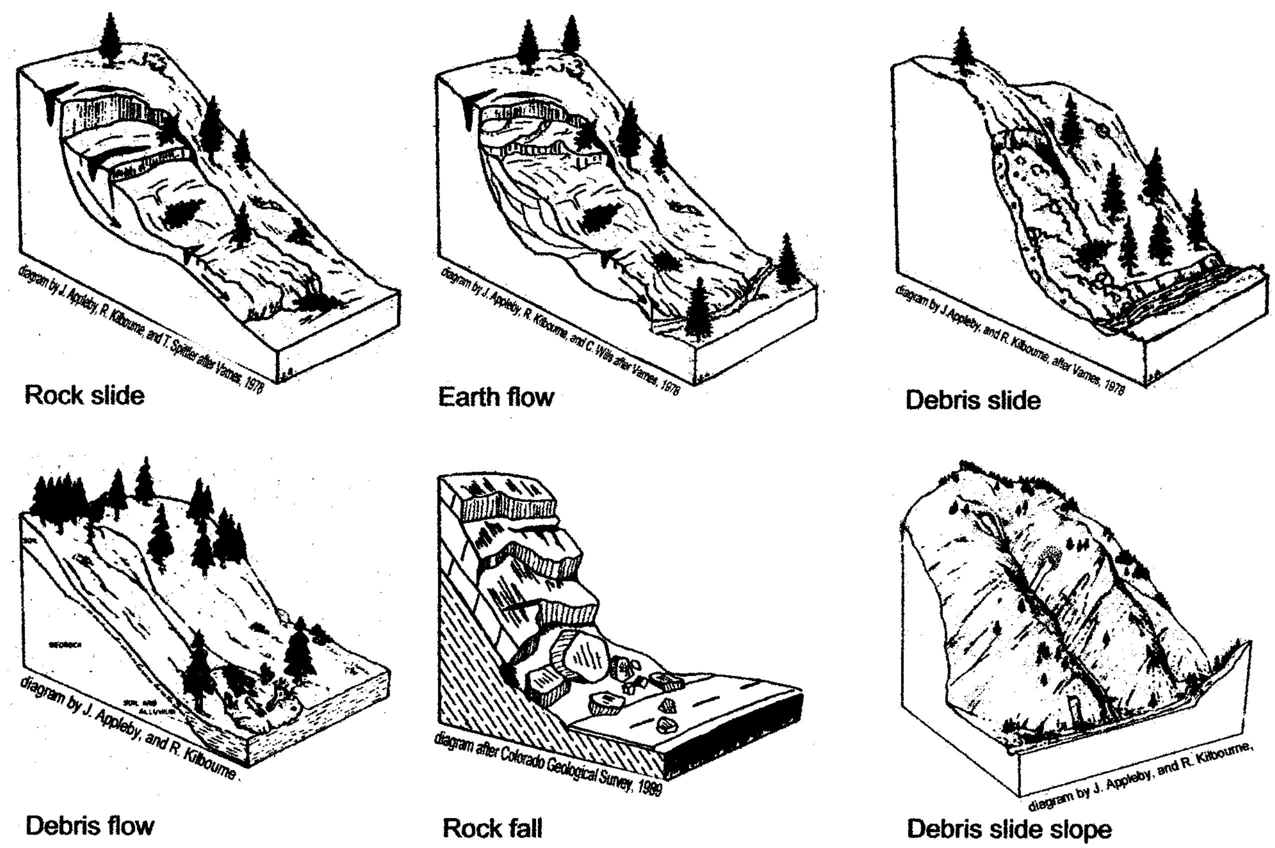

Figure 3. Diagrams of basic types of landslides, note that the example rock slide could be more completely described as a rock slide-flow and that the example earth flow is the type typically found in northern California; a deep composite slump-flow with numerous, ephemeral slide planes.

ROCK SLIDE: A slide involving bedrock in which much of the original structure is preserved. Strength of the rock is usually controlled by zones of weakness such as bedding planes or joints. Movement occurs primarily by sliding on a narrow zone of weakness as an intact block. Typically these landslides move downslope on one or several shear surfaces, called slide planes. The failure surface(s) may be curved or planar. In some older classification systems, slides with curved failure surfaces are commonly referred to as slumps, while those with planar failure surfaces are called block glides.

EARTH FLOW: A landslide composed of mixture of fine grained soil, consisting of surficial deposits and deeply weathered, disrupted bedrock. The material strength is low through much of the slide mass, and movement occurs on many discontinuous shear surfaces throughout the landslide mass. Although the landslide may have a main slide plane at the base, many internal slide planes disrupt the landslide mass leading to movement that resembles the flow of a viscous liquid. Earth flows commonly occur on less steep slopes than rock slides, in weak, clay-rich soils or disrupted rock units.

DEBRIS SLIDE: A slide of coarse grained soil, commonly consisting of a loose combination of surficial deposits, rock fragments, and vegetation. Strength of the material is low, but there may be a very low strength zone at the base of the soil or within the weathered bedrock. Debris slides typically move initially as shallow intact slabs of soil and vegetation, but break up after a short distance into rock and soil falls and flows.

DEBRIS FLOW: A landslide in which a mass of coarse-grained soil flows downslope as a slurry. Material involved is commonly a loose combination of surficial deposits, rock fragments, and vegetation. High pore water pressures, typically following intense rain, cause the soil and weathered rock to rapidly lose strength and flow downslope. Debris flows commonly begin as a slide of a shallow mass of soil and weathered rock. Their most distinctive landform is the scar left by the original shallow slide.

ROCK FALL: A landslide in which a fragment or fragments breaks off of an outcrop of rock and falls, tumbles or rolls downslope. Rock falls typically begin on steep slopes composed of hard rocks and result in piles of loose rubble at the base of slope.

DEBRIS SLIDES and DEBRIS FLOWS are commonly found on a landform called a DEBRIS SLIDE SLOPE, which represents the coalesced scars of numerous landslides that are too small to depict on a map of this scale. These landforms are generally very steep, and have developed in areas of weak bedrock mantled with loose, thin soils and covered with sparse vegetation.

Activity of landslides

Each landslide is classified based on the recency of activity into one of four categories based on the system of Keaton and DeGraff (1996) (Figure 4).

Active or Historic: The landslide appears to be currently moving or movements have been recorded in the past. The classification system of Keaton and DeGraff (following Varnes, 1978) uses the term active to mean active in the past year. We therefore combine active with the classification dormant-historic and use the designation "active or historic" on our maps. In order to be mapped as active or historic there must be records of movement of the slide or disruption of man-made features (roads, fences, etc).

Dormant-Young: Classifications besides "active or historic" are based on landslide geomorphology. For a landslide to be "dormant-young" the landforms related to the landslide must be relatively fresh. Cracks in the slide mass are generally absent or slightly eroded; scarps may be prominent but are slightly rounded. Depressions or ponds may be partly filled in with sediment, but still show phreatophytic vegetation.

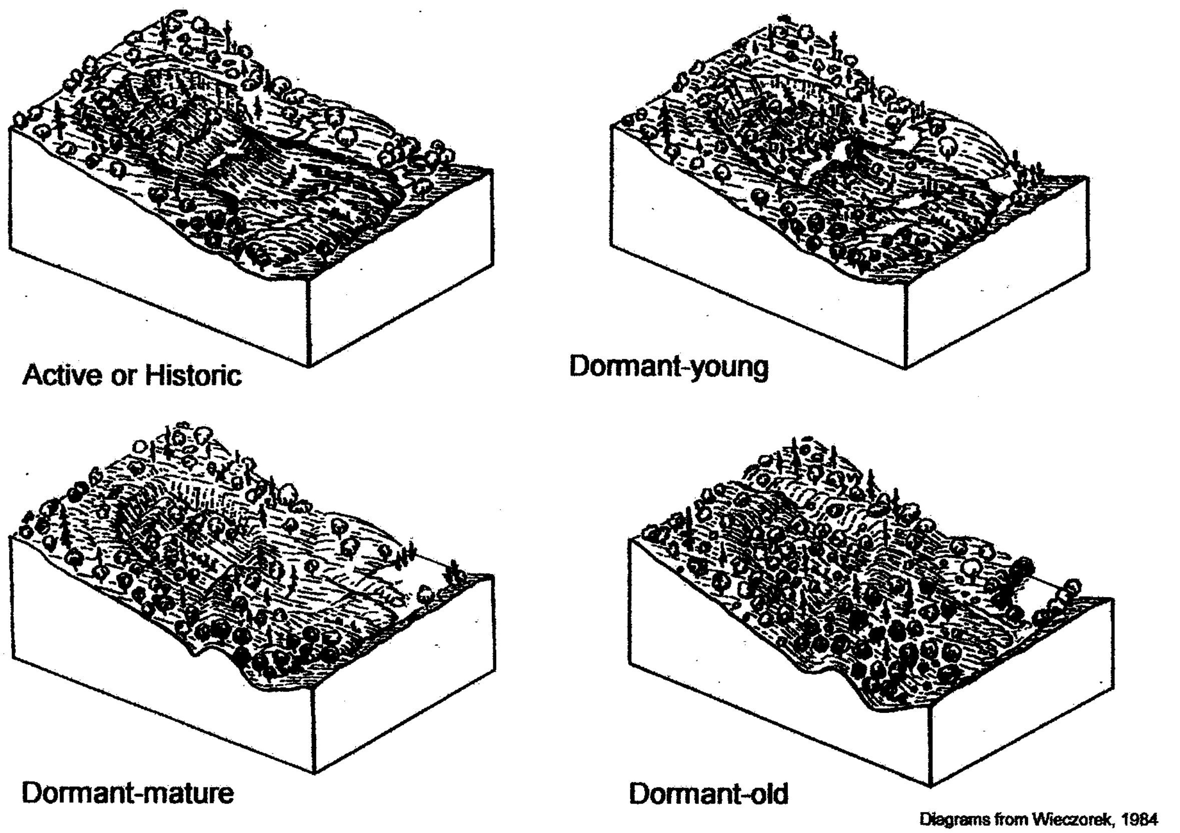

Figure 4. Diagrams showing progressive aging of landslide related landforms.

Dormant-Mature: In dormant-mature slides the landforms have been smoothed by erosion and re-vegetated. The main scarp is rounded, the toe area has been eroded and some new drainages established within the slide area. Benches and hummocky topography on the slopes are subdued and commonly obscured by dense, relatively uniform vegetation.

Dormant-Old: In dormant-old slides the landforms related to the landslide have been greatly eroded, including significant gullies or canyons cut into the landslide mass by small streams. Original headscarp, benches and hummocky topography are now mostly rounded and subtle. Closed depressions or ponds are filled in. Vegetation has recovered and mostly matches the vegetation outside the slide boundaries.

Confidence of interpretation

Each mapped landslide is also classified as a definite, probable, or questionable. Because landslides are mapped based on their landforms, the confidence of identification is dependent on the distinctness of those landforms. Confidence of interpretation is classified according to the following criteria:

DEFINITE LANDSLIDE. Nearly all of the diagnostic landslide features are present, including but not limited to headwall scarps, cracks, rounded toes, well-defined benches, closed depressions, springs, and irregular or hummocky topography. These features are common to landslides and are indicative of mass movement of slope materials. The clarity of the landforms and their relative positions clearly indicate downslope movement.

PROBABLE LANDSLIDE. Several of the diagnostic landslide features are observable, including but not limited to headwall scarps, rounded toes, well-defined benches, closed depressions, springs, and irregular or hummocky topography. These features are common to landslides and are indicative of mass movement of slope materials. The shapes of the landforms and their relative positions strongly suggest downslope movement, but other explanations are possible.

QUESTIONABLE LANDSLIDE. One or a few, generally very subdued, features commonly associated with landslides can be discerned. The area typically lacks distinct landslide morphology but may exhibit disrupted terrain or other abnormal features that vaguely to strongly imply the occurrence of mass movement.

Landslide maps

Each of our landslide maps depicts the three most important landslide classification categories (type, activity and confidence of interpretation) graphically. Our maps of highway corridors differentiate the type of movement by a pattern, the activity by a range of colors and confidence of interpretation by the thickness of the outline of the mapped landslide (Figure 5).

Each landslide is also classified by a number of other factors not presented on the map, but listed in the accompanying database table. The records in the database table include a unique number for each landslide and a listing of the quadrangle name. Other factors recorded for each landslide are: depth, described as shallow, medium or deep, direction of movement, primary and secondary geologic unit and lithology, area and perimeter.

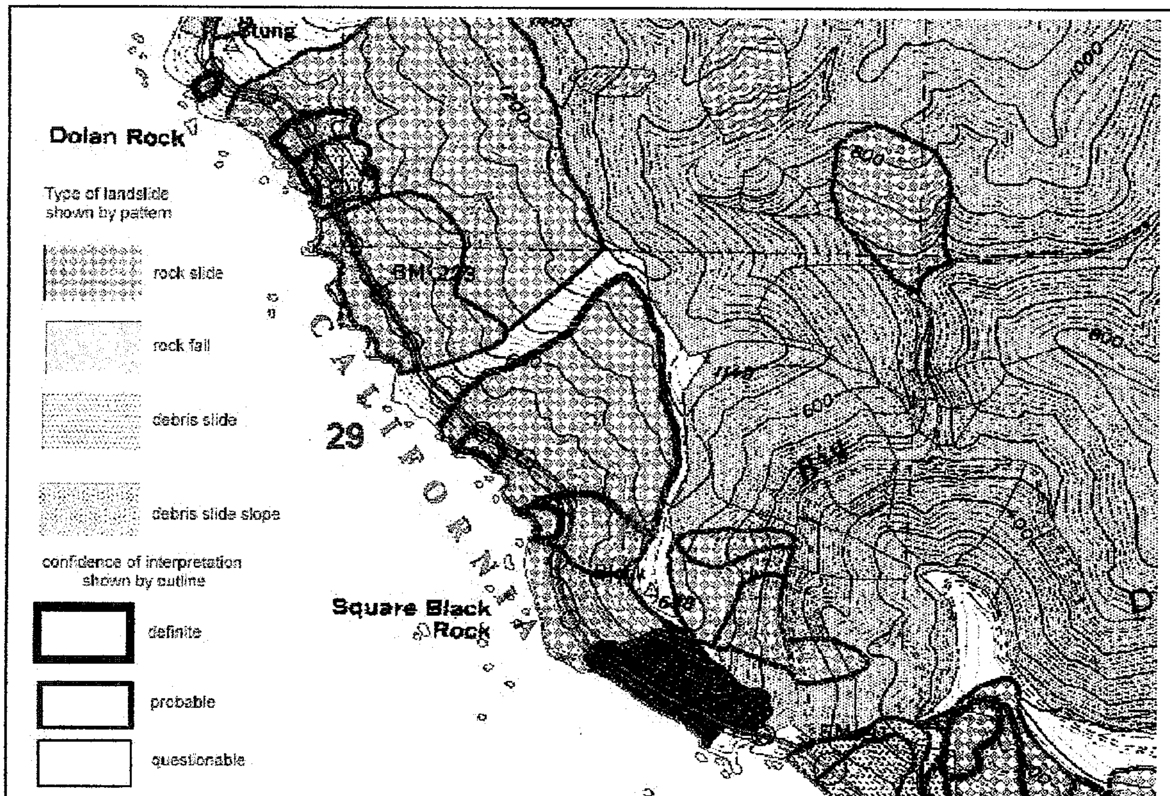

Figure 5. Landslide map of part of the Highway 1 corridor along the Big Sur coast.

Factors that influence landslide hazards along highway corridors

The inclination of slopes, their underlying rock types and geologic structures, landforms, rainfall, and modification of slopes for roadway or other construction all influence the slope stability along the highway corridors that we have mapped to date. Each of these factors influences the type and abundance of landslides, but the geologic materials probably have the greatest effect.

Slopes where we have mapped landslides range from moderate to extremely steep. The steepest slopes are along the sea cliffs along the Big Sur and Del Norte coasts. In Big Sur, some sea cliffs are as steep as 150% and as high as 400 feet. More typically sea cliffs are about 200 feet high and have about 100% slopes. Slopes to the crest of the ridge above the highway are not so precipitous, but many slopes as steep as 50 to 60 % extend to the ridge crests at over 2000 feet. Slopes at Last Chance Grade along Highway 101 in Del Norte County are almost 1000 feet high and average steeper than 60% slope. Most landslides on these very steep slopes involve shallow soil and loose rocks, moving as debris slides and rock falls.

Steep slopes are formed and maintained by the uplift of the mountains combined with wave erosion along the coast. Wave erosion removes loose rock deposited at the base and undermines the base of slopes, triggering landslides. The effect of wave erosion is greatest where steep high slopes extend upwards from the beach, without intervening marine terraces, and where weak rocks are found at sea level. This effect is especially clear in Big Sur on the southwest side of Sierra Hill where weak rocks of the Pismo Formation are exposed at sea level. Erosion of those rocks contributes to the instability of the harder rocks higher on the slopes.

Bedrock geology also has a very strong influence on the types and activity of landslides. The rock units that we have mapped range from massive, hard rocks with few fractures (notably the charnockitic tonalite and granitic rocks of Big Sur) to weak rock with pervasive shear surfaces and fractures (the Franciscan melange). Bedrock geology also has a very strong influence on the types and activity of landslides. Within the Franciscan complex, hard blocks of sandstone separated by weak beds and shear zones, tends to result in large, deep masses of rock that slide on narrow zones of weakness. Melange, consisting of weak sheared clay with little remaining rock-like structure, typically fails as earthflows. Even within areas mapped as melange, zones of serpentine appear to be weaker that the surrounding rocks, leading to large rock slides.

The charnockitic tonalite of Big Sur is less prone to large rotational landslides and forms very steep slopes along the coast. Those slides that have occurred historically, however, have been large and very damaging, notably the 1983 McWay (or J.P. Burns) slide. The granitic rocks on the northern part of the Big Sur coast, the quartz-diorite, granodiorite and granite, are similarly resistant to large landslides, though some slides are found in all units.

Even the weathering characteristics of bedrock units can be important factors in controlling the size and density of landslides. In Big Sur, the charnockitic tonalite, Sur complex gneiss and amphibolite, and the granitic rocks have significant differences in surface weathered zones. The depth of weathering is partly controlled by the original mineralogy of the rock units. Ross (1979) reports that the main mafic mineral in the granitic rocks and the gneiss is biotite, which makes up 12 - 13% of two samples of the quartz diorite and 8% of one sample of the gneiss. The charnockitic tonalite in contrast has only 1 to 5% biotite (Ross, 1979). Biotite typically weathers very rapidly to clay minerals, expanding and breaking down the rock as it does so. We observed that the granitic rocks and gneiss, but not the charnockitic tonalite, typically have deep zones of decomposed rocks at the surface, resulting in common shallow debris flows. The charnockitic tonalite, by contrast, usually has a very thin rubbly soil layer on the lower slopes and very few debris flow scars.

Precipitation is a major factor influencing landslides. But its effect varies across the state. The segment of Highway 101 in Del Norte County passes through one of the highest rainfall areas in California. According to the Oregon Climate Center the area averaged over 100 inches of rainfall per year between 1961 and 1991. The Big Sur segment of Highway 1 receives up to 60 inches of rainfall annually, up to four times as much as the Salinas Valley on the landward side of the Santa Lucia Mountains. The Highway 60 corridor through the San Timoteo Badlands, in contrast received averaged less than 15 inches of rainfall per year . The rainfall in this area, however comes as very intense storms of short duration. Although the average rainfall for any day of the year is less than 0.4 inches, extreme events may result in up to 4 inches of rainfall in 24 hours (Western Region Climate Center, 2000). These shorter term, but very intense rain storms tend to de-stabilize the shallower types of landslides, such as debris slides and debris flows.

Wildfires also contribute to the triggering of debris flows. In areas covered by dense chaparral, a hydrophobic soil layer may form after a wildfire, causing more rapid runoff and a greater potential for mobilization of debris from lower slopes and small channels. On the Big Sur coast, the effect of fire on debris flow potential has been most clearly shown in the Big Sur River watershed, where a fire in 1971 was followed by debris flows in 1972 (Jackson, 1977). Other areas with deeply weathered bedrock and colluvium, mainly the granitic rocks and Sur complex metamorphic rocks, also appear to be susceptible to the fire-flood-debris flow sequence.

The effect of wildfire on potential for debris flows in the Highway 60 corridor across the San Timoteo Badlands appears to be opposite of the expected pattern. Morton (1989) found that a hydrophobic soil layer formed in areas or the San Timoteo badlands burned by a wildfire in 1968, but that far *fewer* debris flow scars were found on burned than unburned slopes after the major storms of 1969. In this area of short, very steep slopes with relatively little debris stored in the channels, the faster runoff due to lack of vegetation and presence of the hydrophobic layer may decrease the potential for debris flows.

The landforms created by landslides, in some cases, help to perpetuate the slides. Closed depressions, troughs and benches that commonly form near the scarps of landslides allow increased percolation of water into the slide mass and along the slide plane, accumulate rainwater and destabilize the slide. Shallow debris slides may destabilize the adjacent upslope area when they move, leading to retrogressive failures.

Highway construction and maintenance across marginally stable and unstable slopes has also contributed to the triggering of new or renewed movement on landslides. Along older highways such as Highway 1, along the Big Sur coast, original construction of the highway left many steep cut slopes above the road. Blasting used during the original construction, left loose and fractured rocks on many steep cut slopes, which has contributed to rock falls and small debris slides.

Even when blasting was not used during construction, creating a cut slope in marginally stable material can trigger small slides. This type of sliding, including retrogressive failures, was probably extensive in the 10 to 20 years after construction of some older highways. More recently, repairs and maintenance have led to similar failures. Along the Big Sur coast at "Pitkin's Curve" (P.M. 21.5) a landslide below the highway led Caltrans to move the highway to the southeast, off of the slide. A cut slope was required to move the highway inland. That cut slope in weak, loose debris and weathered rock rapidly failed, leading to retrogressive failure of the entire slope above the cut to the ridge crest (Figure 4)

Other construction practices that can contribute to landsliding are the placement of fill onto an existing slide mass, and the directing of runoff onto loose materials or unstable slopes. Placement of fill onto landslides may increase the driving forces causing the slide to move. This is clearly happening along the Big Sur coast at the Willow Creek landslide (P.M. 11.8), where Caltrans has moved the highway to the east, so that most of the highway is off the landslide. The relatively flat area overlying the landslide has been used as a disposal site for debris from the Pitkin's Curve area mentioned above. The placement of that debris as fill over the landslide has increased the driving forces on the landslide and probably increased its rate of movement. We observed ridges of pushed up material and open fractures at the toe of the landslide, as well as fractures and scarps at roadway level, indicating active movement of the slide since the fill was placed in the spring of 2001.

Direction of runoff onto loose material or existing landslides is common along roadways in mountainous areas and requires very careful construction and maintenance to avoid. We observed numerous culverts in several of the highway corridors that were contributing to erosion and probably small landslides below the highway.

CONCLUSION

The California Geological Survey has completed geologic and landslide maps for over 120 miles of corridors along major highways in California. Mapping is underway in an additional 45 miles of highway corridor. For each corridor, we have classified landslides by type and recency of activity and described the possible consequences of landslide failure.

Within the varied geologic environments and terrain of California we find that several factors influence the type, numbers and activity of landslides. The physical properties of the geologic units are one of the most important among these factors. On the most basic level, soft, intensely sheared bedrock of the Fransican complex melange tends to fail as earthflows and weakly cemented sandstone of the San Timoteo formation tends to fail in debris flows. Geological details can have improtant influences on the type and activity levels of slides. On the Big Sur coast, large active and dormant-young rockslides are common in Franciscan complex rocks where there are more common shear zones with serpentine along them. Farther north, the Charnockitic tonalite has low biotite content and few debris flows but the granodiorite has higher biotite content and abundant debris flows.

Detailed maps of the geology and landslides along highway corridors in California allows the California Department of Transportation to consider the geologic hazards and potential landslides in planning for construction projects and contingency planning for landslides. Already corridor maps prepared by the California Geological Survey have been used in evaluation of repair or bypass options for a unstable portion of Highway 101 and in the long-term planning for landslide mitigation and repair along Highway 1 along the Big Sur coast. With the completion of maps for additional corridors we hope that we are increasing the ability of Caltrans to understand the geologic environments of landslides and thereby plan for and mitigate the hazards.

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I – 70 Corridor Programmatic Environmental Impact Study

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Abstract

The recent rise in Colorado's population has led to detailed studies for options to increase the transportation capacity of the I-70 corridor. This corridor is the primary access route through the mountains of Colorado for trucking industries as well as recreation and tourist activities. Yeh and Associates was responsible for performing the geotechnical, geological, and hazardous materials evaluation of the programmatic environmental impact study of the I-70 corridor from Denver to Glenwood Springs, Colorado. This project involved identifying and describing the geologic and soil conditions along the highway and the potential hazards and engineering constraints posed to the present and proposed highway system by these conditions.

The geologic hazards identified include; rock fall, debris flows, landslides, avalanches, collapsible and swelling soils, and the hazards related to past mining activity such as heavy metals in mill tailings and mine subsidence. The study area varied but generally encompassed the I-70 corridor and the ground to the ridgelines on either side. Through research of existing maps, aerial photos, and site-specific studies, a geologic hazards inventory map was produced.

The second phase involved observation of each geologic condition that affects the existing I-70 corridor. Preliminary field verification was conducted to verify the hazards and their impacts on current and proposed alignments. Evaluation included the history (cause, origin) and the current condition (size, materials, stability) of the hazard as well as any mitigation procedures previously performed. In addition, alternative routes for tunnels were investigated for future expansion of the highway and/or railway.

INTRODUCTION

The scope of the first phase of the project was to describe known, existing geologic hazards along the existing I-70 corridor from the intersection of C-470 in Denver to Glenwood Springs. The total project length is 144 miles. These hazards include rockfall, landslides, debris flows/ mudslides, avalanche chutes, collapsible soils, mine tailings piles, and mine subsidence. Swelling soils were including in the initial scope of the project, but were later discarded because none were found within the project area. The focus of the study includes the encompassing valleys as well as all the drainages on either side of the valley that contribute to these basins.

The second phase expands on the results of the first phase. Each geologic hazard that affects the existing I-70 corridor was observed and a preliminary evaluation was conducted in the field to verify the current condition of the hazards and their impacts on the current alignment. Three more geologic hazards were added during this phase: 1) rapid settlement, 2) potential for acidic and highly mineralized water in road cuts, and 3) potential for seismic activity. Evaluation included history (cause, origin), the current condition (size, materials, stability), and any mitigation procedures performed. It also included a preliminary field evaluation of risks associated with building a highway near or through each identified hazard area.

A geologic hazards inventory map of the entire I-70 corridor, identifying the location and rating of the hazards was developed from the literature review and field mapping. The map is on a topographic base with a scale of 1:24,000.

Following the field identification phase, a ranking was applied to the geologic hazards based on the following criteria:

- EXTREME (DO NOT DISTURB)
  Impact of hazard on highway is so great that any disturbance of the hazard for highway improvements is not recommended

- HIGH
  Impact of hazard on highway is great

- MEDIUM
  Impact of hazard on highway is moderate

- LOW
  Impact of hazard on highway is low

- NOT PROBLEMATIC
  Natural hazard does not cause a problem to current or future highway improvements to the existing alignment

In addition, possible mitigation alternatives were identified for each type of hazard identified along the corridor. This information can be used to assist in prioritizing projects along the corridor and for cost estimates for the required corresponding mitigation.

Mining Related Hazards in Clear Creek County

A separate investigation was done to identify mining-related hazards along the corridor. The study encompassed hazards such as unstable tailings, contaminated tailings, groundwater contamination, and airborne contaminants. Several individual sites were discussed and evaluated. Work in this investigation was limited to compilation of existing reports and studies. No new lab tests or fieldwork was done at this stage.

Alternative Alignment Evaluation

Yeh and Associates provided input on alternative alignments identified by the project team. Evaluation included the potential impact to identified geologic hazards, geotechnical design considerations and impacts on known mining hazards. In addition, alternative routes for tunnels were investigated for future expansion of the highway and/or railway.

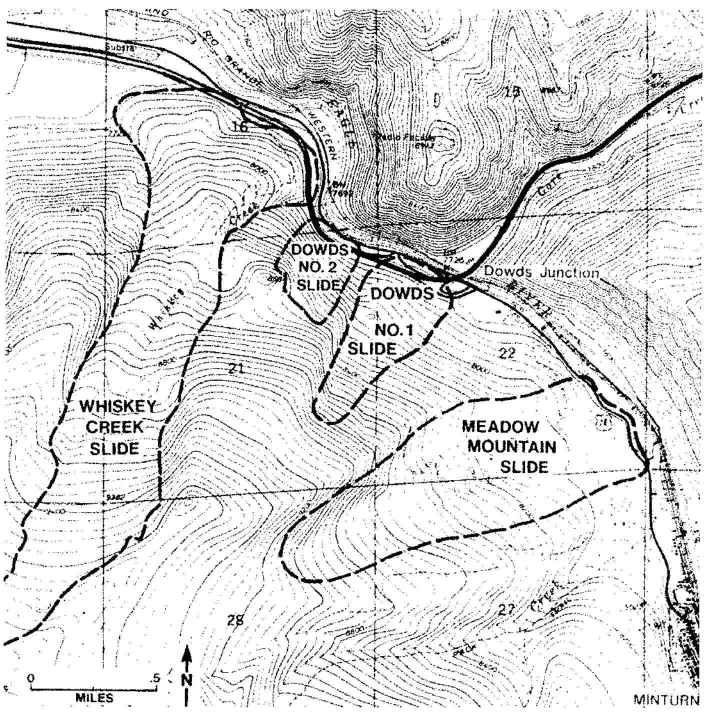

Figure 1. Dowd Junction Landslide Complex, (Jochim et al, 1988)

LITERATURE REVIEW

Data for this project was collected continuously. As data was gathered, it was added to topographic base maps. Sources of information include geologic and land use maps, research papers and publications, site-specific studies, construction and mitigation reports, field trip guides, aerial photo, and interviews with fellow engineers and geologists. Figure 1 is from the Colorado Landslide Hazard Mitigation Plan, one the many references used in the compilation of previous studies.

For information that seemed to have a high level of dependability, the geologic hazards were outlined and colored. For information that was questionable, it was given a question mark. Features that were very uncertain were labeled, but left uncolored. Boundaries for the study area were drawn to include the encompassing valleys as well as all the contributing drainages on either side of the valley. In order to keep the extent of the study area reasonable, only a short distance into each tributary was included. Towards the end of the first phase, aerial photos were consulted in order to verify the information gathered, as well as identify new features.

FIELD CHECKING

The second phase of the study involved field checking the results of the literature review. The staff geologist visually inspected each geologic hazard on the maps, verifying the type of hazard and adjusting the boundaries where necessary. More attention was given to accurately assessing the nature of the hazard than to defining the boundaries in detail. Features that did not concur with their original designation were changed. Sometimes features were eliminated, being reclassified as something other than one of the hazards defined in this study (i.e. alluvial fan, colluvium/ slope wash, soil creep).

"Undifferentiated Hazards" from USGS 1° x 2° map as described in Phase I are questionable areas. If a geologic hazard was located during field checking of these areas, the hazard would be mapped, and the "undifferentiated" designation removed. However, if no recognizable feature could be discerned, then the area was dismissed.

Geologic Hazards

A geologic hazard is defined as a geologic process that is a risk or potential danger to human life or property (Rahn, 1996). The varied and complex geology and geomorphic process has led to the development of the several zones of instability and marginal subsurface material. Although a natural process, these features can pose a risk to humans either directly by an encounter with the hazard or indirectly through effect of the hazard on roadways, railways or buildings. Conditions that may adversely affect the humans and/or the proposed improvements in the corridor include: faults, adverse rock structure, poor rock quality, and existing geologic hazards (debris/mudflows, rockfall, landslides, avalanche, and collapsible soils).

Landslides include the movement of both competent material (rock slides) and incompetent material (debris slides). Debris flows and rock fall are subcategories of landslides, but for clarity, were broken down into differing modes of failure. Landslides are distinguished from debris flows by the method and means for transport of the material downslope. Unlike flows, slides occur along a defined slip surface, and usually have a defined scarp at the head forming a step like appearance.

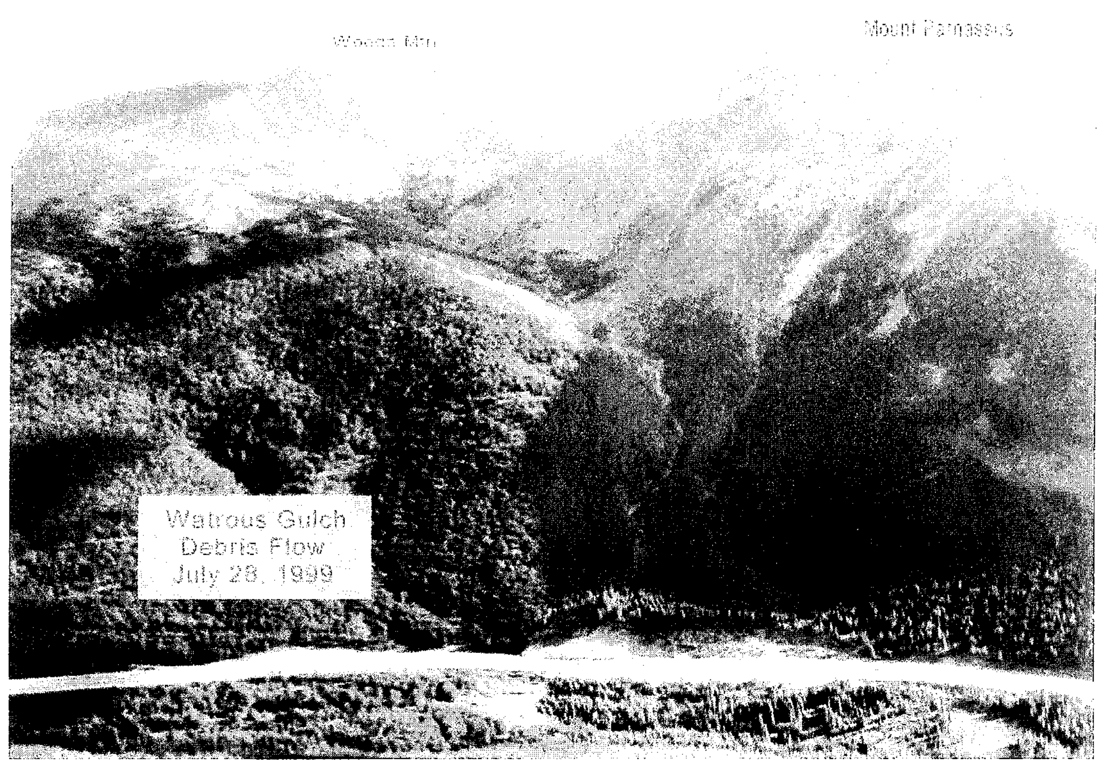

Figure 2. Watrous Gulch Debris Flow

South facing slopes along the Corridor are usually characterized by a lack of vegetation, and thin, dry soil. These conditions are more susceptible to smaller scale debris flows and rockfall. Landslides and large-scale debris flows are more common on north facing slopes that are characterized by dense vegetation cover, a thick soil mantle and higher soil moisture. The exception is The Watrous Gulch debris flow that occurs on the south facing slopes of Mount Parnassus (Figure 2). This flow originates from elevations in excess of 12,000 feet, which contributes to the extreme climatic conditions that cause all of the flows along the Corridor. The frequency of slope failures on north facing slopes is much less than that of south facing slopes for the majority of the debris flows, but the magnitude of any slide is much greater.

There are many conditions that can affect a slope and lead to rockfall and usually more than one factor contributes to a failure. For rockfall to occur there must be geological factors such as rock type, structure, and discontinuity characteristics. Different types of rockfall occur in differing geologic situations, as mitigation differs for different scenarios. The conditions that cause rockfall generally fit into one of two categories. One is the case where joints, bedding planes, or others discontinuities are the dominant structural feature of a rock slope. The other is the case where differential erosion or oversteepened slopes are the dominant condition that causes rockfall (Andrew, 1994). The most active rockfall area identified in the Corridor occurs along the Georgetown Incline (Figure 3). The frequency of rocks impacting the existing highway can occur daily.

Figure 3. Georgetown Incline Rockfall Area

Avalanches in the study area occur where a large mass of snow or ice moves rapidly down a mountain slope. Avalanche chutes appear as elongate, narrow, barren scars on a mountainside normal to the strike of the valley. Often, a fan-shaped deposit is found at the base of an avalanche chute. Other avalanches can form during periods of high wind causing the rapid accumulation of snow on cliff faces. The most active avalanche zones occur near the east and west approaches to Eisenhower Tunnel.

The geologic structure, slope configuration, precipitation, wind, and extreme temperature fluctuations

Figure 4. Avalanche Chutes, West Portal Eisenhower Tunnel

all contribute to geologic hazards along the Corridor. The climate includes wetting and drying, precipitation, freeze-thaw, and snowmelt. The slopes can be highly susceptible to erosion with little vegetation cover on most of the slopes.

Additional hazards were identified and mapped in the second phase. These include 1) rapid settlement, 2) potential for highly mineralized water in road cuts, and 3) potential for seismic activity. The latter two were mapped, but were not field-checked. Several mining-related hazards were researched, but were not mapped. These include Potential for Mine Blowouts, Water Contamination from Mine Tailings & Tunnels, and Airborne Particulates from Tailings.

RATING SYSTEM

This third phase of the project was done concurrently with the field checking. It involved rating each geologic hazard based upon its impact on the existing Highway I-70 alignment, specifically how it affects future improvements to the highway. The Colorado Rockfall Hazard Rating System (Andrew, 1994) was used to identify and evaluate existing rockfall hazards. During the program, numerical ratings were given by maintenance personnel to quantify the frequency and/or size of the observed rockfall events. The scale ranged from 0 to 4, with 4 depicting the most frequent rockfall activity.

There are five degrees of severity that make up the overall geologic hazard rating system. EXTREME (DO NOT DISTURB), HIGH, MEDIUM, LOW, and NOT PROBLEMATIC. Each classification is defined below. A hazard should meet a majority, not necessarily all, of the criteria listed, and each criterion is weighted differently.

- EXTREME ("A")
  - Impact of hazard on I-70 is so great that any disturbance of the hazard for highway improvements is not recommended

- HIGH ("B")
  - Impact of hazard on highway is great
  - Visual evidence of very recent activity [*frequent recurrence interval*]
    - Landslides/ Rock Slides (LS): head scarps and slumps, young vegetation (small aspens, grasses), tilted fences or utilities, surface is hummocky

        - Debris Flows and Mud flows (DF): known recent flow activity, young or no vegetation, mud cracks on surface, usually no buildings present
        - Rockfall/ Talus Slopes (RF): highly fractured bedrock, rock face is at steep angle, talus on slope below, no catchment besides the highway, rockfall rating of "4" on maintenance records (Andrew, 1994)
        - Avalanches (AV): well defined avalanche chute that is deep and contains no trees, limited or no runout zone besides highway
        - Mine Subsidence (MS): known recent subsidence, mines developed in poor quality rock or unconsolidated material
        - Collapsible Soil (CS): none. Not a high-risk event.
        - Rapid Settlement (RS): all. No variation because only applies to a single location.
    - If failure,
        - Long-term loss of service to highway-impedance of full roadway, road not traversable or driver must stop
        - Immediate mitigation needed
    - No mitigation previously done

- MEDIUM ("C")
    - Impact of hazard on highway is moderate
    - Generally less than 500' from highway
    - Visual evidence of somewhat recent activity
        - LS: no visible scarps or slumps, vegetation is intermediate stages (tall aspens, some evergreen), utilities and fences stand straight, surface is hummocky
        - DF: Debris flow deposit still visible, young vegetation and thick colluvium on surface, only minor engineering structures present
        - RF: highly fractured bedrock, little or no talus on slope below, rock face at moderate to steep angle, limited catchment area, rockfall rating of "3" or "2" on maintenance records (Andrew, 1994)
        - AV: chute is not active every year, runout zone may or may not reach highway
        - MS: none. Either HIGH or LOW
        - CS: known recent collapse of soil, cracks in pavement
        - RS: none. Only HIGH.
    - If failure,
        - Moderate loss of service to highway, impedance to less than half of the roadway, driver must slow down
        - Long-term mitigation needed
        - Periodic maintenance required

- Some limited or partially effective mitigation may have been done in the past

- LOW ("D")
  - Impact of hazard on highway is minimal
  - Generally more than 500' from highway (except rockfall)
  - No visual evidence for recent activity
    - LS: no fresh head scarps, vegetation is mature (evergreens), surface is hummocky
    - DF: most north-facing slopes with long recurrence intervals (50-100 yrs)
    - RF: rock is not highly weathered or there is a good catchment area for debris, includes a rating of "1" on maintenance records (Andrew, 1994)
    - AV: sufficient runout zone, but suspended debris may reach the highway.
    - MS: all areas other than those listed in HIGH, good quality rock
    - CS: all areas that are not included in the MED category. Assume all has the potential to collapse.
    - RS: none. Only HIGH.
  - If failure,
    - Little or no loss of service to the highway, impedance of shoulder or less, no noticeable pavement damage
    - No mitigation necessary
    - One-time maintenance needed
  - Includes areas where extensive mitigation has been done which is mostly effective

- NOT PROBLEMATIC ("E")
  - Natural hazard does not cause a problem to current or future highway improvements to the existing alignment
  - If failure,
    - No loss of service to the highway
    - No mitigation necessary
    - No maintenance needed
  - Includes areas where extensive mitigation has been done that is highly effective
  - Rockfall rating of "0" or outcrops that have not been rated by The Colorado Rockfall Hazard Rating System.
  - Includes all unmodified "Undifferentiated Hazards" from USGS $1^{\circ} \times 2^{\circ}$ map
  - Includes areas that are blocked from the highway by a physical barrier (ridgeline, valley, negative difference in elevation, stream). In this case, the hazard can be within 500' of I-70.

- Debris flows cannot be classified as "NOT PROBLEMATIC" unless there is a physical barrier as described above. Theoretically, any of the debris flow channels could reactivate.

Mining Related Hazards in Clear Creek County

Mining hazards in Clear Creek County include groundwater and surface water contamination, contaminated tailings, airborne metal particulates, and the potential for mine blowouts. Work in this investigation is limited to compilation of existing reports and studies. No new lab tests or fieldwork will be done at this stage. Sources of information include CDOT, the Environmental Protection Agency (EPA), the Department of Natural Resources' Division of Mines and Geology, Colorado Department of Public Health and Environment (CDPHE), Colorado Geological Survey (CGS), and various local historical societies.

RESULTS

The length of I-70 within the project area was divided into 14 geologic domains. An assessment of the geologic hazards was done for each section, based on the data gathered during the study. In addition, each section was given an overall rating for the cumulative effect of all geologic hazards encountered in that particular domain.

Each mapped hazard was rated according to the rating system described above and entered in to the geographical information system ARCinfo. Information regarding the size, type of hazard, estimated recurrence interval and rating will also be recorded for each of the mapped hazards. The data them can be easily accessed for evaluating potential impacts future corridor improvements will have, and determining appropriate mitigation alternatives.

Other information provided included a road log for entire length of the corridor. The log highlights problematic areas that will have a significant impact on the design of future highway projects (DO NOT DISTURB and HIGH designations). Other sites are discussed in order to bring the reader clarity about the rating designation given to a particular hazard by listing the criteria that were met. This is only a listing of significant features and is not a complete listing of all the geologic hazards encountered within the field area.

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DETERMINING LANDSLIDE VOLUME INPUT TO THE COASTAL ZONE ALONG THE BIG SUR, CA COAST

Cheryl J. Hapke[1]

Abstract: Along much of the 200 km stretch of the Big Sur coastline in central California, the rugged Santa Lucia Mountain Range extends uninterrupted into the Pacific Ocean, and the Pacific Coast Highway 1 is cut into the steep coastal slope. Weak rocks, steep topography and seasonal high intensity rainfall provide ideal conditions for large landslides, which frequently block, undercut or otherwise damage Pacific Coast Highway 1.

The California Department of Transportation (Caltrans) is currently developing the Big Sur Coast Highway Management Plan in order to explore solutions to the problems faced with keeping the slide-prone Pacific Coast Highway open and safe while minimizing impacts to the shoreline and nearshore waters of the Monterey Bay National Marine Sanctuary. As a contribution to a developing management plan, and in order to advance the fundamental understanding of coastal processes along this stretch of coastline, the focus of this study is to identify the spatial distribution of chronic landslides, and to quantify the historic volume of sediment that enters the littoral system from slope failures.

Two sections of the Big Sur coast were chosen for an initial pilot study to develop and refine techniques of aerial photograph processing and data analysis. Using digital photogrammetry, Digital Terrain Models (DTMs) are created from stereo models generated from historical (1942) and recent (1994) photography. The generated topographic surfaces are then compared to quantify volumetric changes, and to identify the spatial distribution of these volume changes, as well as the location, number and size of historically active landslides.

INTRODUCTION

The 1997-98 El Niño brought very high amounts of precipitation to California's central coast, raising groundwater levels and destabilizing slopes throughout the region. A number of large landslides in the coastal mountains of Big Sur in Monterey and San Luis Obispo counties blocked access along California's Highway 1, closing the highway for several months. Large slope failures along the Big Sur coast section of Highway 1 occur frequently due to the steep topography and weak bedrock (Figure 1). Major landslides occurred in the severe winter months of 1972, 1978, 1983, 1989, and 1998 (Cleveland, 1973; Works, 1984). A large slope failure

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in 1983 resulted in the closure of Highway 1 for over a year for repairs and slope stabilization. Highway repairs as a result of the 1983 landslide cost over $7 million and generated a combined three million cubic yards of debris from landslide removal and excavations for highway realignment (Engellenner, 1984). In 2000, a year with near average rainfall, two landslides resulted in the closure of Highway 1 for weeks at a time.

Figure 1: Large-scale landslides are is common along the Big Sur coast and road closures from slides are a routine occurrence.

The California Department of Transportation (Caltrans) is responsible for maintaining this precarious highway corridor and providing prompt and safe access for both local residents and tourists. Prior to the 1998 storm season, a typical road opening measure involved some disposal of landslide material and excess material generated from slope stabilization measures on the seaward side of the highway. This disposed material, either directly, or indirectly through subsequent erosional events, was eventually transported downslope into the adjacent ocean waters. In addition to the landslides that initialize above the road, slope failures also occur on the steep slopes below the road. This natural process delivers sediment to the base of the coastal mountains where the material is eroded and dispersed by waves and nearshore currents. As a result, any coastal slope landslide, whether through natural processes or anthropogenic means, can result in sediment entering the littoral zone.

The waters offshore of the Big Sur coastline are part of the MBNMS (Figure 2), which was established in 1992. MBNMS staff became concerned with the Caltrans landslide disposal practices for three reasons: 1) National Marine Sanctuary regulations prohibit disposal of material within a Sanctuary or where it will enter a Sanctuary; 2) landslide disposal practices have the potential to bury shoreline habitat, converting marine habitats from rocky substrate to soft bottom; and 3) the disposal practices have the potential to increase nearshore suspended sediment concentrations, possibly impacting coastal biological communities. On the other hand, coastal landslides and streams naturally deliver sediment to the coast providing nutrients to the water as well as material for beaches.

This study characterizes the long-term sediment contributions from landslides along the approximately 200-km-long Big Sur coast in Monterey and San Luis Obispo counties from south of the Carmel River to San Carpoforo Creek (Figure 2). The geographic limits of the study correspond to that portion of the coast where the steep slopes of the Santa Lucia Mountain Range plunge uninterrupted to the Pacific Ocean. The primary research goals are to quantify the historic volume of sediment that enters MBNMS through coastal landslide processes and to assess the spatial distribution of volume losses and gains using historical and recent aerial stereo photographs. The results are essential for the development of a management plan for Highway 1 that strives to maintain this highly scenic corridor while minimizing adverse impacts on nearshore biologic communities. The focus on quantifying the long-term average annual sediment yield to the littoral system from coastal slope failures and examining the spatial distribution of volumetric losses will provide an increased understanding of the relationship between the geology of the region and the pervasive landslide processes.

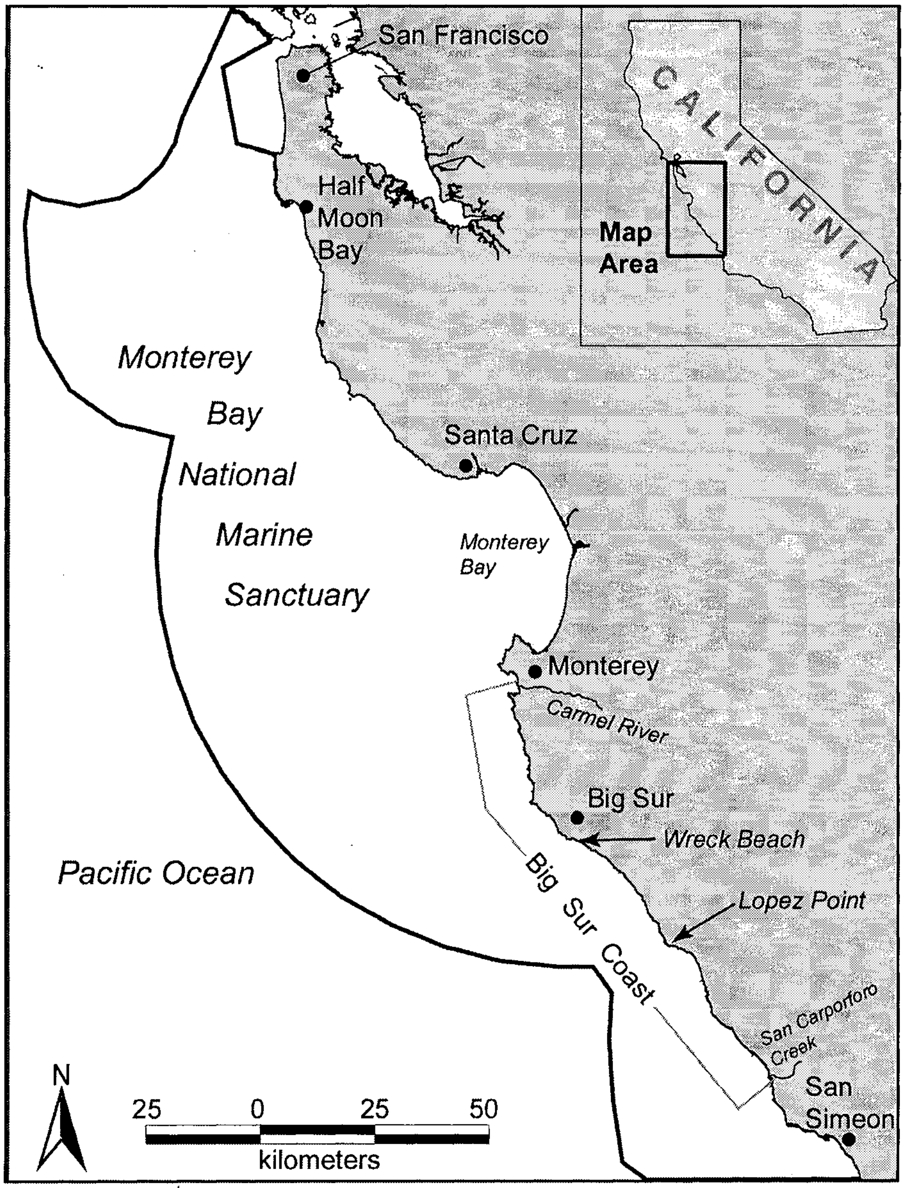


Figure 2: Map showing location of the MBNMS and the Big Sur coast in central California

STUDY AREA

The Big Sur coastline lies on the western boundary of the Coast Ranges geomorphic province, a northwest trending series of mountains and valleys flanking the coast from near Santa Barbara, CA to the Oregon border. In the Big Sur area, the Santa Lucia Mountain Range, part of the Coast Ranges, dominates the landscape.

The mountains rise from sea level, reaching elevations of nearly 1600 m within five km of the coast, making this one of the steepest coastal slopes in the conterminous United States.

The Santa Lucia Range is composed primarily of sheared and metamorphosed sedimentary rocks of the Late Jurassic to Miocene Franciscan Formation, and granitic and metamorphic rocks of the Sur Complex (Dibblee, 1974; Ross, 1976; Hall, 1991). The contact between these geologic units is the Sur-Nacimiento fault, part of the northwest trending San Andreas Fault system. The rocks of the Franciscan Formation tend to be weaker than those of the Sur Complex; therefore, the majority of the chronic landslides occur where the steep slopes are underlain by Franciscan Formation rocks. However, the lithology within the Franciscan Formation varies dramatically, and the softer, highly sheared rocks and mélange are more prone to landsliding while the various sedimentary strata and volcanic rocks form somewhat more stable slopes.

METHODS

The primary tools employed in this study are digital photogrammetry and GIS. Historical and recent vertical aerial photography are processed using digital photogrammetry to produce Digital Terrain Models (DTMs) from 3D stereo models. GIS is used to calculate volume changes and to assess the spatial distribution of the terrain changes. The historical photography is from 1942 (1:30000) and the recent photography was collected in 1994, at a 1:24000 scale. These photographs provide a base for determining a 52-year end-point volumetric change rate for two pilot study areas.

The process of digital photogrammetry requires a specific workflow that results in the production of orthophotographs (digital images from which all distortions have been removed). The distortions inherent in unrectified photography are those related to the camera system, the camera position, and displacements associated with the amount of terrain relief in the area (Falkner, 1995). Once these distortions are removed, the resulting images are orthorectified and can be used to make accurate measurements. In the processing, ground control points are used to tie the imagery to real-world coordinates. A DTM is required to completely remove the effects of relief distortion. The DTMs are built from the stereo images using a TIN

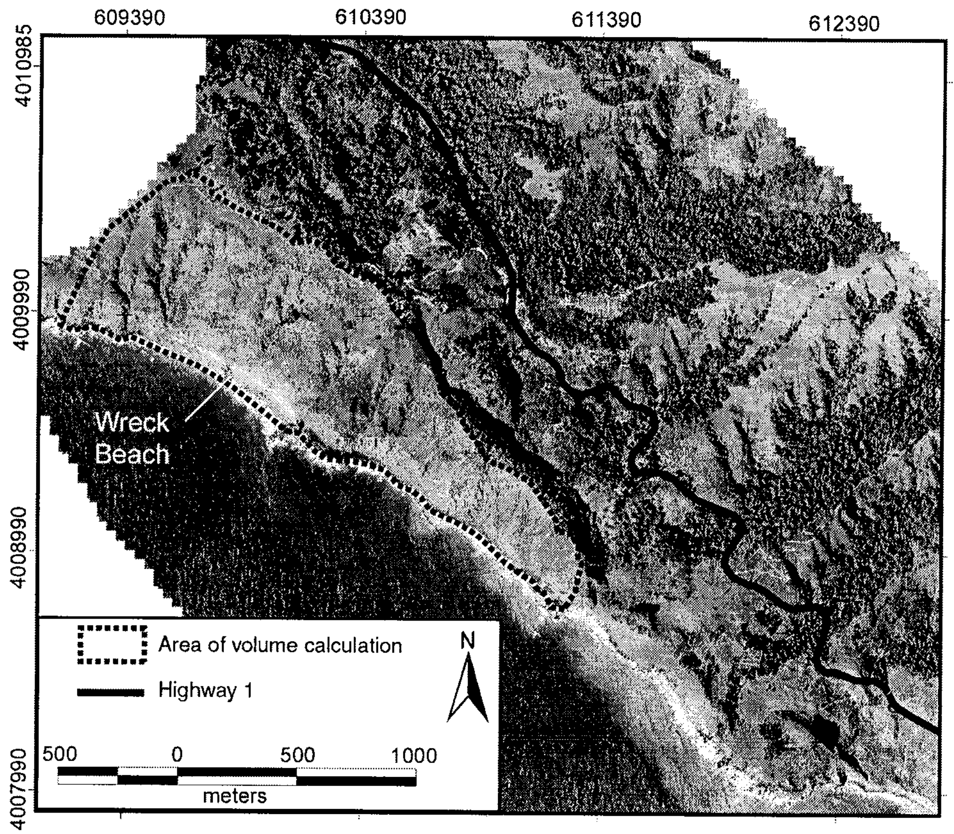
(Triangulated Irregular Network) of elevation points rather than a standard grid model in order to best capture the steep and rapidly changing topography.

Once the TIN models are generated, the topographic surfaces from the two dates are subtracted to produce an overall volume change, calculated from a datum of 1.5 m above mean sea level. The 1.5 m elevation represents the lowest elevation that photogrammetric stereo models can confidently derive elevation without significant visual interference from the movement of waves on the water or in the swash zone on the beach (Hapke and Richmond, 2000). Mosaics created from the orthophotographs are used in conjunction with the volumetric change to map locations and spatial distribution of historically active landslides. The orthophotomosaics are also used to determine whether the material has been completely lost to the littoral system or whether some volume is stored on land at the base of the slope. The downslope areas where deposition of landslide debris has taken place from the older to the more recent photos show up in the topographic comparisons as areas of volume gain.

RESULTS

Two pilot study areas were chosen in order to test and refine the processing techniques and data analysis for the study. These locations, near Wreck Beach and Lopez Point, are shown on figure 2. Both pilot areas are underlain by the Franciscan Formation but the Wreck Beach pilot area is underlain by more resistant sedimentary strata while the Lopez Point pilot area is located within intensely sheared metasedimentary rock and paleo-landslide deposits. Each pilot area was demarcated by identifying from the stereo photography those portions of the coastal slope that show evidence of active landsliding and where the eroded material would have a direct pathway to the ocean at the base of the slope. The two pilot areas are outlined on the orthophotographs in figure 3. The Wreck Beach pilot area is 1.3 km^2, and extends along approximately 2.5 km of coastline; the Lopez Point pilot area is 2.2 km^2 and extends along an approximately 5.2 km stretch of coastline (Table 1). Highway 1 passes directly through the Lopez Point pilot area, but is several hundred meters inland of the Wreck Beach pilot area.

The rate of volume change (net loss in both cases) is calculated over a period of 52 years. Since the pilot areas differ in size, the results are presented as m^3 normalized to the km^2 of each area (Table 1). As shown in Table 1, the volumetric



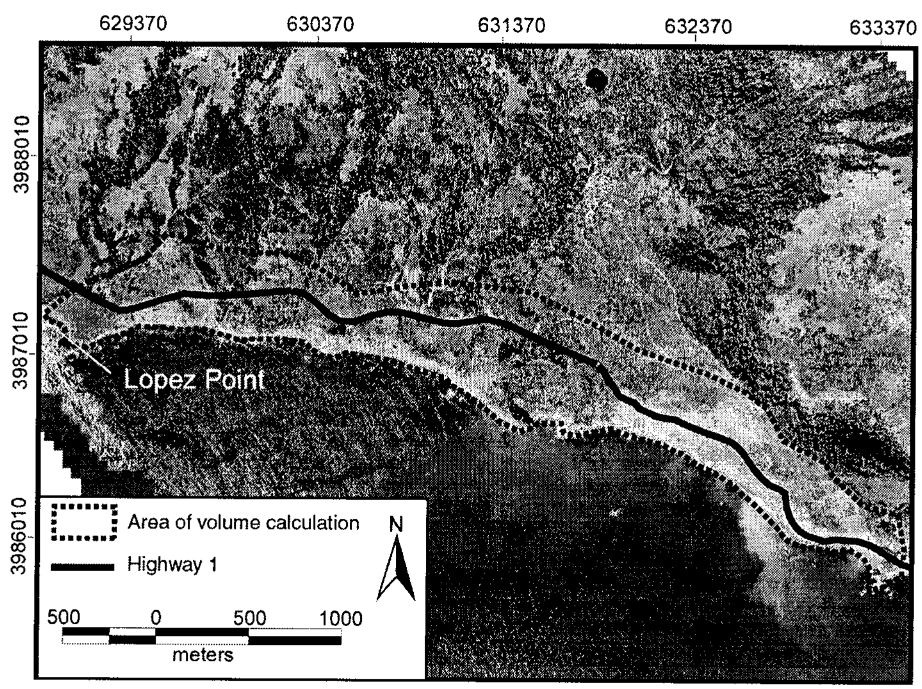


Figure 3: Orthophotomosaics of Wreck Beach (top) and Lopez Point pilot study areas (bottom). The areas within which volumes were calculated are shown as dashed polygons.

loss rate in these two areas differs by more than an order of magnitude. As described above, the geology differs dramatically in these two areas; the northern pilot area (Wreck Beach) is dominated by sandstones and graywackes while the southern pilot area (Lopez Point) is located in weaker materials, including ancient landslide deposits and sheared metavolcanic rocks (Hall, 1991). The existence of a significant embayment of the coast beginning at Lopez Point (figure 2) is further evidence that the material along this portion of the coastline is less resistant and erodes at a faster rate than the Wreck Beach pilot area to the north.

Table 1: Pilot Area Data

	Area (km^2)	Linear extent along coast (km)	Volumetric Loss Rate ($m^3/km^2/yr$)
Pilot Area #1	1.32	2.5	$2.8 \times 10^3 \pm 0.09 \times 10^3$
Pilot Area #2	2.17	5.2	$77.4 \times 10^3 \pm 3.6 \times 10^3$

Error Analysis

The error in the volume calculated for each date is a function of rectification errors, errors associated with the accuracy of the DTM and accuracy of the imagery based on the pixel resolution. The source of the rectification error is the error inherent to vertical aerial photography, including those associated with the camera system, the aircraft attitude, and topography. The rectification software incorporates these errors into the RMS error of the photogrammetric processing workflow. Error associated with the accuracy of the DTM is a function of the scale of the stereo photography (Ackerman, 1996). Finally, error associated with the pixel resolution is directly related to the resolution at which the photographs are scanned; this is simply the visual limitation of identifying an object (or location) that is smaller in dimension than the pixel size of the digital image. Using standard statistics, the error, or variance, associated with the TIN model for each date is determined by:

$$E_t = [(e_r)2 + (e_d)2 + (e_p)2]0.5 \tag{1}$$

where e_r = rectification error; e_d = dtm error; e_p = pixel resolution; and the subscript t is a given time, or date, from which the data are derived. This surface error is translated to an uncertainty in volume by assessing the calculated error over the area within which the volume was calculated:

$$E_{vt} = (E_t * A)/V_t \quad (2)$$

Where A is the area under which the volume was calculated and V_t is the volume calculated for a particular date. This equation produces a percent volume of the total calculated volume that is within the uncertainty range for that date of data. In order to determine the total error in the volume change calculation, the uncertainties for the two dates for a given area are summed:

$$Total\ error = E_{v1994} + E_{v1942} \quad (3)$$

For each pilot area, the uncertainty for the historic (1942) volume was higher than for the recent (1994) volume; this is primarily due to the smaller scale photography and the higher RMS from the rectification processing. The total errors in the volume change calculations associated with the pilot areas are 3.3% (Wreck Beach area) and 4.7% (Lopez Point area).

Contours of TIN Subtraction

In order to visualize the distribution of volume losses and gains in each pilot area, a contoured TIN model of the change (i.e. subtracted surfaces) is created. These contours are plotted on the draped surface of the orthophotograph so that the change can be visualized relative to the slope, to variations in slope, and with respect to Highway 1 (Figures 4a and b). The darker grays in figure 4 are areas of volume gain; the lighter grays represent areas of volume loss.

In Figure 4a (Wreck Beach pilot area), there is evidence that significant landslide material has been retained at the base of the cliffs rather than moved offshore and transported out of the area. Based on the low net volume loss in this area over the past 52 years, it appears that this section of the coast is relatively stable, most likely a function of the underlying sedimentary strata of the Franciscan Formation that are more resistant to erosion and slope failure than other members of the Franciscan Formation.

The volume loss distribution plot within the Lopez Point pilot area (Figure 4b) is dramatically different from the Wreck Beach pilot area, revealing a stretch of coastal slope that is very unstable and prone to landslides with little evidence that material is being stored at the base of the slope. This suggests that the bulk of the volume loss

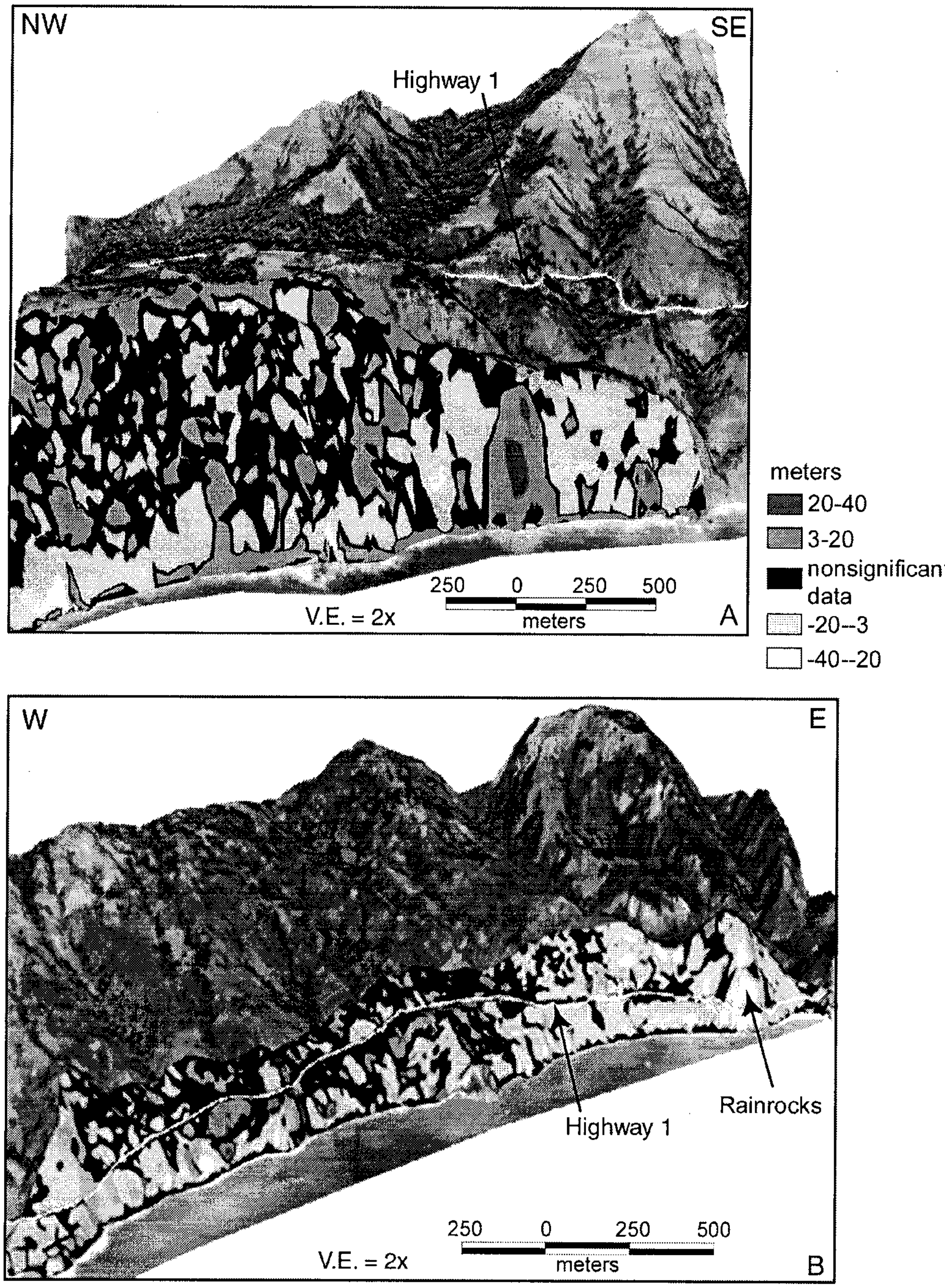


Figure 4: Areas of volume loss (light gray) and volume gain (dark gray) overlain on the draped orthophotographs of Wreck Beach (top) and Lopez Point (bottom) pilot areas. Black represents areas of nonsignificant data. Highway 1 is shown in white.

from upslope is transported offshore and removed from the area by nearshore waves and currents. The underlying geology in this area is the Franciscan Formation mélange and paleo-landslide deposits, both very weak and unstable units. The most active portion of the slope in the Lopez Point pilot area is below the road, with the exception of the region on the east end of the area, called 'Rainrocks' (figure 4b). Debris in this location almost continuously "rains" off the upper slope onto the highway and downslope below the road. Areas of volume gain shown adjacent to the road have been corroborated by Caltrans as being locations of debris stockpile and/or storage of material that awaits permanent disposal.

CONCLUSIONS

The long-term volumetric sediment contribution to the littoral system from coastal landslides can be quantified using aerial photography processed with digital stereo photogrammetry to produce TIN models of the terrain for different years. The TIN models can be subtracted from each other to quantify the net volume change for a given area; volume gains and losses can be plotted as contours of change to assess the change distribution. This technique of dynamic landscape modeling provides valuable information regarding the variability in magnitude and distribution of sediment entering the littoral system that can be useful for the management of landslide material in an environmentally sensitive and remote portion of the central California coast.

Pilot studies completed for two areas exemplify the dramatic variation in volumetric loss rates in an area where the complex geology results in a variety of lithologies being exposed along the coastal slope. Where the slope is formed in resistant interbedded sandstones and siltstones, the volumetric loss rate is nearly thirty times smaller than the volumetric loss rate in an area underlain by highly sheared metasedimentary rocks and mélange.

The results of this study will aid Caltrans and the MBNMS with the evaluation of existing practices of landslide debris disposal along the Big Sur Highway 1 corridor and the development of new management practices designed to minimize environmental impacts.

