

Thesis
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University of Nevada

Reno

Assessment of Rock Bolt Systems For
Underground Nuclear Waste Storage

A thesis submitted in partial fulfillment of the
requirements for the degree of
Master of Science in Geological Engineering

by

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December 1993

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ABSTRACT

A rock bolt performance study was done to determine which bolt systems should be looked at as possible long term underground support for the waste emplacement areas of the Yucca Mountain site.

A literature search was undertaken to develop a data base on Yucca Mountain's site characterization of waste emplacement areas and rock bolt systems and their applications.

An investigation of the effectiveness of rock bolt systems at actual underground mine sites was then addressed. The four sites visited had some similarities in ground conditions to that expected at the emplacement areas. Recommendations for applicable rock bolts systems were made once the data analysis was complete.

In conclusion, passive, flexible, corrosion resistant rock bolt systems should be further tested for possible use in the areas of waste storage. These systems will have relatively large displacement capacities in order to adapt to the possible thermal induced rock mass deformations.

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CHAPTER I

INTRODUCTION

An in-depth review of existing rock bolt systems and their applicability to the support requirements for Yucca Mountain's nuclear waste storage areas was undertaken. The scope of the study was to evaluate the performance of currently available rock bolt systems under varying conditions, so preliminary recommendations of applicable bolts systems can be made. The support systems for the Yucca Mountain Project need to be flexible and fully functional for rock temperatures approaching 200 degrees celsius. The temperature is a function of the thermal pulse induced from storage of waste canisters. The elevated temperatures may increase corrosion, decrease anchorage strengths and increase stress loading on the support systems. Although many studies have been conducted on bolt performance under ambient temperatures, very limited information exists on the reactions of rock bolt systems exposed to temperatures above 40 degrees celsius.

1.1 Previous Work

The most recent investigation of comparative studies on rock support systems for a proposed repository in volcanic tuff has been performed by the USBM (1987). Rock bolts and their probable performance under the expected conditions of the repository were two

of the main focuses of the study. Actual investigations at the test site have been reported by Zimmerman et al, 1988 on evaluations of large scale rock testing undertaken for the G-Tunnel Welded Tuff Mining Experiment. Preliminary rock support recommendations were made after evaluations of the study was complete. Specific recommendations in preliminary ground support parameters to be used at Yucca Mountain have additionally been suggested by Hardy and Bauer, 1991. However, all reports suffered from having not visited specific mines or underground excavations to evaluate rock bolts placed in areas of large rock deformation.

1.2 Description of Research

The review of rock bolt systems included a literature search and underground site visits to active operations. The literature search involved gathering data on Yucca Mountain's site characterization, correspondence with rock bolt manufacturing companies and the critical review of many journal and symposium articles and books written on rock bolts. The site characterization of Yucca Mountain has been an ongoing process for a decade since it was established as