ACKNOWLEDGEMENTS

The authors thank the Caltrans staff, especially engineering geologist John Duffy, for sharing his extensive knowledge of the study area. This research is funded by Caltrans Coastal Highway Management Plan.

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GOING-TO-THE-SUN ROAD – PLANNING THE REHABILITATION OF A NATIONAL CIVIL ENGINEERING LANDMARK

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Historic Perspective

Going-to-the-Sun Road traverses the breathtaking scenery of Glacier National Park, allowing motor vehicle access to some of the most spectacular vistas in the country. Construction of the Road began in 1921. Given the alpine environment, the technology available, and the methods used, it was an awesome accomplishment to complete in 1932. In some places the Road is literally carved out of the cliffs. In addition, the engineers avoided the use of multiple switch-backs typically used in mountain road construction. Only one true switch-back occurs on the entire alignment of approximately 50 miles.

Due to its unique character and historical significance, Going-to-the-Sun Road was placed on the National Register of Historic Places in 1983, was declared a National Historic Civil Engineering Landmark in 1985, and was listed as a National Historic Landmark in 1997. Considerable improvements from the original 1932 roadway took place in the 1930s and 1950s, including asphalt paving of the entire Road in 1957. Prior to 1982, repairs and maintenance were limited and infrequent, and came entirely from the Park's operating budget. In 1982, Congress passed the Surface Transportation Assistance Act, which resulted in the establishment of a road improvement program through a partnership of the National Park Service and the Federal Highways Administration. Since that time, approximately $18 million has been spent on reconstructing primarily the lower sections of the Road. That leaves the upper Alpine Section, the most challenging portion of the Road yet to be addressed. The area's severe environment, increasing visitor use, and constraints of the originally-constructed alignment, have contributed to serious deterioration of the roadway cross-section, pavement condition, guardwalls, retaining walls, and drainage structures.

Directive for Study

In 1999, Glacier National Park adopted a General Management Plan to guide Park management through the next 20 years. An independent engineering study was conducted to evaluate the conditions of the Road and develop alternatives for cost effectively, rehabilitating the Road while preserving the historic character, fabric, width and significance of the Road, while minimizing the effects on natural, cultural, and scenic resources. The purpose of the engineering study discussed herein was to supply an Advisory Committee established by the Secretary of the Interior with alternatives for rehabilitation to make an informed recommendation to the National Park Service regarding rehabilitation of the Road.

Non-Engineering Considerations

Due to the historic nature of the Road, preservation of its character in terms of construction features such as the unique stone guard walls and retaining walls, and the "visual corridor" are of particular concern. Rehabilitation which would allow the Road to meet current standards for width, sight distances, guard rails, and curve radius cannot be done without significant and undesirable impacts to the historic qualities of the Road. During the engineering study, innovative approaches were sought to preserve, intact, the historic features, while providing the structural reconstruction needed to assure long-term integrity of the Road.

Another major consideration for evaluating potential rehabilitation strategies was minimizing negative impacts to the tourist-based economy of areas surrounding the Park. This entailed examining innovative approaches and construction techniques to minimize road closures and traffic disruption. While complete closure of the Road during construction would be the least costly, this option was not considered a viable alternative, due to the tremendous visitor, social, and economic impacts. Five road rehabilitation alternatives were presented in the study, with trade-offs for cost, duration of rehabilitation efforts, and level of traffic disruption.

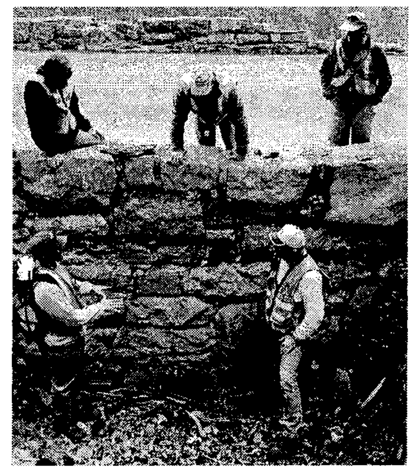
Process of the Study

The steps involved in the study included:

- Condition Assessment
- Engineering Analysis and Site Recommendations
- Development of Rehabilitation Alternatives
- Recommendations for Operation and Maintenance

This paper focuses on the condition assessment, engineering analysis, and site recommendations pertaining to geologic hazards and geotechnical issues. Development of the rehabilitation alternatives intensely examined the options for construction logistics and scheduling, and issues of geotechnical interest are also discussed.

While the condition assessment included review of available previous work and information, the greatest effort was consumed in field reconnaissance conducted in August 2000 and June 2001. The 2000 reconnaissance provided most of the information regarding road conditions. The 2001 reconnaissance defined the means and methods of rehabilitation, including a field review of constructability.



The condition assessment formed the basis of all following efforts by providing a road log of the conditions along the entire roadway. During the reconnaissance, the field team visited key features along the lower sections of the Road, and walked almost the entire eleven miles of the Alpine Section. Visual records of conditions were made via both videotape and still photos to supplement written field notes. A road log format was used to record the location and extent of deficiencies.

Following the field efforts, the project team met to consolidate observations, and to develop rehabilitation strategies and construction logistics.

Assessment of Geologic Hazards and Geotechnical Deficiencies

The assessment of geologic hazards and geotechnical deficiencies was completed as an "inventory" during the field reconnaissance, concurrent with and in coordination with observations of other Road deficiencies by other specialists of the team.

The concerns along the Road included several categories of geologic hazards and geotechnical deficiencies:

- Rockfall
- Unstable soil slopes above the Road
- Sloughing and erosion of soil slopes undercutting the Road
- Slump, landslide, and soil creep features
- Debris flows crossing the Road
- Avalanche chutes crossing the Road

Each of these concerns is described in detail in following sections, along with potential mitigation options and techniques.

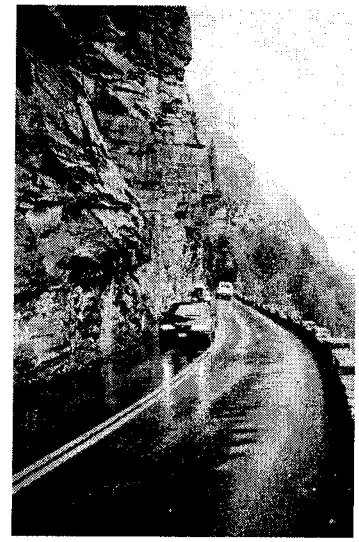
Other Concerns

Other significant (non-geologic) deficiencies included inadequate drainage on many parts of the Road. In many cases, poor drainage was identified as a contributing cause of slope erosion and sloughing undercutting the Road. Structural distress of retaining walls, arch bridges and stone culverts were often the cause of Road deterioration, or threatened integrity of the roadway.

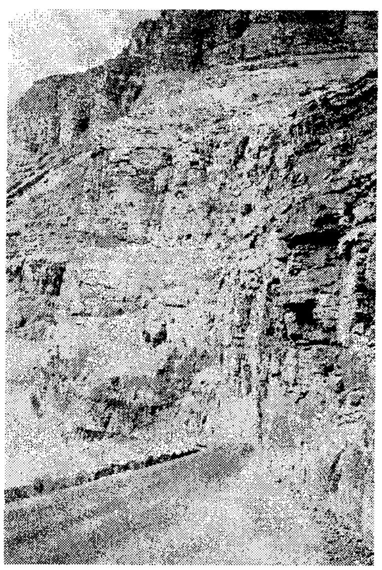
Rockfall

Overview of Conditions

Rockfall hazards exist, in varying degrees, over most of the 11-mile Alpine Section, and in a few areas and a lesser degree in the lower sections. Along many portions of the Alpine Section, the pavement extends completely to the toe of the rock cut, such that there is no shoulder or catch ditch. Many of the rock cuts are very steep and some even overhang the road. In some areas the rock cuts are below steep natural slopes that can potentially generate hazardous rockfall from hundreds of feet above the road.



The most severe area of rockfall concern is just below Logan Pass, where a quarter-mile section of the Road lies directly below steep cliffs that extend over a thousand vertical feet to the top of the mountain. This location has been the site of at least one fatal rockfall, and is regarded as a hazardous area by Park personnel. A foot trail along the cliff directly above the road exacerbates the potential for rockfall onto the road.



Hazards and Risks

The hazards from rockfall consist of rocks detaching from a cut or cliff above the Road, or boulders tumbling onto the road from natural slopes above the cut. The associated risks include rock falling onto the roadway, causing a traffic accident or directly hitting persons or vehicles. Also, a large rockfall might block the road, closing one or both lanes until it could be removed.

Potential Mitigation Measures

Suitable measures for mitigation could include one or more of the following:

- Selective scaling of loose rock from road cuts and the slopes above,
- "Mechanical measures" such as bolting, mesh, shotcrete, and rockfall sheds,
- Reconstruction of the Road to create stable rock cuts and catch ditches,
- Warning signs, restriction of parking and access, and visitor education regarding rockfall hazards.

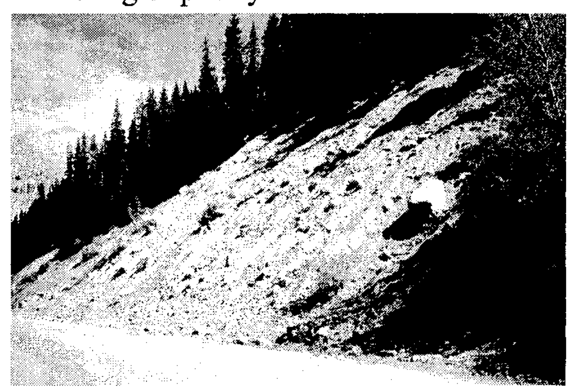
Reconstruction of stable rock cuts would be highly intrusive on the historical character of the Road, and was therefore not considered viable. Similarly, rockfall sheds are not compatible with the desired preservation of the Road's visual character. Other mechanical measures would be less intrusive, but still quite visible. Scaling and signage are the most favorable in terms of historic preservation. In essence, the more effective the mitigation, the more intrusive it would be on the visual experience of visitors and the historic quality of the Road.

Rock scaling was described as the most suitable compromise for providing a reduction in rockfall risk without significant intrusion on visual aspects of the Road. As a side benefit, rock material generated from scaling could be used in rehabilitation of guard walls and repair of retaining walls. The guard walls, retaining walls, and arch bridges historically constructed along many sections of the road are of stonework, primarily of local rock, and are considered key visual, historic features of the road corridor. Rock from a local source is highly desirable for repair of existing walls and construction of new walls or facings.

Unstable Soil Slopes Above the Road

Overview of Conditions

Soil slopes occur above the Road in almost all areas where rock cuts are not present. Only about 1.5 to 2 miles of the 11-mile Alpine Section has soil slopes immediately above the Road. Soil cuts are generally cut back adequately and are stable in the lower Road sections. Soil slope instability is generally in the form of raveling of poorly vegetated slopes, releasing boulders that can roll out of the cut and into the roadway. A secondary concern for maintenance is sloughing of soil onto the roadway. In most locations, soil cuts are small. In one location (near Siyeh Bend), approximately ¼ to ½ mile of soil cuts extend to 40 feet or more above the road. These cuts in glacial till are eroding and raveling badly, releasing boulders from a considerable height above the Road. At present, there is not an adequate catch ditch along the toe of the slope.



Hazards and Risks

The hazards from unstable soil cuts in the Alpine Section consist of rocks eroding out, being released from a cut, and tumbling into the roadway. The associated risks include rock rolling onto the roadway, causing a traffic accident or directly hitting persons or vehicles. Also, blockage of the Road could result from large boulders or a mass of soil entering the roadway.

Potential Mitigation Measures

Suitable measures for mitigation of unstable soil cuts include:

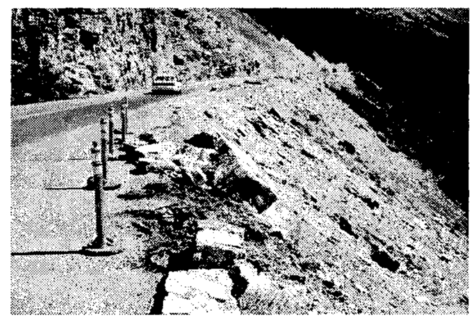
- "Scaling" of boulders from cut slope as they are exposed by erosion,
- Enlargement of cut to provide room for an adequate catch ditch or fence at the toe of the slope,
- Realignment of the roadway in affected areas to provide sufficient width at the toe of the existing cut for construction of a catch ditch or fence,
- Revegetation of the cut slope to retard erosion and further exposure of boulders.

Removal of boulders eroding out of the slope by "scaling" was not considered an effective long-term treatment to reduce the problem, since erosion would continue to expose new boulders and further deteriorate the cut. Cut enlargement is not desirable from the perspective of Road corridor preservation. One of the few locations where the existing road section is wide enough to allow low-impact realignment occurs at the Siyeh Bend cuts. The road template could be shifted away from the cuts to allow for a catch ditch. Alternatively, a catch wall constructed in the style of the existing historic guard walls could be constructed.

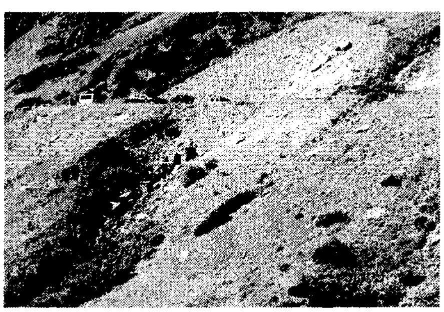
Revegetation of the cut has the advantage of retarding further deterioration of the slope. Given the steep aspect of these cuts and the high erosion potential of the soils, revegetation is expected to require special measures such as the use of revegetation matting to hold the soil while vegetation is established.

Sloughing and Erosion of Soil Slopes Undercutting the Road

<u>Overview of Conditions</u>



Where steep colluvial or fill slopes are present below the Road, erosion and sloughing of poorly vegetated slopes are "eating" back into the roadway section. In a number of locations, the pavement is being undercut by this phenomenon. Historic guard walls have been lost in some locations. The problem is most severe where road drainage discharges on the slope below the Road. At many locations on the Alpine Section, avalanches also contribute to slope erosion. Most important, in almost all cases the condition will worsen, with increasing threat to integrity of the Road, unless action is taken.



This condition is seen to occur on all sections of the Road where steep soil slopes are present on the downhill side, and is not just confined to the Alpine Section. The problem occurs primarily in localized, short road segments at widely distributed locations.

<u>Hazards and Risks</u>

When unstable or eroding slopes undercut the roadway template, deterioration or loss of the Road shoulder and/or pavement is at risk. The loss of the roadway shoulder has already resulted in the loss of some sections of historic guard wall, and more is threatened by the continued deterioration.

In addition, there is loss of structural support to the roadway. In many cases the loss of structural support cannot be replaced simply by reconstructing the fill or outer slope of the Road, but would require more complex measures. This is because many of the outer slopes drop steeply for hundreds of feet before finding a suitable "catch" for a fill toe.

In addition to the loss of roadway integrity, there is risk associated with public safety. Through most of the Alpine Section the Road width is considerably less than the 22-foot standard width that was established at the time of the original paving. Reduction of this already sub-standard width would increase the risk of drivers running off the roadway. In most locations, running off the outboard side of the Road could result in a vehicle tumbling hundreds of feet down the slope.

Potential Mitigation Measures

Suitable mitigation measures are dependent on the extent of deterioration present at individual locations. In many cases, road drainage must be redirected, captured, or controlled to retard further deterioration. Case-by-case mitigation measures may include:

- Where erosion is deteriorating the slope, but is not encroaching on the roadway template, erosion control measures such as revegetation, stone paving, pipe drops, or other armoring may be effective. Where revegetation is planned, erosion or revegetation matting will likely be needed to establish growth of vegetation,
- Where erosion is severe and is threatening the roadway template, more aggressive measures may be needed. Strategies may include soil nailing with an erosion-proof facing to prevent further loss of the cross-section,
- Where sloughing and erosion has already caused loss of the roadway template, and restoration of the cross-section is needed, retaining walls, mechanically stabilized earth (MSE) walls, tie-back anchored walls, or an anchored road slab may be incorporated.

Some of these measures, particularly the structural-type solutions, will have a greater impact on traffic during construction. For example, in most cases, conventional MSE wall construction would require closure of both traffic lanes on a schedule that could not be realistically done under traffic. Therefore, other measures that accomplish the same results, but can be done under traffic may be preferred. These alternative measures might include tie-back anchored walls or cast in place concrete retaining walls which can be done "over the side" with closure of only a single lane. The potential for non-standard design MSE walls was also considered, such as using one-lane-width embedment lengths, to allow construction of one lane at a time.

Slump, Landslide, and Soil Creep Features

Overview of Conditions

Several features are present along the Road which are recently-active landslides, slumps, or areas which appear to be subject to accelerated soil creep. Three of these are discrete landslide features of significant size. One lies along the shore of St. Mary Lake, where wave erosion is eroding the toe of the slide, causing further deterioration and expansion of the failure adjacent to the Road. Horizontal drains were installed in the slide previously, but wave erosion continues to exacerbate the problem.

A second area of slumping occurs along Lake McDonald. The area was previously reconstructed to restore the road template using a soldier-pile tie-back wall, but wave erosion is continuing to undercut the toe.

The third location involves a relatively large landslide mass. The boundaries and geomorphology of the feature are not easily discerned, but offsets at the slide margins across both lanes of pavement attest to continued movement. This feature does not appear to be a direct result of wave erosion (although it lies along Lake McDonald), but a slow-moving creep-type slide.

A relatively large area of what appears to be deep colluvium or ancient landslide deposits is present along approximately ½ mile of roadway west of Haystack Creek. No specific evidence of ground movement was observed, but these deposits may be subject to soil creep, particularly during wet periods.

Along much of the Alpine Section, cracking and evidence of settlement in the outboard lane was observed. This phenomenon appears to be associated with settlement of the fill which typically supports the outboard side of the road template. In many cases the fill settlement appears to be associated with poor road drainage.

Small slumps appear to be developing at a few isolated locations along the Road, evidenced by crescent cracks in one or both lanes of the pavement.

Hazards and Risks

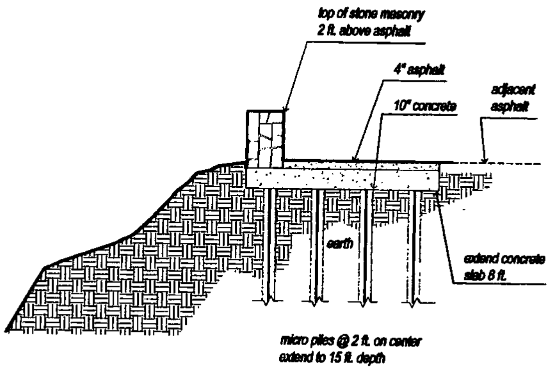
Hazards associated with landslide, slump, and creep features are mostly concerned with the loss of integrity and support to the roadway. Creep-type behavior, due either to soil creep or slow-moving landsliding, will continue to damage pavement, but is not likely to result in a catastrophic failure that would cause massive damage or a hazard to the public, unless driven to sudden large displacements by extreme wet conditions.

Large and small slump features could be prone to sudden movement, depending on the available trigger mechanisms. For most of the suspected slump features, this might consist of unusually wet seasonal conditions. For the two slides along the lake shores, storm events which precipitate high winds and/or increased lake levels could also provide a trigger for sudden movement. At any of these locations, a sudden failure could result in a complete road closure until repairs can be made. Slower, ongoing movement can be expected to cause pavement damage and loss of support to the roadway if not stabilized.

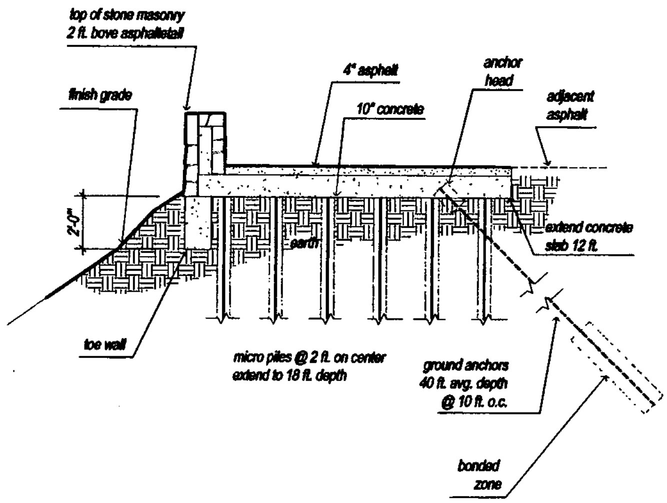
Potential Mitigation Measures

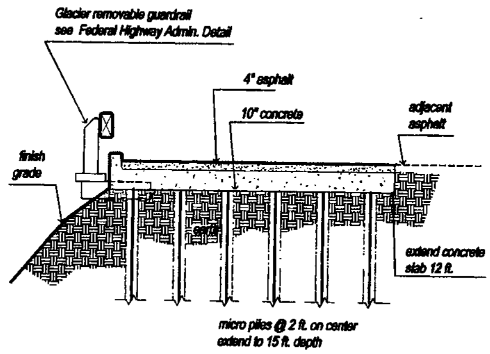
The three discrete landslide features described above require a suitably-scoped investigation specifically to evaluate conditions and define appropriate mitigation strategies.

Deteriorating road sections where localized slumping is developing could be rehabilitated by use of an anchored concrete slab. Treatment of deeper-seated slumps would include



tie-back anchoring of slabs. Several concepts for such techniques were developed during the field-reconnaissance. This “anchored slab template” approach is also proposed to reconstruct areas where the subgrade is deteriorated by water intrusion or subsidence to depths greater than four feet. (Damaged road sections due to unacceptable subgrade less than four feet deep would be restored by removal and replacement with suitable materials.)





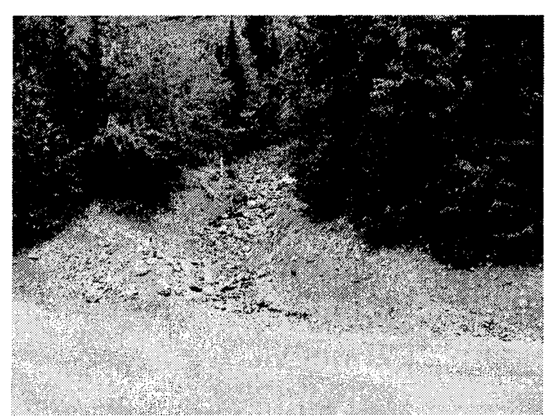
Debris Flows Crossing the Road

Overview of Conditions

Debris flows cross the road at a number of locations. On the west side of Logan Pass, these features are associated with the larger stream crossings. Here, debris tends to clog bridge and culvert underpasses below the Road, damaging these structures, and overflowing onto the Road in the larger events. East of Logan Pass, they occur where steep gullies cross the Road. In most of these locations, culverts carry water flows under the Road and become plugged during debris flow events, resulting in deposition of debris directly onto the Road. The debris flow hazards are found on the lower Road sections, rather than the Alpine Section.

Hazards and Risks

Debris flows constitute a considerable impact for Road maintenance. Significant debris flows will block and close the Road until they can be cleared. Potentially, Road users could be threatened by sudden flow of debris onto the Road.



Potential Mitigation Measures

Debris flows could be mitigated by placing catch-fence type structures or debris basins in the affected drainageways. However, these structures would have a relatively high visual impact in most locations. The potential exists to use warning devices or road closures triggered by debris flows.

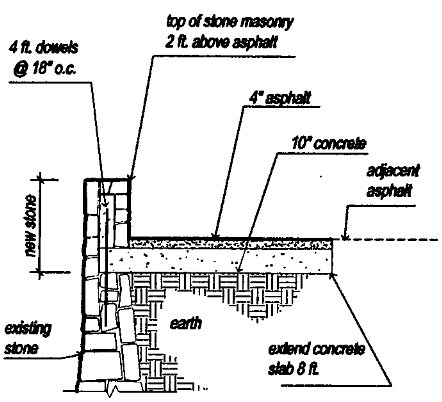
Avalanche Chutes Crossing the Road

Overview of Conditions

Avalanche runs are pervasive throughout the Alpine Section of the Road. Avalanches cause extensive damage in some areas to guard walls and retaining walls, and also contribute to deterioration of outboard pavement edges and slopes by erosion. In addition to damage from moving avalanches, guard walls are also subject to static snow loading pushing down slope and outward at the road edge.

Hazards and Risks

The probability of avalanches is relatively low during the seasons when the Road is open to visitor traffic. Somewhat greater hazards exist when Park personnel initially open the Road in the spring by plowing through large drifts. The impacts of avalanche and snow load damage on guard walls are a considerable contributor to deterioration of the Road's historic features.



Potential Mitigation Measures

In some areas, jersey barriers or removable guardrail is being used to replace guard walls that have been swept away. However, these solutions are at odds with the historical and aesthetic values of the Road. The potential for use of guard wall sections that are removable and are faced with historical-looking stone has been discussed. Perhaps more promising is construction of avalanche-resistant guard walls where avalanche chutes are present, like the ones designed by FHWA.

Status of Planning the Rehabilitation

Some rehabilitation work is currently on-going, as a result of the FHWA's involvement in the identification and rehabilitation design of the most critical areas in need of rehabilitation. Design and construction of the critical areas is expected to continue for the next few years. Inadequate drainage, identified as the single biggest cause of the deterioration of the Road, is currently being addressed by the Park as funding allows.

In November, 2001, the Engineering Study, along with the Transportation / Visitor Use Study and Socioeconomic Study, were presented to the Citizen's Advisory Committee. Presented were five alternatives for rehabilitation which varied based upon funding status and visitor impact. The Committee recommended four alternatives to evaluate in the NEPA environmental impact statement process. The four alternatives all included innovative construction techniques to rehabilitate the Road that consider the historic, cultural, and environmental factors, preservation of natural resources, and visitor impact. The range of alternatives present scenarios that range from a low funding level with a long-term rehabilitation (nearly 50 years) to a high funding level with a minimum term rehabilitation (approximately 6 to 8 years).

The environmental impact statement process was commenced after the November 2001 Citizen's Advisory Committee meeting and is expected to last approximately 18 months. The result of the environmental impact statement process will be an environmentally preferred alternative for the rehabilitation of the Road, with design and construction commencing shortly thereafter.

Snowmass Canyon State Highway 82 Project

Difficult Geological Conditions that Required Innovative Retaining Wall and Foundation Designs

Ben Arndt, P.E., Richard Andrew, P.G., Roger Pihl, P.G.
Yeh and Associates, Inc.

Abstract

Yeh and Associates has been involved with evaluation, analysis and design for improvement of four miles of Snowmass Canyon - State Highway 82, near Aspen Colorado. This section of highway is located in a complex geologic area with slopes in excess of 50^0. Yeh and Associates role on the project was to provide a geologic / geotechnical investigation, slope stability evaluation, and retaining wall design.

The geologic and geotechnical investigation utilized helicopter transported drilling rigs. The subsurface conditions were evaluated by drilling 289 borings with this equipment. Geologic hazards that were identified on the project included landslides, rockfall, and collapsible soils. The competency of some of the bedrock in the area was affected by faulting.

Ground anchor, caisson, and micropile testing was also conducted on the project to determine appropriate design parameters for the design for these structures. The roadway alignment was designed with both cut and fill retaining walls. The retaining walls for the project consisted of soil nail walls, mechanically stabilized earth (MSE) walls, ground anchor / tieback walls, and micropile foundation walls. Because of the steep terrain, MSE wall sections were over 40 feet in height and ground anchor and soil nail wall sections were over 25 feet in height. Inclinometers and piezometers were installed and monitored for earth movements and wall settlements.

Introduction

State Highway 82 in Colorado is the primary two-lane route between Aspen and Glenwood Springs. The highway follows along the general path of the Roaring Fork River that flows from Aspen down to the confluence with the Colorado River. The $93 million project was undertaken to expand a 3.4-mile section of the existing state highway from two lanes to four lanes, creating two lanes up valley and two lanes down valley. The up valley lanes are located on the slopes above the existing two-lane highway. The down valley lanes follow the general alignment of the existing highway. Figure 1 illustrates the general layout of the project area.

The expanded roadway section located in Snowmass Canyon is characterized by steep terrain and variable geologic conditions. In many areas the slopes are in excess of 45^0 and hydro-collapsible subsurface materials are located throughout the project. This paper addresses the geologic and geotechnical aspects of the project that controlled the wall design in the canyon. The paper also describes mechanically stabilized earth (MSE) walls, soil nail walls, ground anchor (tieback) walls, micropile foundation systems, and rockfall mitigation utilized on the project.

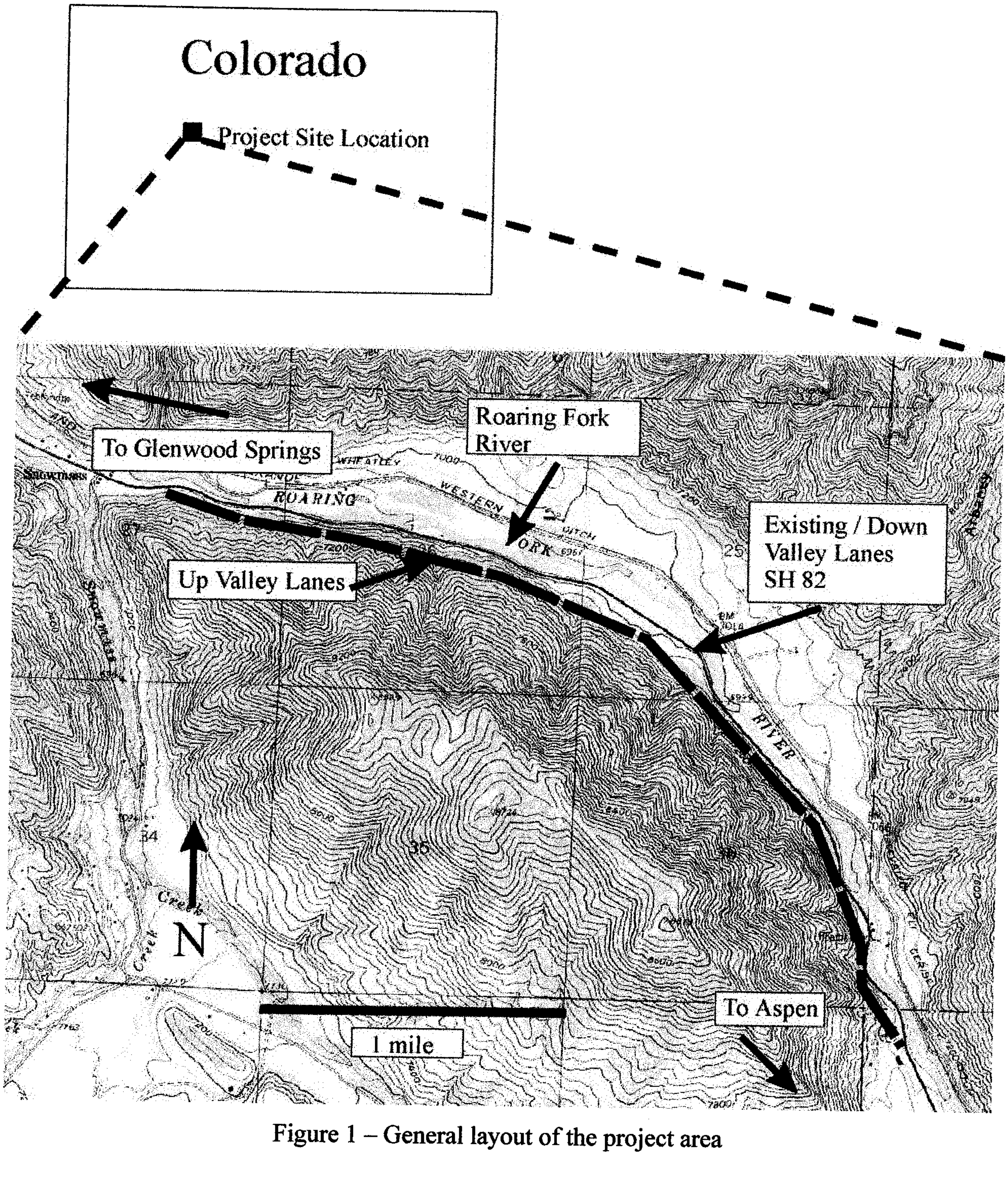


Figure 1 – General layout of the project area

Site Geology

The underlying geology of the project site consists of steep colluvial slopes, alluvial stream deposits and moderately dipping sandstone bedrock. Major and minor faulting is also evident within the project area. The colluvial materials consist of angular, poorly sorted rock debris embedded in a matrix of silt, sand and clay. Most of the colluvial material is present on steep slopes that are in excess of 1H : 1V. A majority of the steep slopes in the project area are comprised of debris fan depositional features. The debris fans appear to be the result of episodic debris flows that formed after intense storm events. Relatively large catchment basins located above the highway act as source areas for the debris flow materials. Thin layers of charcoal deposits are exposed in many of the debris fans that have been cut by the existing alignment. The charcoal remnants are an indication of past fires, which burned the upper slope vegetation and led to mass wasting of surface materials during storm or runoff events. The debris fan materials were typically loosely placed in many areas particularly along the boundaries of the mass wasting events. Figure 2 depicts a typical debris fan road cut profile exposed along the existing highway.

These materials can exhibit the potential for collapse when wetted and can be considered hydro-compactive in some areas. The term hydro-compactive soil refers to subsurface deposits that have the potential to decrease in volume by the addition of water. Typically this decrease in volume is independent of any changes in the vertical loading of the material. The process of soil collapse by inundation with water has been described in a variety of ways and has been referred to as hydro-compaction, hydro-consolidation, collapse, settlement, shallow subsidence, and near-surface subsidence. This volume change generally occurs due primarily to the influence of water. Additional subsidence can be caused by dissolution of disseminated sulfates or other soluble materials within the soil that dissolve when saturated with water.

Alluvial stream deposits, which were deposited in the past by the Roaring Fork River, can be found on elevated terraces on the slopes of Snowmass Canyon and along the banks of the present day river. The alluvial deposits can range in thickness from a few feet to as much as 40 feet. The deposits generally consist of sandy, well-rounded gravels, cobbles and boulders that are well graded or poorly sorted. These materials are highly desirable for foundation materials since they have higher strength and bearing capacity and are not susceptible to settlement or collapse.

The bedrock in the project area is predominately comprised of the Maroon Formation with overlying State Bridge Formation in the up valley section. The Maroon Formation in this area is more than 2500 feet thick and consists of reddish sandstones and siltstones. The State Bridge Formation is similar to the Maroon, consisting primarily of hard reddish brown sandstone with some siltstone. The bedrock is generally competent, ranging from hard, to very hard, except where fractured by faulting. The Rock Quality Designation (RQD) generally ranges from 90% to 50 % in the unfaulted or unfractured areas. In areas were faulting has occurred the RQD is 0% to 10%. Dips and strikes in the bedrock vary throughout the project. In the up valley areas most of the bedrock is relatively uniform with an approximate 25 to 35 degree dip. Figure 3 shows an area with colluvial, alluvial and bedrock materials.

Figure 2 - Debris Fan Deposits.

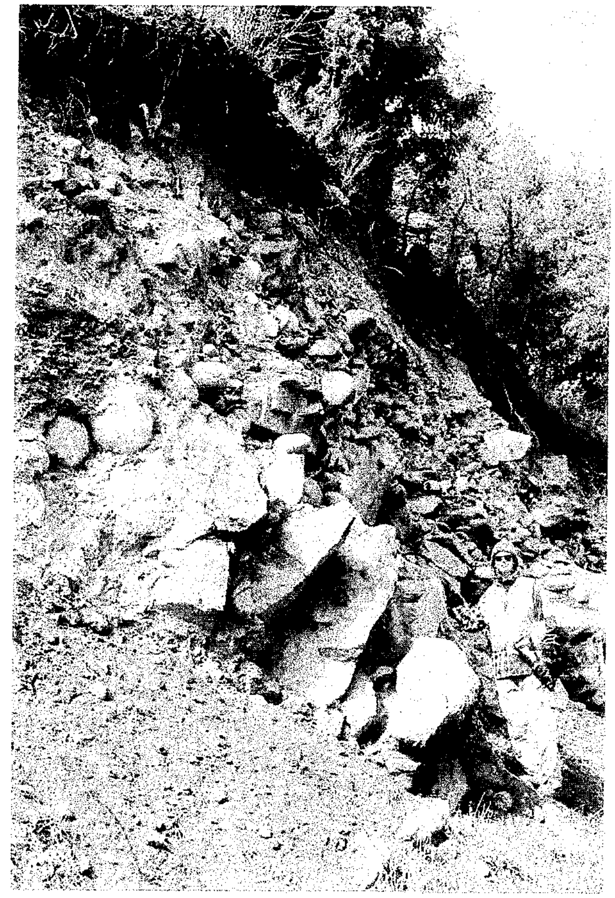


Figure 3 – Colluvial, alluvial and bedrock outcrops.

Generally the area is relatively dry with groundwater levels only being observed near the elevation of the river. Large storm events with greater than normal precipitation tend to produce debris flows in the less vegetated sections of the project.

Geological / Geotechnical Investigation

To characterize the geologic conditions and subsequent constraints to construction it was necessary to implement a geological/geotechnical investigation. The investigation included surface mapping, identification of geologic hazards, and subsurface drilling. Surface mapping was undertaken to identify potential debris fan areas, collapsible soil areas, and identification of subsurface materials along the road cuts. Areas that had the potential for landslides and rockfall were also identified.

Due to the steep slopes encountered on the project it was necessary to use helicopter transported coring drill rigs to access most of the sites. A total of 289 borings were completed over a two-year period. The most extensive drilling period utilized five drill rigs and one helicopter. Split spoon samples were obtained for subsurface materials above the bedrock. Core samples of the bedrock were obtained to determine rock quality. Figure 4 depicts the helicopter used to transport the drill rigs. Figure 5 shows one of the drill rigs operating in a steep section near the Roaring Fork River.

Results from the geotechnical investigation identified areas of potential soil collapse from hydro-compaction, location of alluvial and bedrock elevations, and general subsurface material characteristics. Additionally, a bedrock elevation map was generated from the drill hole information and used in the profile sections for the proposed highway alignments.

Seismicity was also evaluated for the project based on historical research of past earthquake events. This research indicated that at some time in the past the Aspen area likely felt a Modified Mercalli Intensity VII earthquake. This can be roughly correlated to a magnitude 5 earthquake. Additionally, historic evidence indicates that Colorado has experienced magnitude 7.0 or higher earthquakes. For this project and others in the state of Colorado, there has been a recent trend to consider earthquake potential and subsequent hazards more seriously than in the past, so this consideration was applied to the slope stability analysis.

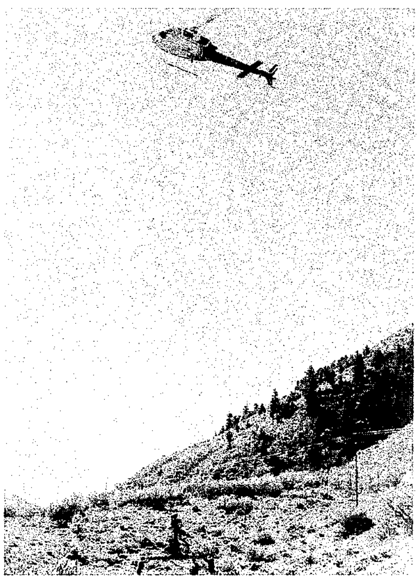


Figure 4 – Helicopter transport

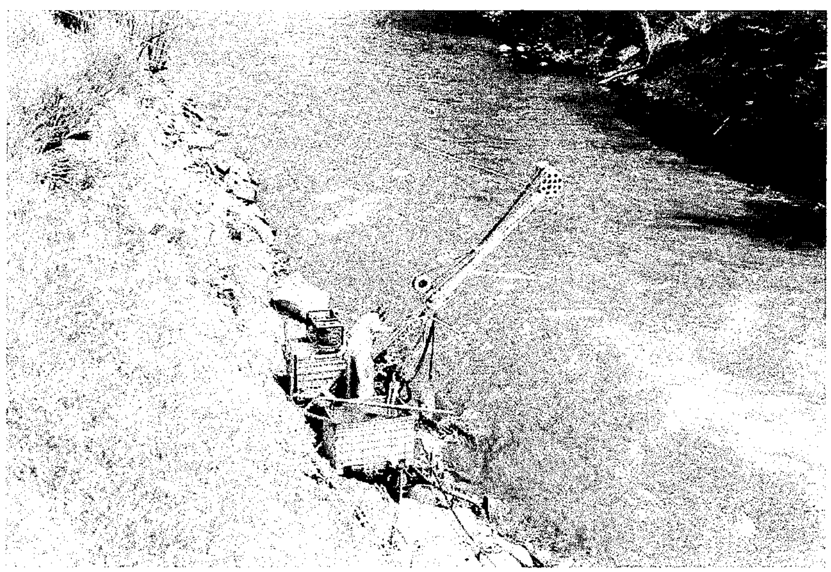


Figure 5 – Drilling on the side of Roaring Fork River

Design of Earth Retaining Wall Systems

The two predominating design constraints for the project were the relatively steep slopes in excess of 1H : 1V for most of the up valley lanes, and the potential for encountering hydro-compactive or collapsible soils throughout the area.

Prior to the retaining wall design phase of the project, it was necessary to evaluate the existing global stability of the area. Soil strength parameters were determined from sample testing of the subsurface materials and from back-calculation of the existing slopes. It was assumed most of the slopes throughout the project were at or above an existing factor of safety of one. Some sections of the project exhibited minor circular failures and sloughing of the colluvial materials, however no large scale circular failures were observed within the project area.

After evaluation of the existing conditions at the site, it was necessary to evaluate and model the slope stability of more than 31 critical sections along the proposed up valley and down valley alignments. Slope stability analysis considered not only the final completed alignment profile, but also the temporary conditions that were anticipated to occur during the construction sequence. Based on this analysis, it was necessary to construct temporary berms along many sections of the alignment to maintain an appropriate slope factor of safety during the construction phase of the project. For the proposed highway alignment a static slope factor of safety of 1.3 and a slope seismic factor of safety of 1.01 was used for the retaining wall systems.

The existence of collapsible soils was also a general constraint to the design that required foundations of retaining wall structures to be located either on alluvial materials, bedrock, or re-compacted structural backfill. The geotechnical data obtained from the drilling program was a valuable tool for delineating the elevations of the colluvial, alluvial and bedrock materials.

Based on the characterization of the geologic and surface conditions at the site it was determined that Mechanically Stabilized Earth (MSE) Walls with soil nails or ground anchor/tieback excavation support was a viable retaining wall system for the proposed highway alignment.

Mechanically Stabilized Earth Walls

Mechanically Stabilized Earth Walls (MSE) were used on the site since they typically tolerate greater settlement and deflection than traditional rigid retaining wall designs. This was especially important since the potential for collapsible soils was present. Where feasible on the project, the base of the MSE walls were located on bedrock or alluvial materials, however, certain colluvial sections of the project site were too thick for reasonable excavation. In these areas subexcavation was necessary. Sub-excavation consisted of removing a vertical portion of the colluvial material and replacing it with structural backfill. In higher wall sections or areas thought to be more prone to collapse, colluvial materials were excavated and replaced with a geogrid "sub-wall" foundation. The "sub-wall" was designed to act as a foundation mattress for the MSE wall.

Figure 6 depicts a generalized profile of the MSE wall system for the project. It was necessary to use either soil nails and/or ground anchors for the temporary and permanent excavation support. The soil nail walls and ground anchor walls were also necessary for global stability.

The MSE walls were constructed using pre-cast concrete panels with geogrid reinforcement cast directly within the panel. The panels were shipped to the project site and positioned with temporary support on the front face of the walls. To achieve the required geogrid reinforcement length it was necessary to attach additional geogrid to the back of the panel face by using a using a "bodkin" interlocking connection system. This system involves intertwining the two geogrid pieces and then inserting a linear member between the intertwined pieces. Figure 7 shows the connection of the panel cast geogrid with the bodkin connection. Backfill was placed in approximate 1-foot vertical lifts. Figure 8 shows the back of the MSE wall during fill placement. MSE walls on the project ranged from 5 to 41 feet in height for an approximate total of 315,000 square feet of wall face.

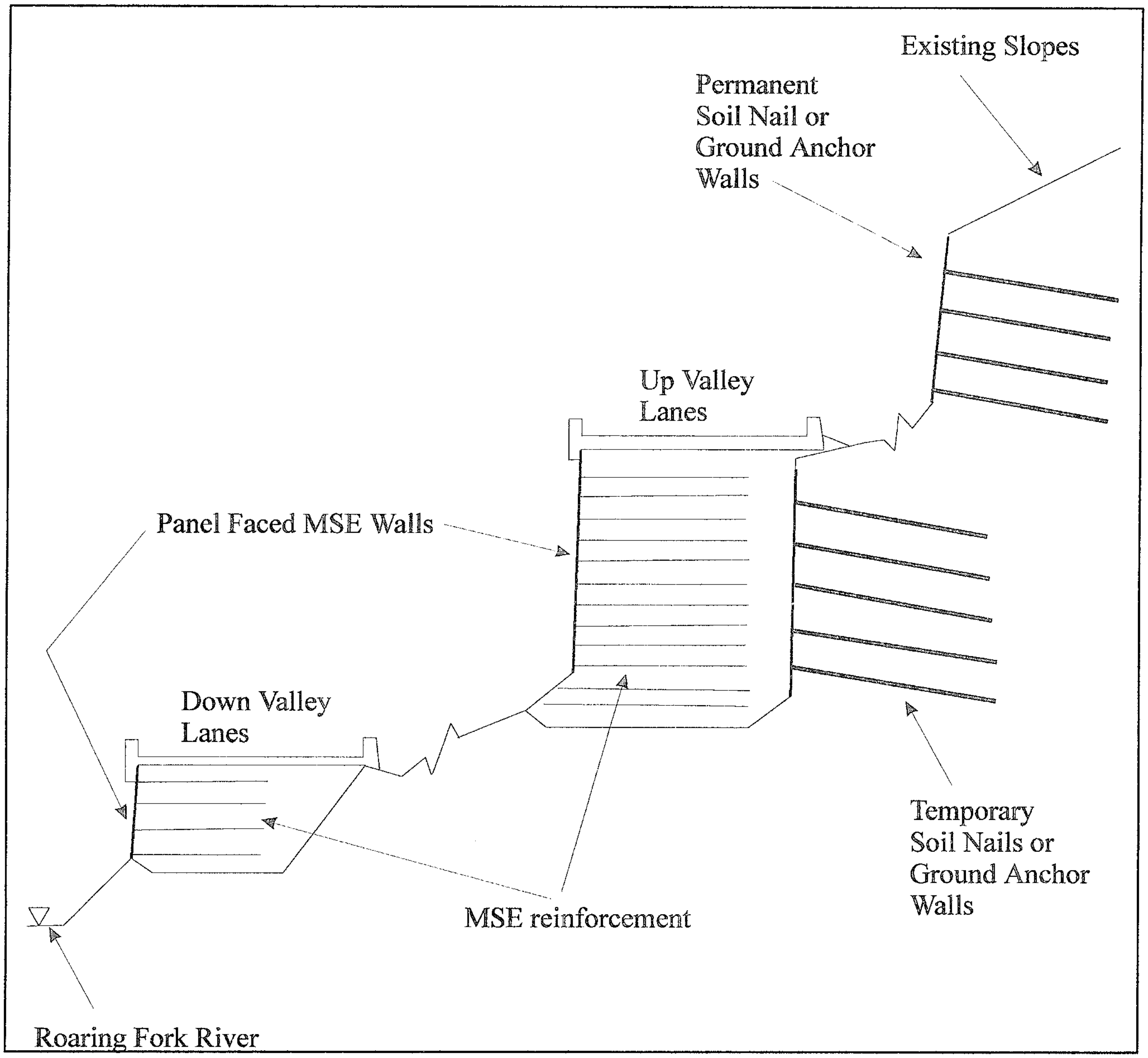


Figure 6 – Generalized Profile of Retaining Wall System for the Project

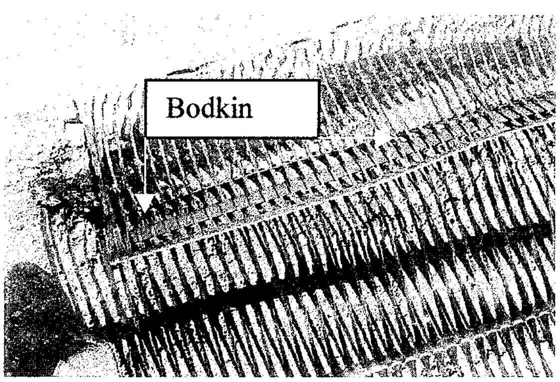


Figure 7 – Bodkin Connection

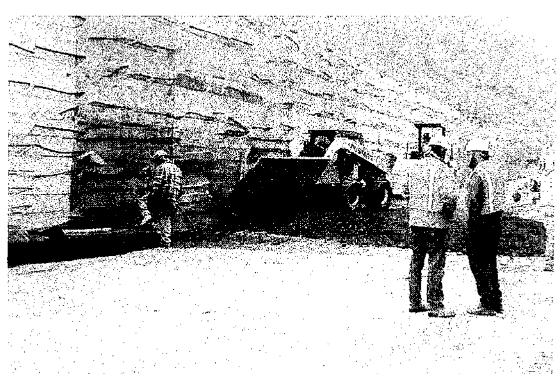


Figure 8 – Fill Placement for MSE Wall

Soil Nail Walls

Soil nail walls were used for excavation and global stability support for a majority of the cuts along the proposed alignment. Both temporary and permanent soil nail walls were used. Final facing of the permanent soil nail walls consisted of pre-cast panels. The soil nail wall heights for this project ranged from 3 feet to 25 feet high with reinforcement nail lengths ranging from 15 to 30 feet. A 4-foot (V) by 5-foot (H) spacing of the reinforcement soil nail bars was used.

The soil nail wall is constructed in a top-down sequence. An initial 4-foot vertical cut was made in the slope. If the cut was not self-supporting, a temporary berm had to be used. Soil nails, which consisted of #8 all thread bar, were placed into predrilled holes on a designed pattern. The holes were grouted either prior to bar placement or by tremie methods. Welded wire fabric with drainboards, to insure groundwater drainage of the system, were then placed over the nails and exposed cut face. The soil nails were then connected together by horizontal and vertical waler reinforcement bars with plates attached to the ends of the nails. Shotcrete was then placed to develop a complete structural reinforcement system. The sequence continued in a top down manner so that no more than 4 to 5 feet of vertical excavation was exposed during construction. As earth pressures develop behind the shotcrete facing system, the load is transferred through the facing system to the soil nail bars. The soil nail bar then transfers the load into the bonded section of the nail. Small deflections at the top of the walls are generally anticipated and designed for. A final precast panel is then attached over the wall face. Approximately 56,600 square feet of permanent soil nail walls were constructed on this project.

Figure 10 shows the installation sequence for the second row of a soil nail wall. Soil nails and shotcrete facing have been completed for the top row. Figure 11 shows a completed soil nail wall in one of the steeper up valley sections of the project. Figure 12 shows the placement of precast panels over the completed permanent soil nail walls.

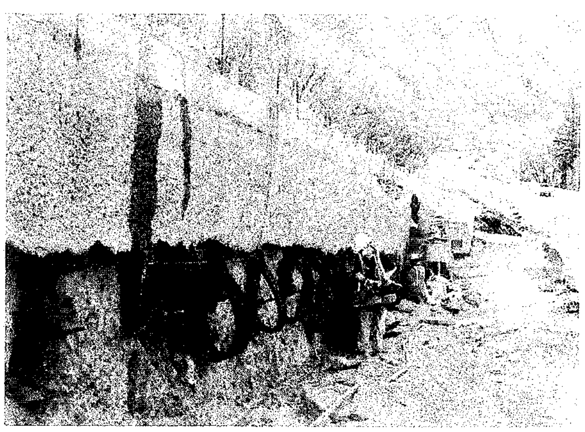


Figure 10 - Soil Nail Wall Construction.

Figure 11 - Completed Soil Nail Wall

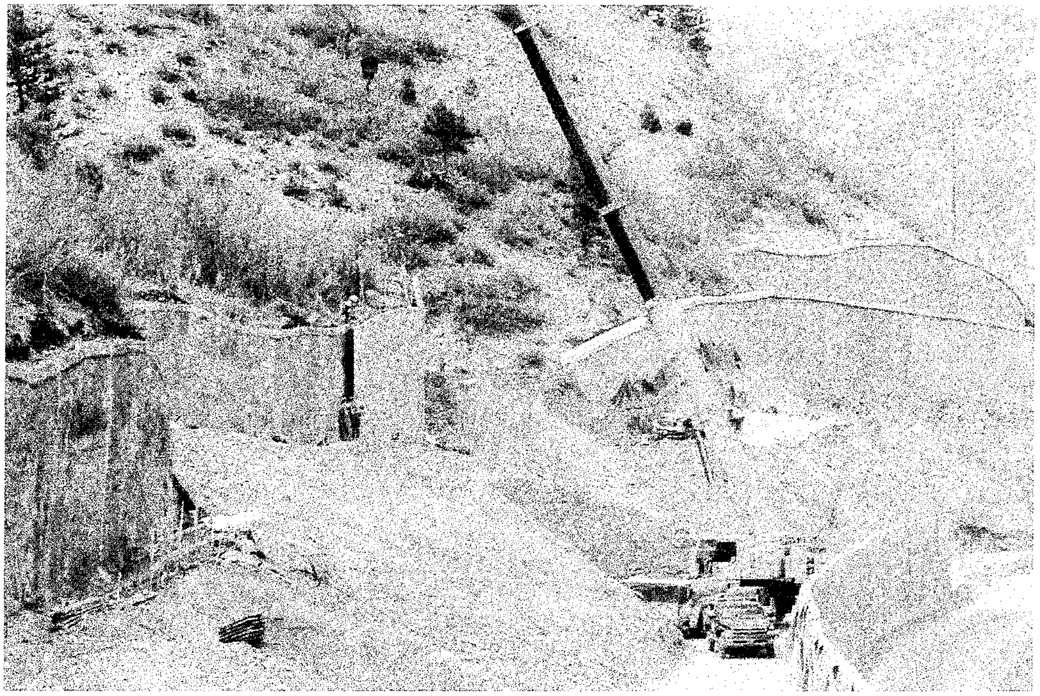


Figure 12 - Attaching final pre-cast panel facing over soil nail wall.

Ground Anchor Walls

Many areas along the existing slopes were too steep to support effectively with soil nail walls. In these areas, it was necessary to use ground anchor (tieback) support. The ground anchors consisted of multiple strand tendons that ranged in length from 35 to 70 feet. Bond lengths of the ground anchors averaged 20 feet. Due to the overall low bond strength of the colluvial materials it was necessary to place the bond lengths of the anchor systems in bedrock or alluvial materials. The ground anchors were set on an 8-foot horizontal by 8-foot vertical spacing. One to three rows of permanent tiebacks were used to provide excavation support and to satisfy global stability where necessary. The ground anchor support panels consisted of 8 foot by 8 foot rebar reinforced sections that were shotcreted in place. The lower section of Figure 13 illustrates a ground anchor panel system prior to shotcrete placement. The observer is looking at a completed ground anchor row above. Figure 14 shows installed ground anchors. Workers are putting a grout containment device on an anchor that is to be installed. A coiled ground anchor is visible next to the workers.

The ground anchor walls were constructed in a top-down manner similar to the soil nail walls. Approximately 35,000 square feet of permanent ground anchor tiebacks were used on this project. Figure 15 shows two completed rows of a ground anchor / tieback wall.

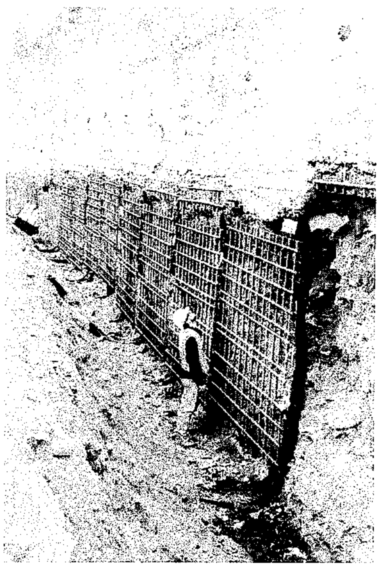


Figure 13 - Anchor Panels

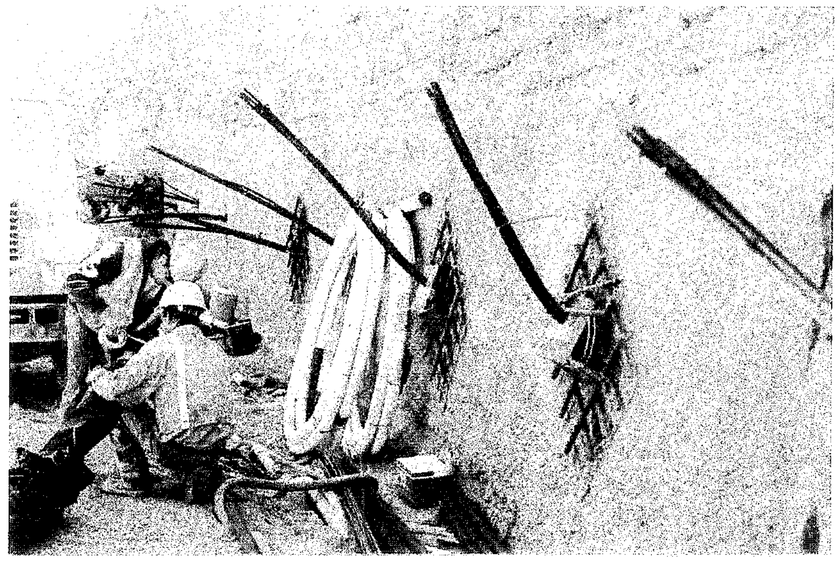


Figure 14- Ground Anchor Installation.

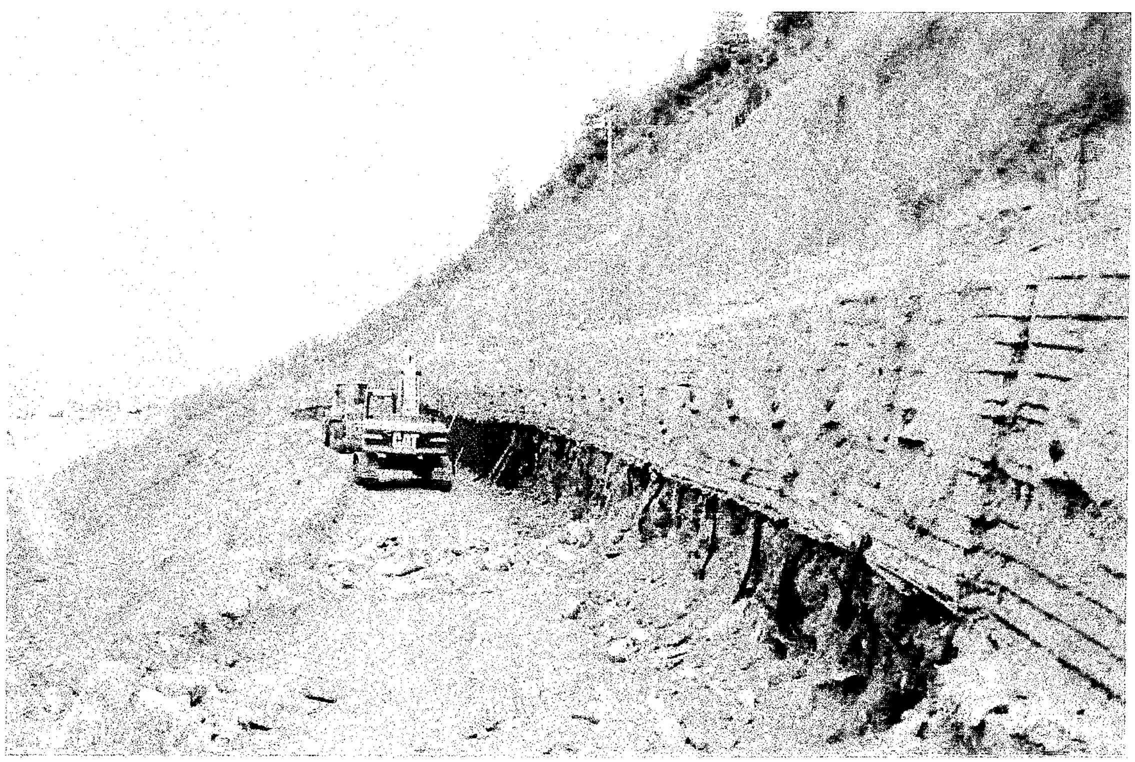


Figure 15 - Two rows of a ground anchor wall.

Micropile Foundation Support

In certain sections of the project where the side slopes were in excess of 1H:1V and the bedrock quality was relatively low, double tee retaining walls with micropile foundation support were used. These walls ranged in height from 14 to 36 feet for approximately 1700 feet of wall length. The original foundation design called for 30-in diameter caisson (drilled shaft) support of the wall system with a 10 to 12 foot spacing between the caissons. The foundation design was modified to use 7-inch diameter micropiles with variable spacing from 2-½ to 8-½ feet depending on the wall height. The micropiles consisted of an approximate 7-in by ½-in casing that was drilled a minimum of 2 feet into the bedrock. A "rock socket" was then drilled into the bedrock to form the bond zone of the micropile. The bond zone ranged from 20 to 37 feet depending on the installation method. A #14 threaded bar was placed inside the 7-inch casing and extended into the bond zone. The inside of the casing and the rock socket were then grouted. Plates were attached to the top of the threaded bar and the system was incorporated into the poured foundation for the double tee walls. To provide additional external wall and global stability it was necessary to incorporate permanent ground anchors into the foundation system. Ground anchors were placed in-between the uphill row of micropiles.

Figure 16 shows the installation of the micropiles. Drilling is ongoing in the background with one row of micropiles visible in the foreground. Figure 17 shows the double tee wall footing prior to placement of concrete. Note the square plates attached to the top of each micropile. The ground anchor tiebacks are placed through the pipes, which are inclined in the left center area of the figure. Figure 18 illustrates the generalized profile of the double tee wall system showing placement of micropiles, ground anchors, and double tee sections of the retaining wall system.

Figure 16 - Micropile installation

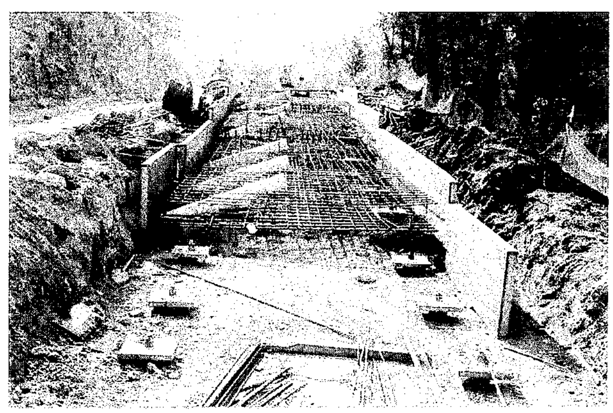


Figure 17 - Double tee footing foundation prior to concrete placement

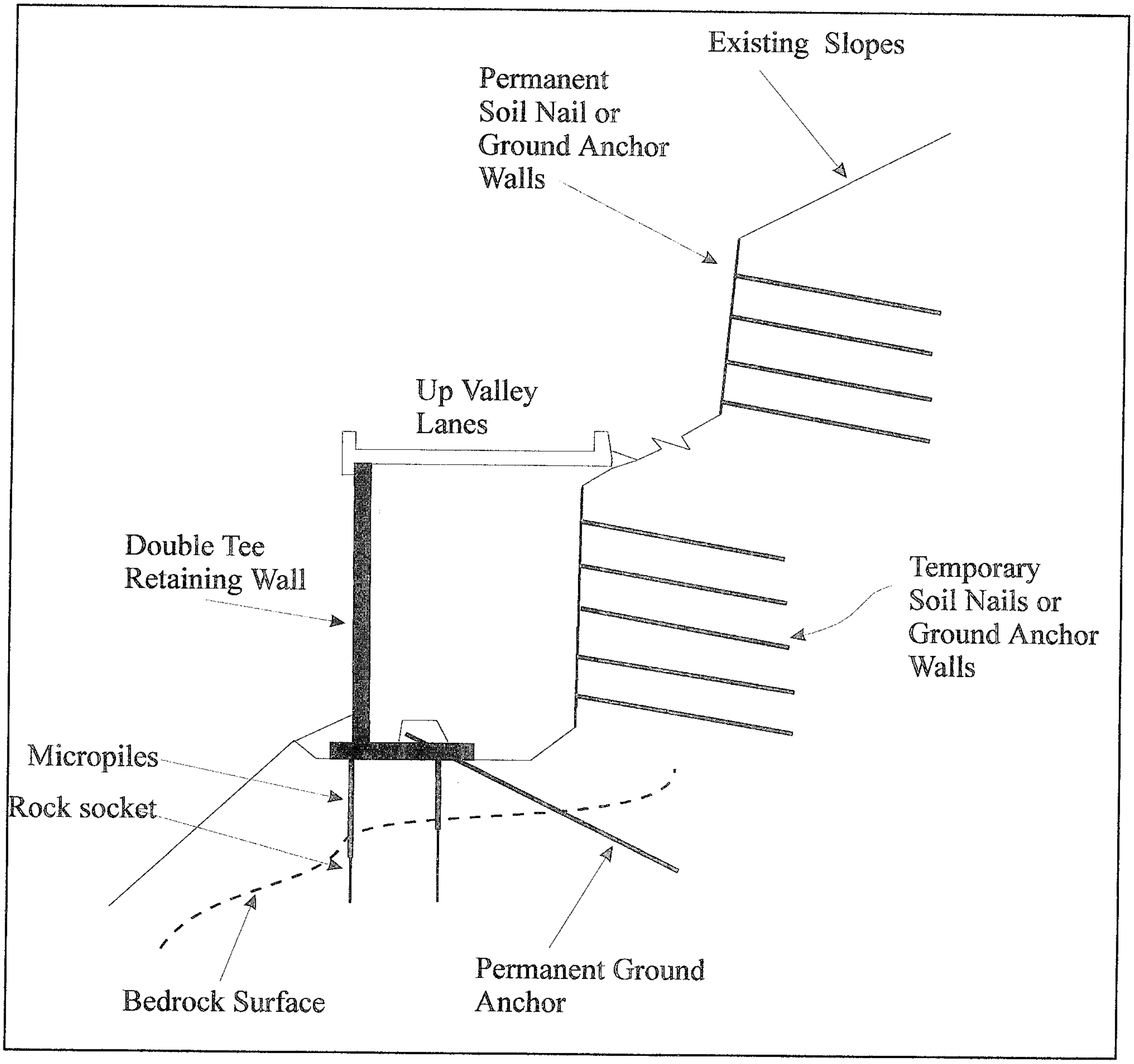


Figure 16 – Generalized Profile of Retaining Wall with Micropile Foundation

Rock Bolting and Draped Mesh

Most of the slopes above the highway alignment consist of colluvial materials, however certain sections of the alignment are next to steep bedrock cliffs. In these areas, kinematic slope stability analyses and the Colorado Rockfall Simulation Program (CRSP) were used to recommend appropriate mitigative options for rockfall. Typically, rockfall mitigation consisted of spot bolting, pattern bolting, and the use of draped mesh in critical sections. Figure 17 shows the use of a crane basket for bolting a rock cut section that will be located adjacent to a bridge structure.

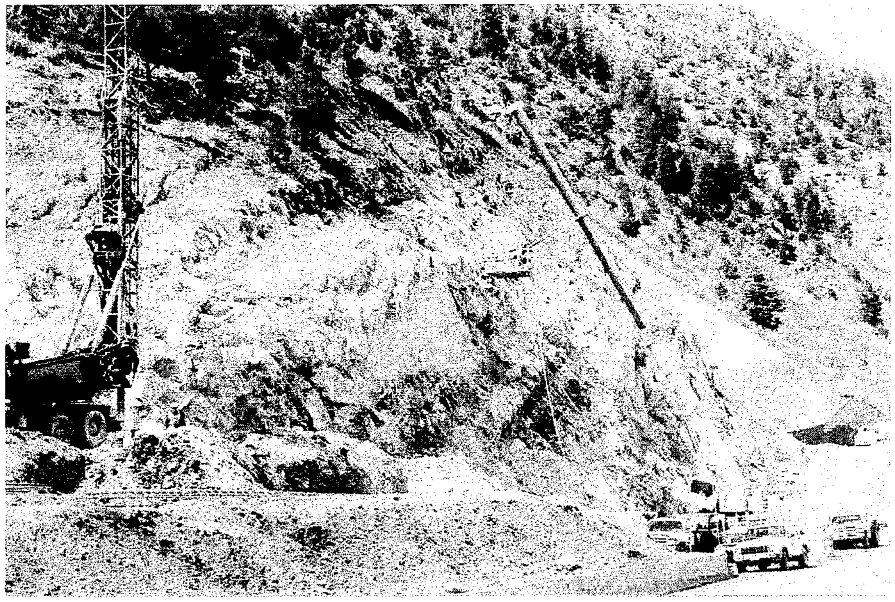


Figure 17 - Rock bolting from crane basket

Summary

The final completion of the project is expected to occur in the fall of 2005. Many of the retaining walls along the alignments have been instrumented with either inclinometers or load / pressure cells. In this manner, the performance of the wall systems can be assessed over time. Based on walls constructed to date, it appears the wall systems are performing in general conformance with the designs.

Overall, the wall systems proposed were based on the subsurface information obtained from the geotechnical investigation. Without the extensive geotechnical investigation, it would have been difficult to predict and design for many of the conditions that are present at the site. The use of helicopter transportation of the drill rigs was an invaluable resource for obtaining information in many locations that were not accessible until actual construction commenced.

NEW EAST SPAN BAY BRIDGE GEOLOGY, SOIL SET-UP AND ACCEPTANCE OF LARGE DIAMETER DRIVEN PILES

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The San Francisco-Oakland Bay Bridge, which is one of the most heavily traveled bridges in the world. The east span of the bridge will be replaced due to seismic safety concerns. The majority of the new bridge will be founded on large 2.5-meter diameter, approximately 60- to 100-meter long pile foundations. Piles foundations will experience tension loads of up to approximately 80 to 90 MN and compression loads of up to approximately 120 to 140 MN during the design earthquake. This paper is based on the Pile Installation Demonstration Project (PIDP) that was conducted as a part of the East Span San Francisco-Oakland Bay Bridge Seismic Safety Project. For the three full-scale pile installations, two different high-energy hydraulic hammers were selected for the installation. However, these hammers may not have sufficient energy to drive these large diameter piles at their ultimate capacity following long-term set-up. Nevertheless, these hammers were selected because, soil plug behavior, the generation of excess pore pressures and remolding of soil during driving were expected to reduce the shaft and end bearing resistance, and therefore reduce the energy required to drive these piles to design penetration. Data collected during continuous installation driving and after a series of re-strikes at different set-up times ranging from 1 to 33 days were used as input data for Case Pile Wave Analysis Program (CAPWAP). These results were used to estimate the soil resistance along the pile length and at the toe of the pile, and the soil pile set-up behavior. Validity of the procedures used to predict the soil resistance to driving and predicted blow counts were verified with the field observed blow counts and the energy that was delivered to the pile. Based on combined CAPWAP analyses, the soil pile set-up with time was estimated and compared with results of other method currently available in the literature for predicting the soil pile set-up rate.

INTRODUCTION

The east span of the San Francisco-Oakland Bay Bridge, which is one of the most heavily traveled bridges in the world, will be replaced due to seismic safety concerns. The proposed new bridge will be constructed along a parallel alignment to the north of the existing bridge. The new bridge will consist of an approximately 460-meter-long transition structure extending from the Yerba Buena Island (YBI) Tunnel to the eastern tip of YBI, an approximately 625-meter-long, single-tower, self-anchored suspension cable, main-span signature structure extending offshore from the tip of YBI, an approximately 2.1-kilometer-long, four-frame Skyway structure extending from the signature structure eastward to the Oakland Shore Approach, an Oakland Shore Approach structure extending about 700 meters from the Skyway structure to the north side of the Oakland Mole and an earthenfill transition from the Oakland Shore Approach structure to the roadways leading to and from the existing bridge. The majority of the new bridge will be founded on large 1.8 to 2.5-meter diameter, approximately 60 to 100-meter long pile foundations. Piles will experience their maximum loads during the design earthquake. Piles are designed for tension loads of up to approximately 80 to 90 MN and compression loads of up to approximately 120 to 140 MN.

The PIDP was conducted as a part of the San Francisco-Oakland Bay Bridge East Span Seismic Safety Project in order to evaluate constructability of these large diameter piles, soil-pile setup behavior and the impact on the project during construction. Three full-scale large diameter steel pipe piles, 2.438-meter diameter, up to 100-meter long, wall thickness 40-70 millimeter, one vertical and two on a 1:6 batter, were driven near the planned bridge alignment in San Francisco Bay, Figure 1.

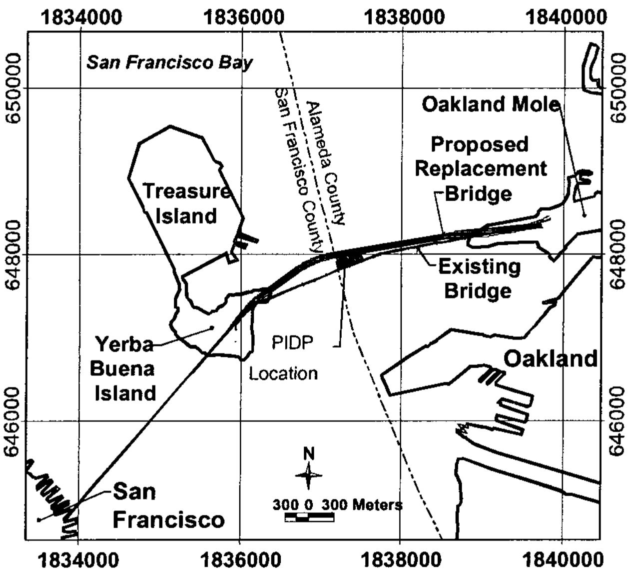


Figure 1. Site Vicinity Map

The PIDP piles are the deepest large diameter piles ever driven in the world (Engineering News Record 2000). The expected ultimate axial compression capacity was estimated based on American Petroleum Institute method (1986, 1993a) to be on the order of 120 MN. Two different high-energy hydraulic hammers MHU500 (550 kilojoules) and MHU1700 (1870 kilojoules) were selected for the installation. These hammers may not have sufficient energy to drive these piles at their ultimate capacity following long-term set-up. However, the repeatedly sheared and remolded clayey soils near the pile tip and wall, the soil plug behavior and the excess pore pressure generation in the Young & Old Bay Mud and Upper Alameda formations during driving were expected to reduce the shaft and end bearing resistance, and therefore reduce the energy required to drive these piles to design penetration.

The force and acceleration time history data due to impact of the hammer was collected during continuous installation driving and after a series of re-strikes at different set-up times with the use of strain transducers and accelerometers attached near the pile top. The CAPWAP (Rausche, 1970) was used to estimate the soil parameters such as soil quake and damping, and soil resistance along the pile length and at the toe of the pile. To compute forces or velocities at the pile top, the pile and soil were divided into many continuous segments, modeled mathematically and an algorithm, which allows for step by step computation of all pile variables along the pile length and in time is followed. The CAPWAP results are based on the best possible match between a computed and measured pile top force or velocity. The soil parameters varied iteratively until a satisfactory match is obtained between a computed and a measured pile top force or velocity.

Wave equation analyses were performed to predict the blow counts with the penetration depth using GRLWEAP (1997-2) program for the continuous driving case. To perform the wave equation analyses, the unit skin friction and unit end bearing for static loading condition is first computed using the method recommended by American Petroleum Institute (API, 1986). The soil resistance to driving (SRD) is then calculated by incrementally reducing the unit skin friction values by multiplying by a factor.

SITE SPECIFIC GEOLOGY

The geologic formations that underlie the PIDP area include in the following sequence (Howard et al. 2001): Young Bay Mud (YBM), Merritt-Posey-San Antonio (MPSA) formations, Old Bay Mud (OBM), Upper Alameda Marine (UAM) sediment and the Lower Alameda Alluvial (LAA) sediment. Fig. 2 provides an illustration of the subsurface conditions encountered along the new bridge alignment including the PIDP area. The subsurface conditions encountered near the test location are illustrated in Fig. 3. The geologic formations and the engineering properties are presented in Table 1.

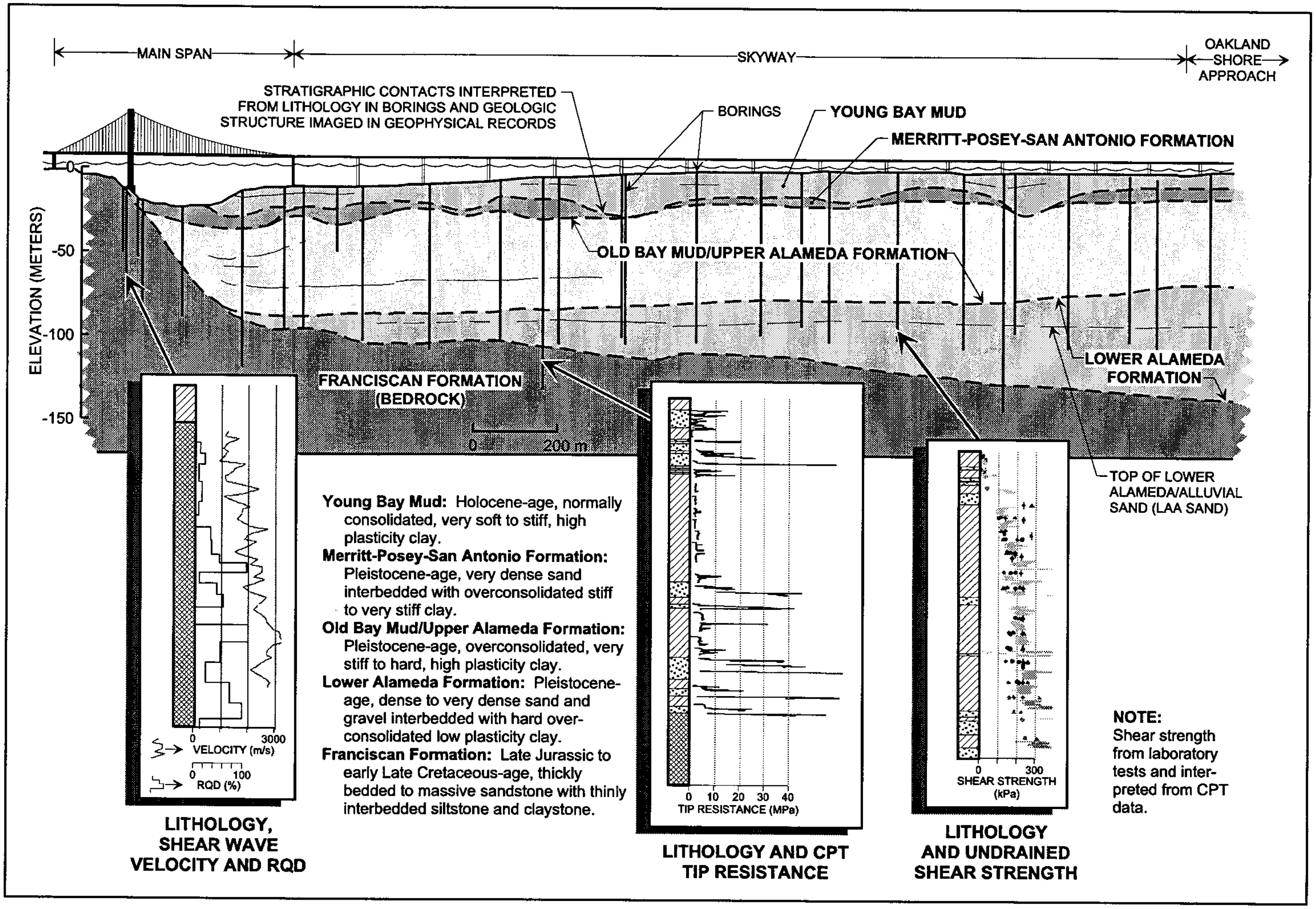


Figure 2. Interpreted Stratigraphic Section Along New Bridge Route

Table 1. Engineering Soil Properties

Geological Formation	Typical Soil	Ø' Degree	Su kPa	γ" kN/m^3
YBM	Very Soft to Firm Clay	-	10 - 65	4 - 7
MPSA	Dense to Very Dense Sand with Stiff to Very Stiff Clay Layers	35 - 42	60 -175	6 - 11
OBM/UAM Sediments	Very Stiff to Hard Clay	-	100 - 250	6 - 9
LAA Sediments	Dense to Very Dense Sand and Hard Clay	40 +	225 - 400	9 - 12

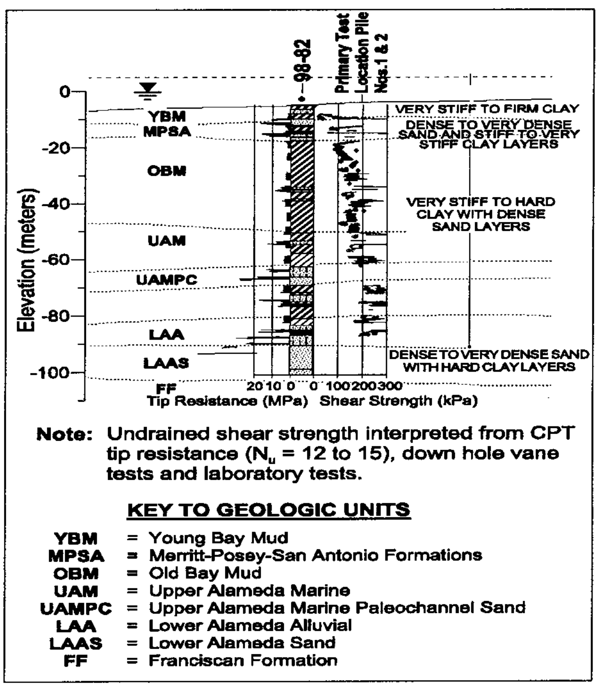


Fig. 3. Typical Geological Formation Cross Section

YBM, the youngest geologic unit in the bay, is marine clay that has been deposited since the end of the last sea level low stand, which was about 11,000 years ago (Atwater et al., 1977). YBM sediments are generally very soft to firm, normally to slightly over-consolidated, high plasticity clay. The YBM includes sand layers and or seams within several depth intervals that may correlate to different depositional periods. The undrained shear strength of YBM generally increases with depths from 2 to 4 kilopascals (kPa) at the surface to 20 to 65 kPa at the base of the sequence. At the primary and alternate test location, the YBM is approximately 4 to 6 and 20 meters thick, respectively.

MPSA formations, which is beneath the YBM, a layered sequence of dense to very dense sand with layers of stiff to very stiff sandy clay and clay is present over portions of eastern Bay. The sequence is generally considered to be composed of late Pleistocene, non-marine sediments deposited during the late Wisconsin glacial stage (90,000 to 11,000 years ago). The Merritt formation reportedly is primarily coarse-grained, aeolian sediments that were blown in from the west during sea level low stands (Rogers and Figuers, 1991).

The Posey member, typically considered the basal member of the San Antonio Formation, is reportedly of primarily alluvial origin and was likely deposited within channels that were active during sea level low stands. Although primarily non-marine deposits, the late Wisconsin glacial stage also included periods of sea level fluctuations that may have produced estuarine environments. The fine-grained layers within the sequence are likely associated with those periods. At the primary and alternate test location, the MPSA is approximately 7 to 8 and less than 5 meters thick, respectively.

OBM sediments, which underlies the San Antonio Formation and overlies the Alameda Formation is considered to be an 80,000 to 130,000 year old marine deposit (Sloan, 1982). The surface of the OBM is extensively channeled. The OBM typically is a very stiff to hard, over-consolidated, plastic clay that includes several, often discontinues, crusts. The undrained shear strength of OBM increases with depth and typically range from 90 to 175 kPa at the top of the sequence to 150 to 250 kPa at the base of the sequence. The sequence also includes numerous "crust" layers with shear strengths 25 to 50 kPa higher than adjacent layers. Those crust layers are interpreted to be old soil horizons that were exposed to air during sea level changes. The OBM is typically about 20 to 25 meters thick.

Alameda Formation generally lies directly above the Franciscan Formation (FF) bedrock in the marine portion of the project area. The Alameda Formation is considered to be of late Pleistocene age (Sloan, 1982) and has been informally divided into an Upper Alameda Marine (UAM), primarily fine-grained marine member, and Lower Alameda Alluvial (LAA), primarily alluvial member (Rogers and Figuers, 1992a and 1992b).

UAM is composed primarily of very stiff to hard, over-consolidated, plastic clay with occasional silt, clayey silt and sand layers. Old Bay Mud/Upper Alameda Marine (OBM/UAM) sediments, which consist primarily of very stiff to hard fat clay. Except for the increased occurrence of coarser-grained interbeds, the UAM clays are similar to the marine clays overlying OBM. The combined thickness of those marine clays is typically about 60 meters. At the primary test location the OBM/UAM is 60 to 65 m thick with an intermediate 6 m thick very dense sand layer known as the Upper Alameda Marine Paleochannel (UAMPC) sand. At the Alternate test location the OBM/UAM is 50 to 55 meters thick.

Lower Alameda Alluvial (LAA) sediments includes a 3 to 10 meters thick cap layer of very stiff to hard lean clay underlain by a sequence of primarily very dense granular alluvial sediments. The PIDP specified pile tip elevations were anticipated to be approximately 10 meters into the LAA-sand.

COMBINED CASE PILE WAVE ANALYSIS

Satisfactory CAPWAP evaluations and estimates of the ultimate pile capacity require that the hammer energy be sufficient to move the pile enough to fully mobilize the available soil resistance. If the energy is not sufficient to mobilize the full soil resistance then the CAPWAP analyses may significantly underestimate the available static pile capacity. However, based on the data collected during continuous installation driving and after a series of re-strikes at different set-up times, the combined CAPWAP analysis (Stevens 2000, Mohan et al. 2002) can be used to better estimate the available static pile capacity even when the hammer is not able to mobilize the entire soil resistance during a series of re-strike(s). The combined CAPWAP approach is illustrated schematically on Fig. 4.

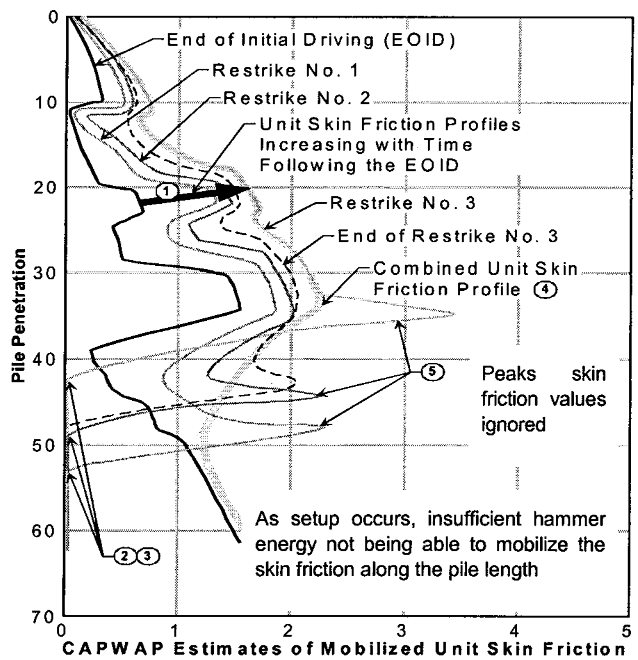


Fig. 4. Schematic Illustration of Combined CAPWAP Analysis

A combined "best estimate" skin friction distribution was obtained by summing the largest mobilized skin friction increment values along the length of the pile during initial driving or subsequent re-strike(s). It is important to recognize that the largest mobilized skin resistance increment may come from the CAPWAP analysis performed at the end of re-strike. This is due to the fact that subsequent hammer blows will breakdown the setup in the upper portion of the pile and mobilize skin resistance in the lower portion of the pile that was not mobilized at the beginning of the re-strike.

The high resistances are seen as abrupt peaks in the unit skin resistance immediately above the portion of the pile for which unit skin friction is zero. This phenomenon may be attributed to the fact that this portion of the pile is no longer being moved enough by the hammer, resulting in wave reflection that may be seen by the model as an end bearing response. The abrupt peaks in the unit skin resistance immediately above the portion of the piles were neglected in the best estimate skin resistance.

PILE CAPACITY/OBSERVED SET-UP

A series of restrikes were conducted on the PIDP piles to better understand the magnitude and distribution of soil resistance along the pile. Three re-strikes were conducted on Pile-1, two re-strikes were conducted on Pile-2, and one re-strike was conducted on Pile-3. Based on CAPWAP analyses results, the trend of increase in total skin resistance (sand and clay) with time is shown on Fig. 5. For comparison, total skin resistance profiles computed based on API design method (1993a, b) is shown on Fig. 5.

Fig. 6 presents the total skin resistance (sand and clay) at each re-strike versus time and predicted static skin resistance based on the API design method (1993a, 1993b). It appears that after 33-days of set up, Pile-1 has the static skin resistance capacity of 70 MN, which is approximately 88 percent of design skin resistance capacity. For Pile 2, the total skin resistance capacity was approximately 67 MN after 22 days of set-up. However, due to the presencs of predominantly clayey soils at the site, it is likely that soil pile setup will continue for several months. Also, it is likely that the skin friction capacity may exceed the predicted capacity of 80 MN by the API design method.

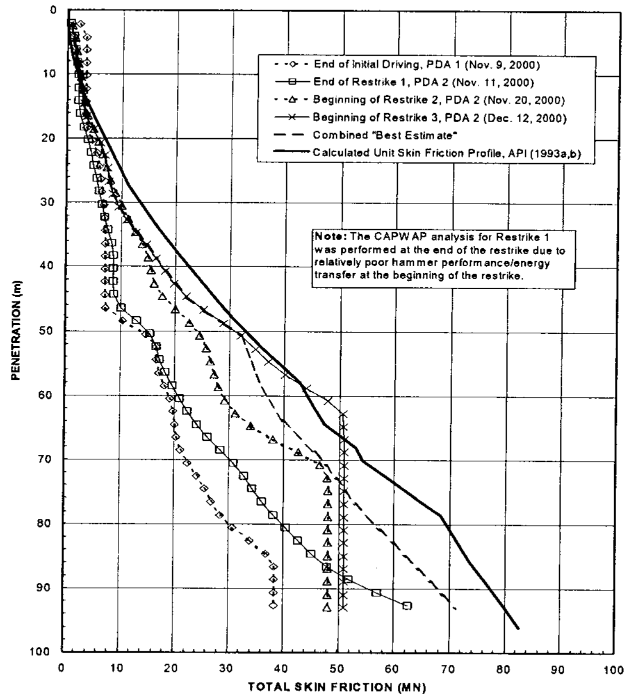


Fig. 5. Pile 1, Total Skin Resistance Profiles from Combined CAPWAP Analysis

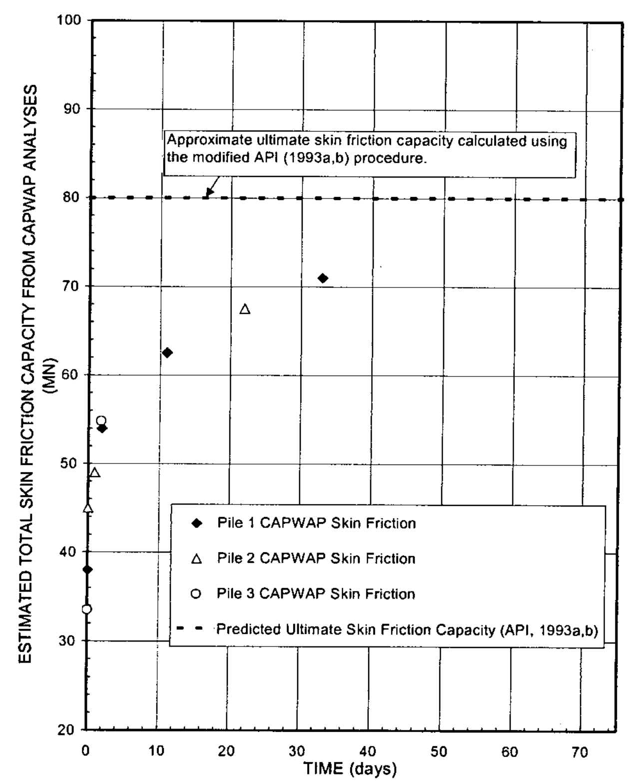


Fig. 6. Total Skin Resistance Capacity from Combined CAPWAP Analysis with Time

PREDICTED SET-UP IN CLAY

The predicted set-up in clayey soils was based on a method developed by Soderberg (1962). This method was based on site-specific radial consolidation coefficient (c_h). The coefficient of radial consolidation was defined by:

$$T = c_h\, t / r_p^2$$

In Soderberg's hypothesis, the time factor (T) was set equal to unity at time t equal to t_{50}. Therefore, the equation may be rearranged as

$$C_h = T\,(r_p^2/t) = (r_p^2/t_{50})$$

The linear relationships describing the range of predicted increase of axial geotechnical capacity with time are identical to that recommended in Bogard and Matlock (1990c).

Lower Bound: $Q(t) = Q_u\,(0.20 + 0.80U)$
Upper Bound: $Q(t) = Q_u\,(0.30 + 0.67U)$

Where percent consolidation (U) given by:

$$U = (t/t_{50})/(1 + t/t_{50})$$

The predicted ranges of set-up for the method is based on an assumed range of set-up factors of 3 to 5. The set-up factor is the ratio of final ultimate capacity to the capacity at the end of initial driving. The horizontal coefficient of consolidation (c_h) was chosen for the Soderberg method based on available consolidation test data in the Young and Old Bay Mud and a series of multiple orientation consolidation tests that were performed as a part of site characterization. The vertical coefficient of consolidation (c_v) was selected from the lower one-third of the available test data range as presented in Fig. 7. Which was approximately 8 square meters per year (m^2/year). Also, based on the multi-oriented consolidation test data in the over consolidated range of stresses; the c_v value was multiplied by the ratio of c_h to c_v (approximately 1.5) to obtain a c_h value of approximately 12 m^2/year.

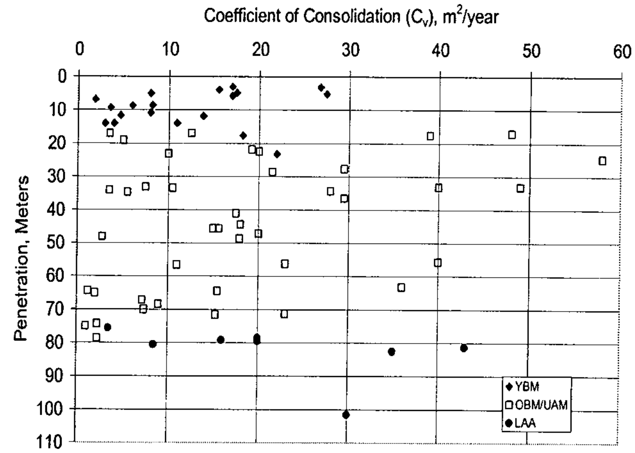


Fig. 7 Coefficient of Consolidation Profile

Fig. 8 presents the predicted set-up that based on Soderberg's method and the calculated setup based on CAPWAP analysis for the three piles.

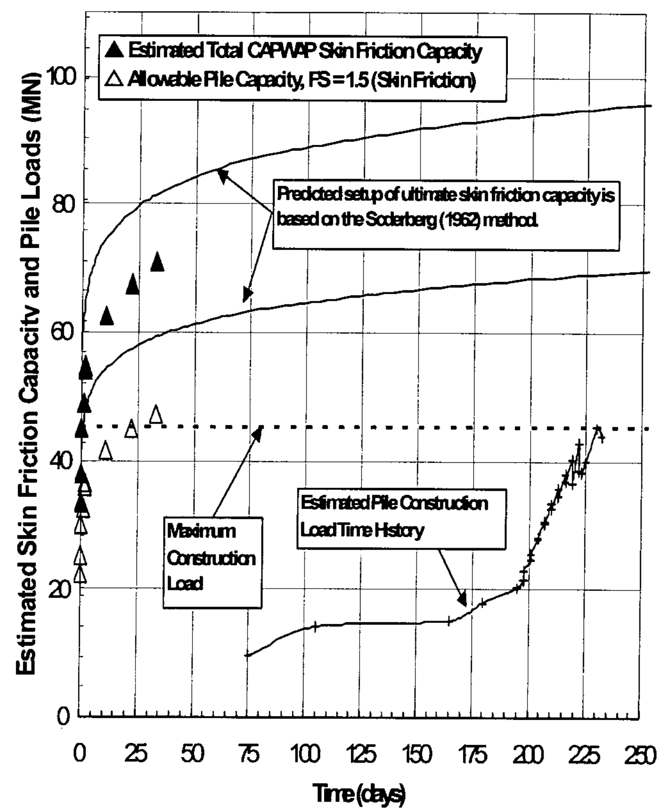


Figure 8. Estimates of Pile Setup From PIDP

The combined CAPWAP analyses may be conservative since the skin friction was not mobilized in the lower portion of the pile during re-strikes, and the skin resistance mobilized on a particular pile segment is assumed to be larger of the actual resistance mobilized during continuous driving (Stevens 2000). However, the skin resistance is likely to continue to increase with time and likely to exceed the calculated (API 1993a, b) ultimate skin friction of pile capacity.

SOIL RESISTANCE TO DRIVING (SRD)

Stevens et al. (1982) recommended that lower and upper bound values of SRD be computed for the coring pile condition especially for larger diameter pipe piles. When a pile cores, relative movement between pile and soil occurs both on outside and inside of pile wall. The data presented in Fig. 9 indicates that all three piles cored through the soil during continuous driving.

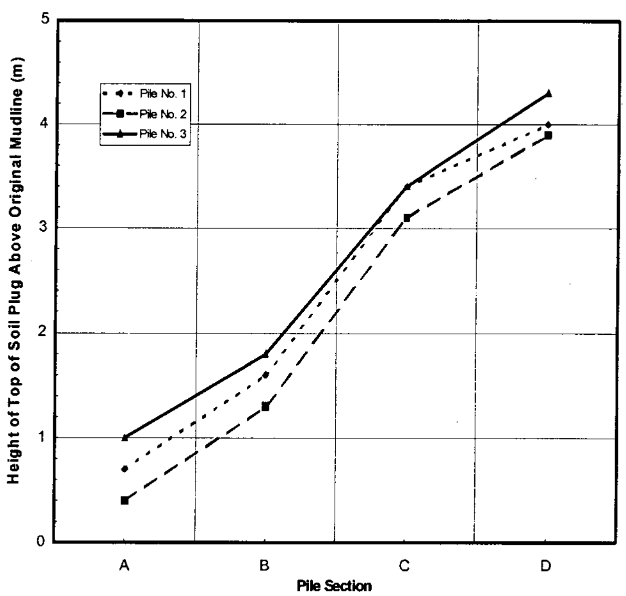


Fig. 9 Summary of Soil Plug Measurements

The lower bound was computed assuming that the skin friction developed on the inside of the pile is negligible. An upper bound is computed assuming the internal skin friction is equal to 50 percent of the external skin friction. For a plugged pile, a lower bound is computed using unadjusted values of unit skin friction and unit end bearing. An upper bound plugged case for granular soils is computed by increasing the unit skin friction by 30 percent and the unit end bearing by 50 percent. For cohesive soils the unit skin friction is not increased and the unit end bearing is computed using a bearing capacity factor of 15, which is an increase of 67 percent.

For sandy soils, the unit skin friction and unit end bearing values that were used to predict the SDR were the same as those used to compute static pile capacity. For clayey soils, the SRD was computed using two methods.

Method-I by Stevens et al. (1982), the unit skin friction was computed from the stress history approach presented by Semple and Gemeinhardt (1981). The unit skin friction and unit end bearing for static loading is first computed by using the method recommended by the American Petroleum Institute (1986). The SRD is then calculated by incrementally reducing the unit skin friction values by multiplying by a pile capacity factor, determined empirically (Stevens et al 1982). The pile capacity factor (F_p) is given by:

$$F_p = 0.5 \times (OCR)^{0.3}$$

The over-consolidation ratio (OCR) was calculated with the following equations.

$$S_u/U_{unc} = (OCR)^{0.85}$$

Where:

S_u undrained shear strength of clay and U_{unc} undrained shear strength-normally consolidated clay

$$U_{unc} = \sigma'_{vo} \times (0.11 - 0.0037 \times PI)$$

Where:

σ'_{vo} effective overburden pressure and

PI plasticity index

<u>Or</u> OCR was estimated from CPT tip resistances with the use of following equation.

$$\sigma'_p = 0.33 \times (q_c - \sigma'_{vo})$$

Where:

q_c cone tip resistance, and

σ'_p preconsolidation stress

$$OCR = \sigma'_p / \sigma'_{vo}$$

Method-II was based on "Sensitivity Method"; the unit skin friction for static loading is first computed by using the method recommended by American Petroleum Institute (1983, 1993a). The SDR is then calculated by incrementally reducing the unit skin friction values by measured clay sensitivities (Dutt et al 1995, Mohan and Buell 2000, Mohan et al 2002), which is the ratio of the undisturbed to remolded clay shear strengths.

WAVE EQUATION ANALYSIS

Wave equation analyses were performed to predict the blow counts with the penetration depth using GRLWEAP (1997-2) program for the continuous driving case. The soil quake and damping parameters recommended by Roussel (1979) were used. The shaft and toe quakes were assumed to be 0.25 centimeters for all soil types. A shaft damping value of 0.19 to 0.36 seconds per meter was assumed for clayey soil. The shaft damping value in clayey soil decreases with increasing shear strength (Coyle and Gibson 1970). The toe damping value of 0.49 seconds per meter was assumed for all soil types.

OBSERVED Vs PREDICTED SRD

Fig. 10 presents results of predicted and PDA observed SRD profiles for Pile 2. Interestingly, the predicted and observed blow count profiles, as expected, correlate well with the predicted and PDA observed SRD. PDA observed SRD was computed based on maximum Case Method and damping coefficient (J) of 0.5. The observed blow counts and SRD spikes at penetration depths of about 45 and 70m were as a result of soil pile set-up that occurred during driving delays such as splicing and welding of pile sections. Therefore, it demonstrates very well that wave equation analyses can be used to reasonably predict the blow counts with the penetration depth provided that similar hammer energies are applied in the model.

OBSERVED vs PREDICTED BLOW COUNTS

Fig. 11 presents the results of predicted and observed blow counts for Pile 2. During continuous driving observed blow counts below a penetration of 35 to 40 meters, tend to follow the lower bound of predicted blow counts based on Method-I (Stevens et al. Method) and are generally bound by upper and lower bounds based on the "Sensitivity Method" (Method-II). The sensitivity method seems to better predict the observed blow counts in the soft Young Bay Mud sediments even in the upper 35 to 40 meters for Pile 2. The observed blow counts spikes at certain depths was as a result of soil pile set-up that occurred during driving delays such as splicing/welding of pile sections. The results also indicate that after 3 to 5m of driving, the setup was broken down and observed blow counts seems to converge with the predicted blow counts. Fig. 11 also demonstrates that if piles are driven to the required design tip elevations, piles can be accepted based on the coring case lower bound acceptance criteria for the given range of hammer energy or efficiencies by either method for clayey soils.

CONCLUSIONS

The combined CAPWAP analysis can be used to estimate the capacity of driven piles with time and to proof-test the piles even if the hammer does not have sufficient energy to drive or mobilize the pile at their ultimate capacity. The combined CAPWAP analysis can also be used to establish the soil-pile setup with time for the clayey soils. It will be valuable data during staged construction in order to establish waiting periods prior to loading the piles. Provided that piles are driven to the required design tip elevations, piles can be accepted based on the coring case, lower bound acceptance criteria for the given range of hammer energy or efficiencies by either method for clayey soils.

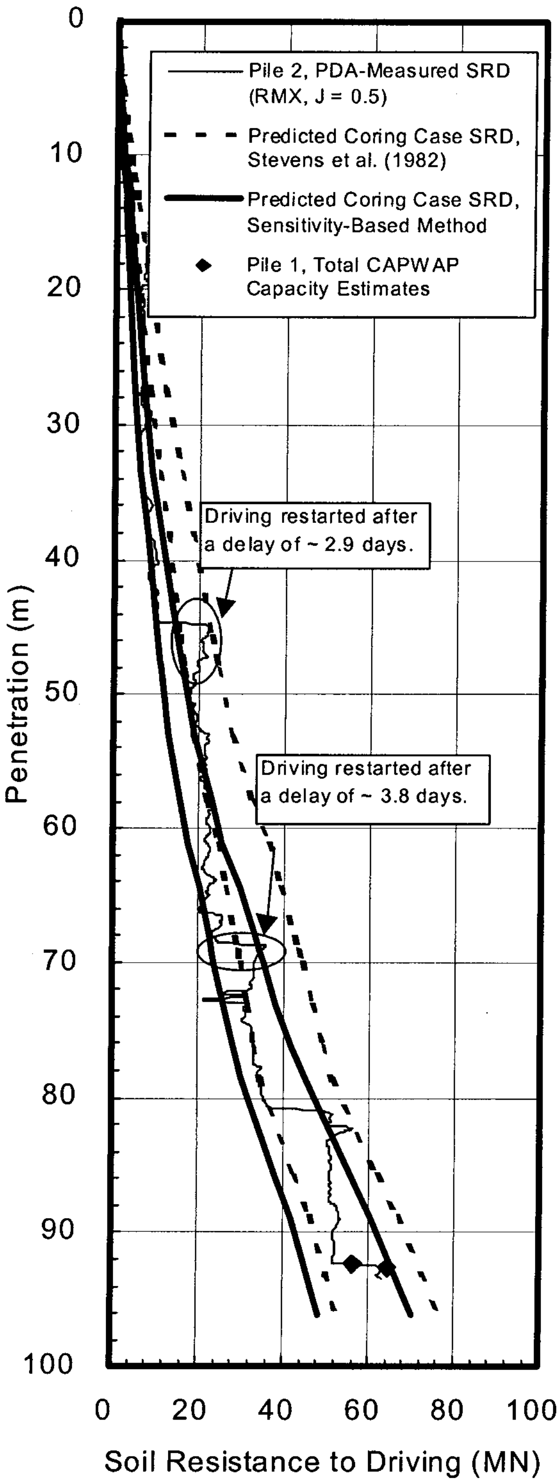
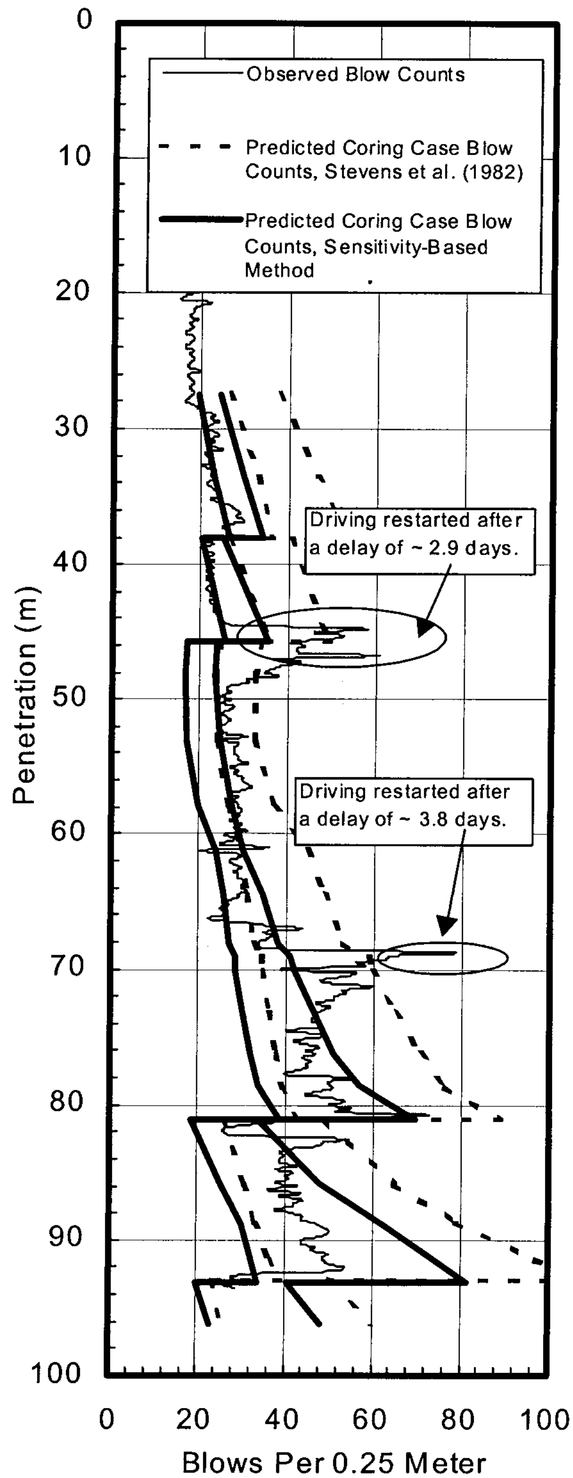


Figure 10. PDA-Measured and Predicted Soil Resistance to Driving Pile No. 2



Note: Blow counts were predicted using a wave equation model with the hammer operated at the PDA-measured transferred energies.

Figure 11. Observed and Predicted Blow Counts, Pile No. 2

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LANDSLIDING ALONG THE HIGHWAY 50 CORRIDOR: GEOLOGY AND SLOPE STABILITY OF THE AMERICAN RIVER CANYON BETWEEN RIVERTON AND STRAWBERRY, El DORADO COUNTY, CALIFORNIA

by

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Abstract

Highway 50 was blocked for 27 days while construction crews removed 350,000 cubic yards of landslide debris following the January 24, 1997, Mill Creek Landslide. At the request of the California Department of Transportation, the California Geological Survey mapped the geology and slope stability of the American River canyon adjacent to Highway 50 for about 15 miles between Riverton and Strawberry. The purpose of this geologic mapping was to identify and locate unstable earth materials underlying the slopes of the canyon and to determine the influence of the geology and geologic structure of the area on slope stability. Over 600 landslides were identified. Over 50 of these landslides moved during the winter of 1996-97. The most unstable slopes are between Riverton and Twentynine Mile Station. The rocks have been weakened by deformation and there are masses of unconsolidated, sandy, material that, prior to this study, had not been recognized. Active landslides occur along or near boundaries between different rock types. Landslides were grouped according to their estimated relative stability to aid Caltrans in setting priorities for mitigation. The area can be subdivided into three domains based on the types and frequency of landslides. Slope inclination, bedrock geology, geologic structure, geomorphology, weathering, vegetation, and precipitation all influence the stability of the slopes along the Highway 50 corridor.

Introduction

Traffic on U.S. Highway 50 (Hwy 50) between Sacramento and South Lake Tahoe (Figure 1) was blocked for 27 days while construction crews removed millions of cubic yards of landslide debris following the January 24, 1997, Mill Creek Landslide (Sydnor, 1997; Spittler and Wagner, 1998). On April 9, 1983, the Highway 50 Landslide closed the highway for 75 days. Highway 50 is a major traffic route crossing the north central Sierra Nevada (Figure 1).

At the request of Department of Transportation, the California Geological Survey (CGS) conducted an investigation of the geology and slope stability of the American River Canyon adjacent to Hwy 50 for about 15 miles between Riverton and Strawberry (Figure 2 a,b).

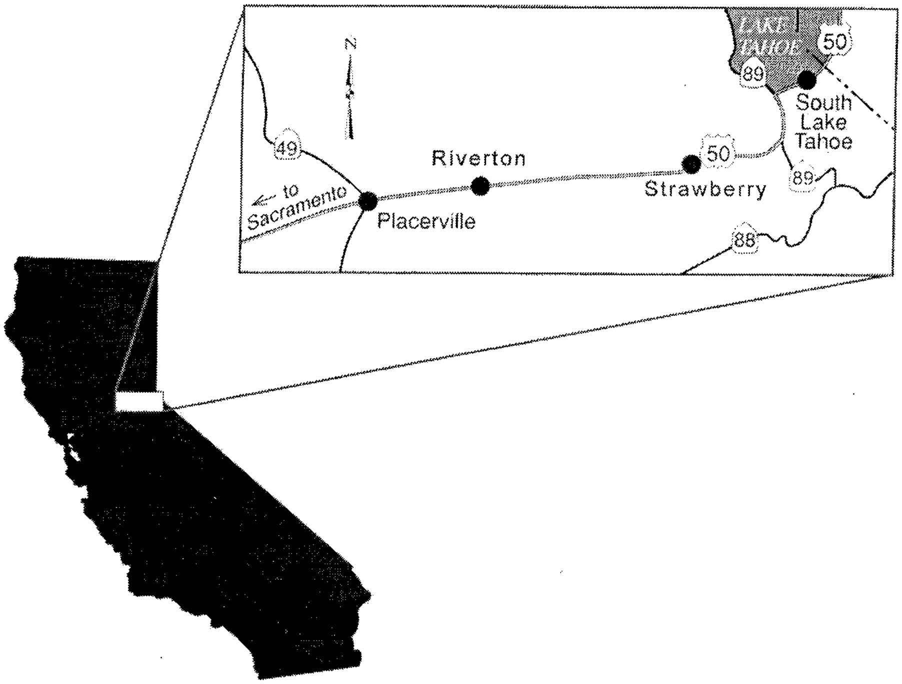


Figure 1. Map showing Highway 50 between Riverton and Strawberry, El Dorado County.

Geologic mapping in the field and interpretation of aerial photographs were employed to identify landslides and landslide source areas that could adversely impact the highway. Mapping was done at a 1:12,000 scale (one inch=1000 feet) on enlargements of parts of the Riverton, Kyburz and Pyramid Peak 7.5' quadrangles. Large scale aerial photographs (1:6,000; one inch=500') flown especially for this project as well as older smaller scale photographs were used to map landslides. The purpose of the geologic mapping was to identify and locate unstable earth materials underlying the slopes of the canyon as well as determining the influence of geologic structure on the slope stability. Aerial photographs and the geologic maps were analyzed to identify landslides most likely to damage or block the highway or to dam the American River causing flooding that could also impact the roadway.

Regional Overview

The Sierra Nevada is a mountain range 50 to 80 miles wide and about 400 miles long, extending from the Cascade Range southward to the Mojave Desert (Fig. 1). The Sierra is a block of the earth's crust that has been tilted westward along a steep fault

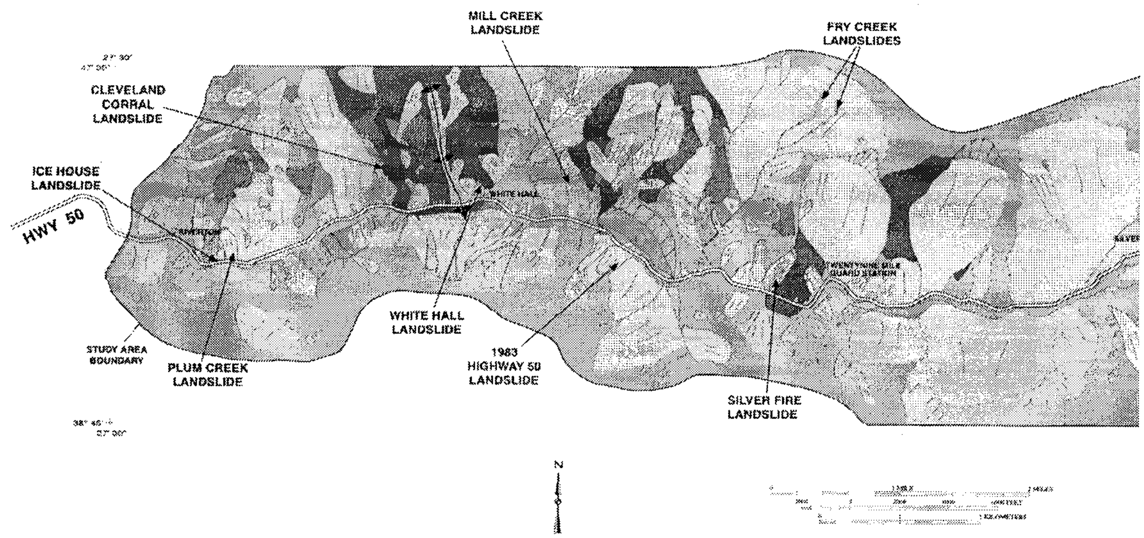


Figure 2a. Geologic map. *Modified from Wagner and Spittler (1997) Plate 2.*

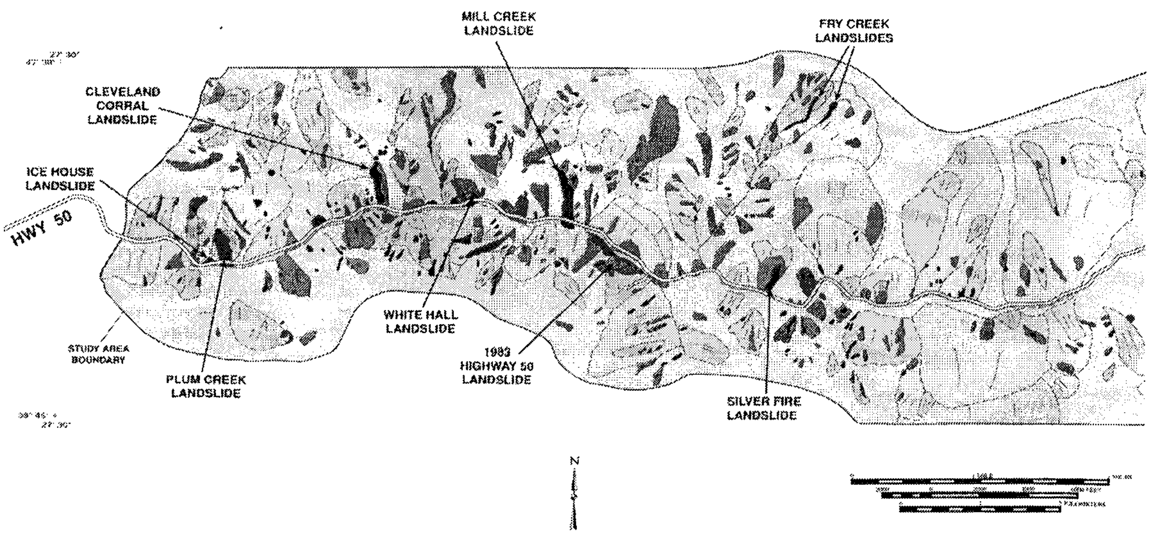
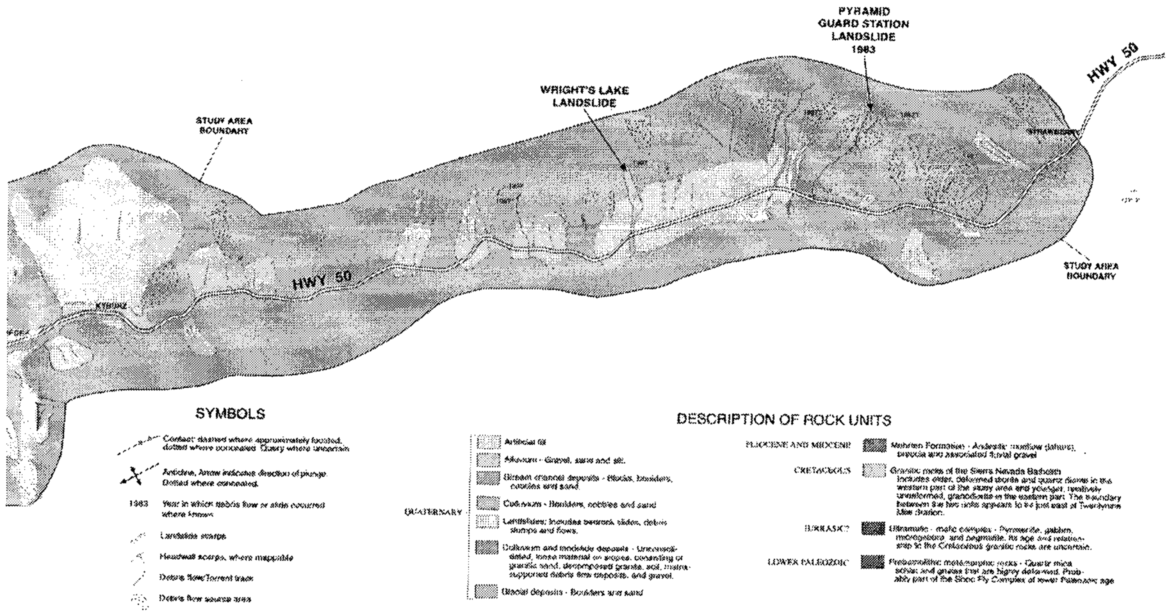
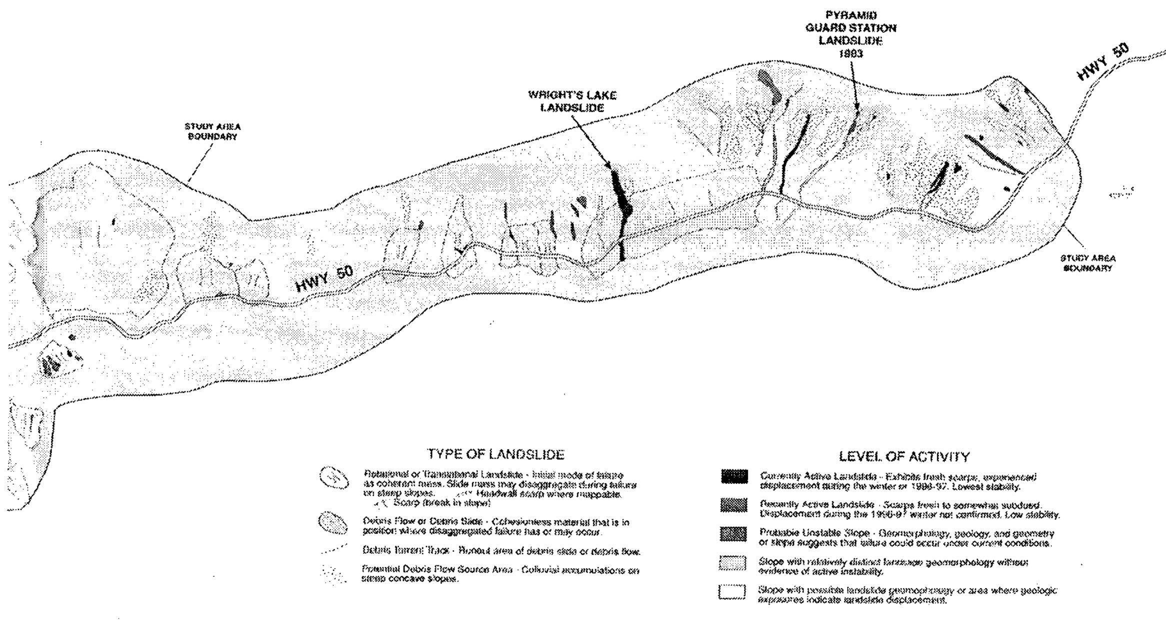


Figure 2b. Landslide map. *Modified from Wagner and Spittler (1997) Plate 3 .*


PYRAMID
GUARD STATION
LANDSLIDE
1983
WRIGHT'S LAKE
LANDSLIDE
STUDY AREA
BOUNDARY
HWY 50
HWY 50
STUDY AREA
BOUNDARY
SYMBOLS
Landslide scarps
Headwall scarps, where mappable
Debris flow source area
DESCRIPTION OF ROCK UNITS
Artificial fill
Cobbles and sand
Glacial deposits - Boulders and sand
QUATERNARY
CRETACEOUS
LOWER PALEOZOIC


PYRAMID
GUARD STATION
LANDSLIDE
1983
WRIGHT'S LAKE
LANDSLIDE
STUDY AREA
BOUNDARY
HWY 50
HWY 50
STUDY AREA
BOUNDARY
TYPE OF LANDSLIDE
Scarp (break in slope)
Debris Flow or Debris Slide
Potential Debris Flow Source Area
steep concave slopes.
LEVEL OF ACTIVITY
Currently Active Landslide
displacement during the winter of 1996-97. Lowest stability.
Recently Active Landslide
Probable Unstable Slope
Slope with relatively distinct landslide geomorphology without
evidence of active instability.
Slope with possible landslide geomorphology or area where geologic
exposures indicate landslide displacement.

escarpment on the east flank of the range, and a relatively gentle sloping west flank. A common perception of many people, geologists included, is that the Sierra is relatively free of landslides, being composed of unweathered, glaciated granitic domes and spires. However, in much of the range there were no Pleistocene glaciers to scour away deeply weathered, weak rocks. During wet years, weak rocks and unconsolidated surficial deposits become water-saturated causing landslides.

The Sierra Nevada is a barrier to moist Pacific storms moving eastward. As the storms pass over the mountains they cool and drop much of their moisture as rain and snow. The 49-year mean annual precipitation at Kyburz is 41 inches. During the two-year period of 1982-83, precipitation was 176% of normal, the wettest period ever recorded in the Sierra Nevada. In 1983, the Highway 50 Landslide closed the highway for 75 days (Kuehn and Bedrossian, 1987). In January 1997 the cumulative precipitation for the winter was 117% of normal when high-intensity, warm rain fell on snow, triggering the Mill Creek Landslide which closed the highway for 27 days. Other landslides in road closures of a few hours or days have also occurred during these years. Major landslides that caused problems during the winter of 1996-97 or are known to be active are the Icehouse, the Cleveland Corral, the White Hall, the Mill Creek, and the Silver Fire landslides (Figure 2 a,b). The Wrights Lake debris flow also closed the highway. Other major debris flows were the Fry Creek and numerous debris flows near Thirtynine Mile signpost west of Strawberry.

Geology

Geologic history of the Sierra Nevada can be divided into three broad phases. During the Paleozoic Era (570-225 million years ago) sediments and volcanic ejecta, destined to become the metamorphic rocks of the Sierra, were deposited in a marine environment. During the Mesozoic Era (225-65 million years ago) molten rock intruded and metamorphosed the Paleozoic rocks far beneath the earth's surface and cooled to form the granitic rock of the Sierra Nevada Batholith. By the earliest part of the Cenozoic Era (65 million years ago to the present), erosion had stripped away miles of the earth's crust to expose the granitic and metamorphic rocks. Later during the Cenozoic, west-flowing rivers cut valleys into the ancestral Sierra Nevada and copious amounts of volcanic material were erupted from volcanoes along the crest and east of the range inundating the ancient landscape. Rivers then began to cut new canyons and were followed by the ice age glaciers during the Pleistocene to put the finishing touches on the spectacular Sierran landscape of today.

Metamorphic Rocks

Metamorphic rocks are the oldest in the area and are exposed in three north-trending masses mostly on the north side of the highway. Dominant rock types are quartz mica schist and gneiss with a lesser amount of quartzite. They are highly deformed, especially along contacts with the granitic rocks, with a strong foliation defined by alignment of platy minerals and compositional layering. Near contacts with the granitic rocks, the metamorphic rocks are highly weathered, which may weaken them.

Ultramafic-mafic complex

Three bodies of an assemblage of ultramafic and mafic igneous rocks were mapped north of the highway between Riverton and Kyburz. The ultramafic-mafic complex includes pyroxenite, gabbro, diorite, and pegmatite. It is associated with granitic rocks, but its structural relationship to them is uncertain. The pyroxenite member of the complex lies immediately above the Mill Creek landslide. Pyroxenite detritus and thick red soil was shed downslope over the area that failed on January 24, 1997 giving the Mill Creek landslide its brilliant red colors. Extensive, largely dormant landslides are associated with the ultramafic-mafic complex.

Granitic Rocks

Granitic rocks of various lithologies and ages (Evernden and Kistler,1970) are the most common rock types in the area investigated. They are dominated by two types of granitic rocks, an older deformed rock, generally of quartz diorite to diorite composition, and a younger, less deformed rock, generally of granodiorite to monzonite in composition. The older dioritic rocks occur in the western part of the area and the granodiortic rocks, occur in the east part.. These rocks are often brecciated and intruded by plugs and dikes of aplite, pegmatite, and granite. Well-developed foliations are defined by the alignment of platy and tabular minerals as well as a pervasive penetrative shear foliation. Foliations in the granitic rocks are parallel to foliations in the metamorphic rocks and they indicate the pluton is tabular. Foliations also indicate there is a major, south-plunging fold near Whitehall (Figure 2a). The younger granodiorite and monzonite are much lighter and coarser-grained rocks. The most common lithology is a biotite-hornblende granodiorite. In places the granodiorite is deeply weathered to grus. There are many pockets disaggregated weathered granitic material on slopes underlain by this unit. Unfortunately, these pockets are only readily observed in burned areas; heavy forest often obscures them. Joints are especially prominent in the granodiorite. Two dominant joint sets trend north. One set dips at gentle angles to the east and the other is near vertical. A third prominent joint set is defined by exfoliation parallel to the slopes of the river canyon. Numerous other, less prominent joint sets are also present.

Merhten Formation

Andesitic volcanic breccia, mudflows and associated volcaniclastic sand and gravel are referred to here as the Merhten Formation. These rocks are part of voluminous andesitic material that was erupted from volcanoes along the present crest of the Sierra from about 18 to four or five million years ago (Miocene and Pliocene) and flowed westward downslope into the ancestral Great Valley. Andesitic deposits inundated most of the landscape of the northern and central Sierra. New drainages were established and the rivers cut new canyons leaving remnants of the volcanics as cap rocks on the ridges.

Colluvium and Grus

Geologic mapping for this project revealed the presence of deposits of poorly consolidated granitic colluvium, decomposed granite, soil, matrix supported debris flow material, and cobble/boulder gravel that have not been identified previously. This material is very unstable and failed extensively during the 1996-97 storms. Small roadcut slumps to large landslides occurred in this material. The most common constituents of the deposits are granitic colluvium and decomposed granite. Both the Icehouse and Mill Creek landslides are composed of decomposed granitic sand and/or colluvium that disaggregated during the sliding. An enigmatic component of these deposits are thick highly oxidized soil horizons. The origin and significance of the soil is unknown. Deposits interpreted to be ancient debris flows are composed of a silty sand and soil matrix with widely separated pebbles. Cobbles of andesite derived from the Merhten Formation are also found in this unit.

Slope Stability and Landsliding

Over 600 landslides are here identified along the Highway 50 corridor between Riverton and Strawberry (Figure 2b). These landslides range from small, recently active debris slides and debris flows to dormant rotational landslide complexes that may have not moved for thousands of years. The mapped landslides include rapid debris slides and debris flows, such as the 1997 Mill Creek landslide, relatively slow-moving rotational or translational landslides, such as the 1983 Highway 50 landslide (Kuehn and Bedrossian, 1987), and highly viscous, slow moving earthflows. Potential debris flow source areas, accumulations of colluvium on relatively steep, concave slopes, are also mapped within the corridor.

Landslide mapping followed two approaches: field mapping of the geology, including those landslides that affects the distribution of geologic units, and interpretation of landslides from stereographic aerial photographs. The photo-interpreted landslides were then field checked and modified as needed. Our confidence in the mapping is bolstered by the correlation in landslide pattern and relative density between the two used mapping techniques

In addition to showing the type and location of each of the many landslides, their estimated relative stability is presented to aid Caltrans in setting priorities for mitigation. The classification of relative stability is as follows:

A. Landslide that moved during the winter of 1996-97.
B. Landslide that appears to be unstable under current conditions or that has distinct surface features, but did not move during 1996-97. This includes historic landslides, such as the 1983 Highway 50 landslide.
C. Landslide that may be unstable under current conditions.
D. Slope with relatively distinct landslide geomorphology without evidence of active instability.
E. Slope with possible landslide geomorphology or area where geologic exposures indicate landslide displacement.

Over 50 landslides that moved during the winter of 1996-97 (Group A landslides) are identified. Most of these are small (less than 1/4 acre) debris flows and debris slides that transported material downslope, but had no significant impact on Highway 50. However, eight of these landslides required road maintenance or grading. Group A landslides include those which may pose chronic problems unless remedial work is performed, as well as acute failures that removed the unstable materials.

Group B landslides have distinct geomorphic expression. In general, Group B rotational or translational landslides and earthflows appear to be intermittently active, with episodes of relatively slow, incremental movement separated by periods of dormancy. Movement of these landslides is typically triggered by high precipitation. Group B debris flows and debris slides include both those that have recently failed, as determined from aerial photographs, as well as well defined, discrete colluvial source areas that appear to be highly unstable. As with the Group A landslides, Group B debris flows and debris slides may be extremely unstable, or they may be the scar left following the failure of the unstable materials.

Landslides mapped in Group C should be considered potentially unstable, but are not as likely to fail as those in Group A or Group B.

Group D and Group E landslides are dormant features that have been modified by post-failure erosion. They are not presently active, but grading or other landuse activities could affect their stability.

In addition to distinct landslides, potential debris flow source areas have been identified. These are areas of colluvial accumulation on steep slopes with local areas of topographic convergence that could concentrate runoff or groundwater. Although these features are not in themselves landslides, they represent areas where debris flows or debris slides are most likely to occur. Stream channels or topographic draws below potential debris flow source areas are most likely to be impacted by debris flows.

The distribution and density of landslides is not uniform throughout the Highway 50 corridor. The area can be subdivided into three distinct landslide domains based upon the types and frequency of landslides:

Domain 1 - Riverton to Twentynine Mile Station (See Figure 2a) - This domain includes a high number of different types of landslides. Complex geology, unstable surficial materials, and highly deformed weakened bedrock underlie the domain. Named active landslides in Domain 1 include the Icehouse, Cleveland Corral, Whitehall, Mill Creek, and Silver Fire landslides, all of which appear to have moved during the 1996-97 winter, as well as numerous other unnamed active slides both north and south of the American River (Figure 2b). Also in this domain is the Highway 50 landslide of 1983. In this area geologic structure exerts a strong influence on the slope stability.

Domain 2 - Twentynine Mile Station and Kyburz (See Figure 2a) - This domain is underlain by granodiorite and in the western part by rocks of the ultramafic-mafic complex. Domain 2 appears to be the most stable of the domains along the Highway 50 corridor. The large bedrock landslides in this domain appear to be dormant but they do exhibit well-defined landslide morphology such as head wall scarps, closed depressions and poorly drained areas.

Domain 3 - Kyburz to Strawberry (See Figure 2a) - This domain is underlain by granodiorite. Extensive debris flows have occurred in this domain, the Wrights Lake landslide and numerous other unnamed debris flows. On June 4, 1983, rapid melting of the deep snowpack triggered the Pyramid Guard Station landslide (Connelly, 1988). All of these landslides are debris flows that failed from colluvial soils within topographic swales on the upper sections of the mountain slopes. Areas with similar slopes and materials are mapped as potential debris flow source areas.

Factors Influencing Slope Stability

Slope inclination, bedrock geology, geologic structure, geomorphology, weathering, vegetation, and precipitation all influence the stability of the slopes along the Highway 50 corridor.

The American River canyon along the Highway 50 corridor is relatively steep, with slope inclinations typically ranging from 40 to 70 percent. Within Domain 1, between Riverton and Twentynine Mile Tract, the slopes tend to be steeper along the lower slopes. Slope angles between Twentynine Mile Tract and Kyburz, Domain 2, tend to be less steep and more subdued. East of Kyburz, in Domain 3, the canyon has more of a U-shape, with the steeper ground high on the ridges, and relatively broad flats at the base of the slopes. In combination with the other conditions, the slope angle is a major determinant in the stability of the corridor.

Bedrock geology, particularly along the contacts between different geologic units, is also a principal controlling factor of landsliding in the area. Two large active features, the Cleveland Corral and the Silver Fire landslides, as well as a host of smaller failures, occur along the contact between the metamorphic rocks and the granitic rocks. The metamorphic rocks are frequently sheared and deeply weathered along the contact. Mechanical disruption along these contacts results in weakened material. The fractures also provide an avenue for groundwater where pore water pressures are frequently elevated. The contact between the Mehrten Formation and underlying granitic and metamorphic rocks is commonly is a locus of landslides. Ground water rapidly permeates through the coarse Mehrten volcanic rocks until it reaches the contact with the underlying granitic or metamorphic bedrock. Lateral migration of groundwater along the contact supports the large number of springs that occur along these zones. Warhaftig (1965) showed that decomposition of granitic rock occurs preferentially beneath andesitic volcanic caprock because there is no drying which slows or ceases the weathering process. The combination of deep weathering along the contact and the high pore water pressures is hypothesized to be a control on landslide frequency. The relationship between landslide frequency and geologic contacts has also been observed in this portion of the Sierra Nevada by Trinda Bedrossian (personal communication, 1997).

The structure of the bedrock geologic units also plays an important role in the stability of the slopes. Metamorphic rocks have a pronounced parting parallel to the foliation which is defined by compositional layering. As mentioned earlier, granitic rocks are cut by fractures or joints. Where foliations in metamorphic rocks or joints in granitic rocks parallel the slopes, a dipslope condition may exist where rock masses may project out-of-slope. About a half of a mile west of Whitehall the metamorphic rocks are folded into south plunging anticline. The active Whitehall landslide has developed upon foliations along a margin of the anticlinal fold where the foliations are parallel to the slope. In addition, the rocks are typically fractured along the crest of the anticline. Not surprisingly, there is a long, narrow landslide along the trace of the fold axis, probably due to the fracturing of the rocks and the out-of-slope character of the foliation.

Topographic swales are commonly the sources of material for debris flows in the area. Typically, the granitic bedrock is typically deeply weathered to grus (decomposed granite) within these swales. It appears that the swales are often controlled by the intersecting joint sets in the underlying bedrock. The combined influence of the out-of-slope projection of the joint-defined wedges, the weakening of the bedrock through decomposition, and relatively steep ground has resulted in unstable slopes. Bedrock jointing and weathering apparently controlled the Mill Creek landslide. The Wrights Lake landslide also appears to have been controlled by jointing at the source area. Most of the other debris flows had source areas in the colluvium filled swales, many of which developed above intersecting joint sets.

Vegetation appears to be a major factor on the distribution of landslides in the Highway 50 corridor. Virtually all of the active landslides occurred within areas that had recently burned. The Cleveland, the Silver Lake, and the Pelican Fires all denuded large areas in the affected areas. The relationship between vegetation and landsliding has been long recognized, particularly with respect to clearcut logging and shallow debris flows. Bishop and Stevens (1964), Endo and Tsuruta (1969), Gray (1970), Wu and others (1979), Ziemer (1981A and 1981 B), Ziemer and Swanston (1977), Abe and Ziemer (1991), and Gray and Sortir (1996) document the role of tree roots in stabilizing hillslopes. Although most of the research has been associated with clearcut logging practices, high intensity wildfires, such as occurred in the Highway 50 corridor, will result in an analogous dieback of tree roots. The effects in burn areas will likely be greater, because wildfires are often much larger than prescribed timber harvests, and because regeneration of a new forest is often harder on large, harsh burn areas.

In addition to reinforcing forest soils, forests actively remove water from hillslopes through evapotranspiration. Gray (1970) and Ziemer (1981A) hypothesized that pore water pressures would increase following clearcut harvesting. Piezometric studies by Keppler and others (1994) show increases in pore pressures persisting at the soil-bedrock interface throughout a 4-year post-harvest study. This is consistent with observations of increases in stream base-level (low summer) flow for 7 to 8 years following timber harvesting and with annual runoff returning to pre-logging conditions within 15 years (Keppler and Ziemer, 1990; Ziemer and others, 1996).

Where forests have been severely burned by wildfire the change in vegetation type and density approximates that of a vegetative conversion. The use of vegetative conversions to increase water yields has been used for centuries (Adams and Coppock, 1986). The increased likelihood of landsliding following conversion is also well known (Corbett and Rice, 1966).

The final factor that affects the stability of the Highway 50 corridor is precipitation. Shallow debris flows, including the Pyramid Guard Station landslide and the 1983 Highway 50 landslide occurred during periods rapid snowmelt in 1983. The Wrights Lake landslide failed during the high-intensity rain-on-snow event of New Year's eve, 1996-97, and the Mill Creek landslide failed just over three weeks following the New Year's eve event. In all of these cases the landslides were triggered by a rapid input of rain water. Future landsliding will also be impacted by precipitation. Major snowpacks and high intensity and duration storms will trigger new or accelerated failures. A major El

Niño event could have major consequences on the stability of the slopes in the Highway 50 corridor.

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Subsidence on I-70 in Russell County, Kansas Related to Salt Dissolution—A History[1]

Presented by
Neil M. Croxton[2]

Abstract

A short section of Interstate 70 in Russell County, Kansas crosses two active sinkholes. These sinkholes have slowly and steadily pulled down the driving lanes since construction of the highway in the mid-1960's. They are the result of dissolution of a thick salt bed over 1300 feet below the surface. Oil drilling activity has allowed fresh water to pass through the salt, dissolving a considerable volume of it and causing the overlying strata to sink. The two areas of interstate have been regraded at significant cost, and efforts were made in 1986 to stop the subsidence at one of the sinkholes, but the lanes continue to drop. Eventually, a nearby bridge will have to be replaced because of the subsidence. The Russell County sinkholes continue to be costly objects of attention to geologists and engineers at the Kansas Department of Transportation.

Discovery of the Crawford Sink

All highway construction in Kansas first has a geology survey. When geology crews began their preliminary studies for this section of I-70 in the early 1960's, there was a large pond along the right-of-way six miles west of Russell. They noticed that the pond appeared rather deep, and although it was situated in a streambed, apparently had no dam. Asking around among local residents, Highway Commission geologists were told that the pond had always been there. An 83-year-old woman who had lived in the area all of her life reported that there had been a pond in that location ever since she could remember. So little additional thought was given to the mysterious origin of the pond. During construction, it was filled in and the highway was built, along with a nearby bridge to carry county traffic over the interstate. Final grading for the new lanes was finished in the spring of 1966.

But something was wrong. Just east of the new bridge, exactly where the pond had been, the subgrade kept dropping. Since it was a stream crossing and a fill section, Highway Commission officials at first just assumed that the fill dirt was settling. But it wasn't just the lanes that were sinking. A few quick level runs confirmed the worst—the new interstate, the pride of Western Kansas, had been built right over a sinkhole.

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[1] Originally presented at the KDOT 2000 Abandoned Underground Mine Workshop in Kansas City, Missouri, April 25-27, 2000.

[2] Neil M. Croxton is the KDOT Regional Geologist for the northwest portion of the state, including the subject area.

The summer of 1966 was a busy one for our geology crews. While I-70's lanes were being paved on either side, geologists and their technicians scrambled to find out what was causing the sinking. Someone thought to check some air photos of the region that were taken in the 1950's. There was no pond. The locals had been wrong—the deep pond with no dam was a relatively new feature. And the State Highway Commission of Kansas had a big problem.

Cause of the Subsidence

I-70 in this area is aligned through the heart of the Gorham Oil Field. It is a very densely-drilled portion of the state, and although sinkholes hadn't caused much trouble at the time, geologists quickly suspected that the subsidence was caused by improper plugging of abandoned wells. Research into oilfield geology and deep groundwater movement led the Highway Commission to the cause—dissolution of the Hutchinson Salt Member.

This salt bed is Permian in age, a member of the Wellington Formation. In western Russell County, the Hutchinson Salt is 270 feet thick and its top is 1300 feet below the surface. Above the salt are three sandstone units—the Dakota Formation, the Cheyenne Sandstone Formation, and the Cedar Hills Sandstone Formation. All three have considerable flows of fresh or brackish water.

Oil wells in the Gorham field were drilled through the salt to a structural high in the Lansing-Kansas City Group and the Arbuckle Group. Hundreds of wells were drilled here beginning in the 1920's; many have since been abandoned. If an abandoned well is not plugged correctly, the fresh water flowing through the overlying sandstones pours down the borehole to replace water taken out of oil-producing layers by nearby active wells. On the way down, the fresh water washes across the salt face, dissolving it. The cavity in the salt grows, and eventually overlying beds sag downward until the depression shows up on the surface.

The sinkhole east of the bridge was named the Crawford Sink, after the Crawford oil lease. There are two wells 50 feet apart at the center of the sink. They are the Crawford 12 and Crawford 16. Both were drilled in 1937 and abandoned in the early 1940's.

Investigation of the Sinkholes

Geologists with the State Highway Commission of Kansas realized that the highway was probably safe as long as the sinking continued. But the I-70 project was too high-profile to take chances with, so during that summer of 1966, before the highway opened, a test hole was drilled. Geology crews drilled a core hole down 240 feet in the Crawford Sink. (This remains the deepest hole ever drilled by our crews). They found only solid bedrock the entire way. The geologist who logged the hole called them, "as perfect cores as you could ever find in that section". They also drilled and dug down to the Fencepost Limestone Member , which is close to the surface here. A structural contour map was drawn that showed the bowl-shaped drop in strata. Officials were reasonably sure that there was not a void under the highway that could suddenly collapse.

About that time, still before the highway was open, construction crews had more bad news: another section of road wasn't holding its profile. This area was a half-mile west of the Crawford Sink. The oil well responsible was the Witt A#1, just south of the right-of-way. This well was drilled in 1937 and plugged in 1957.

Highway Commission engineers and geologists got together late that summer to decide how to proceed. There was serious talk of rerouting the highway around the sinkholes, despite the enormous cost and delays. But the Gorham Oil Field stretches several miles to the north and south. Geologists told officials that there was no way to guarantee that a new alignment wouldn't just put I-70 over other sinkholes. Since there apparently wasn't any danger to the public, the lanes in the subsidence area were paved that fall. Interstate 70 between Russell and Hays opened on schedule on November 16, 1966.

The next summer, the Highway Commission contracted with a drilling company to drill a deep exploratory well at the Crawford site. A crew with Rosencrantz-Bemis Drilling, of Great Bend, Kansas, drilled to 100 feet below the base of the Hutchinson Salt Member, a total of 1670 feet deep. Circulation was lost at 250 feet and never regained, which was attributed to washing of loose material at and below the Dakota Formation. Analysis of a radioactive gamma-ray neutron log indicated that the salt itself was washed along its entire thickness, and had been replaced by material from above. An anhydrite marker bed 350 feet above the salt had, at that time, already dropped 36 feet. Most importantly, however, was that very few voids were found, and none of these were large or near the surface. Officials told the public that the highway was safe, and that the sinkholes would cause only minor damage to the highway.

The 1970's

Nothing was done for a few years. The sinkholes continued to get deeper and broader, and the pond at the Crawford site reformed. The Witt Sink, which formed near the top of a ridge, created a noticeable depression. By 1971, the lanes had dropped so much that they had to be regraded. During the summer of that year, both areas were brought up 5 feet and repaved at the cost of 220,000 dollars. Elevations of the lanes and the bridge were taken every 6 months, making it the most surveyed section of road in the state. The highway continued to drop at almost 6 inches per year. Public relations in the area began to sour when local newspapers figured out that the subsidence showed no sign of stopping. Still, there was no danger, and so people gradually got used to sinkholes under I-70. It became old news.

Overnight on May 1 or 2, 1978, however, a huge sinkhole suddenly opened up in a field 20 miles northwest of the I-70 sinks. This hole, in northeast Ellis County, was also centered on an old well. In a few days, the hole was 75 feet across and 100 feet deep. The press coverage of the nearby sudden collapse forced the Highway Department, now KDOT, into action once again.

Our geologists were still reasonably sure that the gradual subsidence of the highway was a good indication that nothing catastrophic was going to happen to I-70. But, primarily in response to public pressure, new studies of the problem were ordered late that year. The Kansas Department of Health and Environment (KDHE) and the Kansas Geological Survey (KGS) helped this time. Not enough money was available to do any deep drilling, but the KGS ran seismic refraction surveys along both the I-70 sinkholes and near the Ellis County collapse. Again, no near-surface voids were found beneath I-70, and few deep voids. No satisfactory conclusions were ever drawn, however, as to why the Ellis County sinkhole behaved differently from the I-70 sinks.

In addition, the KDHE took infrared air photos of the Gorham Oil Field to try to identify new sinkholes that might be developing. The idea was that shallow surface depressions would hold water after rainfall, and therefore show more lush vegetation. Regular air photos were taken early in the morning and late in the afternoon, to try to find new sinks highlighted by shadows cast by the low-angle sunlight. Neither of these endeavors yielded much useful information. The *Hays Daily News* took credit for instigating the investigations; the uproar finally died down.

Attempt to Stop Subsidence at the Witt Sinkhole

By August of 1984, the lanes at the Witt sink were back down 8 feet. It was starting to cause sight distance problems—engineers were afraid that a stalled car at the bottom of the depression would be rear-ended. So the Witt was again regraded and repaved at a cost of nearly 500,000 dollars.

KDOT engineers in that district were beginning to get more and more concerned. There didn't seem to be any end to the subsidence, and public relations in the Hays-Russell area were a serious problem. In January, 1986, the state again contracted with a drilling company. The goal this time was to stop subsidence at the Witt sinkhole by shutting off the flow of water across the salt.

A hole was drilled 4 feet west of the old Witt A#1 well, to a depth of 1948 feet. This is approximately 400 feet below the base of the salt. Circulation was lost at 157 feet and never regained. A thickness of 124 feet of the salt was dissolved at that time, having been replaced by soft, residual material. No major voids were found during the drilling. Casing was set and cemented.

The casing was perforated between 1930 and 1940 feet, in a limestone zone below the Wellington Formation. The limestone was acidized and fractured in an attempt to establish communication with the original hole.

After this cement had set, the casing was again perforated, this time at the base of the salt between 1538 and 1548 feet. Fresh water was then pumped at 4 to 5 gallons a minute in an attempt to wash through the basal salt and reach the old hole. After 400 barrels of water had been used, there was a pressure drop. Then, 200 cubic yards of cement with additives was squeezed to try to plug the old well at the base of the salt.

Over 30 cubic yards of cement were pumped down the new hole, until pressure built up. Satisfied that the breach below the salt had been sealed, officials resumed their frequent surveys of the lanes. And in fact, it worked—for a while. For 6 months, the lanes didn't move. But somehow, water got around the plug and the highway quickly resumed its subsidence of 5 to 6 inches a year.

In 1988, more cement was pumped down both holes at the Witt sink. This cement was saturated with salt in the hopes that it would bond to the salt face itself. After "lubricating" the hole and cavity with 200 sacks of bentonite, drillers pumped almost 100 cubic yards of the salt-rich cement down the holes. Eventually, pressure built up to 300 psi in both wells at the same time. The voids in the immediate vicinity of the of the boreholes were filled, and KDOT geologists were again cautiously optimistic. Again, their optimism was short-lived, because water soon ate another hole in the salt and the subsidence continued.

Nothing has been done at the sinkholes since this attempt to stop movement at the Witt sink. A drilling program of several holes in a circular pattern around each original well hole would probably give the best chance of stopping the subsidence. No one can guess the quantities of cement or other fill material that would be required to fill the voids at depth. Of course, the tremendous cost of such an undertaking is a concern, especially since there is no guarantee that it would succeed. In the current economic climate, it seems unlikely that another attempt will be made to halt the subsidence in the foreseeable future.

The Bridge

The highway in the vicinity of the Witt sink can be regraded as many times as necessary. The Crawford sink is a different matter. It can't be regraded any more because of clearance requirements under the nearby bridge; the bridge is sinking, too. One end of the 2-pier concrete structure has dropped over 6 feet since it was built in 1965. At the south abutment, the east curb is now 2 feet higher than the west curb. Strangely, about the only indication that the bridge is under any stress at all is narrow cracks in the curbs just over the piers. Except for its noticeable tilt to the east, the structure looks like any other 35-year-old bridge in Kansas.

The Future of the Russell County Sinks

After over a decade of inactivity, work is once again planned for I-70 in the vicinity of the sinkholes. The Crawford sink is the biggest concern--either it will finally subside enough to become a sight distance problem, or water will begin to cover the roadway during storms. A current project proposal calls for removing the bridge over the highway at the Crawford sinkhole and closing the county road. The roadbed, which in addition to dropping has, in recent years, developed a distinct tilt to the south, can then be brought back to grade. The highway will also be regraded through the Witt area. This reconstruction project is currently scheduled to start in June, 2003.

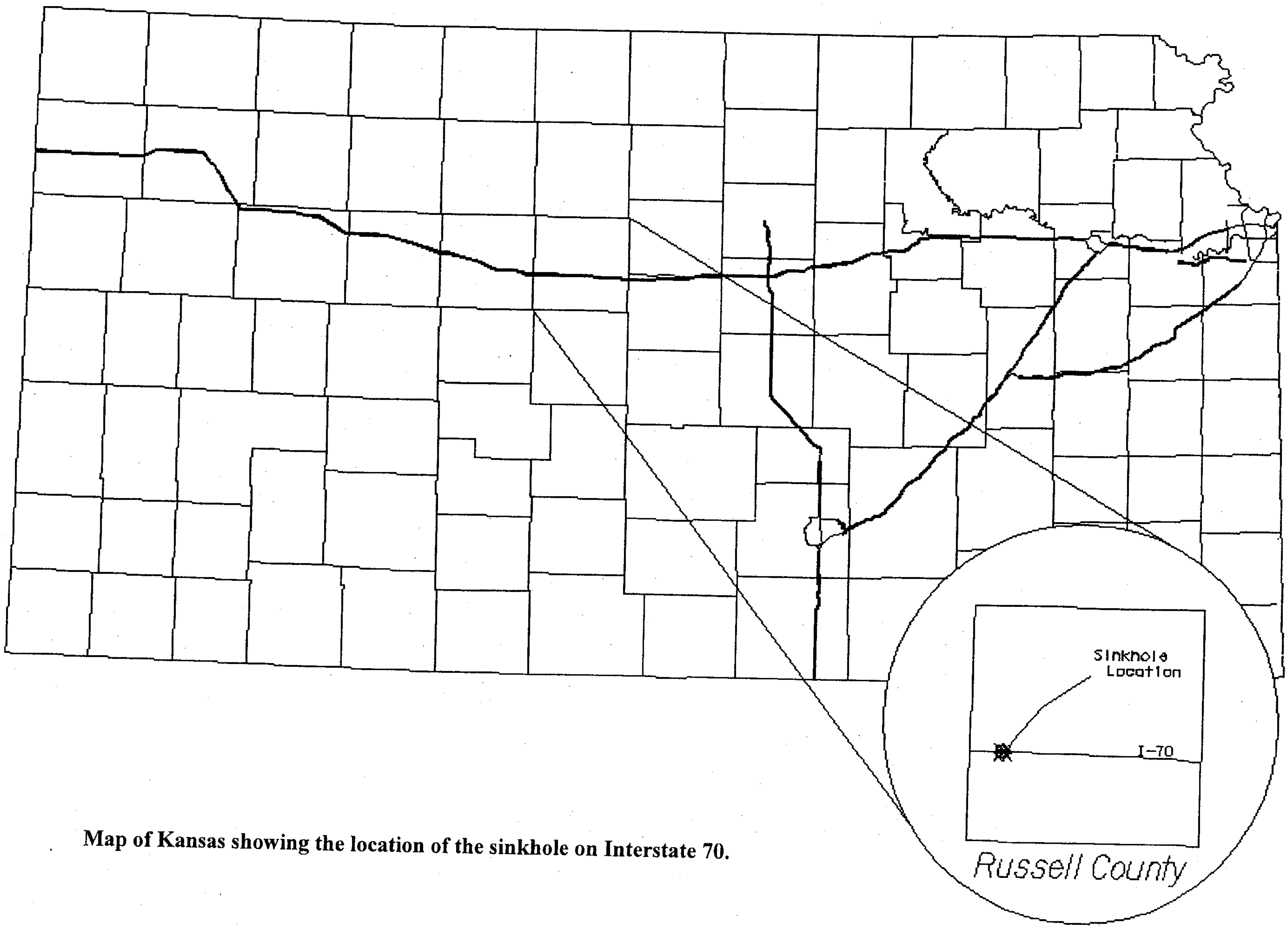
Meanwhile, traffic continues to go over the sinks at a count of over 11,000 vehicles a day. The Witt sink continues to drop at 5 inches a year; the Crawford is subsiding at about 4 inches a year. They will keep sinking until the Gorham Oil Field is finally abandoned and water is no longer drawn out of the strata below the salt.

Summary

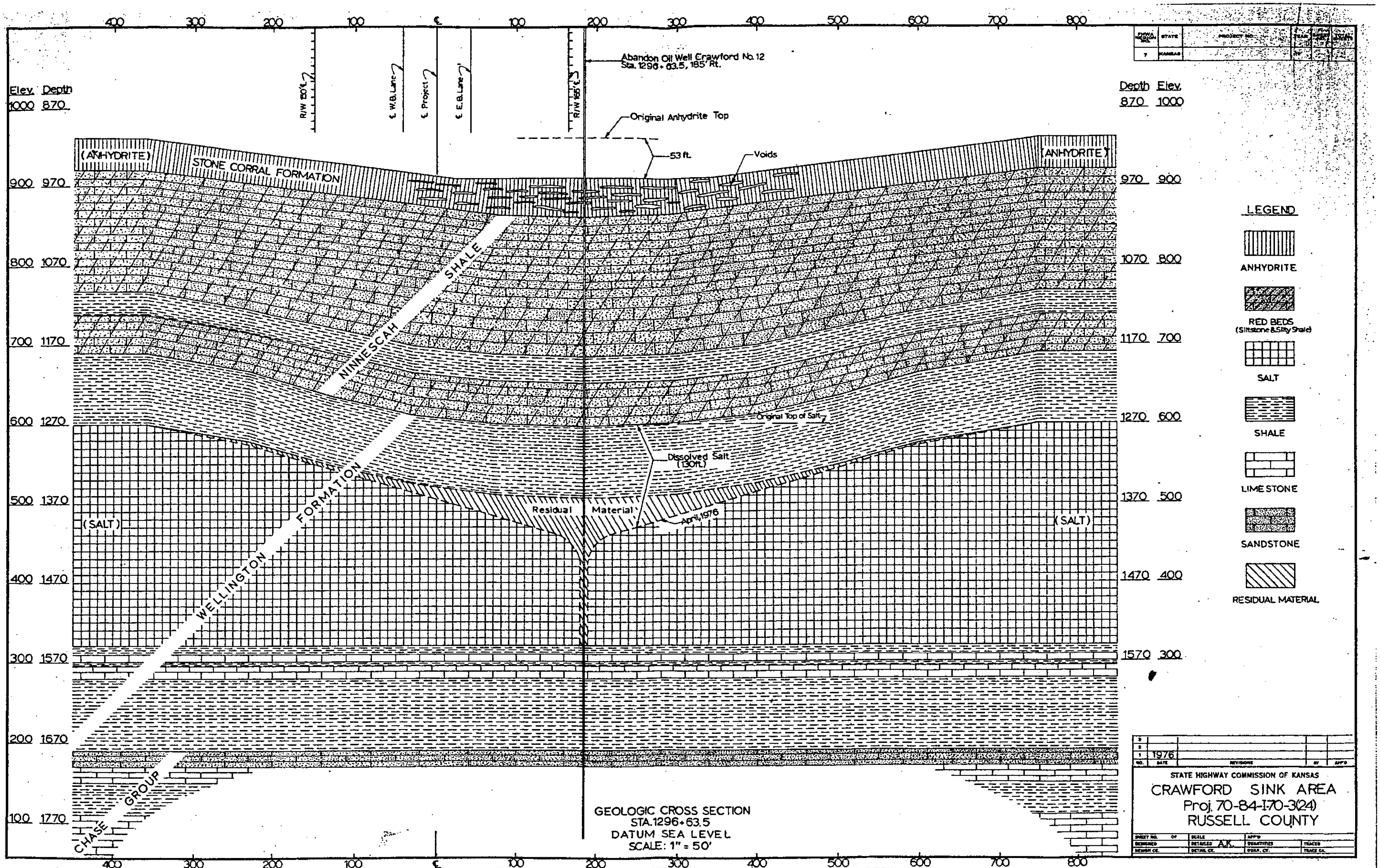
Improper plugging of abandoned oil wells in Russell County led to the development of two large salt-related sinkholes beneath Interstate 70. The Kansas Department of Transportation has struggled with costly, embarrassing repairs and a failed attempt at remediation during the 35-year history of the highway. Despite knowing in detail the cause of the subsidence, an economical solution has eluded us. At present, KDOT plans to simply regrade the lanes for as long as possible. The sinkholes will continue to consume our time and resources well into the 21st century.

Acknowledgments

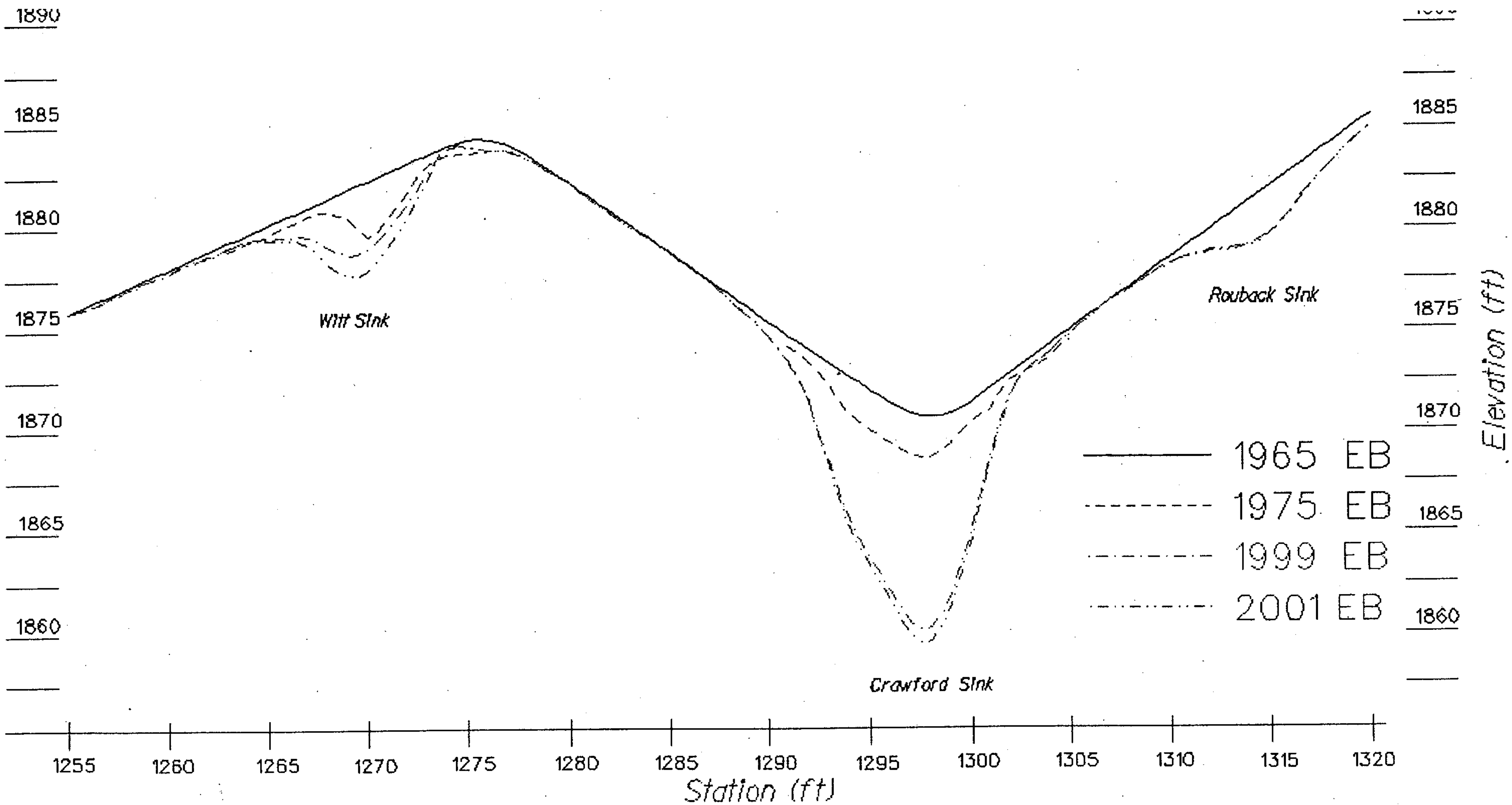
Most of the information presented in this paper was taken from records and reports on file at the KDOT Geology Section in Topeka, Kansas. In addition, the author would like to acknowledge valuable information gained from interviews with the following individuals, all of whom work or have worked for the KDOT: Mr. Wally Taylor, Regional Geologist, northwest region (retired); Mr. Larry Rockers, Chief Geologist (retired); Mr. Ron Sherard, Area Engineer at Hays; and Mr. Wes Moore, District Maintenance Engineer at Norton. Recognition is also given to the Photo Section in Topeka for their assistance in preparing the photographs for this paper, and to Jeff Geist, R.J. Crow, Herb Streit, Bruce Havercamp, and Lynn Byrne for their help in preparing the diagrams.



Map of Kansas showing the location of the sinkhole on Interstate 70.



Cross-section of the Crawford Sink, drawn after the initial deep study in 1967.



Profile of the eastbound lanes of I-70 in the area, comparing elevations of the original grade and those surveyed in May, 2001. The small "Rouback Sink" to the east moved slightly in the 1970's but has since stabilized.

Interstate 77 Abandoned Underground Mine Remediation Project

L. Rick Ruegsegger, P.E.[1], Kirk D. Beach[2] and Frank Rahmlow[3]

ABSTRACT

During 2001, the Ohio Department of Transportation (ODOT) completed a significant project for the purpose of remediating unstable surface and subgrade conditions related to an abandoned underground coal mine lying beneath Interstate 77 in northeastern Ohio. This project was located within the glaciated boundary of Summit County. An ODOT district engineer identified surface subsidence features related to mine subsidence in late March 2001. An abandoned underground mine map indicated that 800 feet of the interstate highway was underlain by coal mine workings. This mine extracted the Brookville #4 coal within the Allegheny Formation. An ODOT subsurface investigation found mine workings at a depth of 25 to 35 feet below the road surface. The overburden consisted of three to four feet of highly broken and jointed limestone overlain with thick, interbedded deposits of sand and/or clay containing a high groundwater table. An ODOT Abandoned Underground Mine Inventory and Risk Assessment of the location determined it to be a high risk site requiring immediate mine stabilization work. An emergency contract was established and construction began in May 2001. Drilling program modifications were required to address the sand formations and the high ground water table. Time Domain Reflectometry instrumentation was installed to monitor the roadway which continued to carry the 75,000 ADT traffic. The resulting emergency project construction included air rotary and sonic drilling, and placement of cement flyash grout in mine workings beneath 2000 feet of roadway. This project required 25,000 cubic yards of grout and was completed in October 2001 at a cost of $8.4 million.

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INTRODUCTION

In March 2001, the Ohio Department of Transportation (ODOT), District 4 Abandoned Underground Mine Inventory and Risk Assessment (AUMIRA)[1, 2] coordinator contacted the ODOT Office of Geotechnical Engineering (OGE) to report an Interstate 77 (IR77) location evidencing possible surface subsidence related to abandoned underground mines. The site was located in Summit County, Ohio and was underlain by the Brookville #4 coal within the Allegheny formation. This roadway section was scheduled for a major new project for the purpose of adding lanes and pavement replacement. District 4 had become aware of the possible mine subsidence conditions as the result of performing a reconnaissance to screen this future project site for mine problems.

Ohio Department of Natural Resources, Division of Geological Survey (DGS) records indicated that two abandoned underground mine maps for this vicinity were available (Figure 1). These maps were for the Shaffer Overholt Mine (Map #St-34) operated by the Overholt Coal Co. until 1936, and the Haurer Mine (Map #St-35) which was operated by R&T Coal Co. until abandonment in 1937. These mines utilized room-and-pillar mining techniques which typically resulted in an extraction rate of more than 50 percent.

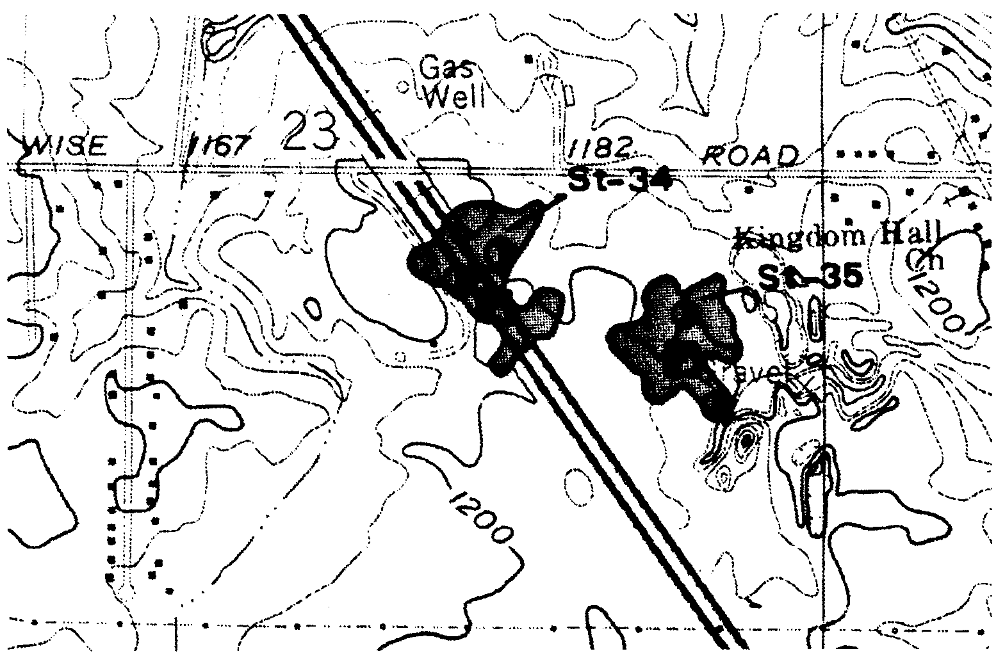


Figure 1: Project Vicinity Map Showing Available
DGS Abandoned Underground Mine Maps (St-34 and St-35)
N.T.S.

The two underground mine abandonment maps for the vicinity were obtained from the DGS. The DGS could only provide paper copies of these maps. The U.S. Department of Interior, Office of Surface Mining (OSM), Appalachian Resource Center, National Mine Map Repository in

Pittsburgh, Pennsylvania had the same mine maps available in the form of electronic image files. OGE recovered these maps and transmitted them to ODOT District 4. The District 4 Office electronically superimposed the mine maps on the existing roadway plan sheets. The resulting plan view provided the approximate location and orientation of the abandoned underground mine underlying the roadway (Figure 2). This overlay of abandoned mine maps on the roadway drawings indicated that approximately 800 feet of I-77 were underlain by the Shaffer Overholt Mine (Map #St-34). Another 700 feet of I-77 were located immediately adjacent to the Haurer Mine (Map #St-35).

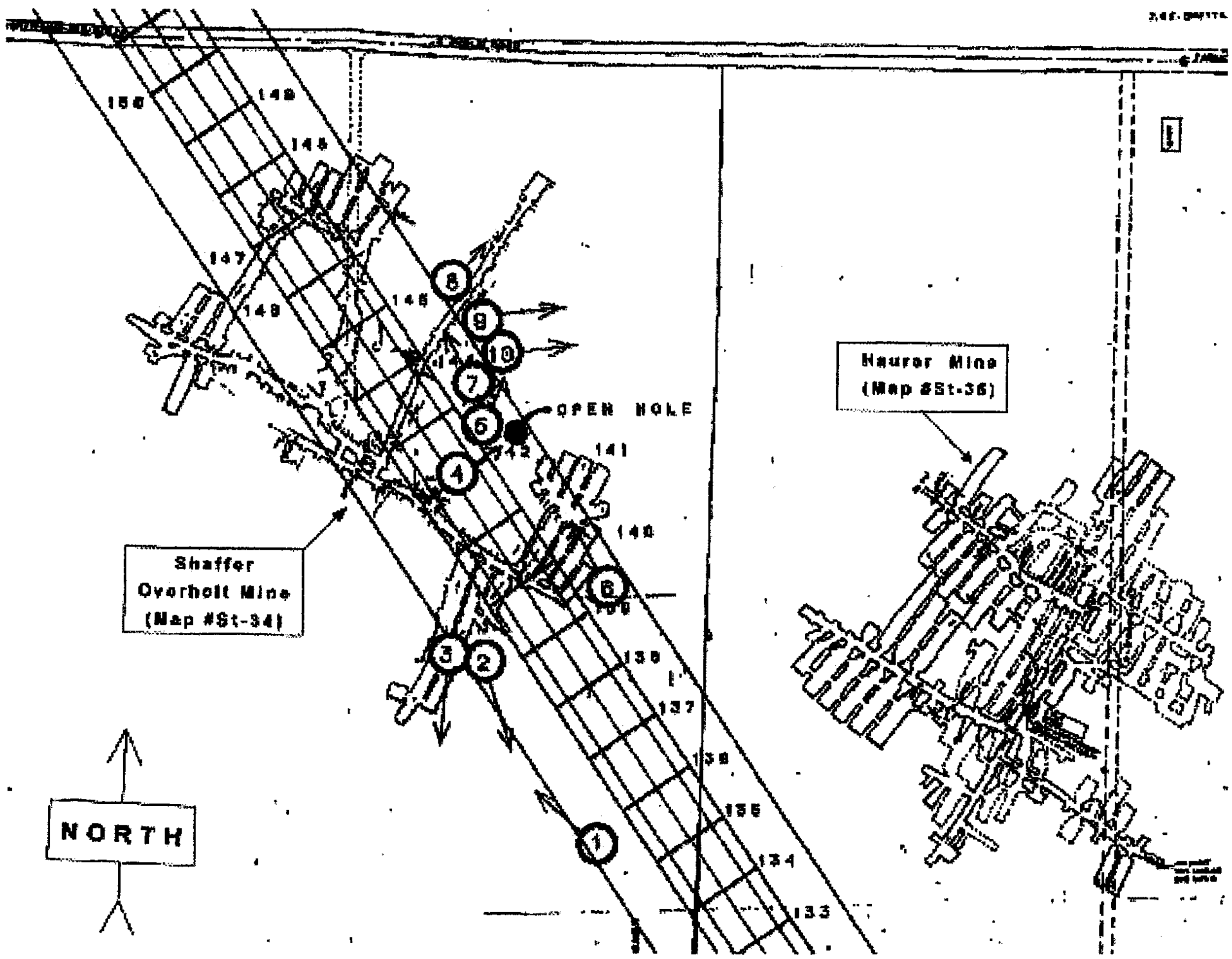


Figure 2: Plan View Showing Approximate Location of the Mapped Abandoned Underground Mines
N.T.S.
(Note: Numbered circles with arrows indicate locations and perspectives of field reconnaissance photographs not included in this paper.)

SITE RECONNAISSANCE

A joint site reconnaissance by OGE and District 4 was next conducted to review available information, to view observable site features, and to determine subsurface investigation requirements. The plan sheet displaying the superimposition of the mine maps on the roadway was used as a guide for locating and interpreting observable site features.

A recent surface deformation in the southbound ditch line was found to be the general location of a possible haulage shaft per the mine map overlay on the roadway plan view. Based on the shaft depth obtained from the mine map, an estimated mine void height of 5 feet, an estimated shaft diameter of 10 feet and an angle of draw of 35 degrees, the diameter of the potential danger zone around such a shaft was estimated to be approximately 60 feet. The mine map/roadway overlay sheet indicated that a similar potential shaft danger zone existed at a mapped pump shaft location beneath the northbound driving lanes. At another location, a water-filled surface depression near the northbound right of way fence (noted as an "open hole"on Figure 2) was suspected of being the location of an unmapped mine shaft with a similar danger zone. The actual existence of these possible shaft danger zones depended on whether the mapped shafts were properly backfilled at the time of abandonment, during roadway construction, or later as roadway maintenance work. Surface subsidence features were also observed in the adjoining fields located east and west of the right of way. This roadway location was found to have a very high risk profile per the AUMIRA risk assessment criteria for surface deformation sites. OGE recommended that a priority site investigation and appropriate site monitoring should be performed for this site per the ODOT AUMIRA manual.

Surveying of 100-foot stations was immediately requested. Station locations were located using PK nails driven into the asphalt pavement on both outside shoulders. Surveying to locate the three (3) mine shafts shown on the available mine maps was also ordered. The district was requested to compile all existing documentation of the area (e.g., construction diaries, maintenance work records of filling sinkholes or mine shafts, etc.). The district search for documentation found that the only ODOT records available for this location were the original roadway construction drawings. Based on review of the available information and site reconnaissance observations, a limited subsurface investigation was recommended to determine the stability of the I-77 subgrade.

GEOTECHNICAL INVESTIGATION

An OGE drill crew and rig performed a subsurface investigation to "ground truth" the mine map overlay on the roadway plan view. This limited drilling program entailed the installation of five (5) boreholes along IR77.

Two (2) boreholes were located on the northbound shoulder adjacent to the St-35 mine map. Three (3) boreholes were drilled in the area of the St-34 mine map. Two of these borings were on the northbound shoulder, and one was drilled on the southbound shoulder. The proposed boring along the edge of the southbound shoulder was located 150 feet away from the possible shaft location in the southbound ditch line. All boreholes were located in the soil at the edge of the paved outside shoulder and were drilled to an elevation below the Brookville coal and underclay elevation. Borehole depths ranged from 40 to 55 feet. The water table was encountered at a depth of 6 feet to 10 feet below the ground surface. The unconsolidated material was sampled at 5-feet intervals. Bedrock cores were recovered and stored. Upon completion of each hole, neat cement grout was tremie placed into each borehole up to the ground surface.

The overburden interval between the roadway and the top of the Brookville #4 coal was found to range from 25 to 39 feet. The overburden was found to be primarily composed of glacial deposits overlying a lesser amount of bedrock. The depth of glacial deposits overlying bedrock ranged from 22 to 35 feet in depth. These glacial materials were predominantly sandy silt and gravelly sand. One boring also reported a 3 ½ foot boulder zone immediately overlying the top of bedrock. The bedrock underlying the glacial materials and above the mined coal ranged from 3 to 4 feet. This bedrock portion of the overburden consisted of a carbonaceous limestone that was highly broken and jointed. The Rock Quality Designation (RQD) for the bedrock overlying the coal was consistently "very poor" (RQD = 0) providing little structural support. This low rating was due to the thin bedded nature of the rock and the high degree of jointing. The Brookville #4 coal which was mined beneath this overburden interval was found to range from 4 to 7 feet. This coal was immediately underlain typically be a carbonaceous siltstone ranging from 1 to 4 ½ feet. Two of the five borings encountered the top of mine voids at depths of 25 and 35 ½ feet. The most alarming of these borings recorded 22 feet of unconsolidated glacial deposits overlying 3 feet of the broken, highly jointed limestone. In abandoned room and pillar mine workings, remnant pillars and blocks of coal, as well as water pressure in the case of flooded mines, provide support for the overlying rock. The localized mine subsidence occurrences were surmised to be the result of pillar failures, roof rock failure, and changes in water pressure within the abandoned underground mines.

One of the criteria utilized in the AUMIRA process to measure the likelihood of subsidence on an evaluated site is the ratio of unconsolidated material to bedrock in the overburden. This ratio for the five borings ranged in value from approximately 5 to 9. The AUMIRA process identifies a value greater than 1 for this ratio as an indicator that a higher risk of mine subsidence progression to the ground surface exists if abandoned underground mines were present.

An AUMIRA Detailed Surface Deformation Site Evaluation was performed for this location. The site was found to score at the highest level of risk possible for most evaluation criteria. This AUMIRA evaluation confirmed the need to perform remedial construction to arrest the mine subsidence activity along the roadway section.

REMEDIATION

A drilling and grouting program was chosen as the mine remediation method to be used to stabilize the roadway. An emergency force account construction project was quickly initiated in April 2001. OGE provided a draft set of drilling and grouting specifications, but no formal construction plans or specifications were made a part of the project documents. ODOT determined that the drilling and grouting program would be conducted as two phases of work. The first phase of this work would involve the drilling of lines of barrier boreholes into the mine void system in a pattern that would define the perimeter of the area requiring stabilization. Lower slump grout would then be pumped into these holes to create a containing barrier around the area to be stabilized. Next, a pattern of boreholes would be drilled across the area within the barrier hole lines and higher slump grout would be tremied into the mine void system contained within the area bounded by the completed barrier borehole lines.

This remedial approach had previously been used by ODOT in 1995 to stabilize Interstate 70 (IR70) in Guernsey County, Ohio when mine subsidence caused collapse of an eastbound driving lane and closure of the roadway. This form of remediation when used to stabilize IR70 in 1995 had induced subsidence events within the right of way adjacent to the drilling and grouting operations. These 1995 subsidence events did not pose a threat to the traveling public because all traffic had been detoured off the IR70 roadway during construction.

ODOT searched for an acceptable detour that would similarly remove the traffic from the possible danger of remediation-induced subsidence events on the IR77 roadway. However, no acceptable route was available for detouring the 75,000 average daily traffic volume to another location while construction was undertaken. Therefore, ODOT had to provide for the unimpeded passage and the continued safety of the normal four-lane traffic flow through the 2000 foot length of the project work area while completing the project work.

Time Domain Reflectometry (TDR)[3] cables and instrumentation were installed to monitor the roadway to provide an ongoing monitoring system to protect the safety of the traveling public within the mine remediation project area. TDR cables were placed horizontally beneath the lanes carrying the continuing traffic. The installation of these TDR cables was accomplished by utilizing horizontal drilling equipment and techniques commonly utilized for domestic utility installation. Four (4) lines of TDR cables were utilized to monitor any vertical movements of the subgrade below the 2000 foot lanes carrying the traffic continuing traffic through the construction project area.

Based on 1995 ODOT experience with remediation-induced subsidence events on IR70, drilling of the unconsolidated portion of the IR77 overburden was not to be accomplished by air rotary methods. The contractor initially proposed to use rotary drilling with water to accomplish this portion of the drilling. Upon arrival of the drilling equipment, it was determined that it was logistically unfeasible to provide the necessary water supply to perform the amount of water rotary drilling to perform the unconsolidated overburden drilling required for the project. The only readily available source of water was the flooded underlying mine complex. All parties agreed that the pumping of water out of the mine could in itself trigger unpredictable new subsidence activity along the roadway section.

Due to the lack of an available water supply for water rotary drilling, initial barrier borehole drilling was attempted utilizing air rotary drilling. Unexpected concentrations of well-sorted sands were encountered in some boreholes. This condition caused boreholes to collapse before casings could be installed. The use of air rotary drilling also caused lateral displacement of soils. This displacement of soils induced subsidence at boreholes being drilled and also at previously drilled adjacent boreholes. Therefore, air rotary drilling was immediately halted.

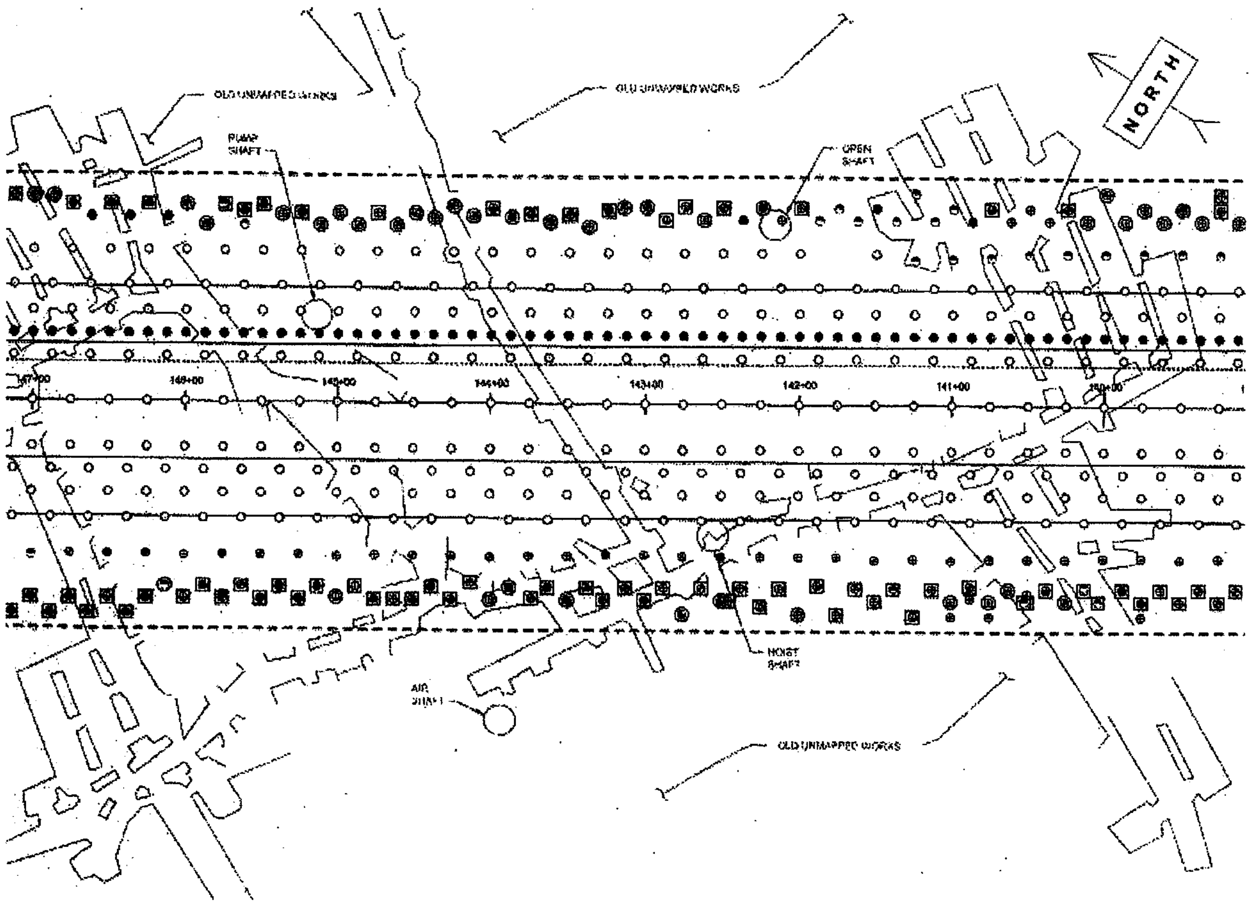
Sonic drilling, which advances a casing using vibratory energy and a small amount of water, was thought to be the only drilling method that might be a viable option for the given nature of the overburden. An available sonic drilling rig was found in an adjacent state. ODOT requested the sonic drilling rig to mobilize to the site and to demonstrate its capability to construct cased boreholes through the unconsolidated overburden and into the bedrock.

The sonic drilling rig successfully demonstrated its ability to construct boreholes which had casings that extended through the unconsolidated overburden and were socketed into the top of the bedrock. One sonic drilling demonstration borehole required additional time for completion. This slower drilling progress was later determined to be the result of the project's first encounter with a zone of glacial debris containing boulders as large as one to two feet in diameter. The field demonstration proved that sonic drilling equipment was capable of non-destructive drilling of the unconsolidated overburden and construction of casings socketed into bedrock. These equipment capabilities would allow for the completion of boreholes by air rotary drilling of the bedrock, through the cased boreholes and into the mined coal interval. This demonstrated drilling sequence would allow the envisioned drilling and grouting program to go forward. ODOT decided to retain the sonic drilling equipment for immediate work drilling the first barrier boreholes.

The early barrier borehole drilling established the general drilling and grouting approach that was utilized for the entire project. This general drilling approach for constructing cased boreholes into the underlying mine void system included:

1) sonic drilling advancing an 8-inch casing into the top of bedrock to create a bedrock socket
2) utilization of the 8-inch sonic casing to install and grout a 6-inch PVC casing into the bedrock socket.
3) air rotary drilling utilizing a 5 ½-inch bit inside the 6-inch PVC casing to complete drilling through the bedrock overburden and into the mined interval
4) grouting of the mined interval and the broken/caved bedrock overburden utilizing a 4-inch tremie placed down into the mined interval.

Additional sonic and air rotary drill rigs were mobilized to the site, and the schedule for drilling and grouting of barrier holes was accelerated. The construction plan called for two parallel lines of barrier boreholes to be drilled along each edge of the right of way. Boreholes in each line of barrier holes were to be drilled at 25 foot center-to-center spacing. The parallel lines of barrier boreholes were spaced 10 feet apart. Boreholes in the adjacent lines of barrier boreholes were staggered 12 ½ feet (Figure 3).



<u>Figure 3: Typical Barrier Line and Production Borehole Spacing</u>
N.T.S.
(Note: Various symbols at borehole locations reflect drilling and grouting status.)

Grouting of the barrier boreholes entailed tremied placement of 8-inch slump barrier grout in each borehole. The barrier grout mix consisted of 23% cement and 77% sand . Most boreholes required several iterations of grouting which were performed over several days. The volume of grout placed in the barrier boreholes was limited to 25 cubic yards per each iteration of placement. Drilling of barrier borehole lines was initiated within the area underlain by the mapped abandoned mine workings. Then, the barrier borehole line drilling was advanced up station and down station along the right of way as a means for determining the actual limits of all mapped and unmapped abandoned mine workings. This approach reflected the past ODOT experience that when remediating such sites, the actual underlying mine workings had commonly been more extensive than the available maps indicated. During this portion of the work, hydraulic communication due to interconnected mine workings occurred between cased barrier holes at distances as great as 1200 feet. Through the advancement of barrier lines along the roadway, the actual extent of roadway underlain by abandoned underground mines was determined to be 2000 feet in length. This length was significantly greater than the 800-foot section of roadway which the available abandoned underground mine map indicated was underlain by abandoned underground mine workings.

The drilling of production boreholes within the area bounded by the grouted lines of barrier boreholes was next begun. Drilling methods for these boreholes were the same as previously described for barrier borehole drilling. Production boreholes were generally placed on a grid pattern at 25-foot spacing (Figure 3). Grouting of the production boreholes entailed tremied placement of high-slump (flow cone test: 15 to 25 seconds) grout in each borehole. The grout mix consisted of 11% cement, 45% sand and 44% flyash. An Ohio EPA UIC Permit to Install and Permit To Operate A Class V - Injection Well Area Permit for Well Code 5X13 was required prior to placement of the production grout because of its flyash component. Most production boreholes required multiple iterations of grouting performed over several days. Since this production grouting was within the previously established barrier lines and because the intent of this grouting was to accomplish maximum lateral placement of grout within the barrier lines, grout volumes placed in these boreholes were not limited. The maximum volume of grout tremied into the abandoned underground mine workings by any production borehole was approximately 800 cubic yards

ODOT recognized that this stage of the work was of a much more serious nature for two reasons. First, the required grade location of the work was adjacent to, or in between, lanes of continuous interstate highway traffic. Secondly, the required subsurface construction included lateral placement of grout below the adjacent lanes carrying traffic, with the possible occurrence of induced subsidence events. The existing pavement was composed of reinforced concrete slabs with an asphalt overlay. The strategy adopted to best protect the safety of the traveling public was to initially continue the traffic on the existing pavement. No traffic was to be diverted off the existing pavement underlain by reinforced concrete slabs until the subgrade beneath the new lane location was first stabilized through drilling and grouting.

Due to construction operations and traffic requirements, the northbound traffic was diverted onto the northbound built-up shoulder at one point in the remediation work. No mine stabilization work had been performed under this temporary lane. A small sag developed later in the pavement of this lane. The traffic was diverted from this lane and ground penetrating radar (GPR) subsurface imaging was used to investigate the area of pavement settlement. The resulting GPR investigation did not reveal any shallow voids in the unconsolidated overburden immediately below the pavement. Borings at this same location did encounter extremely soft soils in the subgrade to the depth of approximately 18 feet. But, these borings did not encounter any discernable voids.

During the production drilling and grouting phase of the project, drilling of angled boreholes was required for several project reasons. Angled boreholes were drilled to verify conditions beneath the foundations of an existing bridge structure carrying Wise Road over IR77 within the project area. These boreholes encountered no mine voids. GPR was utilized to determine the general location of the three mapped and/or suspected unmapped vertical mine shafts within the project area. Angled boreholes were then drilled for the purposes of investigation, and remediation if necessary, of the vertical mine shafts. These angled boreholes encountered either previously placed debris backfill or undisturbed overburden soils and bedrock in the shaft investigation areas. Grouting of these boreholes was attempted using the higher slump cement-flyash grout. However, only a small amount of grout was required at one of the three suspected vertical shaft locations.

The final phase of the project entailed the drilling of angled and vertical confirmation boreholes to determine the success of the mine void production grouting. Project drilling logs and grout placement records were comprehensively reviewed as a comparative study of mine void locations to the distribution of grout volumes placed. Confirmation drilling was performed in concentrated areas of mine voids which appeared to have required lower than expected grout volumes placed. These same boreholes were utilized to complete grouting wherever voids were encountered. Only small amounts of grout were placed in a few boreholes during this confirmation drilling process.

The project was completed in October 2001. Approximately 1400 boreholes were drilled and grouted during this project. The project resulted in the tremied placement of approximately 25,000 cubic yards of grout into the abandoned underground mine workings beneath IR77. A final review of the boring logs revealed that the coal extraction rate for the abandoned mine workings beneath the roadway had been approximately 73%. The final project total cost was $8.4 million.

SUMMARY

Application of the ODOT AUMIRA process to screen the roadway for mine-related risks resulted in this multimillion dollar emergency mine remediation project. The project benefits were the protection of the present day and future public investment in the roadway, and the present day and future safety of the traveling public.

ODOT gained notable working knowledge from this project in the areas of: the uses of sonic drilling; the horizontal application of TDR for subgrade settlement monitoring; the utilization of GPR subsurface imaging for field locating and non-intrusively investigating vertical mine shafts and subsidence features; and the management of unimpeded high volume traffic through a mine remediation project area which encompassed the full width of an interstate highway right of way.

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The Three Dimensional Discontinuous Deformation Analysis (3D DDA) And its Application to Rock Toppling

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Manuscript length: 11 pages

[Abstract]

Since the orientations of discontinuities in the field are not truly perpendicular to the two-dimensional (2D) blocks of simulation, the applications of two dimensional discontinuous deformation analysis (2D DDA) computations have limited accuracy. In order to simulate the three dimensional (3D) block behaviors more accurate, Shi (2001) developed the 3D DDA theory to the blocks with general shape. In this paper, the basic formulations of contributed components of 3D DDA will be presented briefly, and a new 3D DDA program developed by the authors is applied to rock-slope toppling failure at the Amatoribashi-Nishi site to demonstrate the capability of this new method. The results show the ability of 3D DDA to study the mechanisms of slope failure.

[Key Words]: discontinuous deformation analysis, three dimensional analysis, rock toppling

1. Introduction

The 3D DDA theory was developed by SHI (2001) as the implicit algorithm to solve the discontinuous problems with more realized 3D blocks. This new method inherits the 2D DDA (SASAKI et al, 1994) characters as follow: (1) the principal of minimizing total potential energy is used, and the unique solution is confirmed as FEM. (2) dynamic and static problems can be solved by using the same formulations. (3) Any constitutive law can be introduced to the blocks. (4) Any contact criteria (ex: Mohr-Coulomb Criteria), boundary condition (ex: constraint displacement), loading condition (ex: initial stress, inertia force, volume load, etc.) can be considered in the computation.

Although the 3D DDA theory has been developed, it has not applied to the engineering problems until now. The authors develop the 3D DDA program, and apply it to the slope failure problem to demonstrate the capability of this new method in this study.

2. Basic Formulas

3D DDA can be treated as the extension of 2D DDA algorithm to the 3D space. It also supposes the analyzing system as assembly of the discrete blocks. The acceleration contribution is considered to manage the effects of the inertia forces, and the penalty method is used to satisfy the Non-penetration, Non-tension at the contact boundaries. In addition, the behaviors of analyzing system obey the principle of Hamilton, and the Updating Lagrange description is introduced to the time domain. Hence, the 3D DDA is an implicit dynamic analysis method, which is different from the DEM, and the basic formulations of 3D DDA can be written as follows:

2.1 The Displacement Function

Base on the basic idea of 2D DDA (SHI, 1989), the arbitrary 3D block motions can be also divided into tractions, rotations, normal strains and shear strains (Fig.1). Hence, the displacement function has to introduce 12 unknowns (3 Transitions, 3 Rotations, 3 Normal strains, and 3 Shear strains) to describe block motions in 3D. In addition, the block displacement function is equivalent to the complete first order approximation of the displacement, and constant strains and constant stresses are supposed within the blocks.

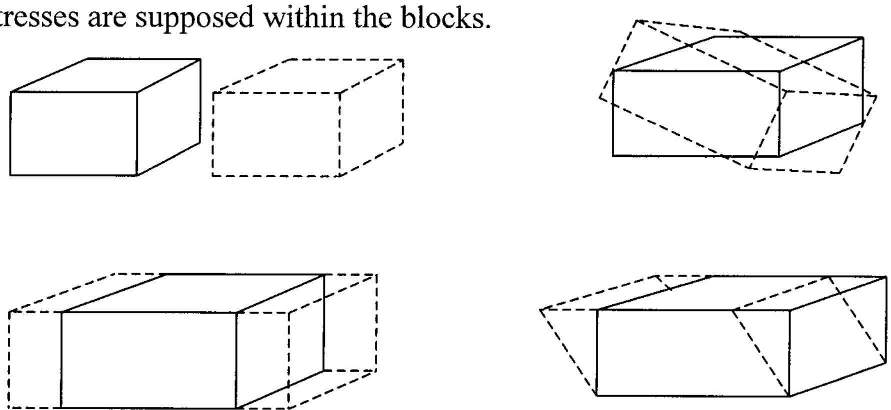


Fig. 1 - 3D Displacements

The complete first order displacement function has the following forms:

$$
\begin{aligned}
u &= a_0 + a_1 x + a_2 y + a_3 z \\
v &= b_0 + b_1 x + b_2 y + b_3 z \qquad \ldots(1) \\
w &= c_0 + c_1 x + c_2 y + c_3 z
\end{aligned}
$$

Where, u,v and w are displacements of a point within the block in x, y, and z direction. x,y and z are the coordinates of a point within the block.

$a_0, a_1, a_2, a_3, b_0, b_1, b_2, b_3, c_0, c_1, c_2$, and c_3 are the parameters.

Assumed that $[x_c \quad y_c \quad z_c]^T$ is the centroid of block, and $[u_c \quad v_c \quad w_c]^T$ is its displacements. Put them into formula (1), and it can turn to formula (2).

$$
\begin{aligned}
u_c &= a_0 + a_1 x_c + a_2 y_c + a_3 z_c \\
v_c &= b_0 + b_1 x_c + b_2 y_c + b_3 z_c \qquad \ldots(2) \\
w_c &= c_0 + c_1 x_c + c_2 y_c + c_3 z_c
\end{aligned}
$$

Subtraction (2) from (1)

$$
\begin{aligned}
u &= u_c + a_1(x - x_c) + a_2(y - y_c) + a_3(z - z_c) \\
v &= v_c + b_1(x - x_c) + b_2(y - y_c) + b_3(z - z_c) \qquad \ldots(3) \\
w &= w_c + c_1(x - x_c) + c_2(y - y_c) + c_3(z - z_c)
\end{aligned}
$$

The rotations can be expressed as

$$
\begin{aligned}
r_x &= \frac{1}{2}\left(\frac{\partial w}{\partial y} - \frac{\partial v}{\partial z}\right) = \frac{1}{2}(c_2 - b_3) \\
r_y &= \frac{1}{2}\left(\frac{\partial u}{\partial z} - \frac{\partial w}{\partial x}\right) = \frac{1}{2}(a_3 - c_1) \qquad \ldots(4)
\end{aligned}
$$

$$r_z = \frac{1}{2}(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}) = \frac{1}{2}(b_1 - a_2)$$

The normal strain:

$$\varepsilon_x = \frac{\partial u}{\partial x} = a_1 \text{ , } \varepsilon_y = \frac{\partial v}{\partial y} = b_2 \text{ , } \varepsilon_z = \frac{\partial w}{\partial z} = c_3 \qquad \ldots(5)$$

And the shear strain:

$$\frac{1}{2}\gamma_{yz} = \frac{1}{2}(\frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}) = \frac{1}{2}(c_2 + b_3)$$

$$\frac{1}{2}\gamma_{zx} = \frac{1}{2}(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x}) = \frac{1}{2}(a_3 + c_1) \qquad \square\ldots(6)$$

$$\frac{1}{2}\gamma_{xy} = \frac{1}{2}(\frac{\partial v}{\partial x} + \frac{\partial u}{\partial y}) = \frac{1}{2}(b_1 + a_2)$$

Hence, the parameters in formula (1) can be computed

$$a_1 = \varepsilon_x \text{ , } b_2 = \varepsilon_y \text{ , } c_3 = \varepsilon_z$$

$$c_2 = \frac{1}{2}\gamma_{yz} + r_x \text{ , } b_3 = \frac{1}{2}\gamma_{yz} - r_x \text{ , } a_3 = \frac{1}{2}\gamma_{zx} + r_y \qquad \ldots(7)$$

$$c_1 = \frac{1}{2}\gamma_{zx} - r_y \text{ , } b_1 = \frac{1}{2}\gamma_{xy} + r_z \text{ , } a_2 = \frac{1}{2}\gamma_{xy} - r_z$$

Denote

$$X = x - x_c \text{ , } Y = y - y_c \text{ , } Z = z - z_c$$

Formula (1) can be written as

$$\begin{pmatrix} u \\ v \\ w \end{pmatrix} = [T(x,y,z)](D) \qquad \ldots(8)$$

Where

$$[T(x,y,z)] = \begin{pmatrix} 1 & 0 & 0 & 0 & Z & -Y & X & 0 & 0 & 0 & \frac{Z}{2} & \frac{Y}{2} \\ 0 & 1 & 0 & -Z & 0 & X & 0 & Y & 0 & \frac{Z}{2} & 0 & \frac{X}{2} \\ 0 & 0 & 1 & Y & -X & 0 & 0 & 0 & Z & \frac{Y}{2} & \frac{X}{2} & 0 \end{pmatrix}$$

$$(D)^T = \begin{pmatrix} u_c & v_c & w_c & r_x & r_y & r_z & \varepsilon_x & \varepsilon_y & \varepsilon_z & \gamma_{yz} & \gamma_{zx} & \gamma_{xy} \end{pmatrix}$$

The (D) matrix is treated as unknowns or variables representing the displacements and deformations of the block. By using this formula, the displacements of all points of the block can be calculated.

2.2 Variation of Formulation

3D DDA follows the principle of minimizing total potential energy "$\prod$", which is the summation over all potential energy sources (eq.9) of each block, to generate the simultaneous equations. The total potential energy for blocks in DDA at the step of "n" can be calculated as:

$$\Pi^{(n)} = \Pi^{(n)}_{elastic} + \Pi^{(n)}_{initialstress} + \Pi^{(n)}_{volume} + \Pi^{(n)}_{inertia} + \Pi^{(n)}_{fixpo\,\text{int}} + \Pi^{(n)}_{contact} \quad \ldots(9)$$

Where, "$\Pi^{(n)}$" is total potential energy. "$\Pi^{(n)}_{elastic}$" is potential energy contributed by elastic deformation. "$\Pi^{(n)}_{initialstr\,ess}$" is potential energy contributed by initial stress. " $\Pi^{(n)}_{volume}$" is potential energy contributed by volume force. "$\Pi^{(n)}_{inertia}$" is potential energy contributed by inertia forces. "$\Pi^{(n)}_{fixpo\text{int}}$" is potential energy contributed by fixed point. "$\Pi^{(n)}_{contact}$" is potential energy contributed by contacts when blocks contact to each other.

If there is only one block, the equilibrium equations of each step are derived from the total potential energy "Π" as:

$$\frac{\partial \Pi}{\partial u_c} = 0,\ \frac{\partial \Pi}{\partial v_c} = 0,\ \frac{\partial \Pi}{\partial w_c} = 0,\ \frac{\partial \Pi}{\partial r_x} = 0,\ \frac{\partial \Pi}{\partial r_y} = 0,\ \frac{\partial \Pi}{\partial r_z} = 0$$

$$\frac{\partial \Pi}{\partial \varepsilon_x} = 0,\ \frac{\partial \Pi}{\partial \varepsilon_y} = 0,\ \frac{\partial \Pi}{\partial \varepsilon_z} = 0,\ \frac{\partial \Pi}{\partial \gamma_{yz}} = 0,\ \frac{\partial \Pi}{\partial \gamma_{zx}} = 0,\ \frac{\partial \Pi}{\partial \gamma_{xy}} = 0$$

The formulation form become as the same as Lagrange`s Equation of Motion, which derives from Principle of Hamiltion (HAYASHI, MURA.1971; HUEBNER.1975) for the dynamic system.

$$[M](A) + [\overline{K}](D) = [\overline{F}] \quad \ldots(10)$$

Where, $[M]$ is the mass matrix, (A) is the acceleration matrix, $[\overline{K}]$ is the stiffness matrix, (D) is the displacement matrix, $[F]$ is the force matrix.

The same as 2D DDA, the acceleration matrix, (A), can be treated as the function of displacement matrix and velocity matrix by the time integration, and the formula (10) turns to the one as follows:

$$[K](D) = [F] \quad \ldots(11)$$

However, if there are n blocks in the analysis system, the total equilibrium equations can be shown as formula (12).

$$\begin{bmatrix} [K_{11}] & [K_{12}] & [K_{13}] & \cdots & [K_{1n}] \\ [K_{21}] & [K_{22}] & [K_{23}] & \cdots & [K_{2n}] \\ [K_{31}] & [K_{32}] & [K_{33}] & \cdots & [K_{3n}] \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ [K_{n1}] & [K_{n2}] & [K_{n3}] & \cdots & [K_{nn}] \end{bmatrix} \begin{pmatrix} [D_1] \\ [D_2] \\ [D_3] \\ \vdots \\ [D_n] \end{pmatrix} = \begin{pmatrix} [F_1] \\ [F_2] \\ [F_3] \\ \vdots \\ [F_n] \end{pmatrix} \quad \ldots(12)$$

$[D_i]$ $[F_i]$ are 12×1 sub-matrices, where $[D_i]$ represents the 12 displacement variables, and $[F_i]$ is the loading on block i distributed to 12 displacement variables. The stiffness sub-matrices $[K_{ii}]$ depend on the material properties of block-i, and $[K_{ij}]_{i \neq j}$ is defined by contacts between block- i j.

2.3.Submatrices of Equilibrium Equation

According to the descriptions above, it is necessary to generate the equilibrium equations from the contributed potential energy components, including block stiffness, initial stresses, body forces and inertia forces etc. In fact, the 3D DDA derivative procedures are the same with the one in 2D, and Shi (1989), and Jing (1998) ever discussed about the procedure in detail. Hence, only brief derivations are represented in this paper as follow.

1) Sub-matrix of Block Stiffness

The blocks are assumed as the linear elastic, and the relations between stress, $\{\sigma\}$, and strain, $\{\varepsilon\}$ can be expressed as formula (13).

$$\{\sigma\} = [E]\{\varepsilon\} \qquad \ldots(13)$$

In 3D, the block i matrix, $[E]$, can be denoted as:

$$[E] = \frac{E_{(i)}}{(1+v_{(i)}^2)(1-2v_{(i)})}\begin{bmatrix} 1-v_{(i)} & v_{(i)} & v_{(i)} & 0 & 0 & 0 \\ v_{(i)} & 1-v_{(i)} & v_{(i)} & 0 & 0 & 0 \\ v_{(i)} & v_{(i)} & 1-v_{(i)} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{2}-v_{(i)} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{2}-v_{(i)} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{2}-v_{(i)} \end{bmatrix} \qquad \ldots(14)$$

Hence, the strain energy can be repressed as follow:

$$\Pi_{e(i)} = \frac{1}{2}\iiint [D_i]^T [E_{(i)}](D_i)dxdydz = \frac{V_{(i)}}{2}[D_i]^T[E_{(i)}](D_i) \qquad \ldots(15)$$

Where, "$V_{(i)}$" is the volume of block i. The stiffness sub-matrix can be obtained and added to the sub-matrix $[K_{ii}]$ of the total matrix (16).

$$\langle [K_{ii}]_{12\times12}\rangle = V_{(i)}\langle [E_{(i)}]_{12\times12}\rangle \qquad \ldots(16)$$

2) Sub-matrix of Initial Stress

For the block i, the initial constant stresses can be supposed as:

$$[\sigma_{(i)}^0]^T = (0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad \sigma_{x(i)}^0 \quad \sigma_{y(i)}^0 \quad \sigma_{z(i)}^0 \quad \tau_{yz(i)}^0 \quad \tau_{zx(i)}^0 \quad \tau_{xy(i)}^0) \qquad \ldots(17)$$

The potential energy can be written as follow:

$$\Pi_{initialstress(i)} = V_{(i)}[D_i]^T\{\sigma_{(i)}^0\} \qquad \ldots(18)$$

The contribution to the global force matrix (12) is represented as follow:

$$\langle [F_i]_{12\times1}\rangle = -V_{(i)}\langle [\sigma_{(i)}^0]_{12\times1}\rangle \qquad \ldots(19)$$

3) Sub-matrix of Body Forces

Assumed that the constant body forces $(f_{x(i)}, f_{y(i)}, f_{z(i)})$ are acting on the turned i-block, the potential energy can be expressed as

$$\Pi_{bodyforce(i)} = -[D_i]^T(f_{x(i)}V_{(i)} \quad f_{y(i)}V_{(i)} \quad f_{z(i)}V_{(i)} \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0)^T \qquad \ldots(20)$$

Where, "$V_{(i)}$" is the volume of block-i.

Hence, the sub-matrix of volume forces can be added to the sub-matrix $[F_i]$ in the global force matrix.

$$\langle [F_i]_{12\times1} \rangle = \left(f_{x(i)}V_{(i)} \quad f_{y(i)}V_{(i)} \quad f_{z(i)}V_{(i)} \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \right)^T \qquad \ldots(21)$$

4) Fixed Point

Assumed that the point $FP\left(x_{f(i)}, y_{f(i)}, z_{f(i)}\right)$ is the fixed point in the block i. The displacements of computation results are $\left(u_{f(i)}, v_{f(i)}, w_{f(i)}\right)$ at point FP. The strain energy, Π_{fix}, caused by the constrained spring can be expressed as:

$$\Pi_{fix} = \frac{p_f}{2}(D_i)^T \left[T_i(x_{f(i)}, y_{f(i)}, z_{f(i)})\right]^T \left[T_i(x_{f(i)}, y_{f(i)}, z_{f(i)})\right](D_i) \qquad \ldots(22)$$

Where, "p_f" is the stiffness of fixed spring.

The matrix can be added to the sub-matrix in the global stiffness matrix as:

$$[K_{ii}]_{12\times12} = p_f\left[T_i(x_f, y_f, z_f)\right]^T \left[T_i(x_f, y_f, z_f)\right] \qquad \ldots(23)$$

5) Inertia Force

Suppose that the volume weight of block i is M, the time step of step n is "$\Delta t_{(n)}$", and the velocity matrix for the beginning of each step is "$[V_0]$".The potential energy contributed by the inertia forces can be represented as:

$$\Pi_{inertia} = (D_i)^T M \iiint [T_i]^T [T_i] dxdydz \left(\frac{2}{(\Delta t_{(n)})^2}[D_i] - \frac{2}{\Delta t_{(n)}}[V_{0(i)}] \right) \qquad \ldots(24)$$

Where, matrix $\left[V_{0(i)}\right]$ represents the initial velocity vector of block i for each time step. Hence, the contribution to the global force matrix (12) is represented as follow:

$$\left([K_{ii}]_{12\times12}\right) = \frac{2M}{(\Delta t_{(n)})^2}\left(\iiint [T_i]^T [T_i] dxdydz\right) \qquad \ldots(25)$$

$$\langle [F_i]_{12\times1} \rangle = \frac{2M}{\Delta t_{(n)}}\left(\iiint [T_i]^T [T_i] dxdydz\right)\left\langle \left[V_{0(i)}\right]_{12\times1} \right\rangle \qquad \ldots(26)$$

6) Sub-matrix of Spring

(1) Normal Spring

In 3D, the potential energy caused by the spring between point P_1 of block i and entrance plane $P_2P_3P_4$ (generate the normal inside the block) of block j can be calculated as:

$$\Pi_{Normal} = \frac{k_n}{2}\left(\frac{V_{cN}}{A} + [E_i]^T[D_i] + [G_j]^T[D_j] \right)\left(\frac{V_{cN}}{A} + [E_i]^T[D_i] + [G_j]^T[D_j] \right) \qquad \ldots(27)$$

Where, "k_n" is the normal penalty, "V_{cN}" is the volume of tetrahedron, $P_1P_2P_3P_4$, and "A" is the area of entrance plane.

$$\begin{bmatrix} g_{11} & g_{12} & g_{13} & g_{14} \\ g_{21} & g_{22} & g_{23} & g_{24} \\ g_{31} & g_{32} & g_{33} & g_{34} \\ g_{41} & g_{42} & g_{43} & g_{44} \end{bmatrix}^T = V_{cN} \begin{bmatrix} 1 & x_1 & y_1 & z_1 \\ 1 & x_2 & y_2 & z_2 \\ 1 & x_3 & y_3 & z_3 \\ 1 & x_4 & y_4 & z_4 \end{bmatrix}^{-1} \qquad \ldots(28)$$

And

$$[E_i]^T = \begin{pmatrix} g_{12} & g_{13} & g_{14} \end{pmatrix} [T_i(x_1, y_1, z_1)] / A$$
$$\begin{aligned}[G_j]^T &= \begin{pmatrix} g_{22} & g_{23} & g_{24} \end{pmatrix} [T_j(x_2, y_2, z_2)] / A \\ &+ \begin{pmatrix} g_{32} & g_{33} & g_{34} \end{pmatrix} [T_j(x_3, y_3, z_3)] / A \\ &+ \begin{pmatrix} g_{42} & g_{43} & g_{44} \end{pmatrix} [T_j(x_4, y_4, z_4)] / A\end{aligned} \qquad \ldots(29)$$

Then, add to the global matrix as follow:

$$[K_{ii}] = k_n [E_i][E_i]^T,\ [K_{ij}] = k_n [E_i][G_j]^T,\ [K_{ji}] = k_n [G_j][E_i]^T,\ [K_{jj}] = k_n [G_j][G_j]^T$$
$$[F_i] = -\frac{k_n V_{cN}}{A}[E_i],\ [F_j] = -\frac{k_n V_{cN}}{A}[G_j] \qquad \ldots(30)$$

(2) Shear Spring

Assume that there is a shear spring between point P_1 of block 1 and P_0 of block j, where P_0 is the projection of P_1 on plane $P_2P_3P_4$. Denote (n_x,n_y,n_z) as the normal of plane $P_2P_3P_4$ out of block j, and s as the projection vector $P_0 P_1$ on plane $P_2P_3P_4$.

$$[N] = \begin{pmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{pmatrix} - \begin{pmatrix} n_x \\ n_y \\ n_z \end{pmatrix} \begin{pmatrix} n_x & n_y & n_z \end{pmatrix} \qquad \ldots(31)$$

The potential energy of the shear spring with stiffness "k_s" can be written as follow:

$$\begin{aligned}\Pi_{shear} &= \frac{k_s}{2}(s+ds)(s+ds) \\
&= \frac{k_s}{2}\begin{pmatrix} x_1 - x_0 \\ y_1 - y_0 \\ z_1 - z_0 \end{pmatrix}^T [N] \begin{pmatrix} x_1 - x_0 \\ y_1 - y_0 \\ z_1 - z_0 \end{pmatrix} \\
&+ k_s [D_i]^T [T_i(x_1, y_1, z_1)]^T [N] \begin{pmatrix} x_1 - x_0 \\ y_1 - y_0 \\ z_1 - z_0 \end{pmatrix} \\
&- k_s [D_j]^T [T_j(x_0, y_0, z_0)]^T [N] \begin{pmatrix} x_1 - x_0 \\ y_1 - y_0 \\ z_1 - z_0 \end{pmatrix} \\
&+ \frac{k_s}{2}[D_i]^T [T_i(x_1, y_1, z_1)]^T [N][T_i(x_1, y_1, z_1)][D_i] \\
&- \frac{k_s}{2}[D_i]^T [T_i(x_1, y_1, z_1)]^T [N][T_j(x_0, y_0, z_0)][D_i] \\
&- \frac{k_s}{2}[D_j]^T [T_j(x_0, y_0, z_0)]^T [N][T_i(x_1, y_1, z_1)][D_i] \\
&+ \frac{k_s}{2}[D_j]^T [T_j(x_0, y_0, z_0)]^T [N][T_j(x_0, y_0, z_0)][D_j]\end{aligned} \qquad \ldots(32)$$

Then, add to the global matrix as follow:

$$[K_{ii}] = k_s[T_i(x_1,y_1,z_1)]^T[N][T_i(x_1,y_1,z_1)],\ [K_{ij}] = -k_s[T_i(x_1,y_1,z_1)]^T[N][T_j(x_0,y_0,z_0)],$$

$$[K_{ji}] = -k_s[T_j(x_0,y_0,z_0)]^T[N][T_i(x_1,y_1,z_1)],\ [K_{jj}] = k_s[T_j(x_0,y_0,z_0)]^T[N][T_j(x_0,y_0,z_0)]^T$$

$$[F_i] = -k_s[T_i(x_1,y_1,z_1)]^T[N]\begin{pmatrix} x_1 - x_0 \\ y_1 - y_0 \\ z_1 - z_0 \end{pmatrix},\ [F_j] = -k_s[T_j(x_0,y_0,z_0)]^T[N]\begin{pmatrix} x_1 - x_0 \\ y_1 - y_0 \\ z_1 - z_0 \end{pmatrix} \quad \ldots(33)$$

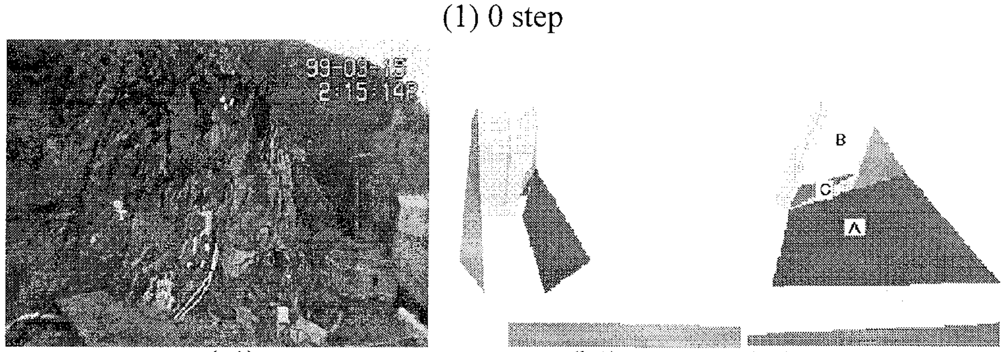
When the frictional resistances caused by the normal spring force is smaller than shear spring force, it slides along the plane.

3. Simulation Example

Japan is a mountainous country, where landslides of many kinds frequently occur due to its local geological, topographical features and climate conditions. Many engineering structures near mountain regions have been under the threat of slope failure, and considerable financial resources are required for maintenance and repairs. In order to get better countermeasure design against slope failure, it is necessary to study their mechanisms. In fact, many methods devoted to this field have been in use for years (SHIMAUCHI et al, 2001), numerical analysis methods being one of them, in particular discontinuous analysis methods.

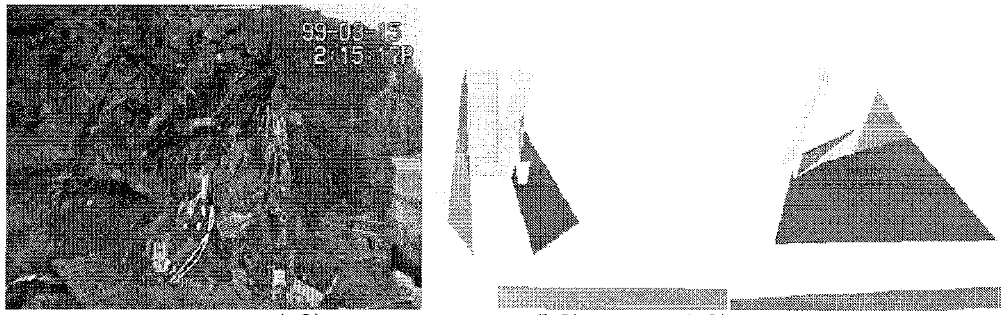
In this paper, one rock-slope failure process at the Amatoribashi-Nishi site in Japan was recorded on video as shown in Fig.-2. According to the landslide classification made by Varnes (1978), the failure can be classified as toppling failure. It is clear however, that this failure process contains also rotation of falling blocks around axes not parallel to the strike of the slope, which is impossible to be simulated accurately by 2D analysis. Hence, the 3D DDA is used for this simulation. According to the video record, it is obvious that the slope in the field can be simplified as 3 types of blocks to the simulation, including fixed block (block A), large failure block (block B) and small failure block (block C) at the foot of block B (Fig.-2).

(1) 0 step



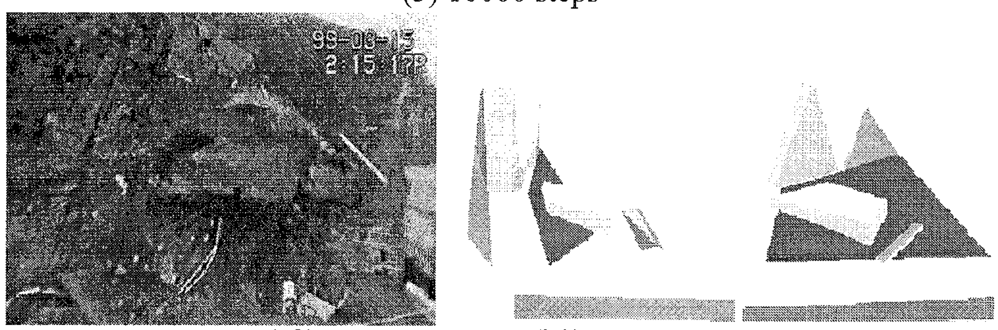
(a1) (b1) (c1)

(2) 2000 steps



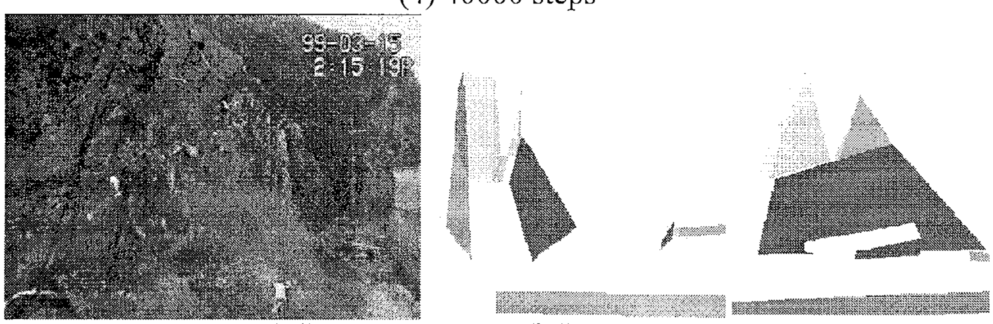
(a2) (b2) (c2)

(3) 10000 steps



(a3) (b3) (c3)

(4) 40000 steps



(a4) (b4) (c4)

Fig 2. Analysis Results of Slope Failure Process

The physical parameters used in this study are from the field or laboratory tests. The parameters are shown in Table 1. The modeled joints correspond with the open joints in-situ. Hence, there is no tensile and cohesion strength for the joints in the simulation.

Table 1 Parameters for the 3D DDA Computations

Item		Value
Analysis Parameters	**Disp. Allow Ratio**	0.0001
	Penalty	50000
	Total Steps	40000
	Max. Time Step (sec)	0.001
Block	**Unit Weight (KN/m^3)**	25.7
	Poisson Ratio	0.2
	Young`s Modulus (GPa)	24.5
Discontinuity	**Frictional Angle (°)**	32.4
	Cohesion (MPa)	0.0
	Tensile Strength (MPa)	0.0

From Fig2, it is clear that the weight of block B is the main driving force causing toppling, and that the interactions between block A, B and C induce the 3D phenomena described above, when block B and C fall down. The simulation offers the engineers investigating the mechanism of the rock-slope failure, an animation of the slope failure process from different directions.

Comparison of the simulation results to the field video record shows that the behavior of large moving blocks (block B) can be simulated very well, especially the toppling phenomenon. On the contrary, the movement of the simulated block C is different from the one in the field. The reason may be caused by the joint assumptions (cohesion, tensile strength) or interactions patterns between block B and C. Jing (1998) mentioned that the chief disadvantage of discontinuous analysis methods is the requirement for knowing the exact geometry of fracture systems in the problem. The unsatisfactory simulation of block C may be due to oversimplification of the system. Hence, in the future it is necessary to investigate the influence of the joint parameters, of the blocks A and C on block B's failure pattern. However, no illegal penetration phenomena happen in this computation, and this indicates the capability of solving different kinds of contact situations by 3D DDA.

Conclusion

In this paper, the basic formulas of 3D DDA are introduced, and it has been shown that the 3D DDA inherits the characteristics of 2D DDA. The derivation of the 3D DDA is not difficult, except the contact judgments. The 3D contact patterns occur in more types and are more complicated than with 2D blocks. However, the example in this paper showed that the new developed 3D DDA program can solve the engineering problem without illegal penetrations. In addition, it also shows the ability of the 3D DDA to reproduce the rock-slope toppling behaviors in the field.

Although the toppling process of most important block B could be simulated well in this study, the influence of the joint parameters (cohesion, frictional angle, and tensile strength) and the interactions among the blocks to the failure pattern have not been studied in detail in this paper. In order to investigate the mechanism causing rock-slope failure at the Amatoribashi-Nishi site, the possible influencing factors described above should be studied in the future.

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Earthquake Input Parameters for Slope Stability Analyses – Science or Sorcery

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At the 52nd Highway Geology Symposium, Humphries, et al (2001) presented an excellent review of the approaches used in seismic analyses of rock slopes. However, whatever the level of sophistication of the analytical model, all depend upon having appropriate design input (ground motion acceleration, time history or response spectrum).

The process is fraught with uncertainties whether a deterministic or probabilistic approach is used. Location is important. Is the design event a western or eastern United States earthquake? Which magnitude scale is used for the design earthquake (Richter, moment, body wave, surface wave, etc.), or is it an older event originally categorized as Intensity (Modified Mercalli or Rossi-Forel) then translated to a magnitude scale to be used in an attenuation relationship? Which attenuation relationship is appropriate for the regional geologic conditions of the site of concern? Close-in differences in the peak ground acceleration (PGA) for recently published relationships can be 50% or more.

Local soil, rock and topographical considerations will also influence design ground motion. Earthquake duration and frequency content will influence slope performance. Thus, should a PGA, or 1 Hz ground acceleration, or a "sustained" level of acceleration be used? Also, it may be necessary to consider the effects of different source parameters and path as well as site effects.

The paper attempts to discuss the impacts of these potential variations and present warnings. However, at this time, there is no perfect answer for a particular site. Design input parameters need scientific input, but economics and judgement (sorcery?) are also important.

INTRODUCTION

In their 2001 paper, Humphries, et al reviews the approaches currently available to engineers and geologists evaluating a rock slope. Whether the slope is rock or soil, the analytical models used can be relatively simple or mathematically sophisticated. No matter how elementary or complex the analyses, appropriate seismic hazard input parameters are required whether as an acceleration value (peak, damped or sustained), a design response spectrum (or spectra) or a design time history (or histories).

Humphries, et al, 2001 defines these steps, simplistically, as:

- Identify the active faults in the region of the project;
- Evaluate the possible length of rupture, type of fault, magnitude of fault and return period of activity for each fault;
- Evaluate the attenuation of the seismic shaking that will occur between the active fault and the project;
- Select an appropriate ground acceleration time history recorded for a similar fault that has been recorded elsewhere and scale it up or down to the appropriate level for the project

site. This becomes the design earthquake or, more precisely, the site-specific acceleration time history, for the project; and

- Carry out the dynamic stress and deformation analysis using the site-specific acceleration for the site.

Any experienced engineering seismologist can perform the first four steps. Unfortunately, if more than one is performing the analysis, there will be more than one answer. As noted in the referenced paper, the seismic design parameters can become the critical "design case" in "areas of moderate to high seismicity".

To aid in better understanding the problems that face the engineering seismologist when asked to gaze into this crystal ball, let us examine some of the natural variations than can affect the analyses. In addition, the standard economic concerns that generally control the extent of the analyses apply here as well. The writer has been associated with engineering seismology investigations that ranged in cost from $5,000 to $1,000,000. There are many possible variations in procedure and the results can be voluminous, hence, only a sample of the problems inherent in a seismic risk evaluation are touched upon in this paper.

PROBABLISTIC OR DETERMINISTIC

Each of these basic methodologies are appropriate, with the deterministic approach being the oldest and probably most judgmental. There are a number of statistical approaches available. The most common being attributed to Messrs. Poisson, Bayes and Gumbel. Presumably, more scientific in concept, probabilistic approaches are often popular, but unfortunately can lead to unscientific results when carried too far by those more knowledgeable of statistics than tectonics.

The first step in choosing any methodology requires some understanding of the degree of risk acceptable, as well as the need for and nature of defending the answers. Other important parameters include the reliability and extent of the available seismic information, sensitivity of the project, potential regulatory guidelines, and the priorities of the owner or sponsoring organization.

The deterministic process is used to predict the size(s) and location(s) of the design earthquake(s) upon the bases of known geology, the historical earthquake database, and the current in situ stress magnitude and orientation, as well as the assumption that the pattern of near-future seismic activity will be similar to that which occurred in the past. Obviously, the length of record is important as well as the judgment of the engineering seismologist in evaluating what size shocks happened where.

The probabilistic approach requires a suitable number of events that some statistical validity can be achieved. The most appropriate statistical approach must be chosen. The most common approach is to assume a Poisson distribution. This assumes a random occurrence in time and space, whether along a fault zone or over a tectonic province. While mathematically simple, the distribution is an anathema to those who believe Perry Byerly's quote: "The farther we are from one earthquake, the closer we are to the next". The "elastic rebound" proponents, the "seismic gap" and the "earthquake clusters in time" theorists, would also find fault with a random distribution of seismic events in time and space. A Baysian distribution attempts to extend any distribution by the use of some judgment. Baysian techniques are often used to evaluate the possibility of the occurrence of events or results of events outside the available history. "These subjective probabilities are used to define what is called the prior distribution for the parameter" (Larson, 1969). Essentially, Baysian statistics assume that any parameter by itself has a probability distribution rather than a single value. An investigator modifies the postulated distribution by his/her experience (i.e., judgment).

An "extreme value" concept (i.e., Gumbel distribution) has also been used. Selecting the largest likely event or characteristic earthquake for such a distribution again requires the judgment of the analyzer.

Thus, both deterministic and probabilistic procedures require judgmental input. The same parameters must be evaluated and the results of the analyses synthesized for eventual use.

SOURCE PARAMETERS

The first step in the slope stability analyses indicated in Humphries, et al, 2001 is to identify active faults. This is appropriate in many plate boundary locations where the length and orientation of active faults are often known. In the eastern part of the United States, the causative faults are not well known and return times may be greater than man's history in the region. Even in well-studied areas such as New Madrid, Missouri and Charleston, South Carolina, the actual size and location of each of the earthquakes of 1811 and 1812 (New Madrid) and 1886 (Charleston) are not well established. In addition, there are numerous smaller events that have been recorded, but only nominally studied (Anna, Ohio; Messena, New York; Giles County, Virginia). For example, what is the magnitude (either M_S, M_L, M_W, mb, or M) of the MMI VIII Giles County, Virginia earthquake of 1897?

In California, where an association of earthquakes and faults can often be defined, it seems new information on "blind" faults and pre-historic earthquakes are becoming common (e.g., Grant, et al, 2002). The question then becomes; how good are our geologic and paleoseismic investigations and how do we take into account our relatively limited historical data? A number of major California shocks (e.g., San Fernando, 1977, Loma Prieta, 1998) occurred on previously unmapped faults. The difficulties become even greater when one tries to define and evaluate faults buried under thick sediments in a subduction zone related to the seismicity in, for example, the Pacific Northwest.

Another concern in evaluating fault or province seismicity is the ability to define the nature of the expected ground motion. In addition to the many magnitude scales for classifying earthquakes (see Table 1 of the next page), the focal mechanism, the depth of the energy release and potential directivity of strike-slip motion must be evaluated in attempting to estimate near-fault motion to define a suitable attenuation relationship.

An example of the possible variations in source parameters is presented in Table 2 of Somerville, et al, 1999. A synthesis of the data presented in Tables 1 and 2 of the Somerville, et al, 1999 paper is provided to illustrate these problems as Table 2, below. Obviously, all earthquake ground motion is not generated in the same manner.

TABLE 1 **Magnitude Definitions**	
M	"Richter" magnitude (less than 600 km epicentral distance) or Magnitude?
Mn	Local magnitude, but computed differently than M_L; also = mbLg
mb	Body wave magnitude
M_L	Local magnitude (M_L = Richter M for shallow events up to about M_L = 6).
M_S	Surface wave magnitude
M_o	Seismic moment
M_W	Moment magnitude

TABLE 2
Source Parameters of Crustal Earthquakes with Fault Plane Orientation
(From: Somerville, et al 1999)

Location	Date	Mech.[1]	$M_o x10^{25}$ dyne-cm	Mw	Slip Duration (sec)	Dip	Strike
Landers, CA	6/28/1992	SS	75	7.22	2.0	90°	355° 334° 320°
Tabas, Iran	6/16/1978	RV	58	7.14	2.1	25°	330°
Loma Prieta, CA	10/17/1989	OB	30	6.95	1.5	70°	128°
Kobe, Japan	1/17/1995	SS	24	6.9	2.0	80° 85°	45° 230°
Borah Peak, ID	10/28/1983	NM	23	6.87	0.6	49°	152°
Nahanni, N.W.T., Canada	12/23/1985	RV	15	6.75	2.5	25°	160°
Northridge, CA	1/17/1994	RV	11	6.66	1.25	40°	122°
Nahanni, N.W.T., Canada	10/5/1985	RV	10	6.63	0.75	35°	160°
San Fernando, CA (Sierra Madre)	2/9/1971	RV	7	6.53	0.8	54°	290°
Imperial Valley, CA	10/15/1979	SS	5	6.43	0.7	90°	143°
Superstition Hills, CA (event #3)	11/24/1987	SS	3.5	6.33	0.5	90°	127°
Morgan Hill, CA	4/24/1984	SS	2.1	6.18	0.3	90°	148°
North Palm Springs, CA	8/7/1986	OB	1.8	6.14	0.4	46°	287°
Whittier Narrows, CA	10/1/1987	RV	1	5.97	0.3	30°	280°
Coyote Lake, CA	6/8/1973	SS	0.35	5.66	0.5	80°	336°

[1] SS = Strike-Slip; RV = Reverse; BO = Oblique (Strike-Slip & Reverse); NM = Normal

ATTENUATION

Ground motion attenuation from fault to structure varies with source mechanism, magnitude of stress drop, nature of the materials between the source and the project, and finally, the geometry and physical characteristics of the subsurface at the site of interest (e.g., Fischer, 1984 and Central United States Earthquake Consortium, 1993).

Attenuation relationships are published regularly by many of the same authors as new data is gathered. Obviously, the engineering seismologist must be aware of all of the relationships to evaluate which input parameters may be most appropriate. Also, different relationships exist for peak ground velocity and peak ground acceleration, as well as damped peak motions. There are relationships for "rock" sites, "stiff soil" sites, "deep cohesionless soil" sites, and sites underlain by soft to medium stiff clay deposits. While dated, Figure 1 (from Seed and Idriss, 1982) below provides an example of some of the problems faced in selecting an attenuation curve.

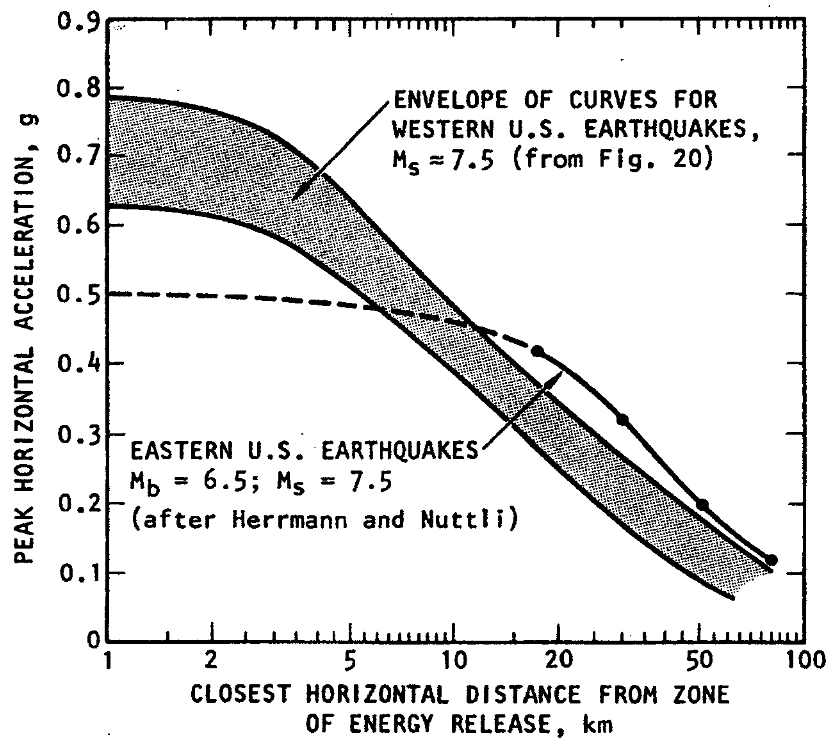
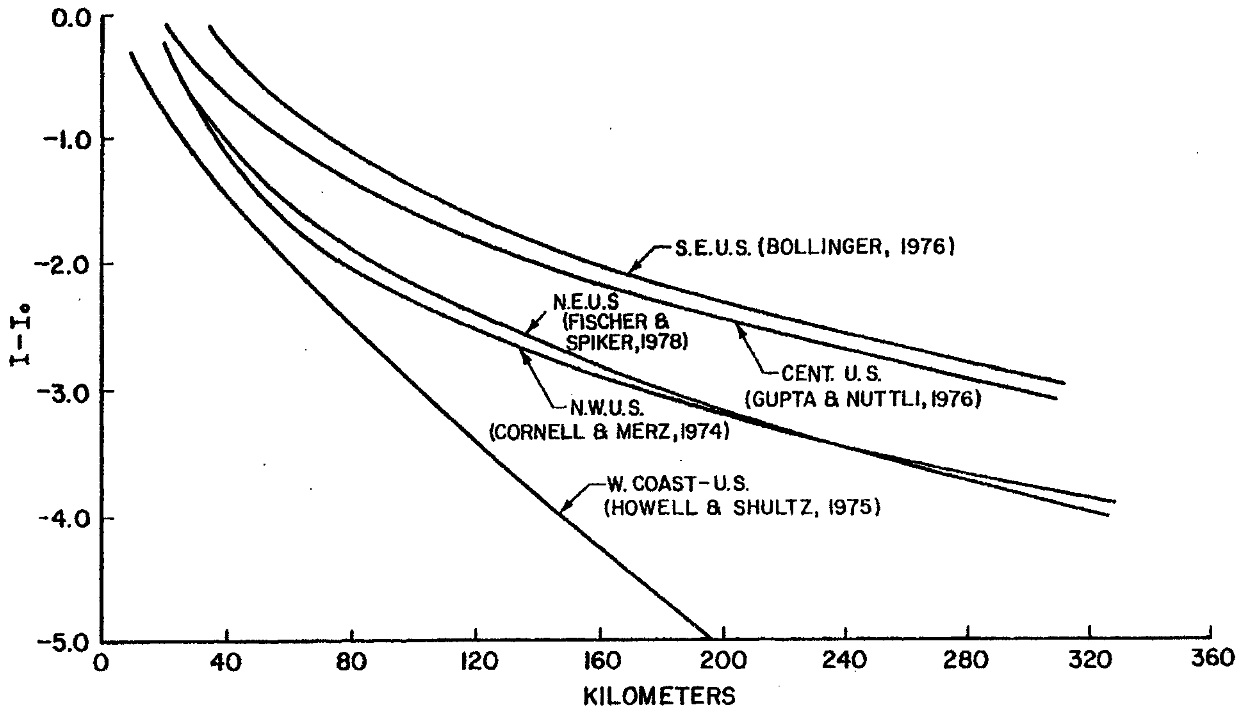


Figure 1 (From Seed and Idriss, 1982).

Seismic instrumentation has only been in existence since about 1900. Hence, most current attenuation curves are based upon relatively recently recorded data (circa 1970 and newer), and obviously, only from the more active areas. Earlier data were only recorded as damage curves (i.e., Rossi-Forel scale, Modified Mercalli Intensity scale, Japan Meteorological Scale [JMA], the Medvedev-Sponheuer-Karnik scale [MSK]). To make this older data more scientific(?), a number of authors have developed scales to convert intensity to magnitude and then use modern accelerograph readings to extrapolate new ground motion attenuation curves to replace the older intensity attenuation curves. However, even these older attenuation relations demonstrate the variation in hazard levels from location to location within the United States (see Figure 2). Attenuation curves of this nature can be used with Intensity versus Acceleration relationships, which suffered the same problems as Magnitude versus Distance versus Acceleration plots.



FORMULATED REGIONAL ATTENUATION-UNITED STATES

Figure 2 (From Fischer, 1979).

A quote from a recent paper on the accuracy of attenuation relationships (Douglas and Smit, 2001) indicates the concerns that must be understood when using them. "Efforts should be made to find reliable and predictable sources, path and site parameters that correlate well with ground motion in order to reduce the uncertainty rather than simply using more complicated functional forms or regression procedures using only magnitude, distance, and crude site characterization".

SEISMIC INPUT PARAMETERS

Input parameters for seismic design can include:

1. Peak Ground Acceleration – generally used in pseudostatic analyses or could be used to anchor response spectra or time histories.
2. Sustained Peak Ground Motion or a dampened Peak Acceleration.
3. Time History – Should be recorded accelerographs from earthquakes of a similar size, source mechanism, stress drop, and region, as well as having been recorded upon similar subsurface conditions, or perhaps a suite of time histories.
4. Response Spectra – Requires similar pre-conditions as Time History, or an "envelope" response spectra. If an "envelope" response spectra design concept is used, the smoothed

spectra should consider damping and the desired degree of conservatism (i.e., envelope all points, the mean, one standard deviation, etc.).

Again, from the above list of input variables, the seismic design input must be selected with care and conservatism. There is no substitute for communication among the design team as well as experienced judgment.

ANALYSES

Advances in modeling and dynamic analyses come almost as quickly as changes in attenuation relationship. However, input (i.e., material elastic/plastic properties) damping (usually variable in relation to ground motion amplitude) and mass must also be defined with a degree of precision and reliability suitable for the model. This is a difficult task with earthen materials subject to dynamic loading. Then, of course, we must realize that any earthquake is three dimensional in nature, generally necessitating more approximations of the actual ground motion impinging upon the slope in question.

SUMMARY AND CONCLUSIONS

This paper has attempted to record some of the problems inherent in developing appropriate seismic hazard parameters that are the required input for highway project designs in areas susceptible to earthquake ground motion. It is in no way complete and the references listed are illustrative only, but it can indicate the length of time that studies within this field has been underway. Progress has been made, but it sometimes seems that this progress only indicated how much further there is to go before the black box approach to evaluating seismic data can be resolved.

The initial engineering seismological steps in any slope stability analyses performed at any site subject to earthquake motion are not simple and routine. No matter what level of sophistication of analysis the slope demands, the task of the engineering seismologist requires an understanding of the vagaries of the earth from source to site; a subject not commonly taught in civil engineering courses. Even if it were taught, the necessity to estimate the effects of conditions still being evaluated by the scientific community requires a great deal of judgment. Engineering seismology truly is science, albeit a multi-disciplined one, but sorcery does appear to exist in developing useful answers, albeit cloaked in the term "judgment".

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Role of megaquarries in future aggregate supply

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Abstract

Much aggregate production in the next fifty years will likely come from very large quarries or "megaquarries." Remote megaquarries served by low-cost ocean or rail transport, are an attractive solution to meeting aggregate demand in large urban areas that have depleted or sterilized local aggregate sources. Megaquarries, however, present challenges for land management and reclamation. Only a few geologic, geographic and economic settings are suitable for megaquarry development. Societal resistance to mining is the single most important obstacle in their development. Megaquarries will affect aggregate cost, operation of transport systems, and the types of aggregate available to customers.

Primary Concern

The availability of affordable aggregate is essential to continued transportation development and maintenance of the modern highway system. While reserves of aggregate available for use are expanded by material and mining technology, they are diminished by increasingly stringent technical specifications that make some traditional aggregate resources unsuitable (Langer and Jahn, 1997). Urban encroachment and restrictive land-use planning is further eroding the stock of available resources, most significantly around the urban interface where the industry is most concentrated. The situation is exacerbated by prices that remain, on a constant dollar basis, relatively unchanged since 1905 (Mason and Welgoss, 2002).

The loss of available resources through restrictive land-use planning is leading to shortages across the nation. Linda Falasco, president of the Construction Association of California as reported by Wolf (2000, p. 22), aptly summarizes the current situation in California as follows:

> In the state that produces more aggregate than any other, it is ironic that the largest problem reported is a lack of available materials. Not because there isn't any more stone or sand and gravel available - the shortages are artificially produced due to some of the most severe permitting conditions in the nation.

Construction aggregate producers have responded with short-run strategies that include acquiring additional operations with significant resources that are already permitted. These acquisitions provide greater economic efficiency for producers having several operations in one region. Nonetheless, longer-run strategies require that companies significantly reduce production costs beyond the savings currently being realized. These

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companies must also address ongoing conflicts with communities around existing operations.

It would be accurate to state that the industry faces a conundrum; the very markets being served are forcing operations to relocate further away, increasing haulage costs that offset savings in extraction costs. As the distance to market increases, rail and water-based transportation alternatives become attractive where available. Access by rail and water transport allows some producers to realize significant cost savings through vastly larger quarries that serve several markets. These quarries provide very large quantities of high-quality crushed stone, and have access to major markets through low cost transportation.

New aggregate operations are becoming more difficult to open because of ongoing issues over comprehensive land-use planning. In most counties and states, land-use plans neither consider nor encourage aggregate production. This has not been lost on producers. Concerns relating to permitting delays and restrictions on the expansion of existing operations and development of new production sites are stimulating industry consolidation. The ongoing interest among foreign buyers in acquiring U.S.-based capacity will continue to move the U.S. industry towards degrees of consolidation already realized in Europe (U.S. Bancorp Piper Jaffray, 2000, accessed 6/08/02, via URL http:// www. piperjaffray. Com/). Because aggregate revenues are tightly linked to heavy construction spending on major infrastructure development projects, the location of new quarries is not independent of national and state-funded highway programs. Given this complex picture, what is the prognosis for aggregate availability in the next 50 years and what role will very-large quarries or megaquarries (MQs) play?

Research Assignment

The U.S. Geological Survey (USGS) is beginning a new study on MQs, an emerging class of very-large quarries that will be an important future source of aggregate in the next fifty years. Some types of sand and gravel deposits may also support large capacity pits (or megapits (MPs) that are comparable to MQs. The new USGS Western Industrial Minerals Project will seek information pertinent to the possible development of MQs and MPs. Sites with production over 5 Mtpa in the US will need to be identified and characterized. Reclamation concerns in MQ development will also be explored. The initial effort will be on areas along the coast and waterways of the western US and adjacent areas. These areas are undergoing rapid urban development commonly associated with existing port facilities. Geology, regional land ownership, and current and allowable uses will be among the layers to be included in a publicly available Geographic Information System (GIS). Markets and surface transportation nets will also be given. The project will also examine possible future economic scenarios and possible impacts they will have on MQ development.

Current situation

Concerns have been raised in the past about aggregate availability in the United States (US). It has been forecast that about one-third of the US will experience an aggregate shortage (Witczak and others, 1971 as cited in Gandhi and Lytton, 1984). When these shortages will occur is not specifically proposed. In September 2001, it was reported that construction aggregate shortages were delaying highway construction projects in the San Francisco Bay to Sacramento corridor in California. Linda Falasco, as cited by Lindelof (2001), noted the shortages in Sacramento County and outlying counties. She said, "There is about 10 years of permitted aggregate supply left." She also noted that an aggregate producer was not able to obtain a permit for a new quarry in Herald, Sacramento County, California.

The Minneapolis St. Paul region fares no better. More than 360 km^2 of land containing good aggregate have been paved over or otherwise excluded from land-use (Southwick and others, 2000.) A remaining 1.5 billion metric ton (Bt) resource, from an original 5.2 Bt, may well overstate available resources, as the estimate does not take into account the impacts of future zoning decisions, or the willingness of landowners to allow aggregate mining. Southwick and others (2000, accessed 05/09/02 via URL http://www. dnr.state.mn.us/ minerals/ abstract.html) note, "Aggregate resources of the metropolitan area could be exhausted as early as 2028, because of loss of aggregate-bearing land to urbanization, together with increased demand." Such shortages will increasingly arise elsewhere nationally.

Historically, most production sites meet aggregate needs for a relatively small area—to a distance of no more than 20 to 260 km. Faced with resistance to establishing new quarries, many aggregate companies can be expected to spend time and political capital to attempt to open new quarries only if reserves are extensive. This will result in fewer but larger new quarries, with a continued trend away from alluvial aggregate pits toward higher quality bedrock quarries. Whereas historically, transportation cost was a limiting factor for aggregate production, "remoteness is no longer an absolute disqualifier for the production of aggregate" (Langer. 1995, p. 303).

The Scottish birth of an idea

Colin Gribble, a geologist at the University of Glasgow, and Ian Wilson, a Scottish entrepreneur, developed the MQ concept in the late 1960's. Both were involved in planning a third airport for London at Maplin Sand on the Essex coast in England. Wilson suggested that the aggregate needed for construction could be obtained from Peterhead, a site identified by Colin Gribble on the east coast of Scotland. While the idea of the airport faded, the concept of obtaining aggregate from bedrock in Scotland did not. Wilson and Gribble prepared a report for Ralph Verney (see Verney, 1976), an advisor to the environmental secretary. The report identified 16 possible locations suitable for superquarry development, primarily on the Scottish west coast and islands, where Wilson held much of the mineral rights! One of the sites was Glensanda, and it is the location of the only operating superquarry in Scotland (Trade and Environmental Database, 1995, accessed 5/25/02 via URL http://www.american.edu/TED/QUARRY.HTM.) A principal assumption of the report was that future aggregate quarries must be located near the coast so that inexpensive marine water transport of the aggregate is possible. The report for Verney also suggested that future aggregate quarries should be located away from population centers (and their associated environmental concerns) to remote locations. A number of locations were also identified as possible sites of coastal quarries in Scotland by Dalradian Mineral Service for the Scottish Development Department (See Wardrop and Walton, 1996).

Definitions and Types of Megaquarries and Megapits

McNally (1998) defines a MQ as a quarry with a production capacity of 5 to 10 million metric tons per annum (Mtpa) of high-quality crushed stone, with sufficient resources to support 100 years of production, that is, one where 1,000 million metric tons (Mt) of probable reserves are anticipated, and 100 Mt are proven or drilled. Two types of geological situations appear to allow for MQ development and one type allows for very-large pits that extract sand and gravel. The "Type I MQ," based on the requirements proposed by McNally (1998), produces only aggregate. The best geologic candidate would be an essentially unaltered intrusive igneous body, easily crushable, and producing little waste and fines. Smith and Collis (1993) note that volcanic necks and dikes make attractive quarrying targets due to the relatively small area needed for the quarry given the large resource they may contain. Aggregate sources suitable for MQ development need to be homogeneous, volumetrically extensive, and suitable for mining. Engineers designing aggregate and cementing agent mixtures for use in construction can accommodate a range of aggregate characteristics as long as those characteristics are consistent from one shipment to the next. Those preparing construction mixture design have historically emphasized the structural strength and durability of the aggregate-bearing product (i.e. ready mixed concrete). Prior to the advent of Superpave highway standards, these designs were less concerned with the source of contained aggregate. Many in the industry now support performance-based standards for the aggregate itself, a change that will have significant impact on the source materials quarried.

An attractive characteristic of MQs is the relatively small "footprint" they leave on the land. Given an ostensible quarry depth of 100 m, only about 4 km^2 would be occupied (McNally, 1998). Indeed, Lafarge Redland's proposed MQ at Harris, Scotland (since rejected in 2001) was to impact only 459 ha (A.L.P. Ambrose Minerals Planning & Development Consultancy, Winter 2000, A.L.P. Ambrose News, p. 4., accessed 5/9/02 via URL www.alpambrose.co.uk/news.html). By comparison, the Foster Yoeman Glensanda coastal MQ, occupies a total area of 4,500 ha. This MQ provides 6 Mtpa but has reserves sufficient for 150 years at 15 Mtpa as reported by the owner overview (Foster Yeoman Ltd., 2000, accessed 05/09/02 via URL http://www.yeoman-poland.pl/e_ofirmie2.html).

Other candidates for Type I MQ development are large batholiths where care needs to be taken to characterize all phases present and to determine the extent and degree of hydrothermal alteration. Detrimental changes associated with weathering and groundwater must also be assessed. These single product MQs may be developed in any large homogenous rock unit, but most commonly in carbonate rocks. Historically, MQs have also supplied carbonate rocks and other needed material for the manufacture of cement. The Glensanda quarry, which is described below, is a typical Type I MQ.

The second type or "Type II MQ" may also be located in carbonate rocks but produces rock not only for aggregate but a number of different products suitable for other uses such as chemicals, agricultural lime, filler, and cement. These quarries require considerable management to maximize return from the complex mixture of commodities they produce. A number of the examples given below are Type II MQs.

The third type is for MQs developed in sand and gravel. While production sites in sand and gravel deposits are usually identified as pits, they are identified here as a MQ to be consistent. These sites produce material mainly used for aggregate and comparable to material from Type I MQs. Type III MQs are not common. This may reflect the polymictic lithology of many alluvial fans and intercalations of organic and other deleterious materials. The Sechelt pit, described below may have qualified as a Type III MQ for just one year only when production exceeded 5 Mtpa! It is possible that some Type III MQ operations may become either Type I or II MQs given a transition to mining underlying bedrock.

Sand and gravel deposits in alluvial fans, if sufficiently large, are the principal sites to develop Type III MQs. The median volume of alluvial fan sand and gravel is estimated to be 110 million cubic meters (~220 million t). The largest 10 percent are perhaps 1 billion cubic meters (~2 billion t) or more (Bliss, 1998). Most sand and gravel deposits are thin compared to their area, and extraction disturbs large areas relative to comparable production from quarries. The median thickness of alluvial-fan sand and gravel deposits is estimated to be 13 m, and is on average twice as thick as those found in other types of sand and gravel deposits (Bliss and Page, 1994). Some glaciofluvial sand and gravel deposits can also be very large (Bobrowsky and Manson, 1998) and may also be suitable for MPs based on observations from Vancouver Island, B.C. The following discussion of MQs is largely applicable to Type III MQs as well.

Production rates—how big can they be?

Without regard to location or commodity extracted, what is the maximum production rate of material from an MQ given existing mining technology? A survey of very-large mines identified lignite mining in Germany as likely to be the largest mining operation in the world (M. Poulton, written commun., 01/04/2002). The three mining operations in the Rhenish (Rhineland) mining region west of Cologne, Germany produce a total of 91.9 Mtpa of lignite. The largest of the three is the Garzweiler Mine with an output of 31.4 Mtpa in 1998/99. The average stripping ratio is around 4.9 cubic meters per ton of lignite (The web site for the mining industry, [undated], access 04/01/02 via URL http:// www. mining-technology. com/ projects/ rhineland/ index.html). Assuming 2 metric tons (t) per cubic meter for the overburden, these figures suggest that the Garzweiler operation moves about 308 Mtpa of overburden. Together with lignite production, these figures indicate total material moved to be 340 Mtpa! Given a very-large aggregate resource under favorable conditions, technology will allow very large mining operations, and therefore MQs may have very-high production rates.

MQs can be expected to share some of the same advantages and problems found in other types of large surface mines. MQs with extraction of material from intrusive bodies may have some of the same mining-related issues as porphyry copper mines, including concerns about slope stability, etc. Extraction of bedded material like limestone from a MQ may have some of the same problems as those for very-large coal mines. It is likely that underground mining will not be important, although underground facilities may be needed for material handling and quarry services.

Limitations imposed by equipment will not be a problem. In casting a vision of the evolution in mining technology, it is noted that development of jumbo-sized mining equipment has quickened in the last few years. Although ultra-class trucks are not intended for quarry applications, their technology will quickly filter down to haulers in the 150 t range. The advantages will be improved reliability, faster speeds, and lower operating costs (Carter and Drake, 1999).

Megaquarries coastal location and transportation issues

Development of MQ sites in coastal areas may not be possible due to legal requirements set by national or local governments. For example, Spain's coast law of 1988 prohibits all quarries within 200 m of the coastline and "large" quarries from a coastal strip of 1 km. MQs appear to be excluded from coastal England and Wales by government guidelines. Quarries are expected to be "well away from the coast." (Trade and Environmental Database, 1995, accessed 5/25/02 via URL http://www.american.edu/TED/QUARRY.HTM.) Extraction of aggregate may be possible in sites inland from the coastal strip in countries with these restrictions if port handling facilities and transport across the coastal strip is permissible. Regulatory restrictions of these types must be identified early in the exploration process.

Since transportation is the limiting cost in penetrating aggregate markets, those MQs in areas with coastal access and harbor facilities will be the most competitive. U.S. transportation costs in 1999 were 0.06 to 0.3 cents per t/kilometer (t/km) for ocean, 0.6 to 1.7 cents for barge, and 1.7 to 5.6 cents by rail. Truck transportation ran about 6.8 to 11 cents for long hauls, and 9 to 18 cents for short hauls (David Holm talk, Northwest Mining Association speech, 1999). Markets accessible to each quarry will also be larger. A number of distribution yards may be needed hundreds of kilometers away from the primary pits (e.g., Dolese Brothers Co. has aggregate yards in southern Kansas for the redistribution of stone produced from quarries located in southern Oklahoma about 320 km away.) MQs may have significant production economics of scale but much of the cost of aggregate to users is in transportation. The cost to users may actually be greater given the larger transportation distances even if by water (Drew and Fowler, 1999). This cannot be anticipated with certainty in many situations as the cost of aggregate to the user may or may not be less in a real dollar basis, as the aggregate will also require double handling from ship to stockpile and stockpile to truck or rail to the end users.

As production from MQs will cross regional jurisdictions and country boundaries, aggregate companies also seek locations where cultural attitudes and government policies would be more welcoming to future MQs. Presently, product enters the US from an operation in Yucatan, Mexico (Rukavina, 1991) and several Canadian provinces. Imports from British Columbia have climbed from under $4 million dollars in 1997 to $7 million in 2001. Jurbin (2001, p. 20), stated "As permitted reserves near the booming San Francisco Bay area market dry up and the Canadian dollar's value drops, transporting large volumes of material from B.C. has finally become viable." Likewise, trade of crushed stone from New Brunswick has gone from virtually zero in 1997 to $3 million in 2001. Nova Scotia now ships $12,500,000 in crushed stone, a more than 300 percent growth since 1997. Whereas these shipments come from export-oriented quarries, Ontario contributes another $10 million from Great Lakes MQs and smaller operations in

Southwestern Ontario (Industry Canada, lasted modified 01/21/02, Trade data online, access via URL http:// strategis.ic.gc.ca/ sc_mrkti/ tdst/ engdoc/ tr_homep.html).

Megaquarries locations—known, proposed and suggested

Introduction

Possible locations for MQs are dependent on geology (as described above), geography, environmental concerns, market locations and demand. A number of quarries that likely have, or will have, MQ status are already operating in the world. In fact, very large quarries have been present for a considerable time for production of limestone used to fabricate cement. MQs are defined by large production rates and not strictly by the commodity or commodities they produce. However, the MQs discussed here are some in which aggregate is part, and usually a major part, of the production. The list of operations that follows is likely not complete but does help to give some idea of the challenges and issues that may be faced in the development of MQs!

Glensanda, Scotland

A true MQ in Europe is the Glensanda quarry operated by Foster Yeoman Ltd., a family owned operation, in western Scotland. This is a Type I MQ. The source of crushed stone is the Strontian Granite of the coastal batholith composed of a pale pink, medium grained, prophyritic biotite-granite that is suitable for most construction applications (Smith and Collis, 1993). The quarry supplies aggregate to many parts of southern Britain and Northern Europe (Anonymous, 1986) including Poland and Berlin, Germany using barge transport from Swinoujscie, Poland ((Foster Yeoman Ltd., 2000, accessed 05/09/02 via URL http://www.yeoman-poland.pl/e_ofirmie2.html). Other recipients of aggregate from this quarry include builders of the channel tunnel under the English Channel from Dover to France. Aggregate produced from Glensanda was also shipped to an airport in Trinidad in the Caribbean (http:// www. iht. org. uk/ news/ pages. asp? newside=79 &typ=I). This MQ has a production capacity of 15 Mtpa of material (Trade and Environmental Database, 1995, accessed 5/25/02 via URL // http://www.american.edu/ TED/QUARRY.HTM). Permission has been granted for Glensanda to extract over 10 Mtpa, but it is now operating at less than 6 Mtpa as of May 2000. This is due to an inability to find markets (Friends of the Earth Scotland, 2000, access 5/25/02 via URL http://www. foe-scotland. org. uk / nation/ superquarry3 . html.)

Both mining methods and loading capacities will affect production rates. The port facilities at Glensanda are designed to accommodate bulk carriers for transport of crushed stone up to 150,000 deadweight t or cargo weight (dwt); however the average size is expected to be around 60,000 dwt (Anonymous, 1986).

MQs, including Glensanda, can be expected to generate considerable waste. This is the inevitable material generated during crushing that has no market. Of particular concern is production of fines (minus 200 mesh material) that are not marketable. One possible way to use these fines and other waste material is under investigation by Foster

Yeoman Ltd., along with the Scottish Association of Marine Science. They are planning to build an artificial reef for fishery protection and enhancement on the west coast of Scotland off the island of Lismore (The proposed location, [undated], accessed 03/18/02 via URL http:// sams.ac.uk/ dml/ projects/ reef/ overview.htm). The habitat to be created is targeted for valuable species such as lobsters and edible crab. The reef is to be created in water of 10 to 30 m depths with exposure to sufficient water currents to insure adequate oxygen and food but not so strong as to move the blocks. The blocks used to create the reef will be fabricated using fines from the Glensanda quarry. Blocks using 15 different mix designs were developed and have either a simple or complex form. They measure 40 cm by 20 cm by 20 cm, and weigh between 25 kg and 40 kg. ((The proposed location, [undated], accessed 03/18/02 via URL http:// sams.ac.uk/ dml/ projects/ reef/ overview.htm). If approved, a reef containing 1.25 million concrete bocks will be built with an area of one square kilometer and will be among the largest artificial reefs in the world.

Rodel (Harris), Scotland

The proposed Rodel (Harris) quarry on Southern Isle of Harris, Outer Hebrides, Scotland would be a MQ with reserves of 550 Mt and a 60-year life (Wardrop and Walton, 1996). The material to be mined is anorthosite and the average annual production would be 9 Mtpa. The product is intended to supply "road and construction work in the South of England and overseas" (http:// www .leverburgh. co. uk/ quarry.htm). This will probably be a Type I MQ.

Plans for production of aggregate out of the Rodel MQ call for the entire production to be shipped using bulk container barges at a docking facility that will also be used to handle all incoming equipment and supplies. Good transportation design is important for these large operations. Few employees are needed according to the plan for the Rodel MQ, which will require 80 employees at full production (Leverburgh, [undated], accessed 03/15/02 via URL http:// www. leverburgh. co. uk/ quarry.htm). The quarry is to be worked using conventional technology that has proven safe and reliable at other quarries (Wardrop and Walton, 1996).

MQs are difficult to hide because they are operational for a long period of time and will be an unavoidable part of the local landscape. Once mining has ended, sites of past MQ production will be highly visible unless specific efforts are taken. The Rodel MQ (Wardrop and Walton, 1996) is designed so that edges of the quarry will not alter the skyline and that the shape of the quarry will mimic local landforms. At closure, plans call for the MQ to ultimately have the appearance of a flooded glacial cirque or a sea loch. Care is to be taken so that the geometry of the resulting feature has geometry comparable to nearby natural cirques as well as stable slopes to reduce the likelihood of rock falls. These closure requirements will result in a significant reduction in reserves (Wardrop and Walton, 1996). The idea of mimicking existing landforms during quarry reclamation is described by Walton and Allington (1994). The geomorphology of the area surrounding a quarry must be carefully analyzed because surficial geology can create complex outcrop forms. If recreated, they also need to be technically sound for the quarry lithologies. One problem is that natural slopes developed in weaker strata are often steeper than slopes that can be created safely by mining (Walton and Allington, 1994).

A public inquiry about the proposed Rodel MQ administered by the Secretary for Scotland opened Oct., 1994, ran for 8 months, and was the longest in Scottish history at that time (Leverburgh, [undated], accessed 03/15/02 via URL http://www. leverburgh. co. uk/ quarry.htm). Local government and a majority of the local population were initially supportive of the proposed project. Despite all of the efforts to develop the Rodel (or Harris) quarry and the expectation that it would rejuvenate the Harris island economy, the project was refused as of Nov. 3, 2000 (http://www.scotland.gov.uk/ news/ 2000/ 11/ se2846.asp). The document of refusal for development of the Rodel (or Harris) superquarry in Scotland lists reasons that fall into three generals categories: (1) not consistent with government policy of sustainable development strategy and of a cautious approach to superquarry development, (2) unacceptable aesthetic and environmental degradation, and (3) cultural and societal penalties exceed economic rewards offered (Friends of the Earth Scotland, 2000, access 5/25/02 via URL http:// www.foe-scotland.org.uk/ nation/ superquarry3. html.) It seems clear that MQ development is likely to be limited not so much by geology, technology requirements, and geography but by social and cultural attitudes of areas where they would be best located. It will be argued that even isolated areas with limited human use and occupancy and biodiversity should not be considered, as any activity may destroy natural areas thought worthy of protection.

The project was denied despite the fact that Scottish planning policy allows for up to 4 superquarries in Scotland based on projected aggregate demand in England. As a consequence, further development of MQs in Scotland is not likely. One issue is that projections of aggregate demand in England made in the 1990's were high—in fact demand in 2000 was running a little more than 50 percent of those projections. The argument is that additional MQs are not needed. An additional environmental review with public inquiry (as of Feb. 1, 2000) is now required before development of all new harbor facilities, an essential element in coastal MQs. The current UK Labor Government has policies that can be interpreted as discouraging quarry production of all types. An aggregate tax of □1.60 per ton was to take effect in April 2002. These taxes are applied to "tax environmental bads and reward environmental goods." In the "Trust for Tomorrow" document, it is stated "Labor rejects development of any further coastal superquarries." (Friends of the Earth Scotland, 2000, access 5/25/02 via URL http://www.foe-scotland. org. uk/ nation/ superquarry3. html.) Clearly MQs, despite the best efforts, can be difficult to launch.

Milton Quarry, Ontario

The largest Canadian crushed stone producer for 12 consecutive years is the Milton Quarry about 40 km west of Toronto. It is located on the Niagara Escarpment, an assemblage of Ordovician and Silurian carbonate rocks. Products from the quarry supply cement and other aggregate users. Its usual production is between 6 and 8 Mtpa. The quarry supplies about 30 to 40 percent of the crushed stone sold within the greater Toronto area (Aggregate & Roadbuilding, [undated], accessed 04/01/02 via URL http:

//rocktoroad .com/ 96top10q.html; 04/01/2002). This is a Type II MQ. Unlike other MQs, distribution is primarily by land transportation, not water.

Manitoulin Quarry, Ontario

Production from the Manitoulin Quarry, on Manitoulin Island in Lake Ontario is reported to be 5.015 Mtpa for 2000, a 27 percent increase from 1999. This water-based production site is among the fastest growing quarry operations in Canada and qualifies as a MQ. Production is currently from a single 16 m face in the Amabel formation, a white-gray fine-grained dolomitic limestone (Aggregate & Roadbuilding, [undated], accessed 04/01/02 via URL http:// rocktoroad.com/ 99quary. html). At least 50 years of confirmed reserves have been identified. Production includes not only construction aggregate but also metallurgical rock material. Markets include cities in Ontario as well as Saginaw, Michigan, Cleveland, Ohio and Chicago, Illinois (Aggregate & Roadbuilding, [undated], accessed 04/01/02 via URL http:// rocktoroad .com/quarry. html). The material "is highly regarded by ready-mix concrete producers and brick manufactures both for its appearance as well as its physical properties" (Aggregate & Roadbuilding, [undated], p 4,accessed 04/01/02 via URL (http:// rocktoroad.com/ 99quary. html). This site is a Type II MQ.

Texada Island Quarry, British Colombia

Products from this quarry supply markets by marine transport in Los Angeles, San Diego, San Francisco and Seattle. This is likely to become a Type II MQ. (Jurbin, 2001) as production levels are projected to reach 10 Mtpa. Production is from the lower member of the Upper Triassic Marble Bay Formation, a 200 m thick high-calcium limestone with massive beds (Krukowski and Newman, 2001) that is suitable for cement and lime, chemicals, aggregate, and bulk fill material. Production was about 4 Mtpa in 2001.

Sechelt Operations, British Colombia

Historically, the Sechelt Operation has produced sand and gravel at a site located 100 km north of Vancouver. The operation is located in a large glaciofluvial deposit resting on bedrock of granodiorite of the Coast Mountain Plutonic Complex that intrudes into metamorphosed sediment and volcanic rocks (Fred Shrimer, written commun., 04/20/2002). The Sechelt operation is frequently ranked as the largest sand and gravel operation in Canada with production typically around 3.5 Mtpa. In 1993, sand and gravel production peaked at 5.6 Mtpa to help meet the construction needs of the Vancouver International Airport expansion (Aggregate & Roadbuilding, [undated], accessed 04/01/02 via URL http: //rocktoroad .com/article3. htm) and, for that year; this site would have qualified as a Type III MQ! In addition to sand and gravel, the Sechelt Operation also produces about 0.75 Mtpa crushed stone from the granodiorite. Jurbin (2001) reports that expanded aggregate production and processing capacity, loading operations and a marine-based distribution including the western U.S. coast, may allow full production to be 6.6 Mtpa—making this a future Type I MQ site.

Norway

Neeb (1998) suggested that Norway might export more aggregate given continued demand in Northern Europe. The likelihood for this is enhanced if there are increased restrictions on production of marine sand and gravel in the North Sea and elsewhere. Coastal access and suitable lithology nearby are two important factors that suggest that MQ development in Norway is possible. Neeb (1998) reported that about 23 percent of Norwegian aggregate produced from 17 mines (the capacity of which is not given) is targeted for export to England, Denmark, Germany, the Netherlands and Belgium. However, given that 5.9 Mt were exported in 1995 and that the sources were the same 17 mines, the average production would be 0.35 Mtpa per mine. That is considerably less than the 5 Mtpa used to definite a MQ. Neeb (1998) puts forward the idea that demand in 2010 might reach 460 Mtpa for the countries listed above (Neeb, 1998). Given that Norway would continue to supply the same proportion of the market as today, several operations may reach MQ size.

Closing remarks and questions

Within the next 50 years if current business, economic, and population trends continue, the development of MQs and MPs will have a major affect on the aggregate production patterns. Production rates from MQs will be high and they will have huge reserves. MQs may have markets in a number of countries at a considerable distance from the source.

In a world dominated by MQs, considerably fewer operations will be involved. MQs can be expected not only to be the primary source of aggregate in the US but in other parts of the world as well (e.g., Europe, Australia.) Because MQs and MPs have only recently become important, not all MQ-related issues can be anticipated. However, those issues associated with past and existing very-large surface mines and quarries may provide some clues.

One of the biggest challenges faced by developers of MQs is the need to return the extraction sites to ecological values not just comparable to how they were initially found, but to values higher than how they were initially found as recommended by Lafarge Corp. (Quarry Management, [undated], accessed 05/25/02 via URL http:// www. lafarge. com/ lafarge/ lafargeV2. nsf/html/ US40F47FF1 A14F14 F7C1256 AE90048 F971/ $file/ devdurenv3 US.pdf.)

Those who are planning to develop MQs need to comprehend the full complexity of the culture and priorities of the local and national populations of the areas of interest. Equally important is the ability to recognize how these attitudes might influence decision makers. Even in the developed world there is almost a total lack of understanding of the importance of raw materials for the continued operation and maintenance of modern civilizations.

Some final questions are: (1) what will be the likely affects on cost if US/world aggregate production is dominated by MQs? (2) How many companies and how many

MQs worldwide are likely to be present in 50 years under a variety of global development scenarios? (3) Would it be possible for producers to form an MQ-based cartel?

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PETROGRAPHY OF GRAVEL AGGREGATES FOR INDIANA BITUMINOUS PAVEMENT OVERLAYS

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ABSTRACT

Analysis of gravel aggregates, a predominant source in Indiana, determined the factors affecting friction properties when used in highway bituminous overlays. Petrographic, physical, and friction tests were performed. Petrographically, each rock particle was identified using hand lens and HCl. Particles were classified into five groups: igneous, metamorphic, limestone, dolomite, and other sedimentary rocks. For physical properties, absorption, specific gravity, Los Angels abrasion loss, and freeze-thaw loss were considered. To measure friction properties related to skid resistance, aggregate coupons were made from each individual rock type. Initial friction values (IFV) and Polished values (PV) were measured using the British Pendulum tester (ASTM Standard D3319, E303).

Results of the petrographic study indicate gravel samples consist mainly of carbonates (limestone and dolomite). Igneous rock is next abundant, followed by other sedimentaries and metamorphic rocks. Based on British Pendulum test results and correlation with other properties, dolomites in gravel samples have higher IFV and PV values than do limestones and other sedimentaries. Igneous and metamorphic rocks are highest. Crushed faces of gravels are highly correlated with increased IFV and PV. Gravels are a potential aggregate source for bituminous wearing courses on Indiana highways.

INTRODUCTION

Major gravel sources in Indiana occur in landforms deposited directly from glaciers or by glacial melt water streams during continental glaciation. In Indiana gravel samples, 10 to 20 varieties of rocks can be found (Carr and Webb, 1970; Carr, 1971), including carbonate, sandstone, siltstone, chert, shale, iron congregation, gneiss, schist, quartzite, granite, granodiorite, diorite, gabbro, andesite, basalt, syenite, dacite, rhyolite, amphibolite, and quartz (Shakoor and West, 1979; McGregor, 1960).

In the study (West and Cho, 2001; West et al., 2001), six gravel sources were evaluated. A megascopic, petrographic examination was performed and frictional properties including initial fiction value (IFV) and polished value (PV) were determined on coupons made from these gravels. Objectives of the petrographic examination of gravel samples were to describe and classify constituents and determine the relative amounts of these different materials based on ASTM Standard C295.

RESEARCH PROCEDURES

Selection of Samples for Examination

Approximately 2.5 kg of a gravel sample were prepared for sieve analysis by reducing the sample material to the required quantity according to ASTM Standard C 702. From this, by sample splitting, 300 representative particles were obtained and subsequently examined.

The size range from 9.5mm to No. 4 (4.75mm) was chosen and fractions smaller than the No. 4 were excluded from the analysis. It is generally accepted that the coarse aggregate portion largely determines skid resistance for bituminous pavements, and the finer portion provides only ten to thirty percent of the aggregate used in the bituminous surface. All the particles present in the plus No. 4 size fraction were examined when the total numbered less than 300 particles.

Examination of Gravels

Rock particles were identified in a wet surface condition which enhances color and structure of the particles. Hand lens and binocular microscope were used to identify individual rock constituents. Following this, 0.1 N HCl was applied on soft rocks to differentiate carbonates from other constituents. Limestones produced a brisk effervescence whereas dolomite showed slow effervescence or only effervescence when scratched. Ten rock types were chosen according to their abundance in the samples. When particles of indeterminate type were encountered, they were included with known types having similar texture and hardness. The ten rock categories selected are limestone, sandstone, siltstone, shale, chert, granite, diorite, felsite, gneiss, and quartzite.

The percentage of crushed particles was determined according to the definition of a fractured face in the Indiana Test Method (ITM) 204. It reads: Fractured surface: A broken surface constituting an area of at least 25% of the largest cross sectional area of the particle. A fractured particle is defined as one being fractured either by mechanical means or by nature. Natural fractures must be similar to those fractures produced by a crusher.

The ten rock types were consolidated into five major rock types, that is, igneous, metamorphic, limestone, dolomite and other sedimentary rocks. The petrographic analyses for the gravels are summarized in Table 1.

Table 1. Rock type percentage in gravel samples

Id. No.	Igneous Rock	Meta. Rock	Limestone	Dolomite	Other Sed. Rock	% Crushed
GR-1	27.5	11.1	19.5	20.8	21.1	99%
GR-2	35.8	3.6	20.8	21.5	18.3	30%
GR-3	29.7	6.8	14.1	14.7	34.7	10%
GR-4	49.2	8.7	17.5	8.6	16.0	78%
GR-5	9.0	3.7	15.9	40.4	31.0	57%
GR-6	27.8	18.7	6.2	0.4	46.9	93%
Average	29.8	8.8	15.7	17.7	28.0	61%

Results show that gravel samples examined consist mainly of carbonates (limestone and dolomite) ranging from 6.6 percent to 56.3 percent. Igneous rock is the second most abundant constituent, ranging from 9.0 percent to 49.2 percent, followed by other sedimentary rocks ranging from 16.0 percent to 46.9 percent. Metamorphic rocks comprise a portion ranging between 3.6 percent and 18.7 percent of gravel samples. Figure 1 highlights the significant difference in constituent percentage for these six aggregate sources.

Physical Properties of Gravels

The physical test results for gravel samples were obtained from INDOT. These test results are listed in Table 2.

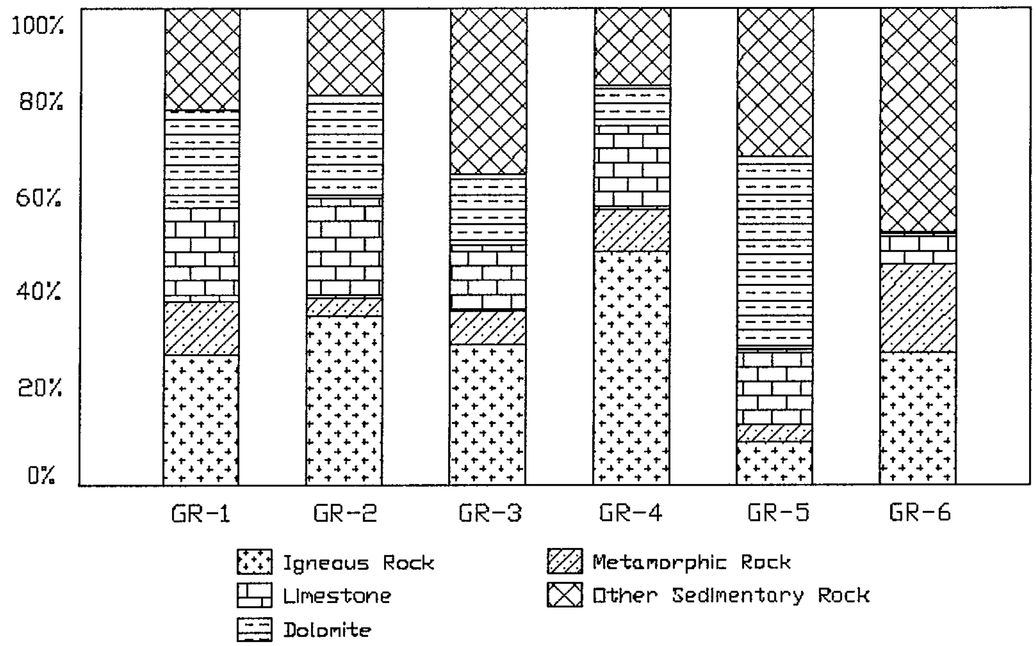

Figure 1. Distribution of different rock types in gravel samples

According to results of the physical properties of the gravels, absorption ranges from 1.21 % to 3.21 %, specific gravity from 2.509 to 2.676, Los Angeles abrasion loss from 22.64 % to 28.00 % and the freeze and thaw loss in brine solution from 4.49 to 9.18. For sample GR-6 only the freeze and thaw loss in water was available, resulting in 2.06 % loss.

Table 2. Physical properties of gravel samples

Id. No.	Absorption	Specific Gravity	L.A. Abrasion Loss	Freeze/Thaw Loss in Brine
GR-1	3.21%	2.590	27.99%	9.18%
GR-2	1.42%	2.593	26.26%	4.49%
GR-3	1.21%	2.676	22.64%	5.67%
GR-4	1.33%	2.638	24.22%	4.64%
GR-5	1.57%	2.645	23.19%	5.77%
GR-6	2.03%	2.509	28.00%	*2.06%
Average	1.80%	2.609	25.38%	5.95%

Note: * Freeze - thaw loss in water

British Wheel Testing, IFV and PV

The British wheel test procedure involves two separate pieces of equipment; the British Wheel machine and the British Pendulum. This test was adopted as a British Standard and later as ASTM D 3319 and ASTM E 303.

The British wheel machine is depicted in Figure 2. It was designed to provide rapid wear and polishing of specimens by a pneumatic tire.

Major aspects of the British Wheel are:

- Road Wheel: Aggregate specimens are prepared and mounted on the 16-in. diameter steel wheel having a flat periphery 2-1/2 in. wide (Figure 3).

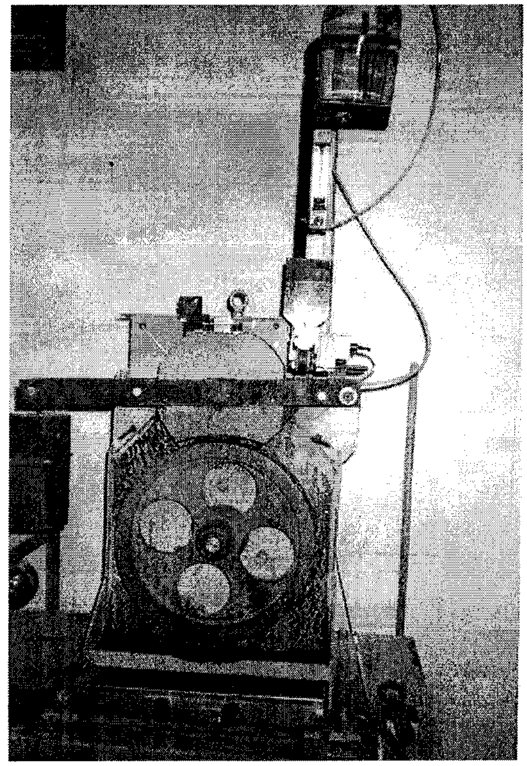

Figure 2. The British Wheel machine

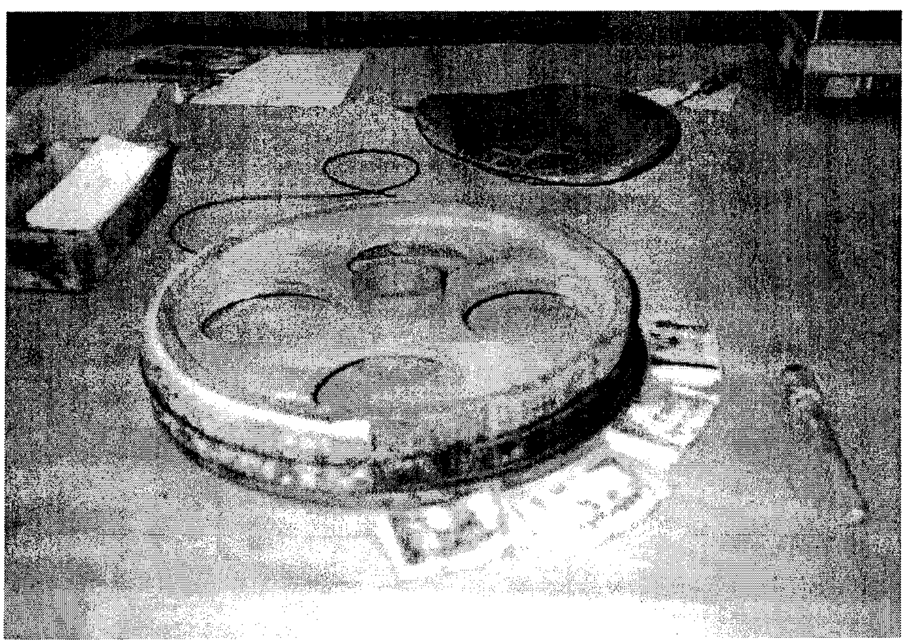

Figure 3. The Road Wheel

- Rubber Tired Wheel: The tire used in the study was modified from the original per ASTM D-3319. The aggregate surface was in contact with a cross-hatch pattern tread tire. Pneumatic tire pressure was 35±2 psi and the tire rests on the road wheel test specimens with a force of 88±1 pound. This is developed by a weighted lever system to which the axle of the rubber-tired wheel is fixed.
- Grid Feed System: Accelerated wearing and polishing is accomplished by the continuous, uniform application of an abrasive between the rubber tire and the road wheel. 150-grit size silicon carbide is vibrated from a small hopper to a chute. Water is supplied to wash the grit onto the wheel.
- Aggregate Specimen: Aggregates were sieved to produce samples less than ½-in. size but retained on the 3/8-in sieve. They are washed, dried, and stored for specimen preparation. Only particles with a relatively flat surface and a minimum 3/8-in thickness were included. Specimens were prepared by closely placing individual aggregate pieces in an inverted mold. Wax was placed on the mold sides and top plate to act as a release agent. Ottawa sand was used to fill the space between aggregate particles leaving the tops of aggregates exposed. Polyester resin mixed with methyl ethyl ketone peroxide (MEKP) was placed in each mold to anchor the aggregates. Each molded specimen was allowed to cure for 3 hours before removal from the mold. The top plate and mold were disassembled to remove a specimen. Sand, placed in the bottom of the inverted mold, prevented polyester resin from penetrating to the bottom of the mold or to the specimen's wearing surface. Specimen preparation is both tedious and time consuming. Prepared specimens are shown in Figure 4.

British Wheel simulates the polishing action of vehicular traffic on the aggregate surface. Fourteen curved coupons, including two samples comprising ten molds and four control specimens, are placed along the circumference of the steel wheel. The small pneumatic tire is placed in contact with the molds on the steel wheel. As the tire and wheel turn grit and water are evenly applied to perform the polishing action.

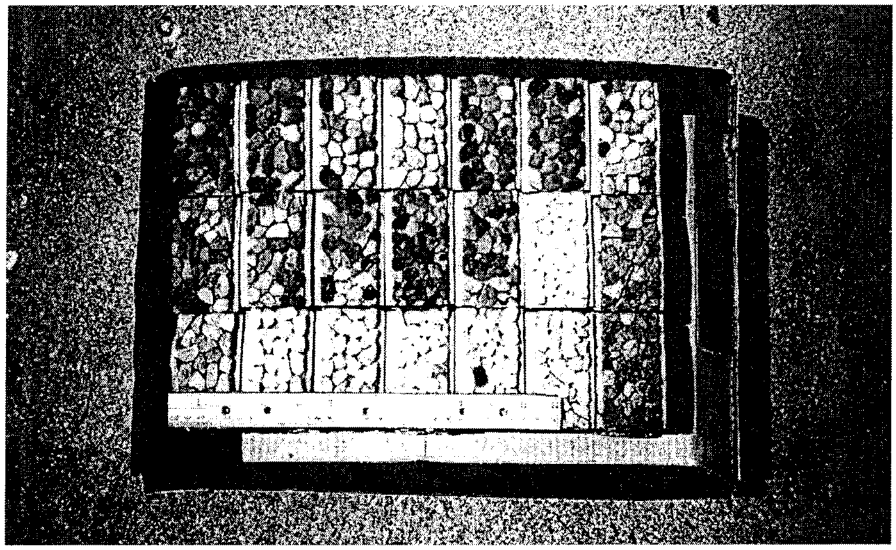

Figure 4. Some prepared specimens

Aggregate coupons are polished for ten hours at which polish equilibrium generally occurs for most rock types. These results indicate the polish susceptibility for a given aggregate source.

Friction values for aggregate coupons were measured before and after polishing using the British Pendulum. It is a dynamic pendulum impact tester, which measures energy loss when a runner slider moves by gravity over the aggregate test specimen (Figure 5). After removal from the polishing wheel, each test specimen is cleaned of grit and locked into the specimen base. Next, the pendulum is released and the rubber slider affixed to the end of the arm makes contact with the surface under a preset normal load and for a specific contact length.

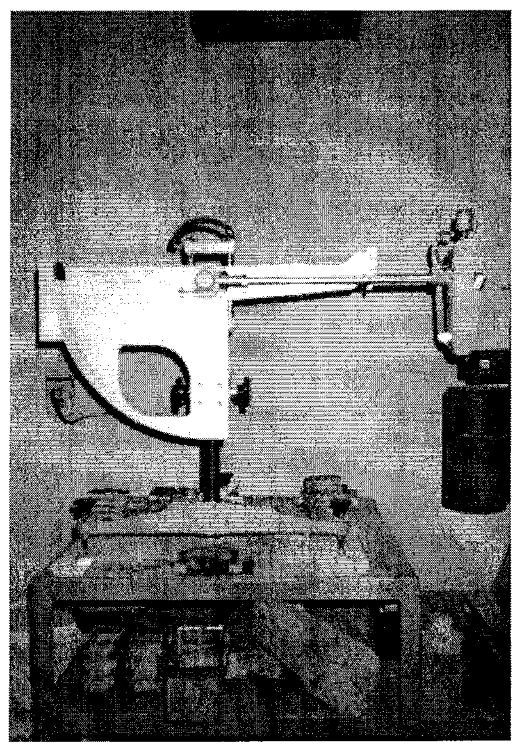

Figure 5. The British Pendulum tester

The angle between the pendulum arm and the horizontal after the slider passes over the surface is a measure of the energy absorbed by that surface. When the pendulum arm is released from a fixed initial horizontal position, it carries a pointer along a circular scale calibrated to read frictional measurements known as British Pendulum Numbers (BPN).

Prior to polishing an aggregate coupon, the friction value is determined with the British Pendulum and termed the Initial Friction Value (IFV). The BPN obtained after polishing by the British Wheel and measured with the British Pendulum is called the Polished Value (PV). The difference between IFV and PV is known as the Wear Index (WI), which indicates a reduction in frictional resistance for the aggregate source.

Aggregate coupons for gravel samples were made using single individual rock types. Initial Friction Value (IFV) and Polished Value (PV) were measured by the British Pendulum tester. IFV and PV for gravel aggregates are listed in Table 3.

Table 3. IFV and PV for rock constituents in gravel samples

Id. No.	Igneous Rock		Metamorphic Rock		Limestone		Dolomite		Other Sed. Rock	
	IFV	PV	IFV	PV	IFV	PV	IFV	PV	IFV	PV
GR-1	43.3	26.3	43.3	28.3	43.3	25.3	46.7	27.3	43.7	25.3
GR-2	32.2	21.0	-	-	33.2	23.1	41.7	24.6	36.6	23.1
GR-3	34.2	21.0	-	-	34.2	24.1	40.2	26.9	35.2	24.1
GR-4	40.3	23.7	-	-	36.3	25.3	42.3	25.3	35.3	23.6
GR-5	40.3	24.3	-	-	35.7	18.6	41.6	24.5	37.2	22.6
GR-6	45.6	29.9	39.6	27.5	46.6	31.4	-	-	35.7	21.6
Average	39.3	24.4	41.5	27.9	38.2	24.6	42.5	25.7	37.3	23.4

Based on Table 3, for the British Pendulum test, igneous rocks in gravels have an IFV ranging from 32.2 to 45.6 and a PV from 21.0 to 29.9. Aggregate coupons of metamorphic rocks were made only from GR-1 and GR-6 gravels because insufficient metamorphic rock pieces in other gravel sources were present. For metamorphic rocks IFV ranges from 39.6 to 43.3 and PV from 27.5 to 28.3. The limestones show IFV ranging from

33.2 to 46.6 and PV ranging from 18.6 to 31.4. The dolomite in gravels show IFV ranging from 40.2 to 46.7 and PV from 24.5 to 27.3. The other sedimentary rocks such as sandstone, shale, siltstone and chert show IFV ranging from 35.2 to 43.7 and PV from 21.6 to 25.3.

As indicated in Table 3, on average, dolomites show higher IFV and PV values than do limestones. Weighted averages of IFV and PV for each gravel sample based on their percentage of rock compositions are summarized in Table 4.

Table 4. Weighted average of IFV and PV for gravel samples

Sample No.	IFV	PV	Remarks
GR-1	44.1	26.3	99% crushed
GR-2	35.3	22.6	30% crushed
GR-3	35.6	23.5	10% crushed
GR-4	39.0	24.1	78% crushed
GR-5	39.1	22.9	57% crushed
GR-6	39.9	25.6	93% crushed
Average	38.8	24.2	61% crushed

Example (GR-1):
IFV = 43.3(0.275)+43.3(0.111)+43.3(0.195)+46.7(0.208)+43.7(0.211)=44.1
PV= 26.3(0.275)+28.3(0.111)+25.3(0.195)+27.3(0.208)+25.3(0.211)=26.3

According to these results, as the percentage of crushed faces on gravel increases, both IFV and PV increase. In Table 5, the percent of crushed gravel faces is more highly correlated with IFV (R=0.89) than with PV (R=0.80). IFV is more dependent on the percent of crushed gravel than is PV. This means that the percent of crushed gravel faces has a diminished effect as gravels undergo polishing.

Correlation of Frictional Properties for Gravel Samples

In order to find the critical factors for frictional properties of gravels, data analyses for correlations between British Pendulum Number (BPN) and other aggregate properties were performed. Using a correlation analysis (SAS program), the statistics and correlation matrix for all parameters of the gravels were developed as shown in Table 5.

The upper number (R-value) in each cell is the coefficient of correlation between two variables defining the cell. R-value measures the degree of linear relationship among variables. The lower number (P-value) in each cell is developed from hypothesis testing, indicating the significance of the correlation; lower P-values mean greater significance.

Based on the correlation analysis and confidence interval, PV correlates well with IFV ($R^2 = 0.70$), absorption ($R^2 = 0.69$), freeze-thaw loss ($R^2 = 0.75$), and the percent of metamorphic rock in gravel ($R^2 = 0.69$). Note that R^2 is obtained by squaring the upper value in the cells for Table 5.

Table 5. Correlation matrix for gravel aggregates

	IFV	PV	WI	AB	SPG	LA	F-T	IG %	META %	LS%	DOL%	OS%	CRUSH %
IFV	1.00 0.00	0.84 0.04	0.92 0.01	0.89 0.02	-0.37 0.47	0.56 0.24	0.85 0.07	-0.18 0.74	0.51 0.31	-0.03 0.95	-0.01 0.99	-0.03 0.96	0.89 0.02
PV		1.00 0.00	0.56 0.24	0.83 0.04	-0.57 0.23	0.71 0.11	0.87 0.06	0.08 0.88	0.83 0.04	-0.33 0.52	-0.49 0.32	0.23 0.66	0.80 0.05
WI			1.00 0.00	0.77 0.08	-0.16 0.76	0.35 0.49	0.73 0.16	-0.33 0.53	0.18 0.73	0.18 0.73	0.34 0.51	-0.20 0.71	0.78 0.07
AB				1.00 0.00	-0.48 0.34	0.74 0.09	0.94 0.02	-0.21 0.69	0.47 0.35	0.07 0.90	0.00 0.99	-0.02 0.98	0.72 0.11
SPG					1.00 0.00	-0.89 0.02	-0.36 0.55	-0.01 0.98	-0.76 0.08	0.49 0.33	0.49 0.32	-0.40 0.43	-0.62 0.19
LA						1.00 0.00	0.58 0.30	0.11 0.83	0.66 0.16	-0.14 0.79	-0.40 0.43	0.09 0.87	0.67 0.14
F-T							1.00 0.00	-0.34 0.57	0.68 0.21	0.14 0.82	0.15 0.82	0.08 0.89	0.59 0.29
IG %								1.00 0.00	0.13 0.81	0.21 0.69	-0.67 0.14	-0.49 0.33	0.03 0.95
META %				Symmetrical					1.00 0.00	-0.74 0.09	-0.76 0.08	0.58 0.23	0.68 0.14
LS %										1.00 0.00	0.53 0.28	-0.93 0.01	-0.22 0.67
DOL %											1.00 0.00	-0.28 0.59	-0.27 0.61
OS %												1.00 0.00	0.04 0.94
CRUSH %													1.00 0.00

In Table 5, the correlation coefficient between IFV and PV is 0.84. This indicates that PV measured after extensive polishing is still dependent on the initial BPN. Higher absorption and higher freeze-thaw loss are related to higher PV. This may suggest that aggregates that weaken easier develop an irregular aggregate surface, yielding a higher PV. Gravel aggregates with a higher portion of metamorphic rocks show higher PV and those with a higher percent of crushed particles provide higher IFV. Results suggest that crushed gravels in Indiana are a potential source of aggregates for bituminous wearing courses on state highways.

ACKNOWLEDGEMENTS

Research support for this project was provided by INDOT, FHWA and JTRP (Joint Transportation Research Program of Purdue University, West Lafayette, Indiana).

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Rock Excavations for Upgrading High Volume Roadways

Michael Vierling, Richard Cross, Jay Smerekanicz,
and Peter Ingraham

Widening of roadways and reconfiguration of interchanges in high volume traffic areas and critical corridors under live traffic conditions pose significant challenges to transportation officials and contractors. The New York State Thruway Authority and New York State Department of Transportation have faced significant challenges to roadway widening/reconstruction and bridge replacements in the greater New York City area. Recent projects have required re-cutting of slopes up to 115 feet high adjacent to roadways with average daily traffic counts of over 120,000 vehicles per day. Planning for maintenance and protection of traffic, staged construction, community relations and careful preparation of contract documents and blasting specifications are required to complete projects requiring substantial rock excavation and rock excavation support. The paper outlines recent construction in the New York City area including the $200M Tappan Zee Toll Plaza to Saw Mill River Parkway project in Tarrytown, New York. This project highlights the need for careful planning to maintain live traffic and a tight schedule for construction of 15 new rock slopes and replacement of 6 bridges in the highest traffic volume corridor on the Thruway. The paper discusses construction specifications, contract management, liquidated damages for traffic delays beyond specified limits, community relations, coordination of construction with traffic control operations, close in and production blasting issues, choreography of the shot/cleanup/reopening of the roadway, and rock support issues during bridge replacement.

INTRODUCTION

The New York State Thruway Authority (Thruway) and New York State Department of Transportation (NYSDOT) have undertaken a series of complex projects involving widening and reconfiguration of interchanges in high traffic volume areas on I-87 and I-287 (Cross Westchester Expressway) in the greater New York City metropolitan area. These interstate roadways, including I-95, are the main northern arterials leading to New York City. The I-87/I-287 corridors are maintained by the Thruway Authority and are part of a 600-mile long patron-supported system. With average daily traffic counts of over 134,000 vehicles per day, the Thruway system is actively managed by engineers, maintenance and law enforcement personnel dedicated to these roadways to ensure that patrons are provided safe and uninterrupted traffic conditions through the system. This paper discusses planning and construction issues for two projects completed in the past three years, and one ongoing project:

- Milepost 18.3 (southbound) on I-87/287 in Nyack consisted of reconstructing a 115-foot high diabase rock slope to add a traffic lane, improve slope stability and catchment area, and improve sight distance;

- Interchange 5 Cross Westchester Expressway consisted of replacement of two bridges, reconstruction of three rock slopes ranging from 60 to 110 feet in height, and roadway widening; and

- The Interchange 8 reconfiguration project I-87 to I-287, which has added lanes, provided new collector distributors to accommodate traffic going from I-87 to the Cross Westchester Expressway (I-287), fifteen new rock cuts including a through cut 60 feet high, two rock slope scaling areas, six bridge replacements, two new bridges and fifteen retaining walls up to 60 feet in height.

Each of the projects included rock slope reconstruction in areas where previous blasting predated pre-split blasting techniques, and significant backbreak and dilated joints were encountered. The projects were scheduled to address aging interstate rock slopes identified by the Thruway Authority's Rockfall Hazard Rating System and inventory, and to widen and update interstate infrastructure.

KEY CONSTRUCTION ISSUES

Construction involving rock excavations in densely populated areas with high traffic volume is governed by the need for construction safety, maintenance and protection of traffic (MPT), stakeholder issues, and schedule. In the Thruway system and greater New York City area, blasting in close proximity to residences, businesses, adjacent rail and highway routes, and water, natural gas, fiber optic and power supply infrastructure heightens the need for project safety and increases stakeholder concerns for impacts the projects may have on adjacent facilities. Adjacent projects and traffic incidents may impact the ability to control traffic through construction areas in high volume traffic zones and reduce the availability of lane closures and working room for the contractor.

In a patron-supported system MPT is a critical issue, and construction is often necessarily complicated by the need to allow traffic over bridges being replaced, requiring the construction in several stages. Necessary traffic stops for blasting must be limited in duration to allow traffic to clear in a reasonable timeframe, particularly where lane closures are in place. Utility relocation must also be sequenced with the phases of a project to ensure uninterrupted service.

Stakeholder issues include concerns for personal safety, damage due to blasting, disruption of traffic, and noise, all of which can be related to project schedule and the duration of construction. The construction schedule also affects traffic control and MPT.

PLANNING, DESIGN, AND CONTRACT DOCUMENTS

Construction of interstate highway improvements requires significant planning to address design requirements, staging, safety, MPT, stakeholder issues and to establish a construction schedule. Design of rock slope reconstruction includes evaluation of contractor methods and approach, including removal and casting of shot rock from the slope and scaling, maximum height of lifts and length of blasts, as well as blasting window duration allowed on the Thruway and NYSDOT roadways, and reasonable blasting production rates. Potential lane closures and temporary closures must also be considered to provide room for the contractor to complete blasting, while maintaining sufficient roadway vehicle capacity. Where necessary, temporary rockfall catch fences can be used to reduce lane closure requirements (Figure 1).

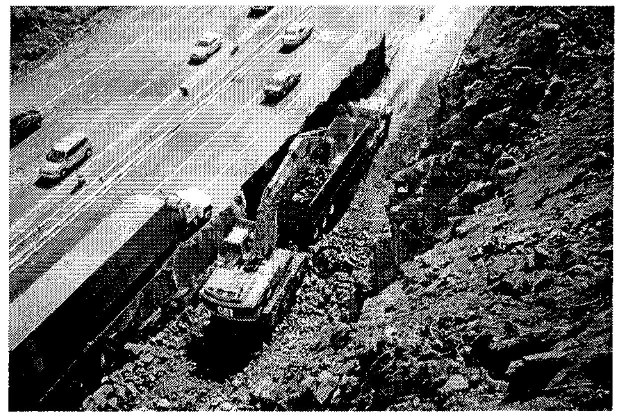

Figure 1 - Temporary Rockfall Catch Fence

Thruway traffic studies have shown that peak commuter traffic subsides between 10 am and 2 pm on weekdays, with the exception of Fridays during the Summer construction season. Accordingly, construction contracts are written to allow blasting between the hours of 10 and 2 pm, Monday through Thursday (excluding holidays). To provide further control to minimize impacts to traffic flow, blast face lengths and heights are usually limited to 30 feet. Although this causes the contractor to have a greater number of smaller blasts, the restriction limits the amount of shot rock to be removed from the adjacent roadway to a reasonable volume. Given the smaller size of the blasts, two to three blasts per day are not uncommon. Per federal regulations, traffic must be stopped for blasting operations; on Thruway maintained roads, this service is performed by the Thruway's troop of State Police. During inclement weather, traffic stops for blasting are not permitted due to reduced traction and sight distance. To maintain traffic flow, traffic stops for blasts are contractually limited to 20 minutes from the time the traffic is stopped for lanes adjacent to the blasting, and 10 minutes for lanes on the far lanes of the interstate. Delays exceeding these limits are assessed liquidated damages on a per lane basis for each 5-minute period the lane is closed. Traffic delays, when kept within these limits, generally take as long to clear as the delay itself; however, delays approaching two hours can take up to eight hours to clear depending on the time of the delay.

Prior to commencing a project, the Thruway contacts local stakeholders and holds informational meetings outlining the scope and schedule of work for a given project. Local municipal officials and emergency services are also contacted and informed of potential impacts on their emergency routes and services. On particularly complex projects like the Interchange 8 project, specifications are developed to include a public relations firm as part of the Consultant Inspection team.

CONSTRUCTION

Interchange 5 Cross Westchester Expressway (I-287) – The Interchange 5 project included reconstruction of a 110-foot high rock slope and adjacent rock slopes up to 60 feet high as part of an interchange reconfiguration and overpass bridge replacement project. Overall slope length was approximately 1000 feet. The rock slope was along a curve in the roadway and reconstructed slope angles varied from 1V:1H on low slope sections to 3V:2H at the tallest part of the slope. The roadway setback was as little as 30 feet from the nearest traveled lane and the tallest part of the slope was adjacent to a deceleration ramp.

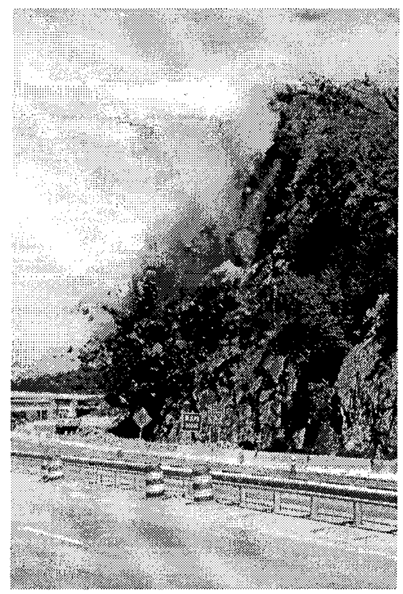

Figure 2 – Cross Westchester Blasting

Traffic control for the project initially allowed a shoulder closure along the tall slope to maintain ramp traffic. The contractor placed concrete barrier along the full slope length to achieve the shoulder closure, and an additional daily temporary lane closure was allowed from 9 am to 3 pm (Figure 2). Several traffic stops in excess of the 20-minute window were required to address high hanging rocks following a blast. Although the roadway was clear and swept, the DOT and Consultant Inspection team geologists had concern for vehicle safety because the catchment area was full of shot rock and additional falling rock would likely fall or roll onto the roadway. Following the long traffic stops, several solutions were reviewed with Thruway Traffic Safety and the contractor. These measures included activating an emergency crossover in the event of a roadway closure that would also facilitate casting shot rock from the crest of the cut, installation of temporary rockfall catch fences to increase the effective catch ditch depth and contain falling rocks, construction of temporary wire rope netting to contain rockfalls, and the construction of a deeper rock catchment ditch to allow more storage volume of shot rock following a blast. The resolution for the high cut issues was to install a temporary rockfall fence, increasing allowable daily lane closures during night and off-peak traffic hours, and reconfiguring a temporary exit ramp to move traffic away from the rock face.

Milepost 18.3, Nyack I-87/287 – This project on the Thruway Mainline just west of the Tappan Zee Bridge over the Hudson River involved recutting a 100-foot high slope in diabase to improve setback from the roadway and improve rockfall stability. The slope was part of a through cut made during original construction of the Thruway. Because of the slope height and staged construction, the contract called for erection of the temporary rockfall catch fence shown in Figure 1. The roadway width was kept at three lanes southbound by shifting traffic to the center barrier and establishing a shoulder closure along the rock cut. The rockfall fence worked well in controlling the size and amount of rock reaching the roadway. Several extended stops were required to address high hanging rock immediately following a blast (Figure 3). However, most of the blasting process went relatively smoothly, and the temporary rockfall catch fence allowed the work to be completed with minimal traffic impacts. Construction issues included deep overburden at the crest of the slope, which could not be graded at 2H:1V within the Thruway Right-Of-Way (see Smerekanicz and Daly, HGS Proceedings 2001). Negotiations with abutters and other stakeholders allowed installation of a soil nail wall with steeply raked top nails without additional Right-Of-Way acquisition. Blast vibration monitoring was complicated by blasting at a nearby quarry, and the primary vibration complaint occurred due to a large quarry blast on a day when there was no site blasting.

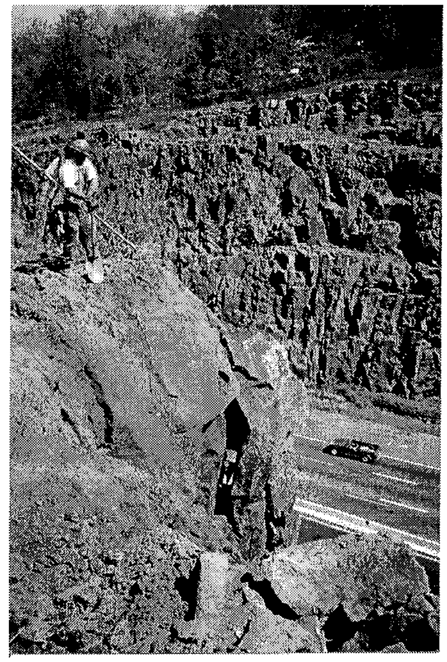
Figure 3 – High Hanging Rock

Interchange 8 Reconfiguration I-87/I-287 – The Interchange 8 Project at over $200M represents a large complex project with 6 bridge replacements, two new bridges, 15 rock slopes, and 17 retaining walls up to 50 feet in height (Figure 4). The project was designed by the Thruway Authority and NYSDOT to reconfigure Interchange 8 to direct most of the traffic from the Tappan Zee Bridge (I-87) to the Cross Westchester Expressway (I-287). When the Thruway was originally constructed, The Cross Westchester Expressway did not exist. When built in the 1950's it provided a connector to I-95 to the east, as well as several other highways and parkways south to New York City. Currently over 80 percent of the traffic crossing the Tappan Zee Bridge exits the Thruway mainline (I-87) via Exit 8 to the Cross Westchester Expressway


Figure 4 – Interchange 8 Blast

over a two-lane exit ramp. Other aspects of the interchange required improvements, including providing collector-distributors to reduce traffic weaving between ramps on the interchange.

The project complexity and location in the highest traffic area (average daily traffic >134,000), dense urban nature of the construction site and tight construction schedule to accommodate staged traffic shifts for bridge replacements, have required extraordinary measures to provide safety, MPT and maintain stakeholder relations. Close communication with Thruway Traffic Safety, State Troopers dedicated to the project, Blasting Inspectors and the Contractor are required to complete blasts in an orderly manner, re-establish traffic flow and maintain blasting production rates to maintain the project schedule.

Early blasting on the project identified several areas where modification of MPT plans and contractor procedures were needed. The first blast at Rock Cut 9 (Figure 5) illustrated the fact that the rock cut was too close to the ramp to avoid having blasted rock fall on the ramp, and the ramp was too narrow for equipment to be able to work rapidly to remove the shot rock. Temporary concrete barriers with connecting pins left in place further complicated cleanup when they were displaced by the blast. The barriers remained connected


Figure 5 – Ramp Closed By Blast – Note Connected Barriers

and several had to be demolished in place in order to remove the blasted rock. This caused a considerable delay in the clean up.

The rock in the roadway shown in Figure 6, caused a two-hour delay and produced a 15-mile backup of traffic on I-87.

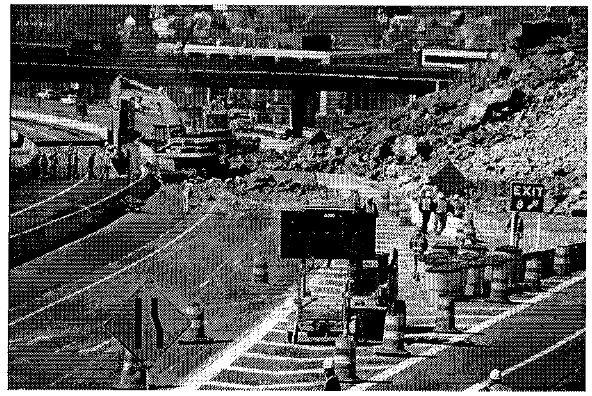

Figure 6 – Rock in Road – Note Unused Left Lane

Subsequent to this blast, Traffic Safety and Project personnel reviewed all phases of blasting procedures. As a result of this review, travel lanes were shifted to move traffic as far away from rock slope excavations as possible, and ramp closures were used to give the contractor more time to clear rock on the ramp, thus allowing the contractor to concentrate on opening the mainline traffic lanes.

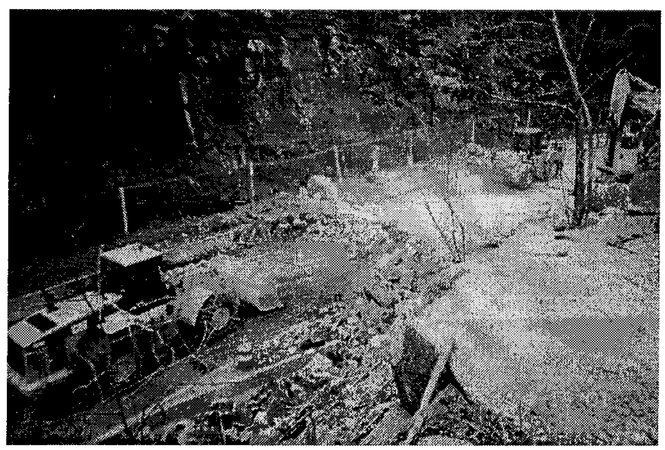

Figure 7 – 980 Loaders Clearing Rock at Exit 8 Ramp

Several other procedures were adopted early in the project to minimize impacts to traffic from blasting operations. The contractor reviewed the effectiveness of the equipment used to remove rock from the roadways and elected to have at least one large loader located at either end of each blast (Figure 7). To avoid problems associated with concrete barriers, barriers in front of a shot were pulled and replaced with plastic barrels prior to each shot. Where possible, blast delay patterns were changed to cast rock in a direction away from the roadway. In order to allow the rock to move more freely (and thus fragment better) the muckpiles from the previous blasts were moved or shaped to control the thrown rock. Through these modifications, the average traffic stop for blasting on the project was 10 minutes.

CONCLUSIONS

Rock excavations involving blasting in high volume traffic areas require extensive planning during design as well as strong communication between the owner, contractor and stakeholders during construction to avoid unacceptable traffic delays. Traffic Safety and design personnel must plan to give the contractor sufficient room to blast and for the shot rock to move into, in the form of long term or daily lane closures. Measures must be put in place to provide a catchment zone for blasted rock, and in the case of high rock slopes over 80 feet, temporary rockfall catch fences should be considered. In confined areas such as narrow on- or two-lane ramps with tall adjacent slopes, ramp detours and extended daily ramp closures should be considered. Practical aspects of rapid clearing of shot rock from roadways used on these projects include sizing equipment for the task of rapid cleanup, ensuring enough equipment is on hand for each shot, and removing pins from temporary barriers in front of a shot, or the barriers themselves, prior to shots that are likely to put rock on the roadway.

To allow blasting production rates to meet rigorous schedules in high traffic urban settings, attention must be paid to stakeholders, MPT, constructability issues and impacts on adjacent facilities. With proper planning and construction practices, the effects of blasting for rock excavations can be minimized by understanding how the blasted rock will behave, when traffic volumes will allow blasting with the least impact to traffic flow, and how to control and remove blasted rock.

Acknowledgements

The authors would like to acknowledge their colleagues at the New York State Thruway Authority, New York State DOT, and Golder Associates, who worked on design and construction of the projects discussed in this paper. Special appreciation is extended to Don Klugo, Jim Cawley, Art O'Donnel, Tina Troici and Dianne Denniston of Thruway Design, Mike Loftus in Maintenance, New York State Thruway District Construction Engineers Charles Johnson, Manuel Gallego and Nick Cavaluzzi, and New York District Director Ramesh Mehta.

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HIGHWAY CUT SLOPES IN ROCK: SPECIALIZED EXCAVATION AND ENHANCEMENT TECHNIQUES

by

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for presentation at the

53rd Highway Geology Symposium, San Luis Obispo, CA

August, 2002

INTRODUCTION

Highway development projects crossing lands with special scenic or recreational characteristics may be required to meet special aesthetic criteria. Generally, the entity imposing the aesthetic criteria is external to the core roadway construction team. Usually, agencies imposing these criteria are land administration entities, external to highway departments, whose primary role is the preservation of historical or recreational opportunities, such as the Bureau of Land Management, the National Park Service, the United States Forest Service, or local governmental agencies with similar responsibilities. However, agencies imposing aesthetic criteria may also be divisions of highway development agencies themselves. For example, in Arizona, the Roadside Development section within the Arizona Department of Transportation has defined aesthetic needs on numerous designated scenic highways.

From the perspective of the highway designer and constructor, the imposition of unfamiliar and sometimes vague design and construction criteria, by external entities whose approach differs greatly from that of engineers, can be problematic. The aim of this paper is to describe how geology, construction engineering, and visual analysis have been used to alleviate and streamline the process of aesthetic attainment.

"AESTHETICALLY PLEASING" CUT SLOPES

Developing Rational Criteria

For rock cut slopes, aesthetic criteria have often been expressed in contract documents and environmental assessments using language such as: "reflect the form, line, color, and texture of

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natural formations", natural-appearing cut slopes", "roughened cut slopes", and "mimic natural land forms. Although this may be meaningful to landscape architects and others who have an intuitive understanding of the desired finished product, road building contractors and engineering designers find such language imprecise and confusing. Others within the highway engineering community may see attempts to "naturalize" the appearance of rock cut slopes as opening the door to slope instability and liability exposure. These individuals prefer slopes excavated using techniques that minimize rock disturbance and produce a planar, uniform, and engineered appearance.

Even though "natural appearing" cut slopes do not preserve rock mass integrity and construction economy to the same degree as rigorously controlled cut slopes, there are aesthetic enhancement techniques that can acceptably balance the visual characteristics with rock slope stability and economy. The key is to develop a working partnership between the highway development and recreational land management agencies *to make the aesthetic criteria deterministic, rational, biddable, and constructible*. The design team must define who exactly is to be served by the improved aesthetics, what their visual perspective is, and how much enhancement is appropriate.

Characteristics of Aesthetic Enhancements

In nature, stable, natural land forms almost always comprise flatter slopes than highway departments wish for their rock cuts. Where the natural landscape is sufficiently rugged that it incorporates cliff faces and natural rock slopes approximating the desired cut slope angles, the condition of the natural rock slopes is generally much more degraded than is desired in the highway cuts. For these reasons, cut slopes cannot be made to exactly look "natural", even if that is what is required in construction bid documents.

To the construction contractor bidding on a rock excavation project, this means that, at the very least, he will not have to make the cut slopes truly "natural". More problematically, a bidder may also believe that since the required "natural" appearance is not strictly attainable, under the pressures of actual construction the door will be open to negotiate a far lesser degree of compliance. Often, to the dismay of many (including the other, unsuccessful bidders whose estimates reflect a nobler intent), he is correct. To avoid this, the land use agency and the highway engineering team must work together to determine what measurable physical characteristics will be used in construction as acceptance criteria; the land use agency must commit to accepting the standard of aesthetic attainment implied by those criteria as expressed in the bid documents; the highway department must commit to inspection, measurement, and enforcement of aesthetic criteria as stridently as for more traditional elements of inspection; and a process needs to be developed to review, and if necessary, attain consensus on modifications *if and only if* geologic conditions differ materially from those assumed during design.

It is important to conduct *visual prioritization* to identify aesthetic enhancements that will actually be of benefit. This is normally carried out by a landscape specialist working as part of the highway department's design team, in partnership with the land management agency. Through the visual prioritization process, the team recognizes the various levels of visual impact and agrees to eliminate from consideration those visual impacts that are not significant. In general, the most significant visual impacts are those that will be apparent for longer than 10 seconds or so.

The Short Range Perspective

Only in the short range view are the texture and fine features of the cut slope important. Serving the short range view means incorporating *textural enhancements* at the rock fabric scale, such as ledges, slope roughening, planting pockets, and boulder salvage. They should only be employed where the visual prioritization shows that their advantages outweigh their disadvantages.

The duration of view, and the perspective of the viewer, should be kept in mind. It has been widely observed by construction contractors, and not without justification, that the driver passing by a road cut at 65 mph is not as likely to appreciate intricate enhancements to rock cut texture as is the construction inspector standing at center line viewing the freshly excavated cut. A driver is much more likely to be affected by the appearance of the road cut ends, which are visible for a much longer period of time.

Slopes incorporating ledges cannot be as steep as slopes without them. Therefore, textural enhancements commonly imply flatter slopes than the maximum that may be justified based on geotechnical criteria alone. This affects economy, safety, and slope performance.

Figure 1 *Sculpted slope in granite with ideal fracture spacings.*

Textural enhancements are most appropriate in masses of hard rock with moderately spaced (1 to 3 ft) fractures (Figure 1). In poorly fractured, massive rocks with fracture spacings more than 10 ft, there are few opportunities for ledging and pocketing along natural joints, so the contractor will have to resort to carving artificial ledges and pockets into the rock. This can be effective visually (Figure 2) or, if not done with focus on the geology and topography, it can lead to exactly the artificiality that the treatment is intended to remedy. In such cases a smooth surface would be preferable. In heavily fractured or strongly weathered, soft rock, ledges can be formed with equipment, but may not be stable or will develop a rounded or humped appearance. In those cases the ledge dimensions should be reduced except where the slope can be flattened enough to exaggerate the features.

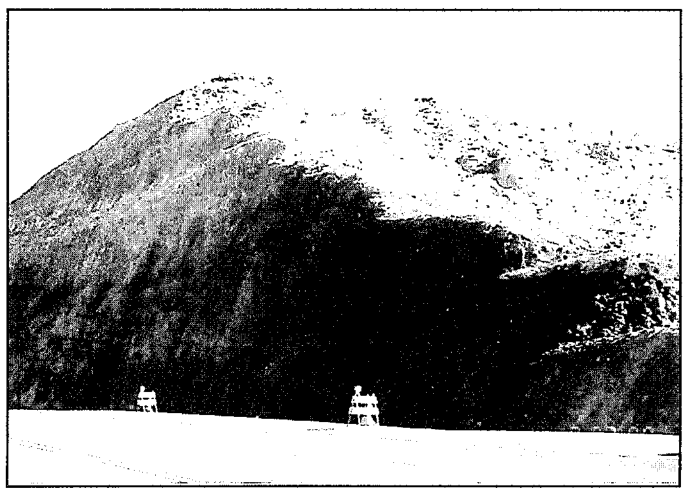

Figure 2 *This cut slope in massive, unfractured conglomerate was smooth blasted. Treatment consisted of only drill trace removal on the main slope, but ledges were cut in natural boulder lenses at the cut end where similar features are in the adjoining natural terrain.*

Unless clearly defined stratification, faults, or other geologic features defining ledge locations can be accurately projected to the finished slope location, the textural enhancements should be shown on the construction drawings as a general range of ledge widths, ledge areas, or pocket quantities. The highway construction plans may show conceptual details, sketches, or photographs of similar features, as guidance to the contractor, and should define the quantity of ledges expected between roadway stations. (As one might expect, if planting pockets or ledges are shown as pay items, contractors are much more enthusiastic about developing them.) During construction, the specific locations and extents of ledges are chosen by the construction contractor, approved by the Resident Engineer, and checked by the design geotechnical engineer.

The ability to develop ledges and roughened slopes implies that some degree of rock mass disturbance and overbreak must occur during initial excavation. Unless the excavation process is controlled, the disturbance and overbreak can lead to long-term slope deterioration. The objective should be to remove rock selectively, reducing the overbreak so that only the closest fractures to the nominal slope (in blasting, this is the last row of drill holes) is affected (Figure 3). What should be avoided is to employ more general (and cheaper) production blasting techniques along the desired slope line, and then to form ledges by excavating selectively with in the general zone of production blasting disturbance. Instead, specialized controlled blasting techniques and machine excavation should be employed, along with a program of continuous review and improvement of the blasting process and the finished slopes.

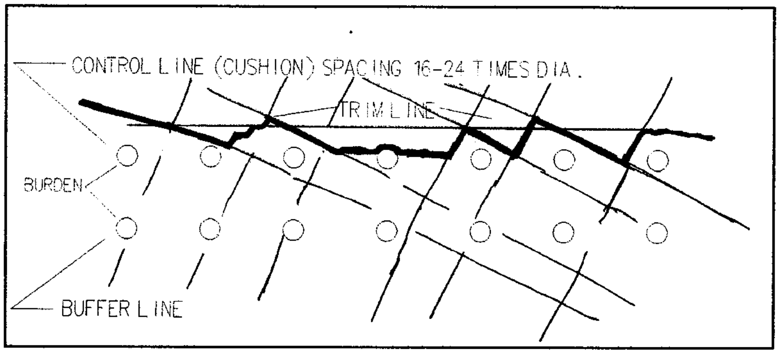

Figure 3 *Rock breakage concept with controlled blasting.*

Roughened slopes with ledges and pocketing are more likely to project rock fall beyond the ditch and onto the roadway. This is because the ledges represent launching surfaces, and the slope flattening necessary to create ledges tends to promote the horizontal rock deflection component. Furthermore, when textural enhancements are employed, it is generally on projects where rock fall control elements like catch benches and rock fences are out of the question. It is very important to thoroughly scale rock slopes that incorporate ledges for planting pockets, and to aggressively round the slope crests.

One way to limit traffic exposure to rock fall, while still providing effective aesthetic enhancements, is to specify a graduated schedule of desired ledge widths indexed to station, cut height, and slope angle. The idea is that ledges can be wider, and slopes can be flatter, where rock cuts are lower. To reduce rock launching and rock fall, ledges should be narrower or absent in the higher portions of rock cuts. This approach provides effective visual enhancement, because the oncoming driver holds the view of the cut end longer than he does the middle. Providing additional ledge width at the end of a cut presents opportunities for revegetation, and also helps warp the cut slope smoothly into the natural terrain. The slope layback associated with additional ledge width at a cut end is not as costly, because the portion of the slope affected by the layback is not as tall. Overall, this presents a good combination of economy, visual enhancement, and safety.

Often, the textural enhancement criteria specify that evidence of the construction technique be eliminated or prevented. Evidence of construction usually means blast hole traces and the marks left by construction machinery. In order for blast hole traces to be omitted from the final cut slope, it is necessary that the rock containing the blast hole traces be removed along with the rest of the production rock. To allow this, overbreak from blasting must extend behind the last row of blast holes, and/or the angle of the finished slope must be steeper than the last row of blast holes. In massive, sparsely fractured rock, the blast hole spacing may be close to or less than the fracture spacing, in which case blast hole traces are almost unavoidable. In these cases it may be necessary to remove portions of blast hole traces by chipping with a hoe ram, excavator bucket, or pneumatic hammer.

The most common sources of undesirable machine scars are corner bits on bulldozer blades, teeth on excavator or loader buckets, and ripper shanks. These marks are fairly easy to avoid except in soft/massive rock. Where the rock is soft and massive, it may be necessary to remove the machine scars by rubbing with a plate bucket attached to an excavator or loader, or (preferably) using a high-pressure water spray.

The visual significance of blast hole traces and machine scars must be considered in terms of the degree of slope roughness attained. Often, the importance of blast hole traces is overstated; not all blast hole traces are deleterious. Blast hole traces that may be evident when the cut is viewed from a stationary position out in the roadway, may disappear from the perspective of the oncoming driver who sees the same rock cut slope in profile (Figure 4). The bid documents should specify a level of blast hole trace reduction that is in accordance with the visual prioritization, taking into account the overall slope roughness, the rock mass fracture spacing and blockiness, and the perspective of the viewer. In most cases blast hole traces need not be completely eliminated or removed. Requiring complete elimination of blast hole traces can mean rock mass disturbance that is not desirable for safety, economy, or slope performance.

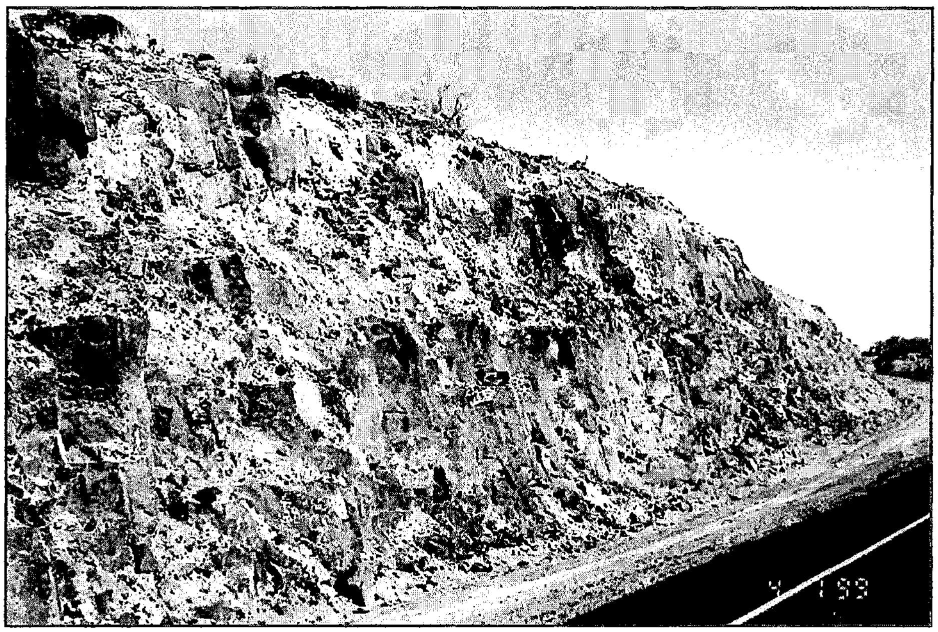

Figure 4 *This slope in basalt was blasted using techniques that reduced rock mass disturbance but left behind some drill traces. However the drill traces are not apparent from the driver's perspective.*

The Long Range Perspective

Probably the most effective method of making rock cuts fit into the natural terrain is to incorporate slope variations and grading features that cater to the long-range perspective. These techniques include major slope warping; expanded slope rounding; laybacks at intercepted drainages; ditch width transition variations; varying the slope angle and revealing important geologic features having topographic expression (such as erosion-resistant dikes or sills, or, in sedimentary terrain, ledge-and-slope topography); false cut embankments and median berms; and the application of rock stain.

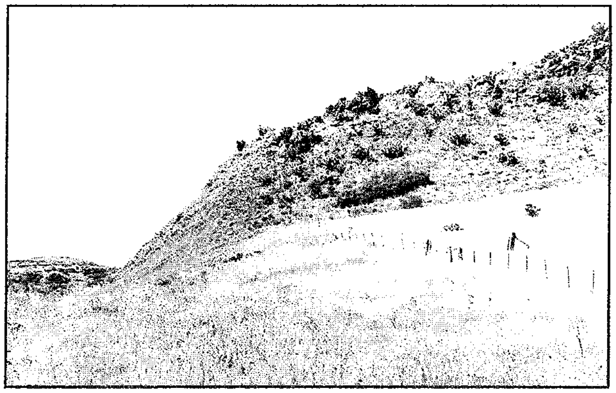

Figure 5 *Slope warping on I-17 in Arizona*

Major slope warping (Figure 5) consists of varying the slope angle at the cut ends to provide a smoother transition to the natural terrain. The warping can be defined through a slope offset and angle table, contour grading plans, or by providing an equation relating cut slope height, distance, and slope angle. Most contractors are familiar with the staking techniques involved, which are standard for golf courses, industrial parks, and landscaped open space.

Expanded slope rounding (Figure 6) is mentioned here because highway standard specifications often do not require slope rounding in rock, or because the standard rounding radii tend to be small, if the colluvial cover is thin. By expanding the rounding of the slope crests, a smoother, more natural transition to the natural terrain is obtained. It is fairly readily done where colluvial cover is thin by providing a row of "satellite" blast holes behind the trim line proper. The rounding zone does not present rock launching features and is an opportunity for ledges and revegetation.

Figure 6 *Note how rounding extends behind the nominal catch point, giving a smoother, natural transition*

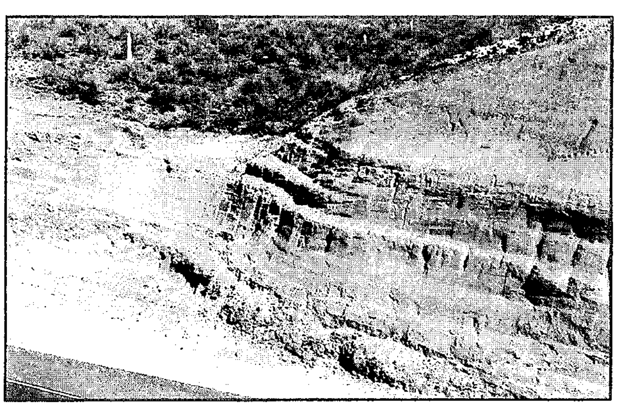

Figure 7 *Drainage layback (with ledging) in lapilli tuff*

A very suitable land form replication technique in cut slopes is the creation of drainage intercept laybacks (Figure 7). On major roadway projects, rock cuts often pass through a series of ridges, without daylighting except at the ends. The topographic lows intercepted by the cut may be considered candidates for drainage intercept laybacks, even if no stream bed or stream sediment exists. Providing drainage intercept laybacks is an extremely effective technique visually, because it breaks up the uniformity produced by the cut slope design template, and because it recognizes natural land form processes.

Ditch width transitions present an opportunity for slope variation at locations where the required cut ditch changes. Rock fall retention requirements may cause cut ditch requirements to be the greatest in the center and the least at the ends (Figure 8). This can impart a concave appearance that is unlike the convex features normally found in nature. Consequently, it may be effective to vary the cut ditch at the ends, to mitigate this effect.

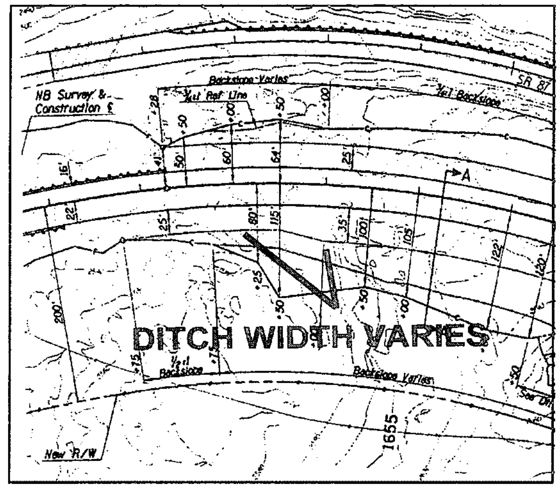

Figure 8 *Ditch width transition detail on Arizona State Route 87*

Slope angle variation is used in areas of long, monotonous cuts, at geologically significant locations, or drainage intercepts. Slope angle variations can be most conveniently shown in the plans through a slope exception table, or a tabulation of station and offset to the slope toe and catch point. On the State Route 87 improvements in Arizona (Figure 9), slope angles were varied on a cut more than 100 ft deep in a variably weathered granite. This gave expression to prominent knolls that were formed along intrusive bodies and that presented contrasts in rock mass competence. Special blasting and staking techniques were used to accentuate resistant intrusive dikes. The effect was to remove the sense of confinement within the rock cut, and to create a cut slope that effectively mimics the surrounding, rugged terrain. This project won the National Environmental Excellence Award in 1999.

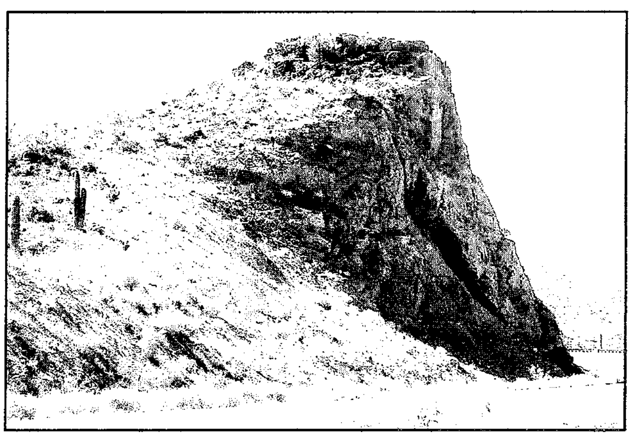

Figure 9 *Slope angles were varied from 0.5:1 to 1.5:1 to highlight the prominent knolls on SR 87 in Arizona.*

Along divided highways, leaving false cut embankments (Figure 10) or creating median berms (Figure 11) can screen the oncoming lanes and provide variety, as well as opportunities for revegetation and landscaping. It is important that adequate shoulder runoff and recovery width be provided, and that the features be designed and constructed such that they do not represent launching ramps for errant vehicles. Done properly, they can reduce guardrail consumption and earthwork quantities. False cut embankments should not be left with a smooth, planar, or excessively regular surface.

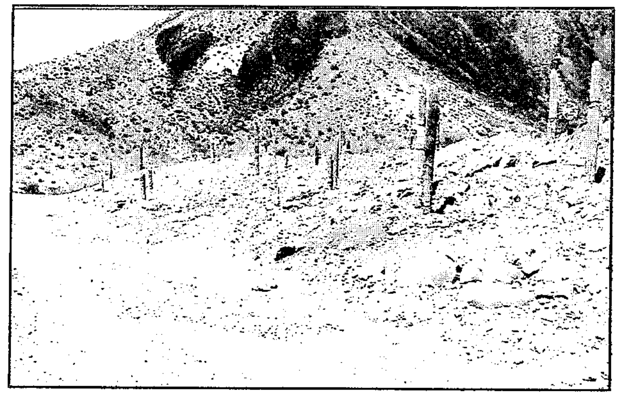

Figure 10 *False cut embankment treated with salvaged boulders and vegetation.*

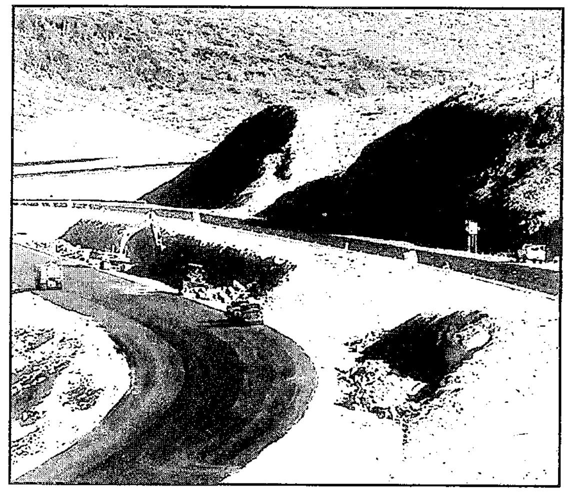

Figure 11 *Median berms on US 93 were finished with natural textures and revegetated*

Larger scale enhancements are not only effective visually, in that they represent topographic variations that occur in the natural terrain, but they also can be shown on project grading plans, so the economics are improved. The quantities of excavation can be captured within the overall roadway excavation pay item; consequently, the construction of these features stems from the competitive bidding process.

Rock staining is a significant visual enhancement for the long-range perspective. Especially in arid areas, chemical weathering of natural rock outcrops results in a dark coating, locally known as "desert varnish", which takes thousands of years to develop naturally. Commercial desert varnish stain products can mimic the darkened surface in weeks. Some formulations leach natural minerals, others rely chiefly on their photoreactivity, and some are analogous to ordinary pigmented stain. Rock stain is applied by spray (Figure 12, next page). Compare the fresh rock tone of Figure 12 with that of Figure 1, which is the same material after staining.

There are certain things to keep in mind when considering desert varnish stain. First, not every stain will produce a color that is compatible with the surrounding landscape, and some stains are not effective on certain types of rocks, so it is critically important to prepare test panels for approval before full scale implementation. Some stains tend to lighten over time, so the test panels should be performed early. The intensity and tone of the finished product will vary with stain concentration and formulation. Multiple applications may be necessary. Also, it is important to fully scale and wash the rock surface before staining, to remove any masking by loose material. The permanence of the treatment should be established. Weak rock, and rock that is absorbent, generally are not good candidates for stain; results may be acceptable initially, but the degradation of the stain surface will gradually become apparent. Finally, be sure the long range contrast between the fresh excavation surface and the natural surface is actually due rock weathering characteristics. If not, the stained slopes will not be entirely natural looking.

Figure 12 *Application of desert varnish stain. This rock darkened within 2 weeks after application.*

CONTROLLED BLASTING FOR AESTHETIC ENHANCEMENT

A full discussion of blasting techniques is beyond the scope of this paper so only selected concepts will be introduced here. Options range from ordinary production blasting techniques, where the slope configuration is entirely defined by over break, to controlled "smooth blasting" techniques, where the slope configuration is entirely defined by the position of the drill holes. "Controlled" means that the charge, hole location, and hole orientation are carefully controlled to produce a specific shear plane or rock surface after blasting. Smooth blasting techniques include presplit blasting and trim blasting. Roughened slopes can be produced using "cushion" (buffer) blasting (a variation of trim blasting), horizontal hole blasting, or, for flatter slopes, step drilling. These last 3 techniques can be modified to qualify as "controlled" blasting. Ordinary production blasting can be used to form roughened highway cut slopes but the results will be unsatisfactory as to slope stability, rock fall, and safety.

One technique commonly known as "cushion" blasting strikes a balance between slope roughening and minimized backslope disturbance. Various cushion blasting approaches can be used, differing according to the cushion hole spacing, the buffer standoff distance, loading density, and detonation timing. The best approach is to use buffer holes to control the degree of burden experienced by the cushion blast holes (Figure 13). The cushion blast holes are essentially trim blast holes, but set at wider spacings with slightly higher charge densities. It is essential to conduct a test blasting program, and to continuously assess the relationship between the pattern used and the results obtained.

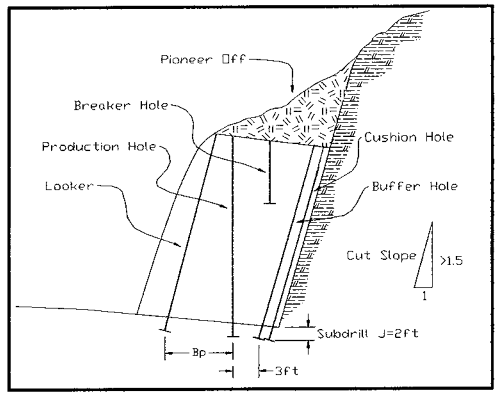

Figure 13 *Cushion blasting arrangement used in a narrow highway cut.*

Flatter slopes can be formed with step drilling (Figure 14). Because step drilling relies on back break between holes to form the finished slope, it is not technically a "controlled" blasting approach. However, where cuts must be developed at slopes ranging between 0.7:1 and 1:1 (H:V), the use of sloped controlled blasting holes is difficult and usually unwarranted. Therefore, on certain projects a pay item has been established to tighten the step drilling pattern and use lighter, distributed charges. On several Arizona projects, in blocky volcanic rock, good results with minimal overbreak have been obtained using this approach, and it has proven popular with contractors who favor the vertical drilling setup.

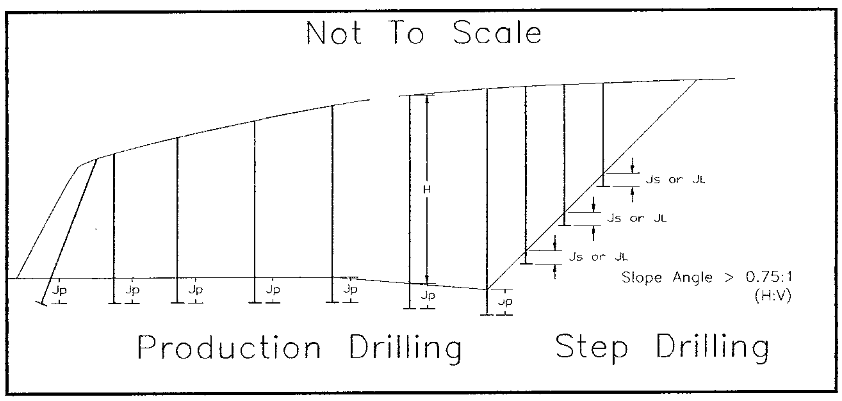

Figure 14 *Cross Section of general step drilling approach.*

In massive rock where drill traces absolutely cannot be tolerated, or in sliver cuts with poor access to the crest for drilling, horizontal hole drilling may be worthwhile. Horizontal drilling is demanding for the driller, because hole orientation, location, and depth accuracy are more critical than for any other method. In particular, depth control is critical to prevent bootlegs, and special drilling equipment is needed that is capable of the vertical reaches required. Explosives loading into horizontal holes is more complicated, requiring either packaged product or pneumatic loading of bulk product, and special stemming procedures to prevent ejection and rifling.

CONCLUSIONS

Aesthetic attainment on highway projects must rationally evaluate what is necessary visually and what is not. A few drill hole traces are generally not deleterious visually and evidence improved slope stability and reduced rock mass disturbance. In cooperation with the land administration agency, the highway department should use visual simulations and visual prioritization to arrive at measurable, objective criteria for slope roughness and drill hole trace retention.

Criteria for visual enhancement fall into the short range and long range perspectives. The short range perspective includes textural enhancements that are important if the viewer is nearby and will hold the view for an extended period. Consideration should be given to accentuating ledges and pockets on the ends of cuts to reduce the potential for rock launching and obtain more visual benefit. The long range perspective is generally viewed over a longer travel time. The long range perspective is best served through enhancements that mimic natural landforms such as desert varnish stain, slope rounding, slope warping, and slope laybacks at drainages. The excavation can be shown on the plans and included in the earthwork estimates so it is accomplished at a competitive unit rate.

Several blasting approaches are available for the creation of enhanced cut slopes in rock. The choice will depend on rock fracture density, rock hardness, and designed slope angle, as well as aesthetic criteria. Controlled "cushion" blasting has been effective in creating rugged cut slopes with natural appearing ledges and pockets, but needs careful design and continuous evaluation starting with a test blasting program, to be effective in changing geologic conditions. Special tightened step drilling and horizontal drilling can be used to advantage where slope angle or access require them. Horizontal holes offer the potential for elimination of hole traces in massive rock, as long as the depths of the holes can be adequately controlled.

DESIGN OPTIONS FOR ROAD WIDENING IN MOUNTAINOUS TERRAIN

Chris Mah, P.Eng., Julia Matsubara, E.I.T., Stephen Barrett, P.Eng.[1]

Abstract: Many roads in the mountainous terrain of British Columbia are located in valleys on steep slopes with rock fall hazard potential, beside environmentally sensitive areas. Because of ever increasing traffic volumes and the need for improved highway safety, many highways in British Columbia require widening in areas constrained heavily by topography, geology and the environment. This paper presents four design options for widening highways in such areas, two of which are highlighted in case histories.

1.0 DESIGN OPTIONS

Figure 1 illustrates the design options discussed in this paper. It should be noted that this is only a sample of available construction solutions, and many more hybrids of the examples shown exist. However, it is hoped that this selection highlights some of the available design options and the associated construction issues that need to be taken into consideration during design.

1.1 Pushing Back an Existing Cut

The design option most commonly employed is pushing back an existing cut at an optimum angle compatible with the geological structure, rock strength and site topography. It is standard practice in British Columbia to design these cuts without any benches, in order to reduce the potential bounce trajectories for rock fall. The cut angle is also typically optimized to reduce the amount of rock support required to stabilize the excavation to an acceptable factor of safety. The acceptable factor of safety varies depending on the potential consequence of failure, but typical is 1.5 for a standard highway cut.

Because of the steep mountainous terrain in British Columbia this method is not always practical to construct, as the optimum cut angle that minimizes the amount of support can result in excessive cut heights. This can result in large volumes of excavated material, which can be difficult to dispose of without significant haulage costs. Another difficulty is the need for wide catch ditches at the base of such high cuts, unless drape nets or rock fall fences are used at the crest to mitigate the rock fall hazard. During construction, the development of an access road up to the initial pioneering bench can also be difficult and can cause a severe rock fall hazard for highway users below, unless temporary rock fall fences are installed between the access road and the highway. Because of the rock fall hazard, the initial few benches are usually very slow to construct, as small controlled blasts are required to keep excavation volumes small.

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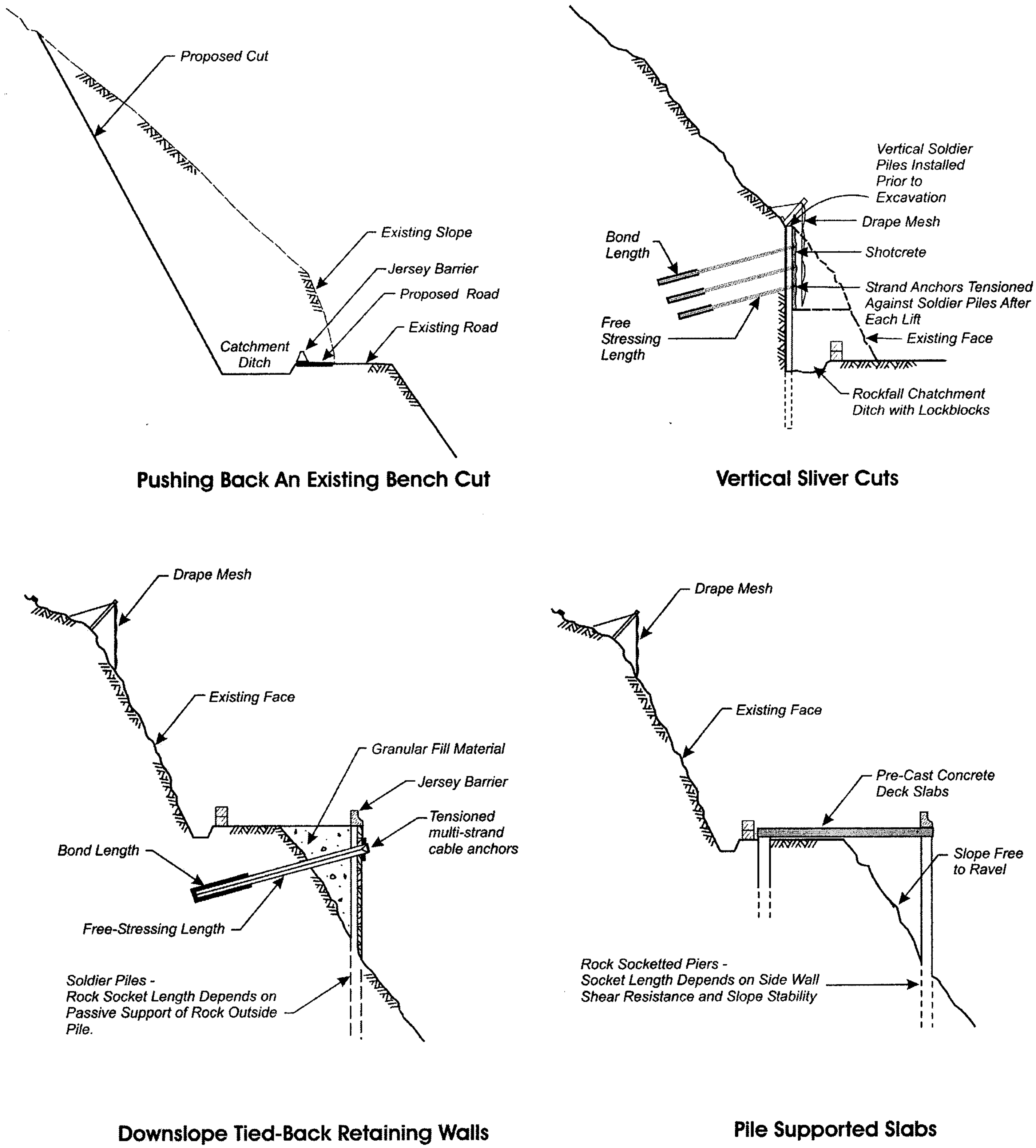

Figure 1: Four Design Options For Highway Widening in Mountainous Terrain

1.2 Vertical Sliver Cuts

Vertical sliver cuts address many of the disadvantages of pushing back existing cuts at flatter angles. However, in order to achieve acceptable factors of safety for the stability of the final cut face, a higher level of rock support is usually required. The example shown in Figure 1 is a typical solution for a sliver cut in poor quality rock or in a fault zone. Here a tied-back retaining wall is used to support the slope as excavation proceeds. Usually these walls consist of steel I beams installed in pre-drilled holes, which are tied back with successive rows of anchors after each lift is taken. If access is available to construct a pioneering bench then traditional drilling equipment can be used to drill the holes for the I beams. Alternatively, down the hole hammer (DTH) drills supported off cranes can be used. However, the rock has to be sufficiently stable to allow the hole to remain open during the drilling process or the drill steel can be lost due to hole collapse behind the bit. If hole collapse is a concern, then drill access to the crest of the cut is required, as temporary casing will be required to keep the hole open.

Temporary support of the rock between the I beams is usually provided by timber lagging, with a permanent reinforced concrete facing added upon completion of the excavation. As with all retaining structures, the design of adequate drainage behind the wall is important. In cases where a retaining wall is required, excavation is usually by mechanical means in order to avoid damage to the structure. Rock fall protection in the form of drape nets at the top of the wall is usually also required, with a provision for a small rock fall catch ditch at the toe. The effective depth of the ditch can be increased by the placement of a Jersey barrier or Lockblocks adjacent to the ditch.

When designing vertical sliver cuts, we have used the following two design techniques in the past to determine the length of the tie back anchors and the required anchoring force for the wall:

1. Conventional two dimensional slope stability analyses techniques using programs such as SLIDE[2] to calculate the magnitude of the external pressure required to increase the factor of safety to an acceptable level.

2. Modeling the existing toe of the slope as a "buttress", to determine an equivalent support force. The required support force is then calculated by multiplying the equivalent support force by an appropriate factor of safety. This technique is useful for large slopes, where the designer would alternatively have to artificially limit the height of critical slip surface, in order to generate reasonable analytical results.

1.3 Downslope Tied-back Retaining Walls

In situations where a highway crosses an area of poor quality rock, such as an ancient landslide or an active rock fall area, developing highway widening solutions that do not cut into the slope or involve people working on the slope above the highway become attractive. One such solution is to construct a

[2] SLIDE is a commercial 2D limit equilibrium slope stability program available from ROCSCIENCE INC. Demonstration programs and ordering information can be downloaded from their website at www.rocscience.com

retaining wall in areas where sliver fills can not be constructed downslope of the existing highway. While such a structure will add a surcharge to the slope, the overall impact on the slope's global stability is usually less than pushing the existing cut back. Downslope retaining walls will also usually require less support than vertical sliver cuts in such areas.

As with vertical sliver cuts in poor quality rock, the wall can be constructed using I beams installed in predrilled holes downslope of the highway. If it is difficult to develop temporary construction access down to these locations, it again may be possible to drill the holes remotely from the highway using a DTH Hammer supported from a crane. Another solution is to construct the wall from one end, with the previously constructed section used as a working platform to construct the next section. In the example shown in Figure 1, tie-back anchors have been used to provide lateral support for the wall. The need for additional rock fall protection will depend on whether the widening will allow a sufficiently wide ditch to be constructed at the toe of the existing cut.

Stability analysis of this wall system should determine if the vertical soldier piles have sufficient lateral load carrying capacity to support the applied shear and bending moments. This will involve an analysis of the lateral load carrying capacity of the rock socket, with longer sockets required if there is a potential for the rock to weather or if unfavourable oriented joint sets are present at the site.

1.4 Pile Supported Slabs

A second downslope design option is to construct a pile supported slab out over the slope. This design solution works well in areas where the additional surcharge provided by a retained fill would destabilize the slope or produce an unacceptable factor of safety. Here rock socketed piers would be used to support the structure, with the structure itself constructed from prefabricated reinforced concrete or steel elements. When designing the rock sockets, it is recommended that they be designed in sidewall shear only, as it is usually difficult to ensure the base is clean due to the remoteness of the socket sites and the need to provide a sufficient depth of cover to prevent future undermining of the foundation. It is also important to check that the socket has enough lateral confinement to mobilize the design shear strength along the socket walls.

2.0 CONSTRUCTION ISSUES THAT SHOULD BE CONSIDERED IN DESIGN

2.1 Blasting

When blasting is required to excavate the rock, controlled blasting techniques such as pre-splitting or cushion blasting should be used to limit the blast damage behind the final face. All blast holes should be drilled approximately parallel to the final face, with holes either drilled horizontally from the ends of the cut or drilled vertically down from the working bench. In order to control fly rock, blasting mats should be used and the holes appropriately delayed. If the duration of highway closures is an issue or there are environmentally sensitive areas at the site such as fish bearing rivers or lakes, the detonation sequence should be designed so that the burden is ejected longitudinally along the cut, rather than out towards the highway and/or water body. When specifying highway closures for blasting, in addition to the time required to fire and check the blast afterward, sufficient time should also be allowed for removing the blasted rock. If this leads to unacceptable closure times, consideration then needs to be given to creating a sufficient buffer zone at the base of the cut, to contain any rock fall from the mucking operation. This can require construction of temporary rock fall barriers at the toe of the slope.

2.2 Construction Access

When considering design options to widen a highway in steep mountainous terrain, consideration needs to be given to construction access. In general, construction access is usually poor and sometimes is limited to only rope access techniques. In such cases, drilling equipment should be light, hand portable units, although this limits the length and diameter of holes that can be drilled both for blasting and installing rock support.

2.3 Rock Cut Stabilization and Protection

Rock slope stabilization and protection methods can be divided into three categories as shown on Figure 2, namely rock reinforcement, rock removal and rock fall protection (FHWA, 1998). On a typical rock cut construction project, a combination of all methods is usually required, but the degree to which each is required depends on the conditions at each site. For example, in cases where the rock quality is good and the joint orientations favourable, scaling and spot bolting is generally sufficient. However, in cases where the rock quality is poorer or is susceptible to weathering, pattern bolting and shotcreting may be more appropriate. In cases where the rock quality is very poor, rock reinforcement or removal may not be practical and rock fall protection may be the only alternative.

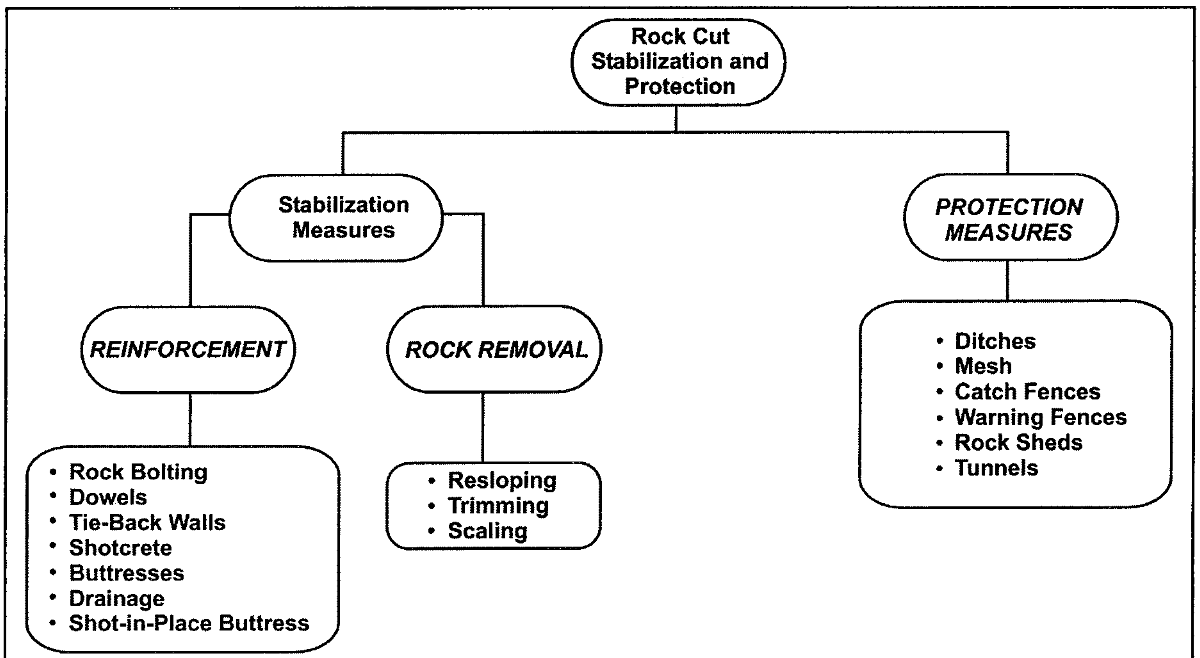

Figure 2: Categories of Rock Slope Stabilization and Protection Methods

2.3.1 Rock Reinforcement

Different types of rock slope reinforcement methods, ranging from tensioned bolts to concrete buttresses, are shown in Figure 3. These techniques limit further relaxation and loosening of the rock mass that may take place as a result of excavation, unloading and weathering of discontinuity surfaces. Once the rock is allowed to relax, there is movement, resulting in a loss of interlock and a decrease in shear strength.

Rock reinforcement techniques are particularly effective when they are used to pre-reinforce a slope prior to the start of excavation or before taking the next lift.

2.3.2 Rock Removal

Figure 4 shows different rock removal methods to stabilize the surface of rock slopes. Typical removal methods include resloping zones of unstable rock, trim blasting of overhangs and scaling. While effective in the short term, rock removal methods tend to be less effective in the long term, due to the ongoing relaxation and weathering of the face. Consequently, if they are relied on as the only means of slope stabilization, either good rock fall catchment is required at the toe of the slope or regular scaling is required to mitigate the rock fall hazard. The exception to this is if the rock quality is good and the joints are favourably oriented, then very little maintenance work will be required in the future.

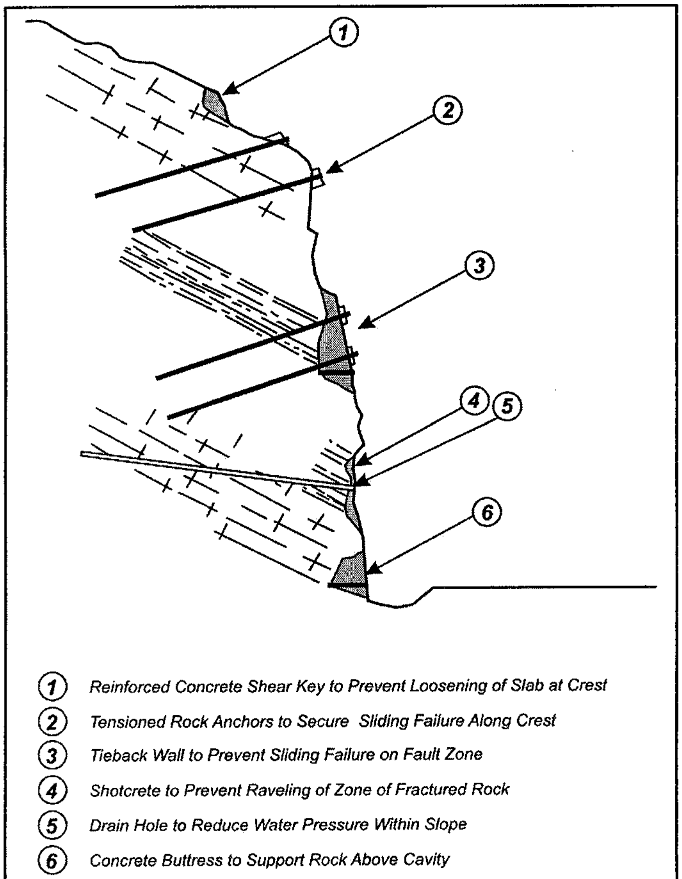

Figure 3: Rock Slope Reinforcement Methods

2.3.3 Rock Fall Protection

Figure 5 illustrates a variety of rock fall protection methods, all of which are gaining increased usage in British Columbia, as effective means to manage rock fall hazards. Three of these systems are briefly discussed below.

Lockblocks Walls – This is a cost effective means of stopping rock falls

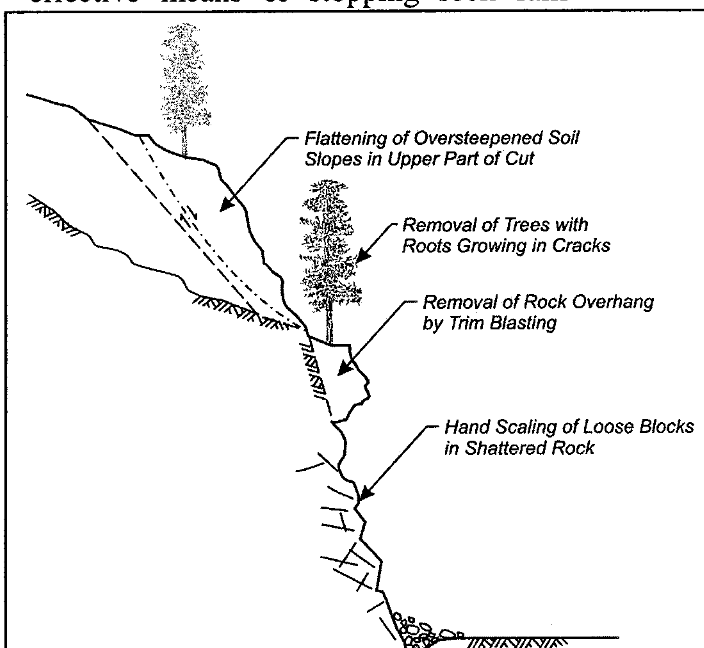

Figure 4: Rock Removal Methods

where there is adequate space at the toe of the slope to construct the wall. The wall can either be used to create a catch ditch or to effectively deepen an existing one. If space is not available, a small toe cut can normally be designed to create it.

Rock Fall Fences – Rock fall fences are usually designed to stop or slow rocks falling past a specific point. High energy fences are composed of non-rigid nets that are able to withstand the impact of a rock fall by absorbing the impact energy. Fences are now commercially available that can withstand impact energies of up to 3000 kJ, for a maximum fence height of 5 m (Geobrugg, 2002).

Drape Nets – Drape nets consist of wire mesh hung over the face of a rock slope in order to limit the rock fall trajectory. The drape net is not attached elsewhere along the slope, to allow the falling rock to work its way down the slope in a controlled manner. The type of mesh chosen for the net is normally based on the expected rock fall dimensions and energies. Ideally, the drape net should be installed as close to the source of the rock fall as possible, so the impact energy is minimized.

3.0 CASE STUDIES

To highlight some of the design and construction issues discussed in this paper, two case studies are presented below. These are:

1. the Pioneer Road 40 Road Realignment at Carpenter Lake, BC; and

2. the upgrading of Hwy 12 in the vicinity of the Texas Creek Slide.

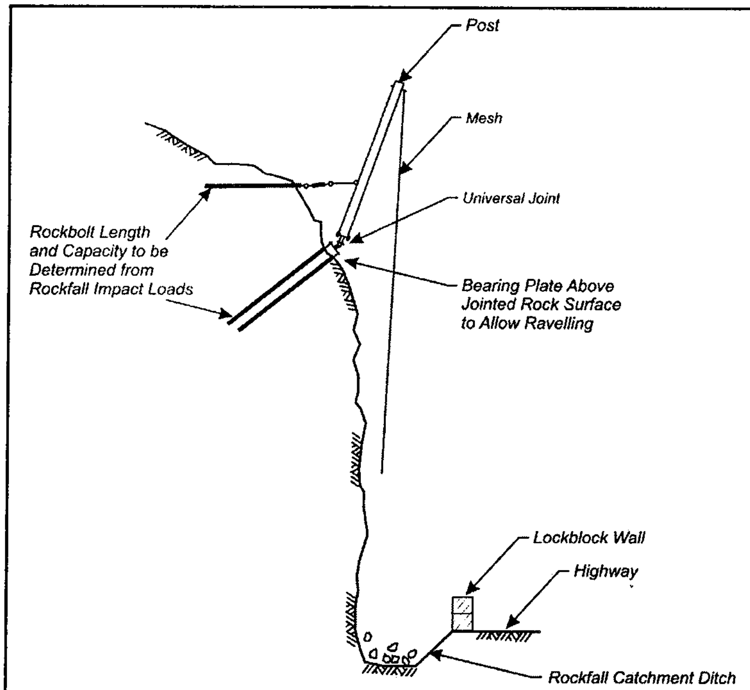

Figure 5: Rock Fall Protection Methods

3.1 Pioneer Road 40 Upgrade

Pioneer Road 40 is located west of Lillooet, B.C. and provides road access to the communities of Bralorne and Goldbridge. The road is located between near vertical rock slopes up to 200 m high and a water storage reservoir (Carpenter Lake) retained by the Terzaghi Dam, an earthfill structure constructed in the late 1950's. There have been several fatalities on this section of the road, attributed to its poor alignment, narrow width, poor sight lines and rock falls.

Golder was asked to review the Ministry of Transporation's (MoT) original design concepts and to suggest alternatives. Golder's recommended conceptual design involved small sliver cuts along the existing inside rock toe, and lowering the road grade to provide a wider road corridor. The revised alignment included the complete removal of a 10 m high "hump" and blind corner, and widening to a full two lane and a full drainage ditch along a 0.5 km segment of the road.

Blasting restrictions were negotiated with BC Hydro, the owner of the adjoining dam facility to facilitate the work with minimal impact on the adjoining dam and reservoir. Key provisions of blasting included containing blast-generated debris so that a minor amount of debris entered the reservoir. Debris considered acceptable included occasional individual fragments of up to 1 m^3 in size, and masses of small debris no greater than 5 m^3 in volume. Blast vibrations were limited to a maximum of 50 mm/sec at the dam's key structures, including the spillway and the splashwall at the dam crest. Blasting was carried out up to a minimum distance of 45 m from the splashwall at the north abutment of the dam. In addition to the blasting constraints, MoT had to open the road three times a day at scheduled intervals, as a detour was not available.

The rock slopes at the site consist of a light grey, coarse grained, competent, granodiorite. The rock is moderately to widely jointed, with the primary joint set oriented unfavourably towards the reservior slope below the road. This results in the formation of large slabs, which have the potential to slide out of the slope, towards the reservoir.

As excellent exposure was available at the site the geotechnical investigation for the highway realignment program consisted of geological mapping of the existing rock cuts and natural slopes. The mapping results were used to confirm the observed potential kinematic mode of failure and to provide data on the strength of each of the controlling joint sets. This information was then used to check the stability of individual blocks and to design rock support in cases where the factor of safety was found to be unsatisfactory. Prior to the start of cutting and road lowering, all of the existing slopes through the section were scaled and trim blasted or spot bolted where necessary, to either remove or support any potentially loose blocks. All scaling was done from the top down, using three to five man scaling crews on ropes. Bolting was either done by the scaling crew or by a track mounted tank drill working off the original road grade.

The road excavation consisted of a combination of small 3 to 5 m high sliver cuts and a series of 2.4 to 3.7 m deep road blasts to lower the grade (Figure 6a). Only a 10 m long segment of road could be blasted at one time in order to ensure traffic openings. With the requirement for traffic movement, traffic at times drove over pre-drilled holes, and at other times across broken muck. The explosive used was 'Emulex 920' and each blast was delayed with 25 ms EZTL Primadet nonelectric detonators. After each blast, the muck and scaled material was loaded onto articulated rock trucks and hauled to the opposite side of the reservoir for use as rip-rap.

In some sections of the site, there was a concern that overhanging blocks or panels of rock below the existing roadway would fall into the reservoir when the blast was shot. To prevent this, these blocks were prereinforced with temporary bolts to give them additional strength to survive the blast without sliding into the reservoir. The lateral extent of each blast was also restricted in these areas to within 1 to 2 m from the outer edge of the road. This technique worked well, and the remaining knob of rock was removed in a controlled manner after each blast using a pneumatic rock breaker.

For the road blasts, pre-splitting was used to limit the blast damage behind the new face and the blast was designed to allow the fragmented rock to be ejected longitudinally down the road away from the reservoir and slope. In order to control fly rock, blasting mats were placed over the entire blast area. The mats were secured with chains and anchored to the rock with mechanical anchored bolts (Figure 6b). Spacers were placed between the mats and the road surface to protect the EZTL detonation delays from abrasion to avoid potential cut offs.

The blasting for the minor slash cuts along the inside slope toe comprised drilling horizontal holes along the strike of the slope and using cushion blasting techniques to minimize the vibration and blast damage to the new face. Again, blasting mats were used to minimize flyrock and spacers were placed between the mats and the slope surface to protect the detonator delays.

These techniques were successful in creating the design road alignment at the desired road grade. Subsequent to this program, several tied-back cast in place retaining walls have been constructed at the site, to fill in gaps along the rock slope crest where there was not sufficient width for the outer lane of the new roadway.

3.2 Upgrading of Highway 12 at Texas Creek

A 1.4 km length of Highway 12 crosses the site of an ancient rock avalanche called the "Texas Creek Slide," located on the east side of the Fraser Canyon approximately 20 km south of Lillooet, B.C., Canada. The steep slopes above the highway consist of cut slopes up to 40 m in height and natural slopes continuing for over 400 metres above. Below the highway, the bedrock slope continues down approximately 125 m to the river (Figure 7a).

The rock fall frequency on this section of highway is very high and no catchment exists other than the existing road surface. There is also a problem with ongoing loss of the road running surface and shoulder, due to ongoing ravelling and erosion of slopes below the road.

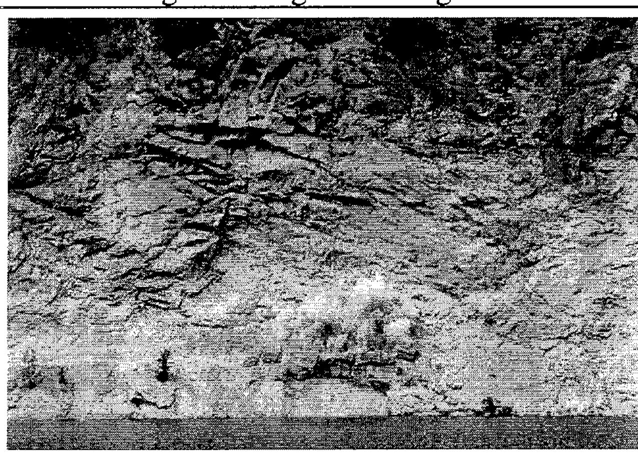

Figure 6a: Blast to Lower Road Grade, Pioneer Road 40

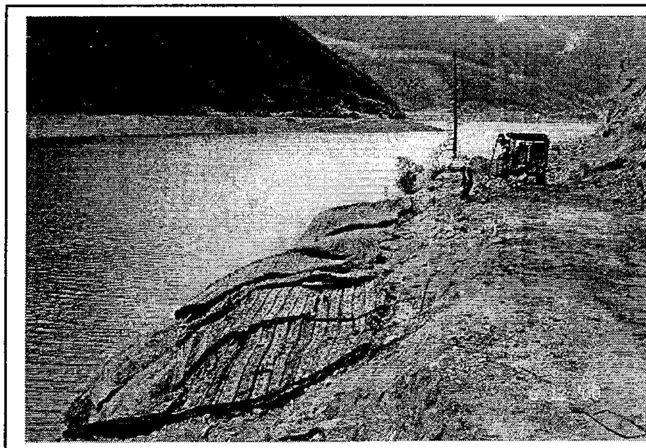

Figure 6b - Blasting Mats Used to Limit Flyrock into Reservoir, Pioneer Road 40

The bedrock at the site is comprised of closely jointed and thinly bedded moderate to steeply dipping argillite, which has been heavily faulted and sheared due to past tectonic movements. Consequently, the rock mass is relatively weak. Rock falls at the site tend to be small, but have high energy due to the height of the fall. Occasional larger falls occur when panels and slabs break away from the main rock mass.

Currently the highway has insufficient width for two lanes and is restricted to 30 kph across the site. To upgrade the highway through this section to a two lane, 70 kph design standard all four of the design options presented in Section 1.0 of this paper were considered, in addition to consideration of various tunneling options. After considering all of these alternatives, a tunnel with rock sheds was determined to be the only feasible alternative that could provide the required roadway configuration and mitigate the rock fall hazard. However, as the available funding was insufficient to construct the tunnel and rock sheds at the tunnel portals, a decision was made to improve three segments of the existing roadway where it is supported by rotten timber cribbing. Rather than replace the existing timber cribs and fills with conventional retaining wall structures, Golder's recommended approach was to span the segments of cribbing with low clearance bridges, resting on prepared concrete abutments (Figure 7b).

For the original road corridor upgrading study, the geotechnical investigation comprised geological mapping of the bedrock outcrop exposures and putting down both vertical and inclined boreholes. The results of the geological mapping were used to develop a geological model of the site, as input to rock mass stability modelling. The recovered cores from the boreholes were used to assess the rock mass quality beneath the existing roadway and behind the existing cut slope. Two inclined holes were also drilled near the portal areas of the proposed tunnel alignment. In addition, the investigation also included installation of two test anchors to assess the bond strength of the bedrock at the site.

Because of the high rock fall hazard and workspace constraints at the site, a number of precautionary safety measures were implemented during the investigation. These measures included erection of a temporary rock fall barrier around the drill rig, full-time traffic control personnel and an on-site Level 3 First Aid Attendant.

The investigation for the replacement of the existing crib walls involved geological

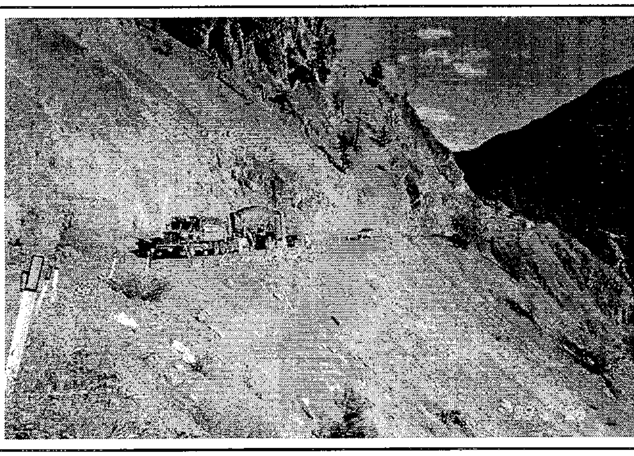

Figure 7a: Temporary Rock Fall Protection Used During Geotechnical Site Investigation, Highway 12 at Texas Creek

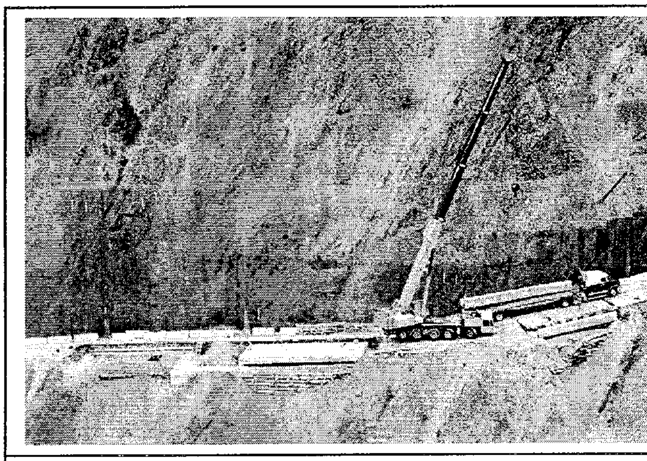

Figure 7b: Overview of Timber Crib Replacement, Highway 12 at Texas Creek

mapping of the road shoulder, and a geophysical investigation (ground penetrating radar) to confirm the extent of the cribwork and to size the proposed bridge spans. After consideration of structural alternatives, a design using precast concrete box beams was accepted by the MoT. The box beams were placed on cast in place concrete abutments. For additional support, passive dowel anchors were installed through the abutments into the rock below.

The low clearance bridges were designed to allow for minimal disturbance of the existing timber cribbing, which was to remain in place after construction. A surface cover of aggregate was placed on the bridges as a protective layer from rock fall, and placed such that the new asphalt tied into the existing road grade. The bridges were also aligned to avoid interference with a buried fibre optic duct along the inside of the road.

Key considerations of this design were to minimize the length of road closure necessary for construction, and to minimize exposure of the construction crew to rockfall hazard. To mitigate this hazard, a temporary rock fall fence was installed at the site, suspended between several large wood telephone poles. Installation of the bridges was completed, without incident, in the fall of 2001.

ACKNOWLEDGEMENTS

We would like to acknowledge the contributions of the Golder Associates Kamloops staff, Ty Garde, Greg Reid, and Tom Kneale, as well as Duncan Wyllie of Wyllie Norrish Rock Engineers who provided the technical direction on both case studies. We would also like to acknowledge Terry Harbicht of the Ministry of Transportation who reviewed the paper and authorized its publication. Lastly, we would like to acknowledge Sonia Skermer of Golder Associates who prepared all of the drawings.

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GEOLOGIC SETTING AND FACTORS INFLUENCING THE DESIGN AND CONSTRUCTION OF THE NEW CARQUINEZ BRIDGE AND CROCKETT INTERCHANGE STRUCTURES, CALIFORNIA

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Marine claystone and minor lenses of marine siltstone, sandstone and shale of the Late Cretaceous Panoche Formation, a part of the Lower Cretaceous to Upper Jurassic Great Valley Sequence, underlie the new Carquinez Bridge and Crockett Interchange structure locations in the northeastern San Francisco Bay region. The structure of the bedrock formation in the south portion of the site is chaotic and discontinuous, both laterally and with depth. The chaotic nature of the bedrock is due to the mode of initial deposition as turbidite sequences that are typical of relatively deep-water submarine fan complexes. These rocks have been further disrupted by tectonic shearing by the Late Pleistocene Franklin thrust fault that lies below the site and is exposed at the ground surface about 1/2 km (1/3 mile) south of the new structures.

The chaotic bedrock structure results in discontinuous, relatively hard cobble- to boulder-sized blocks of more competent formational rock within a highly sheared, very intensely weathered to decomposed rock material resulting in a weak matrix (block-in-matrix sedimentary-tectonic mélange). Overturned beds were observed on a hillside cut exposure near the leading southwestern edge of the Franklin thrust fault. The bedrock typically swelled vertically into the core barrel during exploration drilling operations (providing generally greater than 100 percent recovery), indicating expansive bedrock under local stress conditions. The exposed formational materials also exhibited a marked slaking behavior, which would affect structure foundation pile/rock side shear resistance. The site geologic constraints influenced feasible foundation types, appropriate foundation design methodologies and construction considerations.

Slope Stabilization Utilizing High Performance Steel Wire Mesh in Combination with Soil Nailing and Anchoring

by

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ABSTRACT

One popular and effective approach to achieving long-term slope stabilization is covering the slope with flexible steel mesh facing. In North America these systems are typically anchored only at the top, allowing the mesh material to drape freely down the slope. The weight and friction of the mesh material provides stability, and allows controlled downward movement of material. Typical European installations provide deeper stabilization by holding the mesh to the surface with anchors or soil nails throughout. These designs are largely dependent on the ability of the system to transfer forces from the facing material to the anchor points. The low tensile strength of conventional wire mesh has led to the use of steel wire rope nets, but these nets tend to be relatively expensive.

These limitations have been overcome by the development of a cost-effective diagonal wire mesh manufactured from high tensile strength, highly corrosion-resistant wire. In extensive testing this mesh has demonstrated a strength approaching that of wire rope nets. Additional development has produced an anchor plate that optimizes force transfer from mesh to anchors. These factors allow the mesh to be pre-tensioned against the slope, which restricts deformations in critical surface sections and prevents movement along planes of weakness. Newly developed dimensioning models yield an engineered design of these systems, including anchor design. Numerous such systems have been installed throughout Europe and the U.S. A review of material properties and system performance will be presented in addition to brief case histories of two U.S. installations.

INTRODUCTION

Anywhere a highway, railroad, or other infrastructure enchroaches upon a slope, instability is bound to be a significant and recurring problem. Limited right of way frequently mandates the creation of over-steepened or truncated slopes. Other contributing factors can include groundwater conditions, the structural geology of the slope, or environmental factors such as heavy rainfall or erosion. These factors lead to two main types of instability: surficial degradation of the slope, and deeper instability along discontinuities.

Before selecting what type of mitigation is most appropriate for a particular slope, it is necessary to distinguish between surficial problems and deeper instability. Surface instability is characterized by material moving down the slope under the influence of gravity. Depending upon the site conditions, this material can include soil, mud and debris, or rocks and boulders. Deeper instability consists of the movement of a mass of material along planes of weakness.

A wide variety of mitigation measures are available to address stability concerns. Surficial problems can be addressed by use of a slope matting material (jute mesh, wire mesh, wire rope nets, etc), shotcrete facing, catchment barriers, re-vegetation of the slope, and other methods. Deeper instability typically necessitates more extensive mitigation measures such as pattern anchoring both with and without a facing material (meshes, shotcrete, concrete panels, etc.), retaining walls, or excavation of the unstable material.

HIGH-TENSILE STRENGTH WIRE MESH AS SLOPE PROTECTION

The use of wire mesh and wire rope nets is an effective and widely accepted method of providing protection to the surface of slopes. These materials are typically draped freely on a slope in order to control downward movement of material. To provide protection against deeper instabilities, these flexible materials must be pinned throughout the slope with a pattern of anchors. Until recently, the wire meshes used in these types of applications have been manufactured from wire with a tensile strength of approximately 500 N/mm^2. These relatively low-strength meshes do not have the high tensile strength that is required to retain an unstable slope and they ultimately require an anchor spacing that does not prove to be economical. Wire rope nets provide the strength necessary to maintain a cost-effective anchor spacing, but they are comparatively expensive and require anchor points to be installed in specific locations.

The development of a high-tensile strength wire mesh manufactured from steel wire with a strength of more then 1,800 N/mm^2 offers the possibility of anchored constructions that provide an effective and economical solution to unstable slopes. This mesh has the high strength necessary to enable greater spacing between anchors, leading to a lower overall installed cost. Additionally, research and testing has resulted in a dimensioning model designed to optimize the high strength of the mesh. This enables the design and construction of an engineered system that takes full advantage of the capabilities of the high strength mesh.

THE TECCO® MESH SYSTEM – COMPONENTS

High tensile strength wire mesh

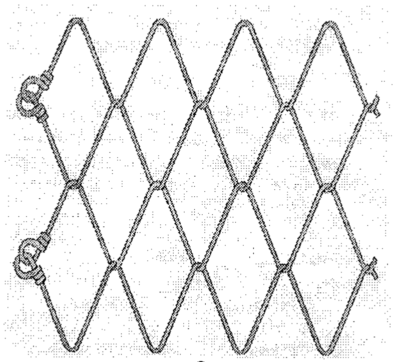
Fig.1: TECCO® Mesh

Wire Strength:	> 1770 N/mm^2 (257 p.s.i)
Wire diameter:	3 mm. (0.125 in.)
Corrosion protection:	supercoating (95% Zn/5% Al)
Coating deposit:	min. 200g/mm^2 (0.3 lbs/in^2)
Mesh opening:	83 mm. (3.3 in.) x 137 mm. (5.4 in.)
Mesh height:	15 mm. (0.6 in.)
Mesh strength	
Longitudinal:	150 kN/m
Transverse:	75 kN/m

The mesh is produced in rolls measuring 3.5 m (11.5 ft.) wide by 30 m (98.4 ft.) long. The gentle deflection of the wire created by the bending process used to form the individual meshes results in a three-dimensionality in the mesh which improves the connection to the subsoil and aids in re-vegetation of the slope. The 150 kN/m longitudinal strength of the TECCO® Mesh represents a significant improvement over the 50 kN/m strength of conventional meshes made with comparable mesh opening size and wire diameter.

Spike Plates

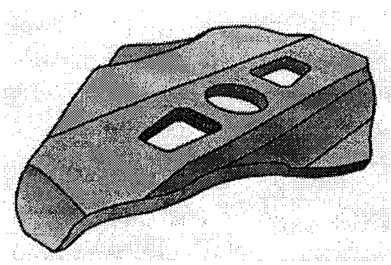
Fig. 2: Spike plate

To insure optimal transfer of forces from the mesh to the anchors, a special spike plate has been developed. These plates are placed over the anchor heads and tightened in order to pre-tension the mesh onto the slope. The plates are galvanized and have dimensions of 330 mm x 190 mm x 10 mm (13.0 in. x 7.5 in. x 0.4 in.).

Compression Claws

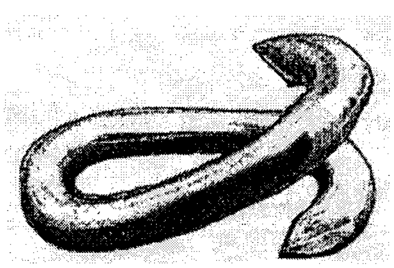
Fig. 3: Compression Claw

In order to take full advantage of the high tensile strength of the mesh, a special fastener is required to connect the individual mesh panels together. These claws are hot dip galvanized and consist of 6 mm (0.25 in.) thick steel spiral.

Anchors

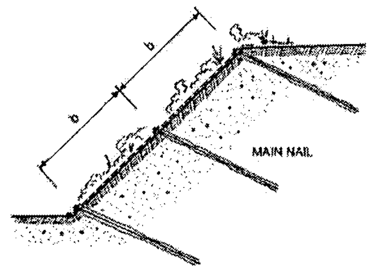

Fig. 4: Anchors

The main anchors of the system are installed in a grid typically ranging from 2.0 m to 4.0 m (6.6 ft. to 13.1 ft.) horizontal and vertical spacing. Actual spacing depends upon the results of site analysis and modeling. Anchors consist of commercially available steel bars ranging from 25 to 32 mm. (1.0 to 1.25 in.). Self-drilling grout injection anchors are also permitted. Additional anchors may also be used to provide added support at boundaries and in low points and hollows.

SYSTEM DIMENSIONING

The Ruvolum® dimensioning concept has been developed specifically for these types of systems. Using the material properties of the mesh along with the characteristics of a given slope as input, this model determines the optimum anchor spacing and dimensioning as well as the optimum mesh tensioning required to provide stability to the slope.

The dimensioning concept consists of two parts:

- Investigation of global instabilities parallel to the slope (Fig. 5)
- Investigation of local instabilities between the nails (Fig. 6)

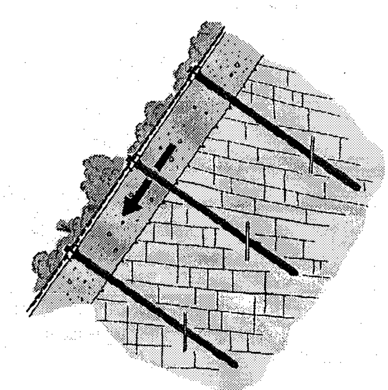

Fig. 5: Global Stability

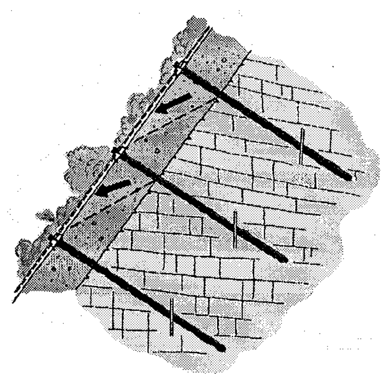

Fig. 6: Local Stability

SYSTEM CONCEPT

The anchors and mesh act together as a system to provide stability to the slope, preventing deformations in the top layers and restricting movement along planes of weakness. As a result of the high strength of the mesh, it is possible to pre-tension the system against the slope. This pre-tensioning enables the mesh to provide active pressure against the slope, preventing break-outs between the nails.

Compared to other alternatives (particularly hard facings such as shotcrete or large retaining structures), this system offers the following advantages:

- Very effective
- More economically feasible
- Allows greening of the slope – resulting in a natural, aesthetically pleasing appearance
- Very long useful life (components have a 100 year design life)

SYSTEM INSTALLATION

The slope is first prepared by cleaning loose debris, leveling as much as possible, and shaping as needed.

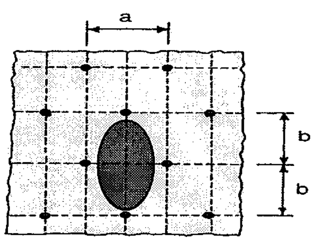

Fig. 7: Anchor grid

The anchors are then installed in a defined grid (see Fig. 7). The mesh does not require the anchors to be placed in specific locations so it is possible to adjust anchor locations (while staying within the design spacing) to compensate for irregularities in the terrain. Additional, short anchors may be installed between the grid anchors to insure a tight fit of the mesh against the slope. This also helps the mesh conform to the terrain, limiting its visibility.

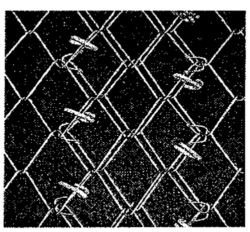
Fig. 8: Mesh connection

After the anchors have been installed, small depressions are created around each anchor in order to accommodate the installation of the spike plates. The mesh is then laid on the slope, and cut to size as needed. Adjacent panels of mesh are connected with the compression claws (see Fig. 8). The mesh is then pre-tensioned with a defined force against the anchors by tightening the spike plates down into the depressions (see Fig. 9).

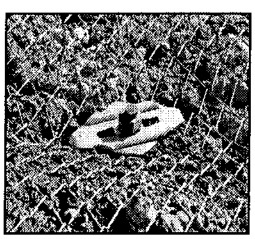
Fig 9: Pre-tensioning.

This pre-tensioning firmly presses the mesh against the slope, preventing deformations, slippage, and pop-outs of the slope material.

Depending upon site conditions, it may be advantageous to lay the mesh on the slope prior to anchor installation. After the mesh is installed and tensioned, the slope is ready for hydro-seeding or other re-vegetation measures.

CASE HISTORY 1 – BOULDER CREEK, CA

In late 2000 the California Department of Transportation (Caltrans) turned to an anchored high strength steel mesh system to stabilize a chronically failing slope. The high levels of seasonal rainfall in the region resulted in this cut slope having a history of shallow slope failures that produced mud and debris flows, creating a hazard for motorists and a maintenance nuisance. The project site is at mile 13.5 of Santa Cruz County Route 9 in Boulder Creek, CA (between Santa Cruz and San Jose). Installation of the system was begun in late 2000 to mitigate the problem before the winter rains began for the year. Other options were considered but rejected due to a variety of reasons, including: cost, length of time to install, the desire to save as many trees as possible, and visual impact. As a result of the aesthetically sensitive nature of this area, the desire for a visually pleasing solution was a key consideration.

The system was installed at the top areas of the head and lateral scarp faces. The head scarp is approximately 35.6 m. x 10 m. (117 ft. x 33 ft.) and the lateral scarp is approximately 25 m. x 8 m. (82 ft. x 26 ft.), resulting in a total coverage area of approximately 556 m^2 (5,985 ft^2). Moving down the slope, the inclination varies from 60 degrees to 45 degrees. Because some of the lower portions of the site are not as steep, these areas were covered only by jute matting and traditional wire mesh.

Modeling of the slope using the Ruvolum dimensioning concept indicated an anchor spacing of 3.3 m x 3.3 m. (10.8 ft. x 10.8 ft.) on center both horizontally and vertically, resulting in a total of 104 anchors. As installation progressed, it became evident that a total of 180 anchors would be necessary. The additional anchors were located at the direction of the project engineer in order to insure a tight fit of the mesh to the irregular terrain.

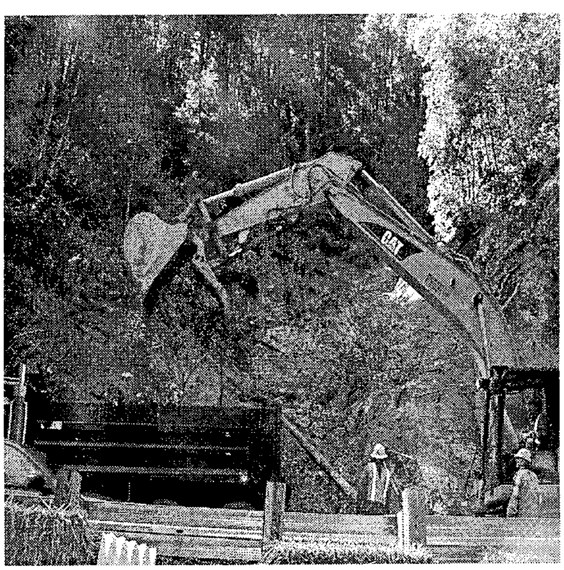

Fig. 10: Slope cleaning phase

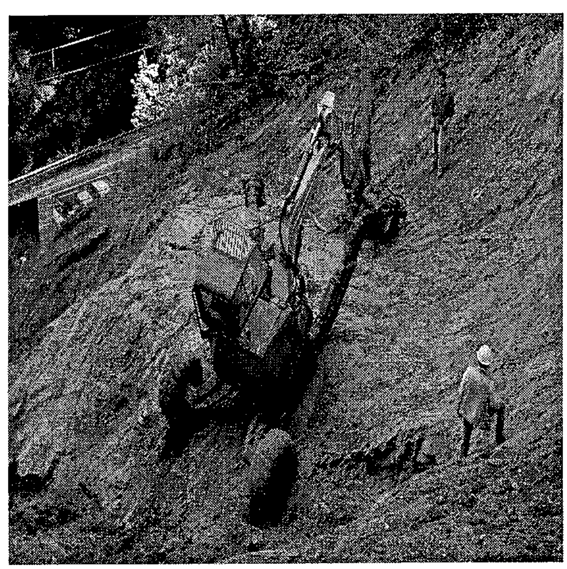

Fig. 11: Anchor drilling

The slope was prepared by removing loose debris and mud (see Fig. 10). The excavating equipment was re-fitted with a drilling attachment to drill the 76 mm. (3 in.) diameter anchor holes (see Fig. 11). The 28 mm. (1.1 in.) diameter threaded anchor bolts were then inserted and grouted into the holes to a depth of 4 m. (13.1 ft.). The anchor specifications indicated by the dimensioning model were a tensile strength of 500 N/mm^2 (72.5 p.s.i) with a breaking strength of 370 kN (42 tons) and a tensile load of 308 kN (35 tons). After installation of the anchors, small depressions were created around the anchors to allow for the spike plates.

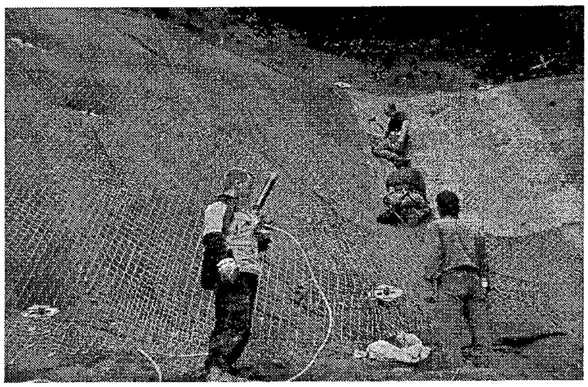

Fig. 12: Spike plate installation

Prior to installation of the mesh, the slope was seeded and covered with jute matting. The mesh was then rolled onto the slope, and tensioned against the surface by tightening the spike plates down into the depressions (see Fig. 12). Fig. 13 is a view of the slope immediately after mesh installation, and Fig. 14 shows the slope after hydro-seeding was completed.

Fig 13: Project site immediately after mesh installation on head and lateral scarp faces

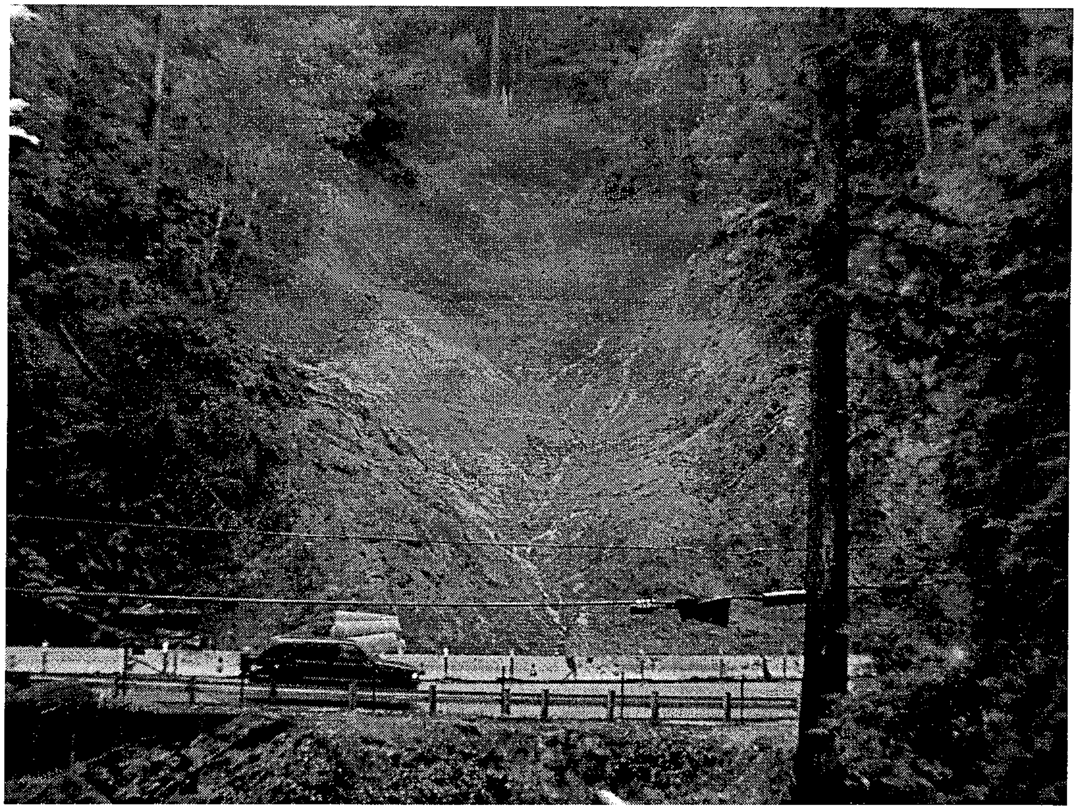

Fig. 14: View of slope after hydroseeding

This system has now been in place through two full winters, and the slope has remained stable. Without the stabilization system, the extreme rainfall events that occurred within this period would likely have resulted in the loss of the head scarp, along with many trees. Re-vegetation has been successful, adding to the surficial protection and providing an aesthetically pleasing slope. Fig. 15 shows the site 16 months after installation.

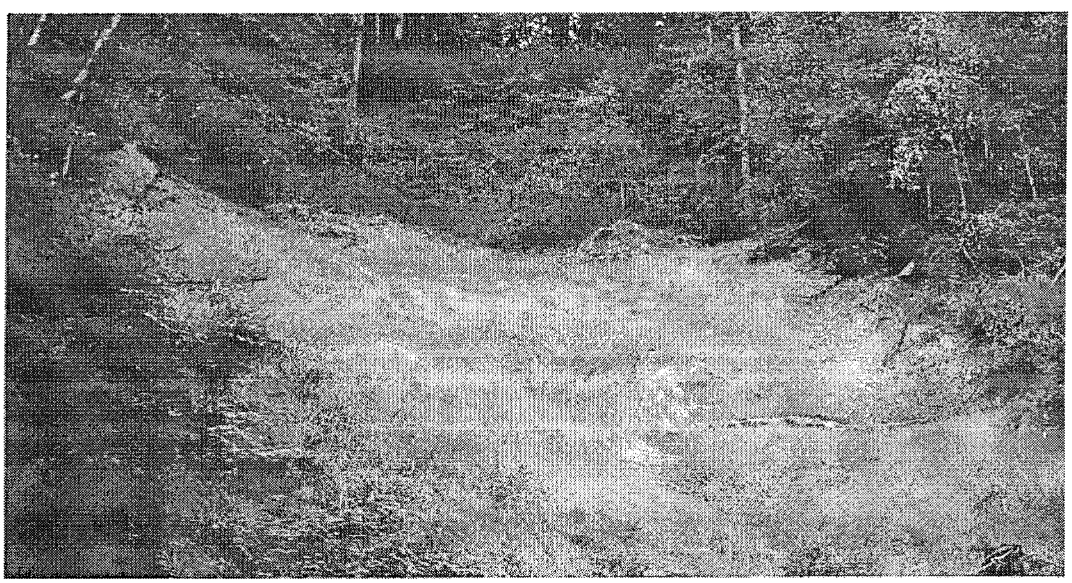

Fig. 15: Mesh is almost completely overgrown – 16 months after installation

CASE HISTORY 2 – PRESCOTT, AZ

In May 2001 the Peridot Retirement and Assisted Living Community in Prescott, AZ chose an anchored high strength mesh system to stabilize a cut rock slope adjacent to a parking lot. The slope was approximately 11 m. (36.0 ft.) high and consisted of highly fractured volcanic rock. The project owner ultimately selected the anchored mesh system over a reinforced concrete wall on the basis of its better aesthetic appearance and lower installed cost (approximately half that of the wall option).

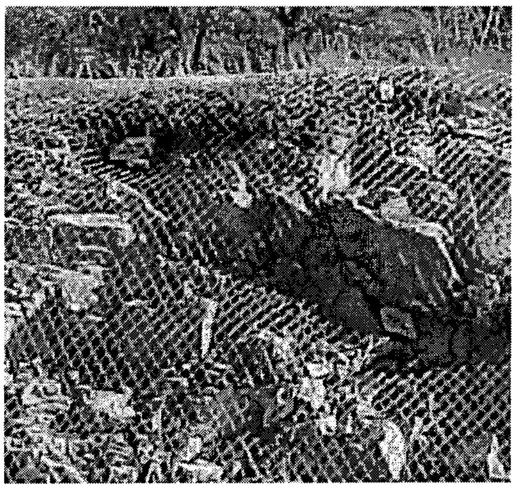

Fig. 16: Anchors located in shallow points

Analysis and modeling of the slope with the Ruvolum dimensioning concept resulted in a specified anchor spacing of 2.2 m. (7.2 ft.) and anchor depths varying from 2.2 m. (7.2 ft.) to 7.4 m. (24.2 ft.). The rock was highly fractured, resulting in a very irregular slope face with frequent pop-outs and sharp angular projections. Wherever possible, anchors were located within shallow points of the slope (see Fig. 16). This greatly facilitated tensioning of the system onto the surface with the spike plates. The durability of the mesh was evident in this installation, as it exhibited no signs of rupturing after being tensioned around the sharp rock edges.

The system has been in place for approximately one year, and the slope has remained stable. Re-vegetation has been minimal as a result of the arid climate, but the system has had a limited visual impact that has met with great approval on the part of the owner.

Fig. 17: Mesh installed on slope

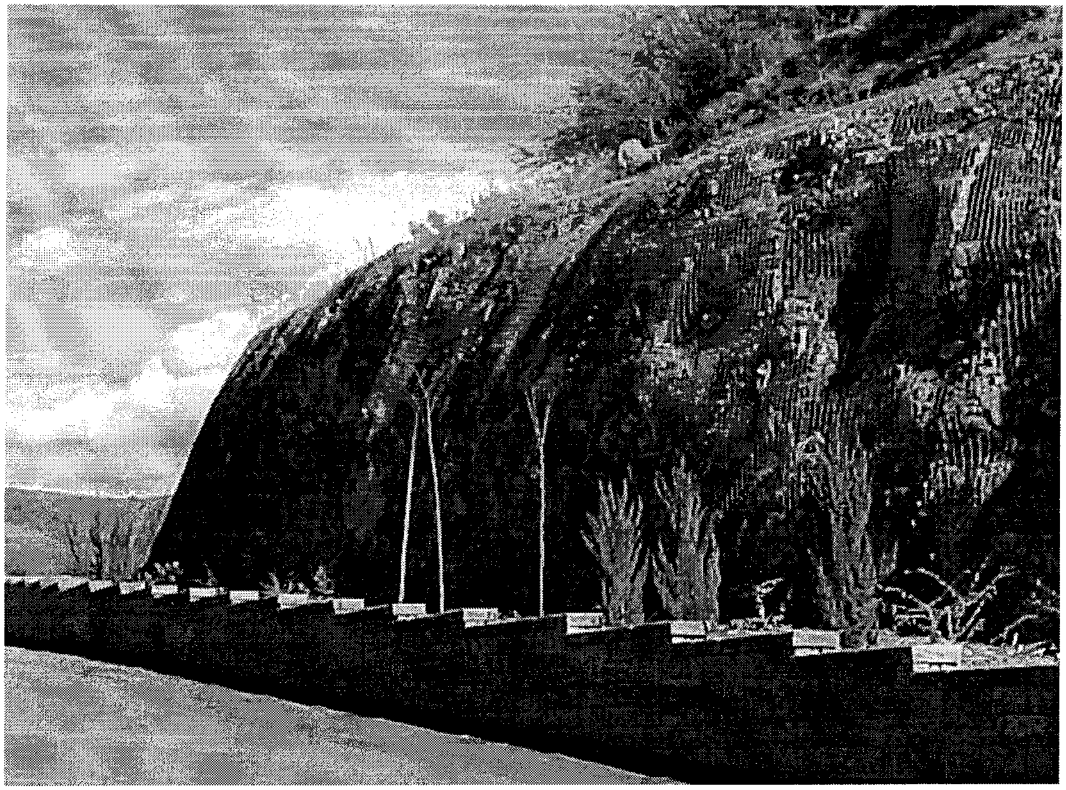

Fig. 18: View of completed installation – the mesh conforms to the slope, limiting the visual impact

CONCLUSION

Anchored slope stabilization systems using high strength steel wire mesh as a facing material are an effective and economical means of protecting unstable slopes. These systems have demonstrated effectiveness in both rock and soil slopes. The high strength allows an economical spacing of anchor points; and the open mesh structure eliminates the need for drainage, and facilitates greening of the slope. As a result, this mesh offers the advantages of both rigid constructions and traditional meshes. This concept is only just beginning to gain acceptance in North America, but it has been successfully applied in Europe for several years.

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Chemical Grouting Beneath Cocodi-Cola Highway and Airport Boulevard

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ABSTRACT

The paper presents the results of the chemical grouting utilized to stabilize the soil prior to excavation of the Haret Hreik-Sands tunnel beneath the Cocadi-Cola Highway and Airport Boulevard, in Beirut, Lebanon. The tunnel face/crown was successfully injected with a silicate base grout by means of inside tunnel micropiling. A discussion of the injection methods used as well as technical evaluation and pertinent field observations are provided.

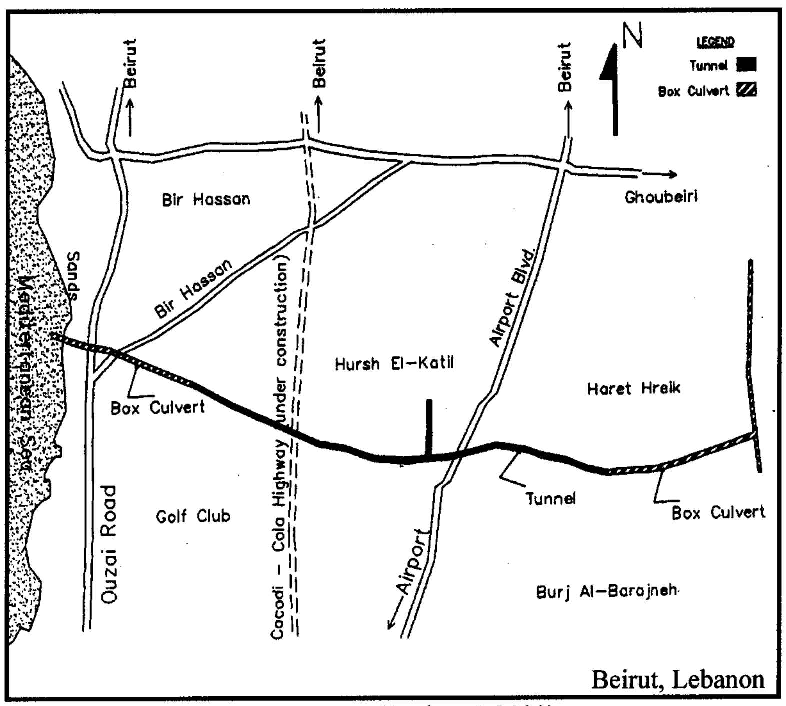

Figure 1: Project Location Map (Scale ~ 1:2500)

INTRODUCTION

Project Background

The tunnel construction is a part of an overall project to collect storm water from low areas in the eastern suburbs of Beirut, Lebanon and discharge into the Mediterranean Sea near the old Sands Beach (see Figure 1). The tunnel is approximately 1.6km and crosses at approximately 12m beneath the Cocadi-Cola Highway and approximately 18m beneath the Airport Boulevard. These two main roads are critical for the daily commute between the City of Beirut and the entire southern portion of Lebanon. As such, it was essential that no disruption of traffic result from tunnel construction.

The tunnel alignment trends primarily beneath an existing golf club and public easements. The soil cover along the tunnel course varies generally between 8m and 22m. The upstream and downstream portions of the stormwater network are conventional reinforced concrete box culverts constructed in an open cut excavation.

The tunnel project was funded by the Lebanese Council for Development and Reconstruction (CDR) in an effort to improve the infrastructure of northern and southern Beirut suburbs. The contractor for the tunnel project was Impressem (Italy) – Batco (Lebanon), J.V. The project was designed and supervised by Dar Al-Handasah (Shair & Partners), an international consulting firm based in Beirut, Lebanon. The project construction occurred in a 30-month period beginning in September 1995, completing in February 1998.

The original design anticipated limited zones of potentially collapsible sand. Stabilization, utilizing conventional micropiling from inside the tunnel utilizing cement base grout, was planned. After an excavation of 30m from the tunnel west portal, a major collapse of the tunnel occurred which created a surface crater with a radius of approximately 8m and a depth of more than 3m. As a result, other soil stabilization techniques were explored to provide a safe working environment. Hence, the tunnel face/crown was injected with a silicate base grout and the project was successfully completed.

Soil Conditions

The tunnel passes through Quaternary sand deposits with varying amounts of silts. A more thorough geotechnical investigation at the start of construction revealed the likelihood that 900m (60%) of the tunnel course would pass through cohesionless sand conditions rather than somewhat cohesive soil as originally anticipated (see Figure 2). Based on the results of the field and laboratory testing performed for this project, the soil along the tunnel course may be classified as follows:

Yellow-brown medium to fine sand, with less than 5 percent silt content, is classified as SP sand per the Unified Soil Classification System (USCS) and A-3 material per AASHTO soil classification system. Based on the results of the field and laboratory testing, this sand has generally a coefficient of uniformity of 2.0 (see Figure 3) and a void ratio ranging from 0.50 to 0.55. The coefficient of permeability is on the order of $5x10^{-2}$ to $5x10^{-3}$ cm/sec. The N-value is typically greater than 30. This sand required pre-treatment to allow for tunnel construction.

Red-brown fine silty sand, with up to 25 percent silt, is classified as SM sand per USCS and A-2-4 material per AASHTO. This sand exhibits some apparent cohesion due to high overburden pressure and the presence of iron oxide. The coefficient of permeability for this soil is typically less than $5x10^{-3}$ cm/sec. The fine silty sand was expected to remain stable during tunnel excavation.

Cemented/Calcareous sand exhibiting cementation bonds varying from weak (broken by hand) to strong (broken by hammer) with a compressive strength of up to 100 kg/cm^2. This sand was considered stable based on the proposed method of excavation.

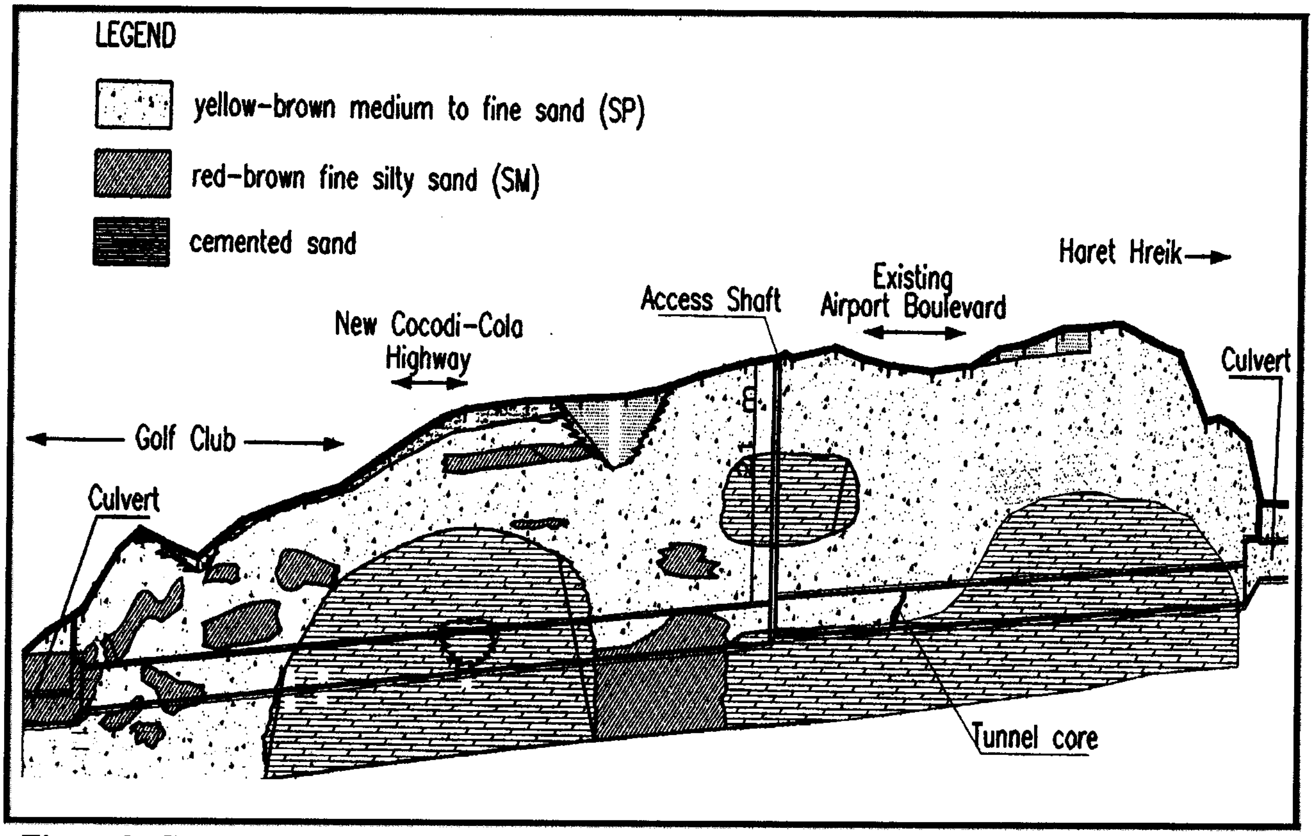

Figure 2: Geologic Profile Along Tunnel Alignment (Horiz. 1 cm = 100 m, Vert. 1 cm = 7 m)

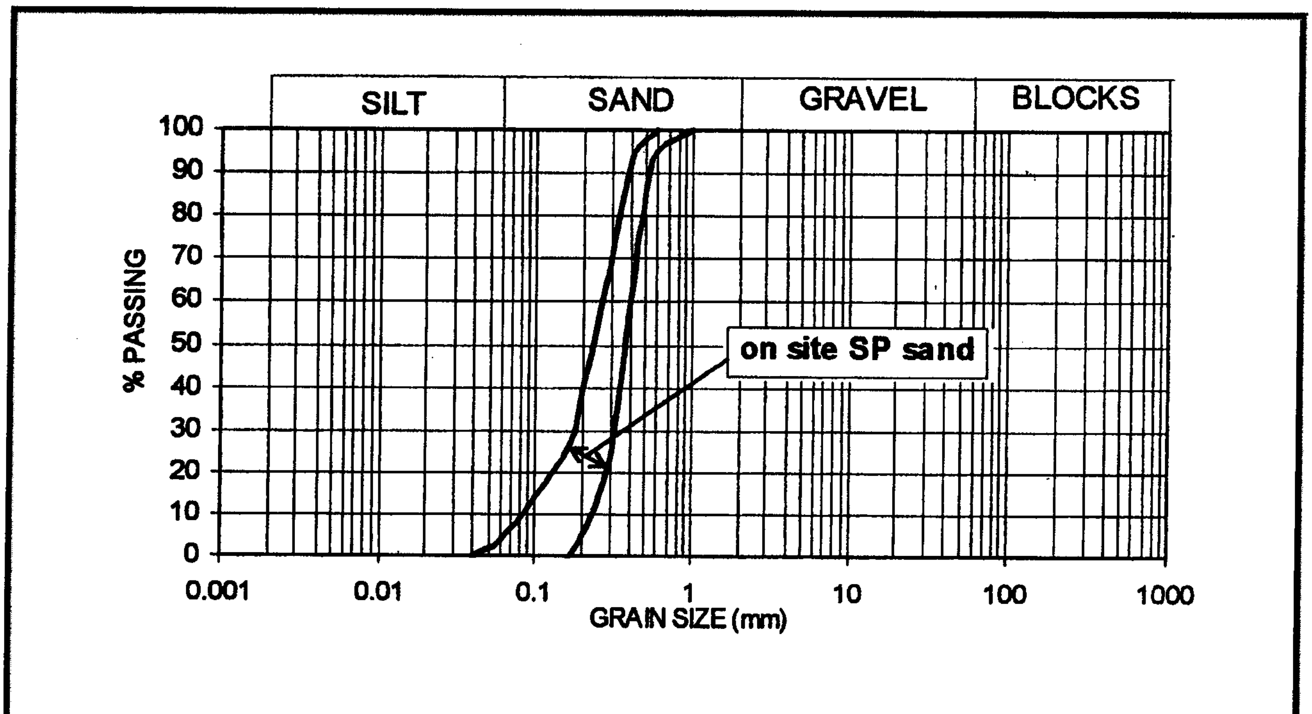

Figure 3: Grain Size Distribution of Grouted Sand

Tunnel Excavation

The construction method implemented by the contractor was based on the principles of the New Austrian Tunneling Method (NATM). The tunnel had a 4.3m horse-shoe shape section excavated generally by hand digging or by an excavator. A typical tunnel section of 80cm to 150cm in longitudinal profile was excavated prior to placement of initial lining. The actual length of the unsupported section varied depending on the soil conditions encountered. The initial lining, which consisted of NP 120 type coupled steel beams and a minimum of 15cm of shotcrete/wire mesh, was applied immediately after excavation. The 40cm concrete final lining was installed at a later stage upon the completion of the entire tunnel excavation. Stress-strain monitoring stations were installed at critical locations along the tunnel alignment to affirm the adequacy of the initial lining system. The tunnel construction method is further illustrated on Figure 3.

SOIL STABILIZATION BY CHEMICAL GROUTING

As previously mentioned, the construction method requires a tunnel excavation section (maximum 150cm) to be unsupported for a short period of time to allow for the placement of the initial lining. The presence of the medium to fine sand made this process difficult without some kind of soil stabilization measures. A silicate base grout was to be injected into the soil to create a cementitious bond between the sand particles and ultimately yield into a stable tunnel face and crown.

There are several methods of injecting chemical grouts into the soil (References 1, 2, and 3). These methods are generally selected based on several factors such as soil conditions, degree of continuity of the grouted zone, availability of qualified personnel and grouting equipment, cost and time constraints, etc. The injection method that was successfully used on this project was Inside Tunnel Micropiling, referred to herein as ITM method. Another method referred to herein as Micropiling from Ground Surface (MGS Method) is also discussed in this paper. However, the MGS Method was abandoned in the early stages of the project due to inadequate grouting results.

Micropiling from Ground Surface – MGS Method

This method was mainly considered by the contractor to avoid further delays in the project completion time whereby the ongoing tunnel excavation can proceed without disruptions due to the grouting operation. Except for limited stretches beneath the Cocadi-Cola Highway and the Airport Boulevard, the implementation of the MGS method was feasible from a site accessibility viewpoint.

The MGS method consisted of excavating vertical micropiles filled with the grout mix with no pressure applied. The micropiles extended from ground surface to a predetermined depth so that the the tunnel face and crown were fully grouted. The micropiles were installed in a staggered configuration spaced between 0.8m and 1.2m center to center. The grout was typically poured in two stages. First stage consisted of pouring 250 liters of grout to treat soil below tunnel crown, whereas the second stage consisted of pouring another 250 liters of grout to treat soil above tunnel crown (see Figure 5).

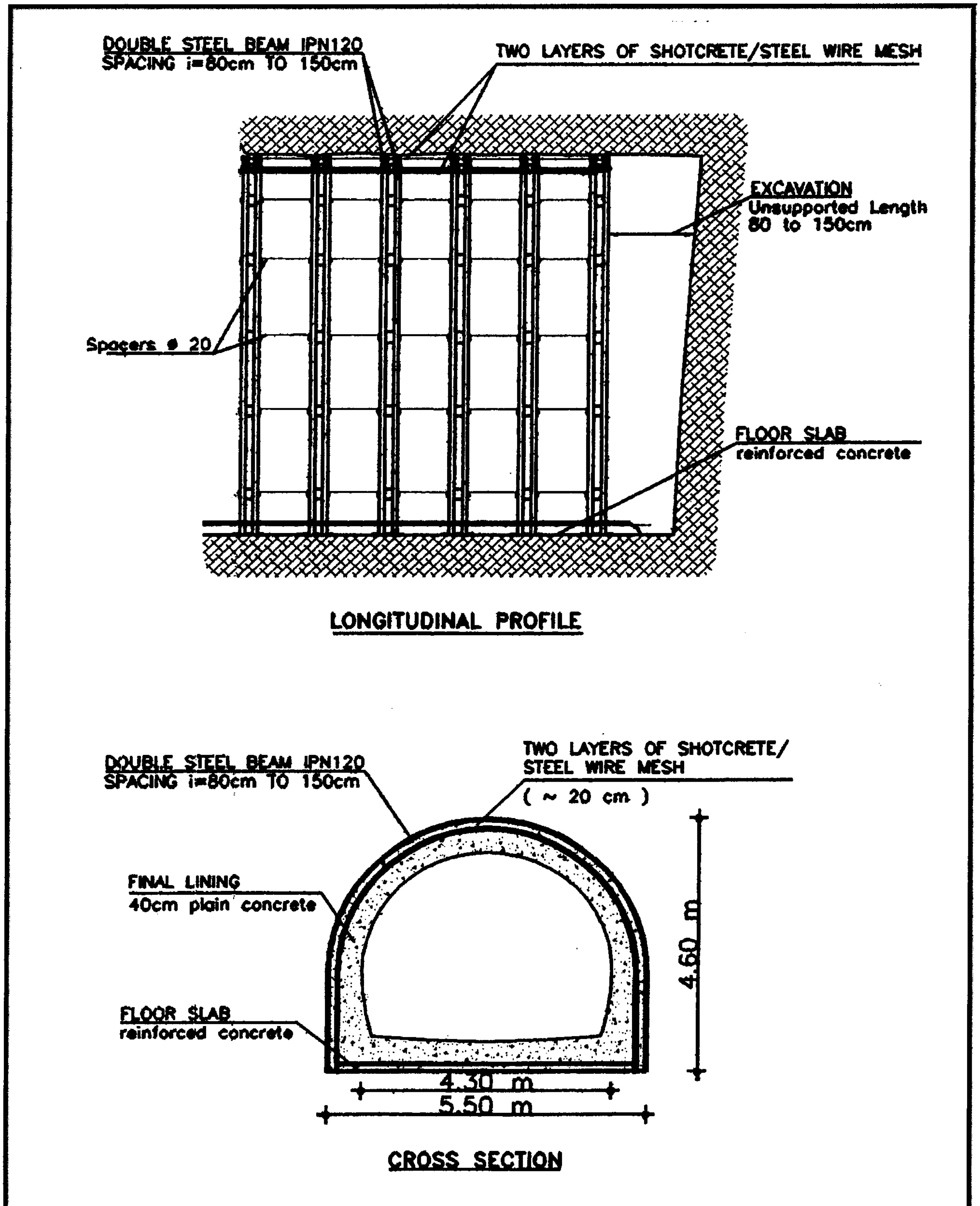

Figure 4: Tunnel Construction Method

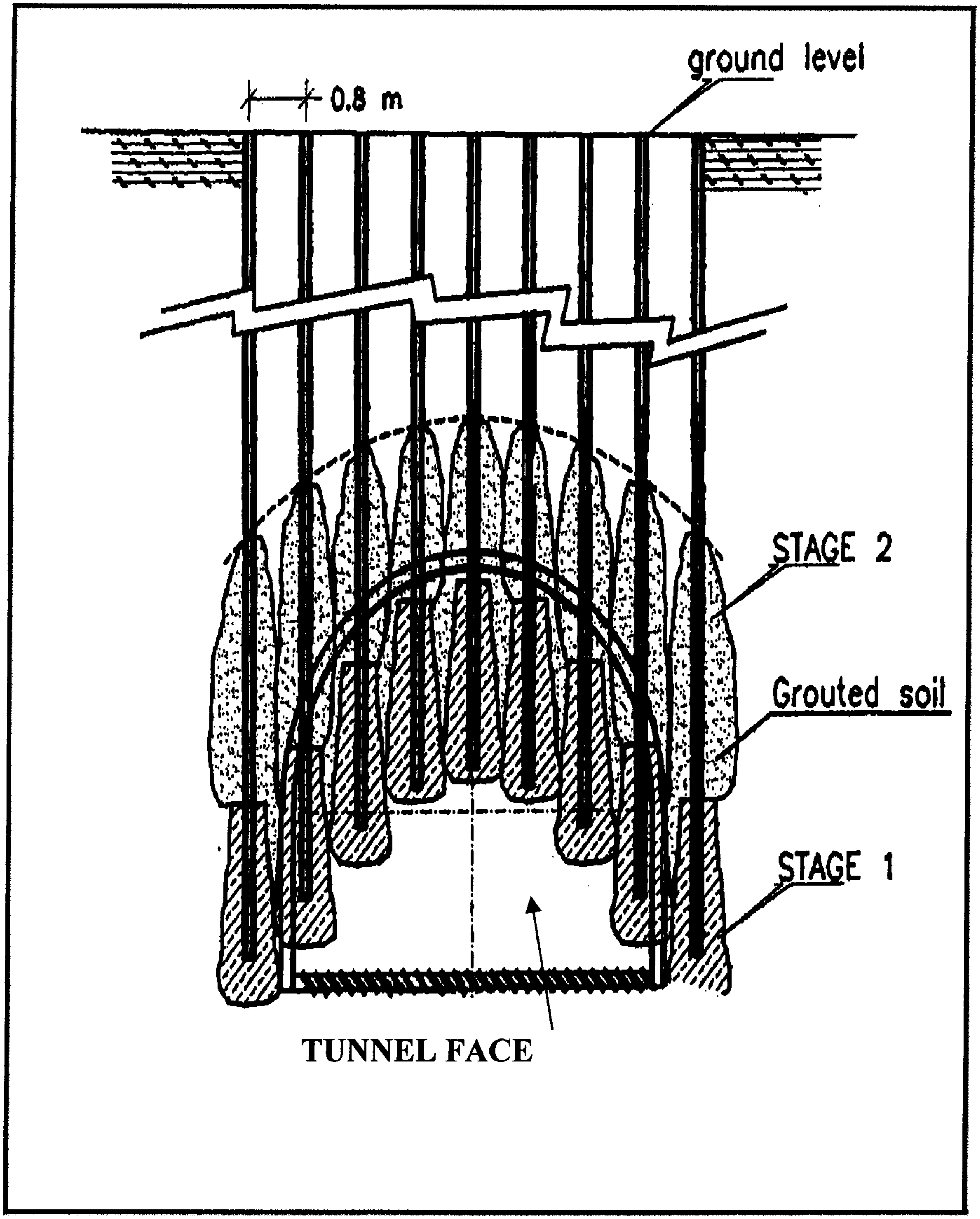

Figure 5: Micropiling From Ground Surface – MGS Method

Several in-situ tests were carried out on the silicate base grout using sodium bicarbonate as a hardening additive. The sodium bicarbonate was the only available hardening agent in the local market and considered very economical. The two-solution process was implemented to inject the grout into the soil. The sodium silicate was first poured into the holes and the sodium biocarbonate was then added as a second step. After advancing approximately 50m using this method, two relatively small soil collapses occurred in the tunnel crown due to ineffective grouting. As such, this method was abandoned and other injection methods were explored to provide safe working environment.

Based on the field observations during tunnel excavation as well as the test trials, the gel time was very difficult to control and varied typically between 15 minutes and 45 minutes for the same mix proportions of 1 liter of silicate diluted in 1 liter of water and 70 grams of sodium bicarbonate. As such, the two-solution procedure was not fully successful in achieving sufficient or complete mixing of the two chemicals in the subsurface soils, hence resulting in loose zones where the presence of silicate grout was very evident.

Inside Tunnel Micropiling – ITM Method

Te ITM method was implemented for the remainder of the tunnel course and consisted typically of 21 holes drilled from the tunnel face for a 9m stretch to complete one injection cycle. After proceeding with tunnel excavation for 7m, another injection cycle is implemented allowing for 2.0m overlap between two consecutive cycles. As shown on Figure 6, most of the holes are located along the upper part of the tunnel face and drilled at an inclined angle to create a grouted hood along the tunnel crown.

The Durglass system was used to inject the chemical grout into the soil. This system consists of a solid 1-inch PVC pipe wrapped by three fiberglass plates (see Figure 7). Another small PVC pipe section is attached to the Durglass assembly to allow for an initial round of grout injection in the first 0.5m of the perforated hole for the purpose of stabilizing the tunnel face prior to the start of the actual injection cycle.

In this method, the one-solution process was implemented to inject the sodium silicate and the reactor to form a gel. A more reliable hardening agent called Durcisseur, a product of Rhone Poulenc, France was used in this method. Based on manufacturer's recommendations and after several field and laboratory trials, the following mix proportions were used:

- Sodium Silicate (SiO2 Na2O): 42% of Volume
- Water (H2O): 50% of Volume
- Durcisseur (Type 600B): 08% of Volume

The gelling time for this mix was experimentally measured to be between 20 and 25 minutes. However, this time increased to at least 30 minutes during the injection operation. The grout mix had a density of 1.16 g/cm^3 and initial viscosity of 5 cP at temperature of 20°C.

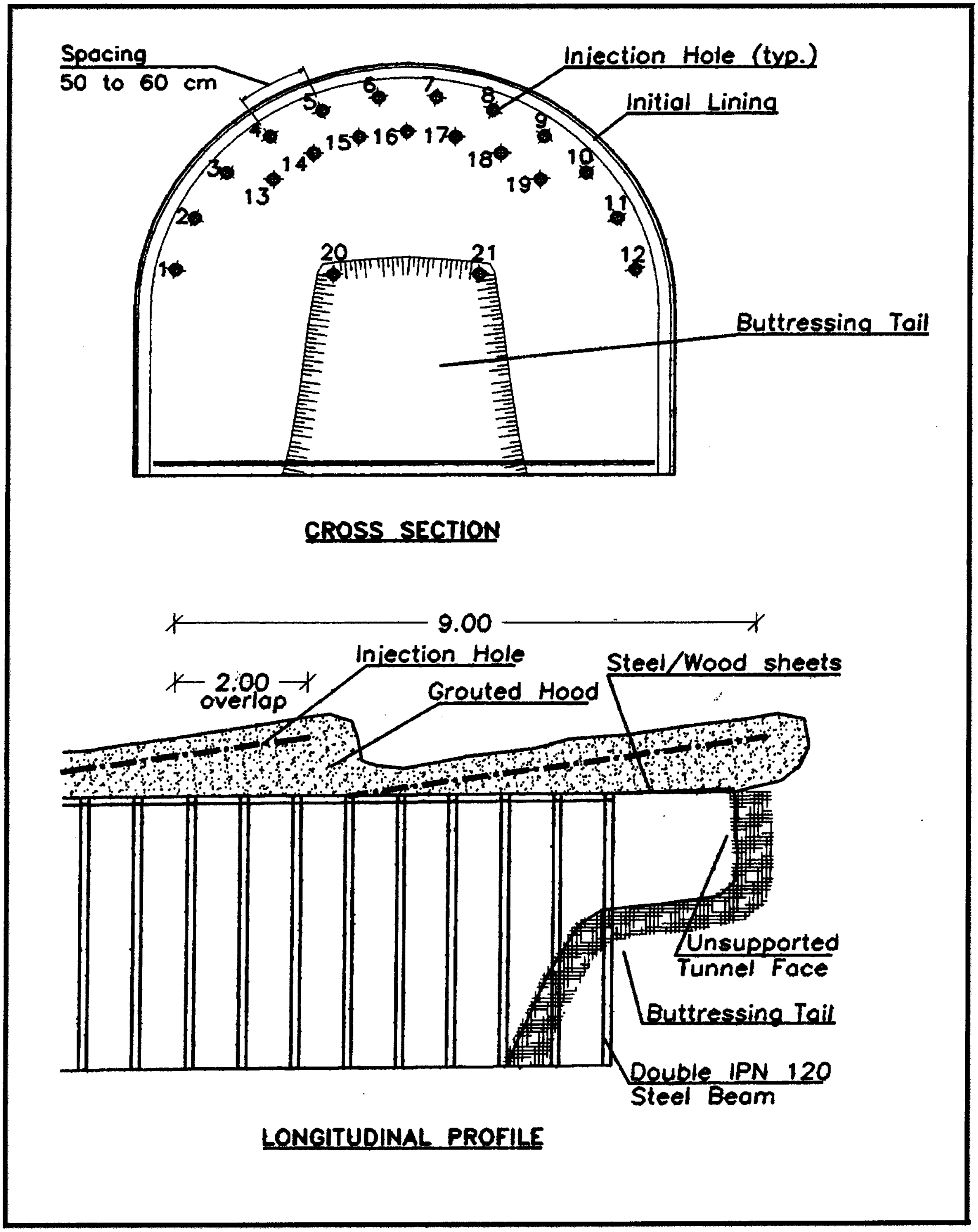

Figure 6: Inside Tunnel Micropiling – ITM Method

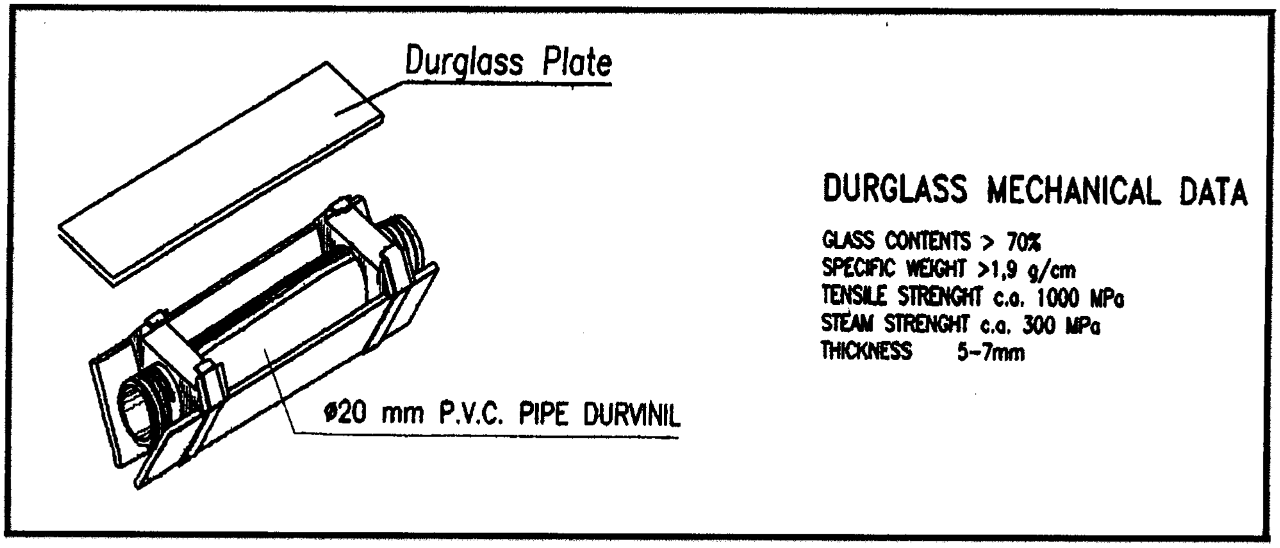

Figure 7: Durglass / PVC Injection System

The injection operation proceeded by rapidly mixing 250 liters of grout in a mechanical mixer to avoid hardening of the grout mix during the injection process. This operation of injecting 250 liters of grout required typically about 5 minutes to completion. The grout was pumped at a pressure of 3.0 to 3.5 bars using a positive displacement pump (piston pump). The injection operation would typically cease once pressure reaches 4.0 to 4.5 bars, which indicates that all voids within the zone of influence of the grouted hole have been filled.

TECHNICAL EVALUTION

General

Ideally, the injected grout is to fill inside the 120 mm diameter hole and start permeating the surrounding soils under the applied pressure. In uniform isotropic soils, and assuming Darcy's Law and Newtonian fluid, the radius of grout penetration from the point of injection according to Maag Mathematical model (References 3 and 4) is as follows:

$R = (3 \,.\, r_0/n \,.\, k \,.\, v \,.\, h \,.\, t + r_0^3)^{1/3}$ where
R – radius of grout front at time t
r_0 – radius of the injection pipe/hole
n – soil porosity
k – soil permeability
v – ratio of water viscosity to that of grout
h – equivalent water head (injection pressure)
t – time

According to the above equation, for isotropic soil conditions the extent of grout penetration is directly proportional to the applied pressure and gelling time. For soil permeability of 10^{-2}cm/sec and after a time of 30 minutes, a grouted zone of 1.6 m diameter would be achieved.

Observed Grout Penetration

Based on the result of the grouting operations for about 900m of tunnel, the following observations are drawn as to grout permeation in the medium to fine sand utilizing the ITM method:

- As illustrated in Figure 8, the actual bulb of treated sand around the injected hole varied non-uniformly between 0.2m and 0.8m. The grouted zone is typically larger at the end of the perforated hole where the injected grout permeates under relatively more efficient applied pressure.

- The grout seems to be generally less effective after traveling further from the injection hole. Traces of grout were often noted outside the effected zone with very weak cementation. After several field trials, an average of $1.5m^3$ of grout is typically consumed per the 9m-long hole, which equates to a grouted zone of 0.8m diameter. As such, approximately half of the anticipated volume was effectively grouted.

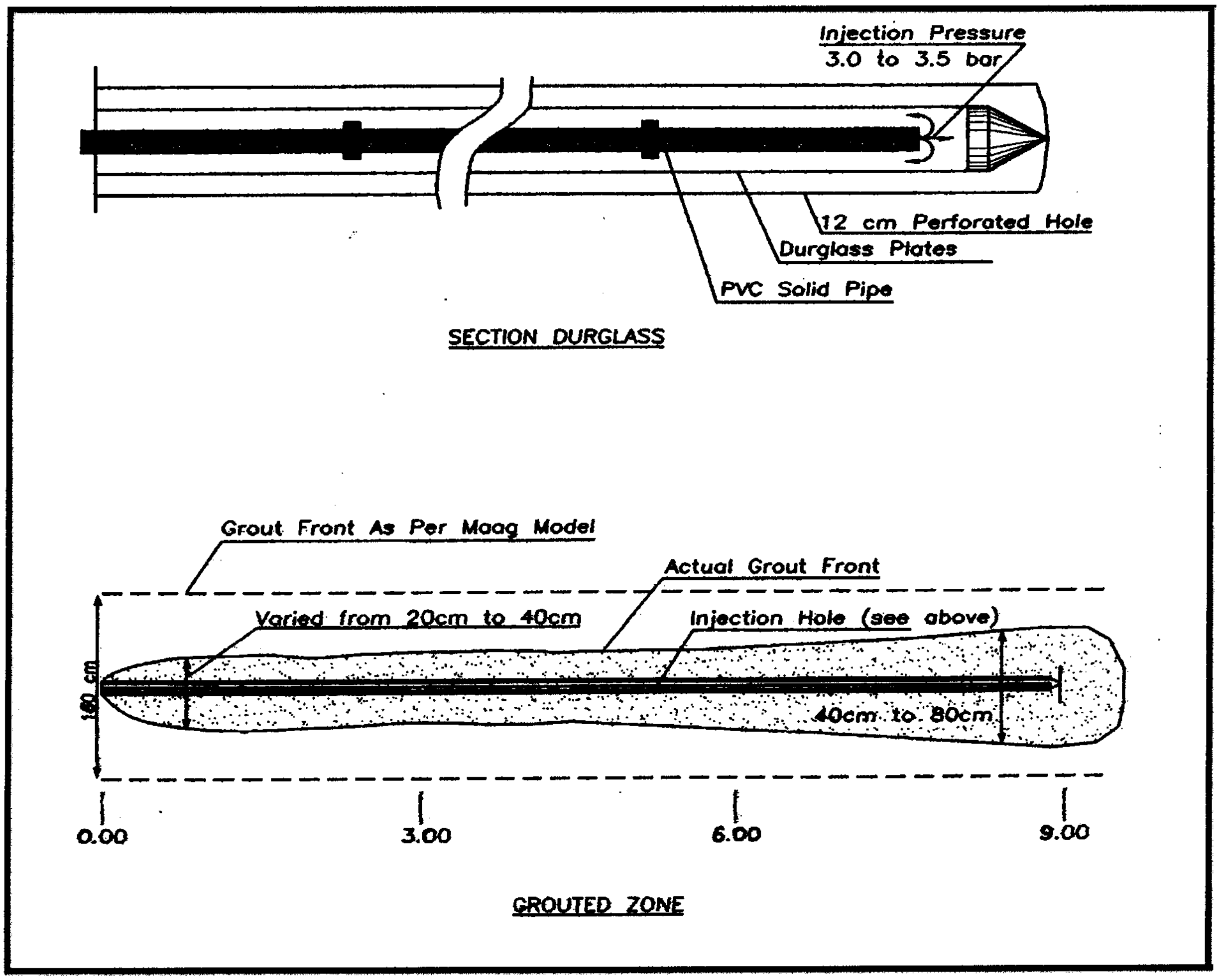

Figure 8: Grouted Zone Based On ITM Method

Compressive Strength of Treated Soil

According to the results of several laboratory tests on core specimens collected from the tunnel face, the treated medium to fine sand has the following unconfined compressive strengths:

SODIUM SILICATE GROUT	COMPRESSIVE STRENGTH (MPa)		
	After 1 day	After 7 days	After 28 days
Sodium Bicorbonate as a reactor	0.5 to 0.7	0.5 to 0.8	0.4 to 0.7
Using Durcisseur as a reactor	1.0 to 1.4	1.1 to 1.5	0.8 to 1.4

From a stability point of view, the silicate grout has introduced adequate compressive strength to the medium to fine sand when using both Durcisseur and sodium bicarbonate. The injected grout seems to introduce strong cementitious bonds hours after application. The results of the laboratory testing further indicated very slight variation, if any, in shear strength of the grouted soil over a period of one month. A hand compressor was typically required to excavate the grouted medium to fine sand.

GENERAL OBSERVATIONS AND COMMENTS

The chemical grouting used on this project was generally successful in stabilizing the medium to fine sand and allows for the implementation of the tunnel construction method. However, from both technical and economic points of view, the following observations can be drawn regarding the injection procedures and materials used:

MGS Method:

- Adequate stability of tunnel crown could only be achieved if an excessive number of micropiles are installed. Supplemental support of steel/wood sheets had to be frequently placed against excavated tunnel crown and sides to ensure soil stability prior to placement of the initial lining.
- The gel time was difficult to control in the two-solution process. As such, effective mixing of the two chemicals in the subsurface soils was not obtained, resulting in loose zones where the presence of silicate grout was evident in the soil voids.

ITM Method:

- The one-solution process used in this method appeared suitable for the medium to fine sand with a permeability ranging between 5×10^{-2} and 5×10^{-3} cm/sec, hence confirming previously published data with respect to grout type versus soil permeability (reference 5). As expected, this grout mix was practically ineffective under the applied pressure in the fine silty sand where the permeability is less that 5×10^{-3} cm/sec.

- Effective grout permeation appears limited in the medium to fine sand regardless of the injection pressure used. A typical pressure ranging from 2.5 bars to 3.5 bars was applied to optimize the effect of the injected grout.
- The Durglass system was generally an effective means of injecting the chemical grout into the soil and provided adequate reinforcement for the relatively thin and sometimes non-uniform grouted hood. The fiberglass plates were crucial in providing additional reinforcement for the grouted hood due to its high bending properties. The Durglass system has no great advantage over other systems if the grouted hood is larger and more uniform.
- Several field trials are generally required to investigate actual grout performance in the specific soil formation to be treated. Injection pressure and the holes pattern may often need to be tempered/adjusted depending on the actual soil conditions. Hence, every effort should be made to investigate thoroughly the soil conditions ahead of the injection operation.

ACKNOWLEDGEMENTS

Thanks are due to Mr. Abed El-Ruhman El-Nahabi formerly of Impresem-Batco, JV, and Mr. Robert D. Edde of DST Consulting Engineers for their assistance in the laboratory testing for this study. Also thanks are due to Mr. Richard C. Leach, PE, and Mr. Timothy E. Metcalfe, CEG of GeoTek, Insite Inc. for their encouragement and technical review of this paper.

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Performance of Permanent Ground Anchors for Landslide Stabilization

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ABSTRACT

Permanent tieback anchors are used widely for stabilization of active landslides. The basic concept is that anchor forces provide resistance to landslide driving forces, thereby increasing the factor of safety against failure. Several types of structural reaction systems are used with anchors. In some cases, each anchor is attached to an individual concrete reaction panel. In other cases, a continuous reaction panel, or "waler wall", acts as a reaction beam for each row of anchors. Design variables that must be determined include anchor design loads, lock-off loads, anchor spacing, bond and unbonded lengths, dimensions of reaction panels, and others. Methods for selecting some of these variables are well-documented in FHWA publications and other sources, but for others, little guidance is currently available. For example, whether to use larger loads with greater spacing versus smaller loads and lower spacing is a design question left up to individual preference.

In this paper, several recent case histories in which anchor loads were monitored for 1 to 5 years after construction are reviewed and analyzed to assess the influence of various design parameters on system performance. Changes in anchor loads over time are evaluated in terms of design load magnitude, observed slope movements, and type of structural reaction system. The Deer Creek landslide stabilization, a Wyoming DOT project that is being monitored by the authors, is the primary focus of this review, but data from several other projects involving monitored anchor loads are included. Recommendations are given for selecting design variables to achieve more efficient and cost-effective anchored stabilization systems.

INTRODUCTION

Permanent ground anchors, or tiebacks, can be used to provide effective and economical methods for increasing the stability of active or potential landslides. Tieback anchors penetrate the failure surface and apply a resisting force directly to the sliding mass. Cost advantages are possible when comparing a tieback anchor system to methods such as concrete walls, buttress fills, reinforced earth walls, excavation, regrading, and relocation (Weatherby and Nicholson 1982). The suitability of an anchored system depends primarily on the geotechnical conditions, especially the strength and stiffness of the soil or rock into which the anchors will be bonded, as well as the properties of the sliding mass. In general, non-cohesive soils with a standard penetration test N-value greater than 10 or cohesive soils with undrained shear strength greater than 0.5 kN/m^2 are considered suitable for anchorage, as are virtually all types of rock. Field performance and proof load tests are used to verify the bond capacity of anchors installed under actual conditions.

Structural reaction systems against which the anchors are stressed may be distinguished on the basis of whether or not they penetrate the failure surface. Examples of systems that cross the failure surface would include anchored soldier piles, sheet piles, or diaphragm walls. In this case, the vertical component of the tieback load is assumed to be transmitted to the portion of the wall below the failure plane, and the embedded portion of the wall provides sliding resistance by development of passive earth pressure. The second type of system is one in which the anchors are stressed against structural members that don't penetrate the failure surface. Typical examples would include individual concrete anchor panels (one panel per anchor) or a continuous concrete beam for a single row of anchors (sometimes referred to as a waler wall). In this case, both the vertical and horizontal components of the anchor force are transmitted to the sliding mass. Also note that anchor forces are the only externally applied resisting forces in this type of system. While both systems are commonly used with success, this paper is focused on the latter type, in which the structural reaction member does not penetrate the failure surface.

Details of anchor design and construction are covered in several existing publications. The FHWA Geotechnical Engineering Circular No. 4, "*Ground Anchors and Anchored Systems*" (Sabatini et al, 1999) is an example of a design manual focused specifically on the use of ground anchors for highway applications, including landslide stabilization systems. However, little guidance is provided in choosing between the range of options available for selection of anchor design loads, spacing, and type of reaction system. It was observed by the authors on several slope stabilization projects that anchor lock-off loads were not sustained over time. This raises several questions pertaining to the cost-effectiveness of using relatively high anchor loads, long-term stability, control of ground movements, and others. The purpose of this paper is, therefore, to share the authors' experience and to examine several other documented case histories of anchored slope stabilization system that provide insight into the performance of such systems with regard to anchor design loads, lock-off loads, and whether a continuous or non-continuous structural reaction system is used.

DEER CREEK LANDSLIDE STABILIZATION

This landslide is located on US Highway 26/89 in the Snake River Canyon of northwestern Wyoming. The general subsurface profile consists of approximately 20 m of sandy clay with gravel and scattered boulders, underlain by very hard siltstone with interbedded shale and sandstone of the Cretaceous Bear River Formation. The soils are colluvial, residual, and man-made fill materials, all of which are derived from the Bear River Formation. Several other landslides occur in the canyon where the road intersects the Bear River Formation and soils derived from it, which in general are highly plastic and have low residual shear strength when saturated.

The Deer Creek slide posed several challenges to WYDOT. Several earlier attempts were made to stabilize the slide, including installation of a permeable drainage wall upslope of the landslide and placement of a toe berm. Neither of these provided an entirely satisfactory solution, with ground movements continuing to cause roadway settlement after these methods were employed.

Constructibility issues included the need to keep the roadway open during construction and minimal disturbance to the surrounding environment, which is adjacent to a national forest. The final configuration was also constrained by the need to minimize the visual effect, especially for recreational users of the Snake River. The stabilization system proposed by WYDOT Geology and Engineering personnel consisted of 139 high-capacity permanent ground anchors, each stressed against a single concrete reaction panel .

The system was designed in-house by WYDOT and called for four rows of tieback anchors (A through D) as shown in Figure 1. The design load was 2360 kN (530 kips) per anchor, based on analysis of slope stability using the computer program XSTABL. In the lower two rows, anchors are battered 30 degrees from horizontal while anchors in the upper rows have variable batter angles ranging between 15 and 20 degrees from horizontal. Each anchor was stressed and locked off against a structural reaction element consisting of a 3 m wide by 3.7 m high by 0.6 m thick (10 x 12 x 2 ft) reinforced concrete panel. A total of 139 anchors ranging in length from 28 m to 56 m (92 to 184 ft) were required. The design bond length of each anchor was 15 m, approximately the maximum length recommended for anchor load transfer in the FHWA ground anchor manual (Sabatini et al., 1999). The average ultimate bond stress in the anchor bond zone was presumed to be 840 kN/m^2 (17.5 ksf). This was based on rock type (siltstone) and unconfined compressive strengths of rock cores obtained from below the slide plane.

Construction began during the first week of March 2000 and involved the following sequence. At each row (A, B, C, D) a working bench was excavated. The heavily reinforced concrete panels were constructed as follows: timber forms were built on the working bench, reinforcing steel was placed in the forms, and concrete (34.5 MPa or 5,000 psi) cast into the forms. When

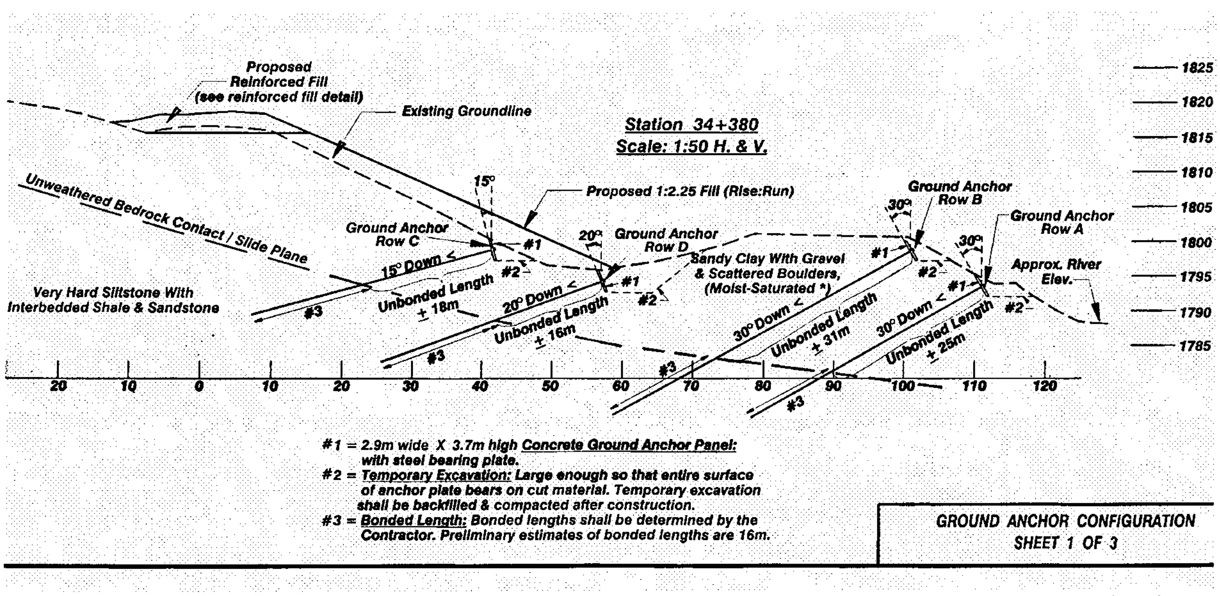

Figure 1. Cross-section with anchors and subsurface conditions, Deer Creek slide stabilization.

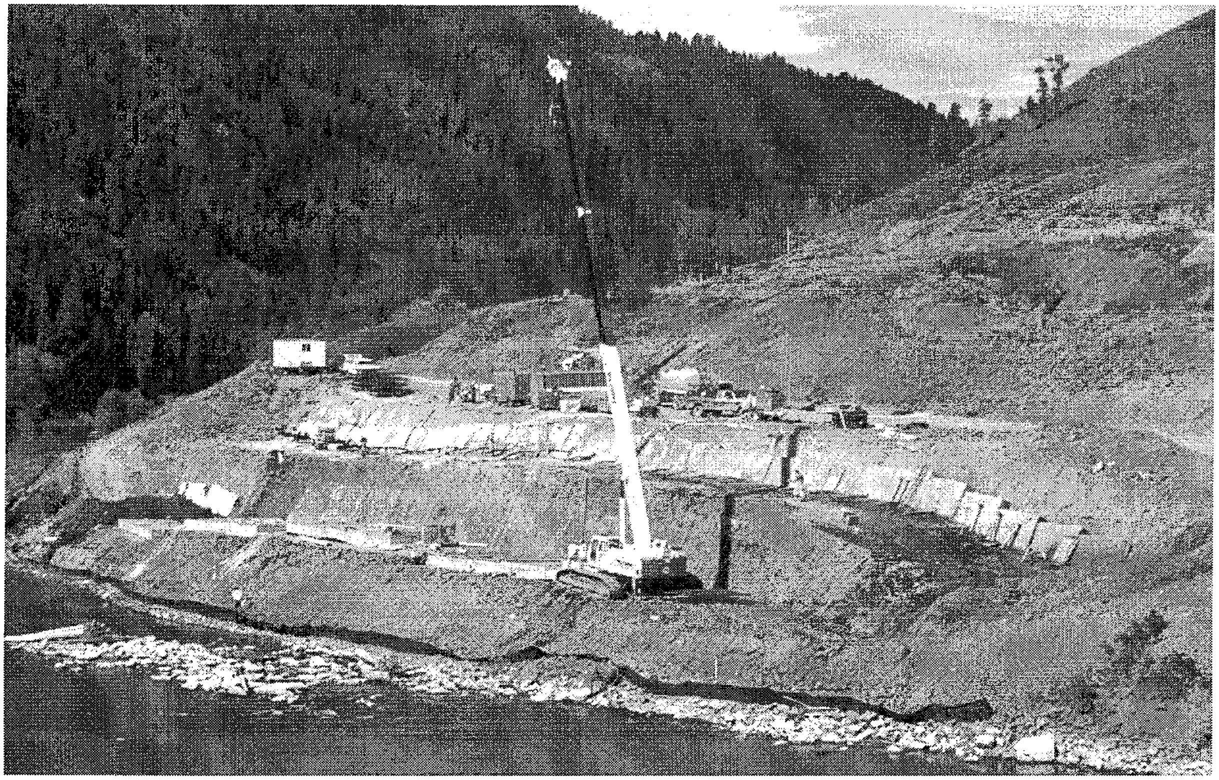

Figure 2. View of construction site showing cut benches and panels, Rows A and B.

the concrete had cured, forms were stripped and each panel was tilted into place against the cut (Fig 2). Anchors were installed through a 203-mm diameter hole in the center of each panel. Drilling was accomplished using a Klemm 806 drilling rig with dual rotary heads (Figure 3). The outer casing was 178-mm diameter and drilling through overburden soil was done using a cutting shoe on the end of the casing. A down-the-hole hammer attached to the inner rod was activated when cobbles were encountered and for drilling the 15-m bond length into hard siltstone, shale, and sandstone. Cuttings were air flushed.

Each anchor consisted of 18 strands (7-wire strand) with double corrosion protection, as recommended by Sabatini et al. (1999) for permanent applications. In the bond length this consisted of grout-encapsulated strands, a corrugated plastic sheath (PVC), and outer grout encapsulation. Along the unbonded length, corrosion protection consisted of a grease-filled sheath around each strand, a grout-filled smooth outer sheath of polyethylene encapsulating all 18 strands, and an outer layer of grout. Following tendon installation, the outer casing was removed while the hole was fully grouted. After curing, each anchor was load tested to 125% of design load then locked off at design load (2360 kN). Selected anchors in each row were subjected to a proof load test that required loading to 150% of design load.

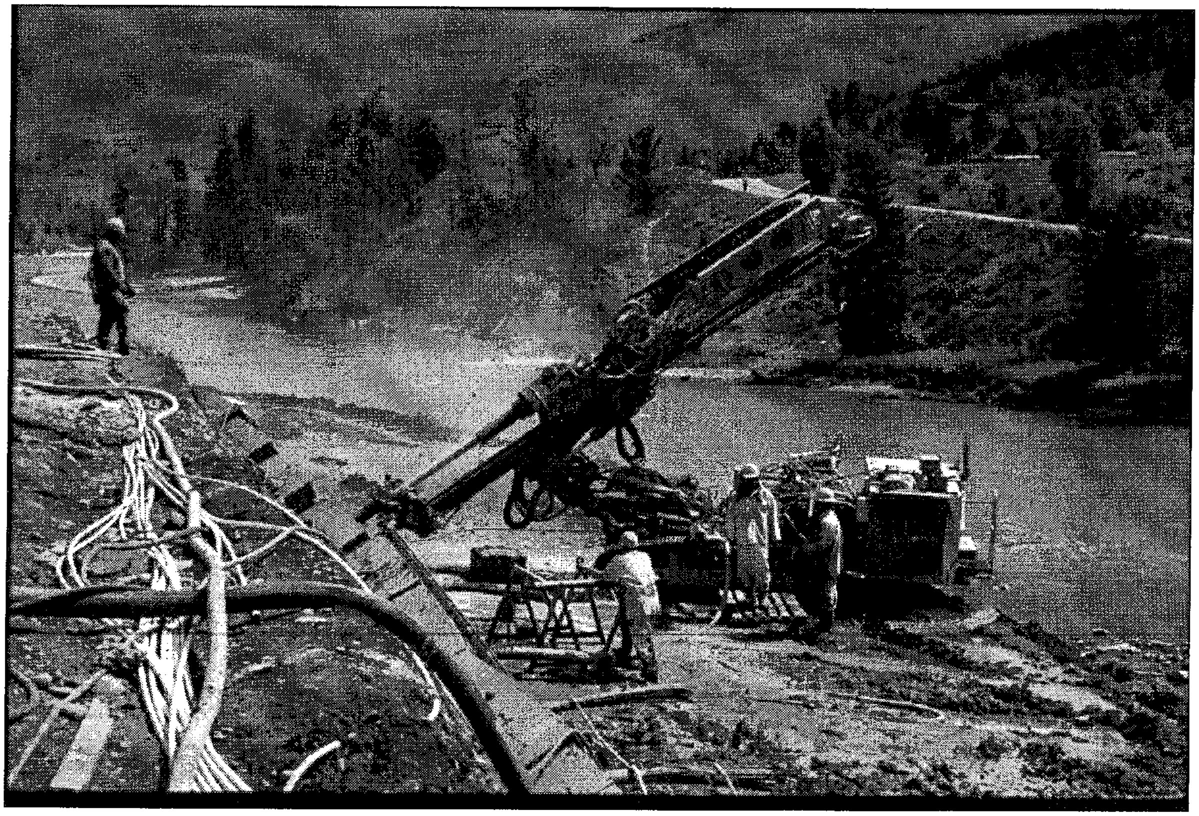

Figure 3. Drilling through anchor panels.

Field Observations, Anchor Loads

During anchor stressing of Row B, which was the first row of anchors to be installed and stressed and which is located in the lower part of the slope, it was observed that the anchor panels underwent large displacements and some anchor panels underwent large rotations. Some required re-setting of the stressing ram once or twice. The throw of the ram was 200 mm, giving some idea of the magnitude of panel settlement during anchor stressing. No measurements of actual panel deformation were made. In response to these large deformations, WYDOT relaxed the required test loads for both proof and performance tests to 100 percent of design load in selected anchors. The most significant problems associated with large deformation occurred at the western end of Row B, which is where the first anchors were installed. As anchor stressing progressed eastward in Row B, and for the other rows, the original test load specifications were met.

The soil profile in the vicinity of Row B where the large panel deformations occurred indicates the panels were bearing against soils of the toe berm which consisted of compacted fill that was placed approximately two years prior to anchor installation. These soils had visibly higher water contents than those further east and were relatively soft. During anchor stressing in Rows A, C and D it was observed that panel deformations were smaller than for Row B and also that the soils on which the panels were bearing appeared drier and stiffer than in Row B.

Three anchors in each row (12 anchors total) were instrumented with permanent load cells placed between the anchor bearing plate and the wedge plate. The compression load measured by the load cell was assumed to be the load transmitted between the anchor head and the bond zone of the anchor. Loads were measured immediately after anchor lock-off and twice per day since.

Figure 4 shows measured anchor load versus time, for one anchor from each of the four rows. All four anchors are located approximately in the middle of their respective rows. The anchor in Row B shows the behavior typical of anchors in this row, with the load dropping off rapidly following lock-off, but ultimately reaching a constant or slightly decreasing load within several months. The anchors in Rows A, C, and D show an initial decrease in load which then levels off. Table 1 summarizes the results of anchor load measurement for all twelve instrumented anchors.

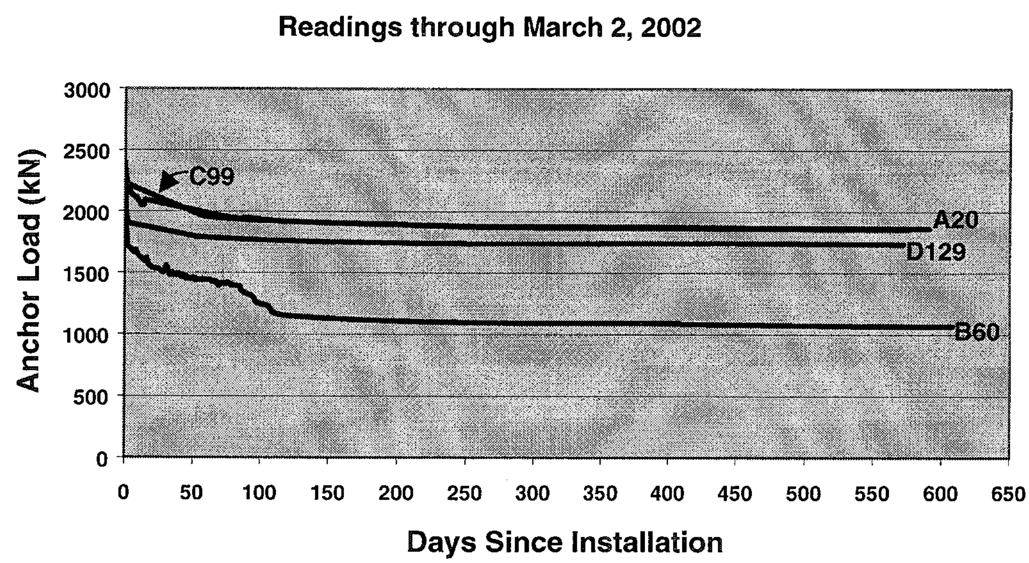

Figure 4. Anchor load variations with time, Deer Creek project.

Table 1. Summary of Anchor Load Measurements, Deer Creek Project.

Anchor ID	Initial Load (kN)	Load at Day 100 (kN)	Load at day 500 (kN)	% decrease after 100 days	% decrease after 500 days
A5	2469	1671	1654	32.3	33.0
A20	2393	1935	1868	19.1	21.9
A36	2197	1810	1753	17.6	20.2
B45	1880	1415	964	24.8	48.8
B60	1997	1242	1073	37.8	46.3
B75	2305	1187	1115	48.5	51.6
C84	2346	1405	1296	40.1	44.8
C99	2337	1927	1860	17.5	20.4
C115	2654	2089	2055	21.3	22.6
D122	2357	1628	1596	31.0	32.3
D129	2357	1767	1741	25.0	26.2
D137	2572	1835	1810	28.7	29.6

CASE HISTORIES OF ANCHORED SLOPE STABILIZATION

Three additional projects were identified in which permanent ground anchors used to stabilize active landslides were monitored. These three projects also incorporate structural reaction elements that don't cross the failure surface and so are similar to the Deer Creek project in that respect. Important aspects of monitored anchor loads for all four projects are summarized in Table 2, and each of the three additional case histories is summarized briefly below. For a full description and discussion of these case histories the reader is referred to Johnson (2002) or may contact the Wyoming DOT for the full research report to be published later this year.

Table 2. Summary of Anchor Loads for Four Case Histories.

Project	Total No. of Anchors	Design Load (DL)		Lock-Off Load			Avg Final Load	
	(instrumented)	per anchor (kN)	per meter of wall (kN/m)	target (kN)	measured (kN)	as % of DL	(kN)	as % of DL
Deer Creek	139 (12)	2360	787	2360	2322	98	1565	66
Stone Point	132 (12)	534	59	534	577	108	557	104
Grapevine	732 (5)	1490	814	1490	1499	101	1187	80
Via Cerro Rebal	1000 (46)	1334	888	761	761	57	905	68
		1512	1008	864	816	54	1096	72
		1624	1082	928	905	56	1085	67

The Stone Point Landslide is located in Spring Valley, near San Diego, CA. In 1987 this landslide caused damaging ground movements to structures in a residential development and a stabilization program was undertaken (Dames & Moore, 1995). The subsurface consists of compacted fill with rock facing underlain by slightly-weathered sandstone of the Sweetwater Formation and moderately-weathered volcanic rock of the Santiago Peaks Formation. The failure surface was modeled by the designers as being a circular surface through the fill and limited to materials above the bedrock. 132 anchors were installed in two rows, one-half and three-quarters of the distance up the slope respectively (Figure 5). Anchors were stressed against individual shotcrete anchor panels, are battered 30 degrees from horizontal, and bonded into the Sweetwater sandstone. Where the sandstone was absent or was highly weathered/fractured anchors were bonded in the volcanic rocks of the Santiago Peaks Formation (Dames & Moore, 2000). Twelve anchors are instrumented with load cells and the measurements summarized in Table 2 are for a period of 1,500 days (approximately 4 years). Seven inclinometer casings are installed in the anchored portion of this slope. Of these, five show zero movement since the

Figure 5. Construction of anchors and panels, Stone Point project. (photo courtesy of URS)

anchors were installed and two show minor movement on the order of 2-3 mm. These measurements indicate the anchored stabilization system has been successful in controlling the Stone Point slide.

The Grapevine Landslide is located in a cut slope above I-10 on the west flank of Grapevine Peak in the Tehachapi Mountains, CA. This project is described in Caltrans reports by Nguyen (1995) and Garofalo (1998). The cut slope is 146 m high at 1.5H:1V. The subsurface consists predominantly of a Mesozoic granitic formation that is highly weathered and fractured. In much of the slide area this material has decomposed into residual soil with high clay content (Nguyen, 1995). The Caltrans design called for 732 anchors divided between six rows. Anchors of each row are stressed against a single shotcrete waler wall (Figure 6) at an anchor spacing of 1.8 m (6 ft). Six anchors, one in each row, are instrumented with permanent load cells. Load cell measurements reported in Table 2 are for a period of 2,000 days (approximately 5 1/2 years). Of the six instrumented load cells, five appear to be functioning properly (Garofalo, 1998). Inclinometer measurements obtained since the anchors were installed show essentially no movement, indicating that the anchored system has been successful in controlling ground movements (Garofalo, 1998).

The Via Cerro Rebal Landslide occurred in 1998 adjacent to a residential area of San Juan Capistrano, CA. The stabilization project is described by Cornelius and Lee (2000) and involved installation of 1,000 permanent ground anchors. Anchors are distributed in 5 levels with 3 rows

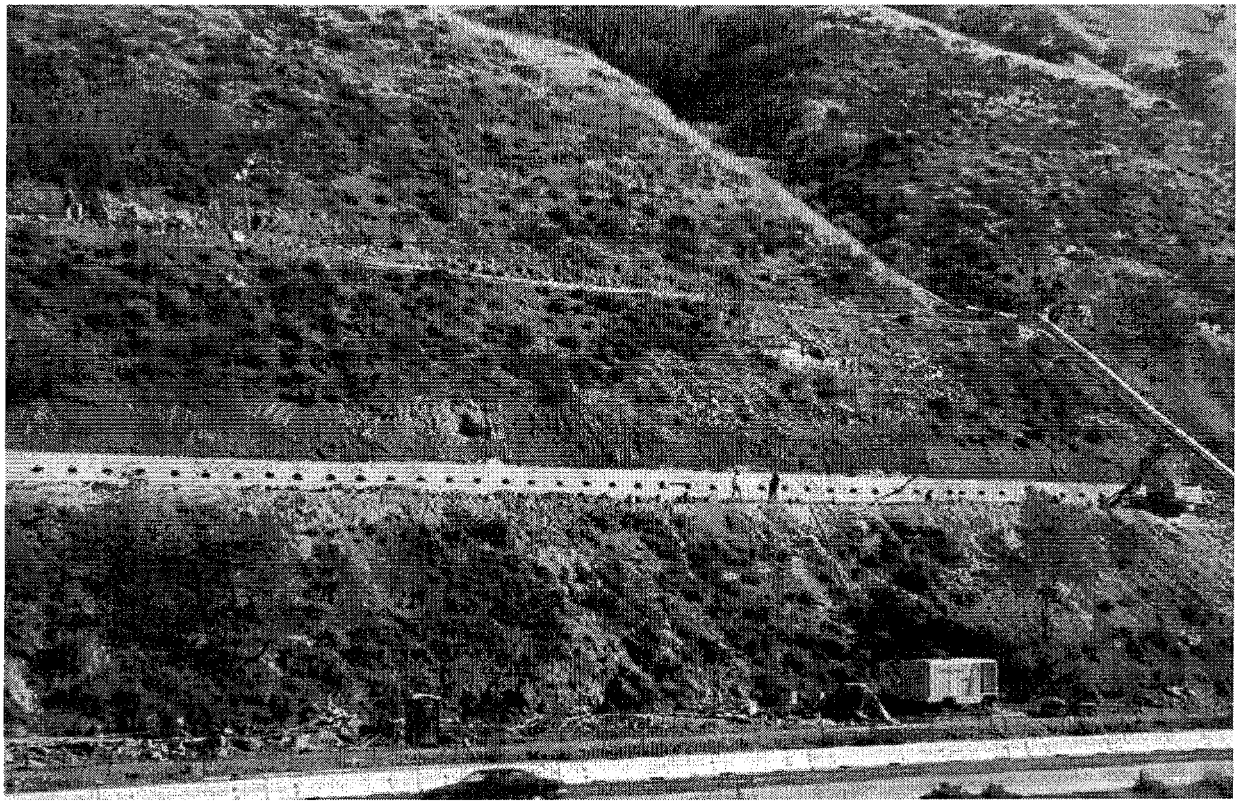

Figure 6. Grapevine project during construction. (photo courtesy of Contech Systems Ltd)

per level. Each row has a continuous waler wall against which anchors were stressed. It is noted that the reason cited for not using individual anchor panels was "low bearing capacity of the soils" (Cornelius and Lee, 2000). Three different design loads were used, as shown in Table 2 (1334 kN, 1512 kN, and 1624 kN). Forty-six anchors are instrumented with permanent load cells and the monitoring results available to the authors and summarized in Table 2 are for a period of 400 days (approximately 1 year) since lockoff. Stability of the slope was analyzed using a limit equilibrium analysis and anchor design loads were determined by requiring a target factor of safety of 1.5. The approach taken on this project for stressing the anchors differs from the other three projects. All anchors were locked off at a load such that the design load, as determined from limit equilibrium slope stability analyses, is 1.75 times the lock-off load. This corresponds to a target lock-off load that is 57 percent of design load. The assumption is that anchor design load will be mobilized when (and if) the slide moves. Figure 7 shows typical change in anchor load over time. No measurements of slope movement are available to the authors at this time.

DISCUSSION OF CASE HISTORIES

As presented in Table 2, the average measured anchor load at Deer Creek (for 12 instrumented anchors) after 600 days of monitoring is 66 percent of the design load. Target lock-off load was 100 percent of design load. However, as noted earlier, there was significantly different behavior observed in Row B compared to the anchors in the other three rows. The average long-term load

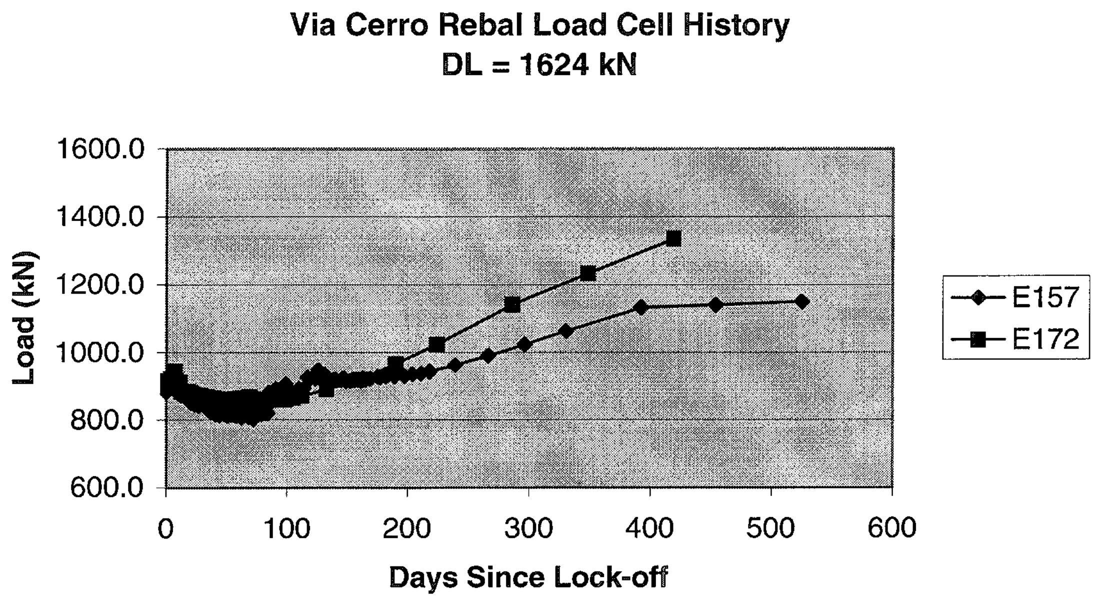

Figure 7. Typical change in anchor loads at Via Cerro Rebal project.

in Row B, based on measurements from three anchors, is 45 percent of the design load. The average of the other nine anchors (Rows A, C, and D) is 74 percent of the design load. The large decrease in load for the anchors in Row B is attributed to softer, more compressible soils underlying the anchor panels in Row B and is consistent with the large settlements observed during construction when load testing and stressing the anchors in Row B. In the following paragraphs, design, construction, and observed performance of the Deer Creek stabilization system are compared to each of the other case histories.

At Stone Point, the most significant design and construction difference is the magnitude of anchor load. Referring to Table 2, both load per anchor, as well as load per unit length of wall, are much lower at Stone Point than at Deer Creek. Stone Point is the only other project beside Deer Creek that utilized individual anchor panels. The target lock-off load at Stone Point was 100 percent of design load, although measured values average slightly higher at 108 percent of design load. Long-term loads show minimal change, averaging 104 percent of design load. Comparing the results of Deer Creek and Stone Point suggests that the use of smaller design loads reduces the amount of load decrease over time, perhaps providing more efficient use of anchors. A higher number of anchors would, however, increase the cost.

The Grapevine slide stabilization is similar to Deer Creek in the magnitude of design load per meter of wall (814 kN/m compared to 787 kN/m at Deer Creek), even though the magnitude of load per anchor is smaller (1490 kN compared to 2360 kN at Deer Creek). Anchor spacing at Grapevine was 1.8 m compared to 3 m at Deer Creek. The other significant difference is the use of a continuous shotcrete waler at Grapevine compared to individual anchor panels at Deer Creek. The average long-term load at Grapevine is 80 percent of design load, compared to 74 percent of design load at Deer Creek (excluding Row B). While this is not a large difference, it may suggest that the use of more anchors at a smaller spacing and lower load per anchor, and/or the use of a continuous waler instead of individual anchor panels, may reduce the amount of load relaxation with time.

Via Cerro Rebal is similar to Deer Creek and Grapevine in the magnitude of design load per meter of wall. Design loads are 888 kN/m to 1082 kN/m, higher than at Deer Creek, but again, load per anchor is significantly less (1334 kN to 1624 kN compared to 2360 kN at Deer Creek) because of the lower anchor spacing of 1.5 m. Via Cerro Rebal also was built using continuous waler walls instead of individual anchor panels. The most significant difference is the approach to lock-off load. As described above, anchors were locked-off at 57 percent of design load. Anchor loads increased following construction, presumably as downslope movements occurred. The average long-term load is 69 percent of design load (for 46 instrumented anchors) approximately one year after construction. As can be seen in Figure 7, some anchor loads appear to be approaching a higher steady-state load. The most obvious aspect of performance is that average anchor loads appear to be coming to equilibrium at levels that are in a range similar to those at Deer Creek and Grapevine when expressed as a percentage of design load. This may indicate that long-term loads may eventually equilibrate within a narrow range regardless of whether anchors are locked-off at the design load or below the design load.

SUMMARY AND CONCLUSIONS

Although many projects involving the use of permanent ground anchors are instrumented for measuring long-term loads, very few measurements have been published. Practicing engineers, in particular in the state DOT sector, don't have a realistic basis for predicting long-term loads in anchors used for slope stabilization. In this paper an attempt is made to examine long-term loads in anchors used in slope stabilization systems that were designed with structural reaction elements that do not cross the failure surface. For all four projects, the designers used traditional limit equilibrium slope stability analyses to determine the required anchor loads to achieve a target global factor of safety. This is the approach described in the FHWA publication on ground anchors currently being used by many state DOT personnel (Sabatini et al., 1999). All four projects are reported as being successful in the sense that ground movements since the end of construction appear to be zero or small enough to be of no concern. Based on the four projects, all of which are reasonably well-documented, the following observations are made:

1. Anchor loads can be expected to decrease over time, relative to design load. Of the four cases examined, three exhibit anchor loads that are in the range of 69 percent to 80 percent of design load. For these three cases, the design load expressed in load per unit length of wall was in the range of 787 – 1082 kN/m. For the single case in which the lock-off load has remained essentially constant over time (4 years), the design load of 59 kN/m is much lower than for the other projects. This observation suggests an empirical relationship between magnitude of design load and load relaxation over time, but the data are insufficient to establish the relationship quantitatively. A single case history (Via Cerro Rebal) in which anchors were locked-off at 57 percent of design load suggests that long-term load per unit length of wall is independent of initial lock-off load. It is noted that locking off anchors below the design load may allow additional slope movement before the anchor loads reach long-term equilibrium.

2. The experience of Wyoming DOT in Row B of the Deer Creek project illustrates the importance of evaluating bearing capacity and settlement characteristics of reaction structures throughout the project, especially if individual concrete reaction panels are under consideration. On this project, large settlement and rotation of the panels during anchor stressing made it necessary to change the proof and performance load test criteria. Long-term anchor loads in Row B are 45 percent of design load, which is clearly below the range expected, based on data from Deer Creek and the other projects. The overall system at Deer Creek appears to be stable and slope movements have been controlled, which illustrates that anchored slope stabilization systems can perform satisfactorily even if all anchors are not carrying the expected level of load. The continuous waler wall at Via Cerro Rebal was selected because of low bearing capacity soils. The following design approach is suggested for areas with questionable bearing capacity or potential settlement problems: (*i*) lower anchor design loads, (*ii*) lock-off loads less than design load, and (*iii*) use of continuous reaction structures such as shotcrete waler walls.

3. Decreases in anchor load are time dependent, for example as shown in Figure 4. Most likely this occurs as the reaction element undergoes settlement, allowing the unbonded portion of the anchor to shorten. Analysis of load-settlement-time behavior of the reaction element may make it possible to predict long-term anchor loads for specific site conditions. This type of analysis would require as input the stress-strain and consolidation properties of the soils on which the reaction panel is bearing. Further research on this topic is needed but may provide useful prediction tools for designers.

ACKNOWLEDGMENTS

The authors wish to thank Mr. Ronald Tanenbaum (Stone Point), Mr. Gary Garofalo, (Grapevine), and Mr. David Lee (Via Cerro Rebal) for sharing information. The authors also wish to thank the Wyoming DOT for their support, especially Mr. Mike Hager, Chief Engineering Geologist, and Senior Engineering Geologists Mr. Jim Dahill and Mr. Mark Falk.

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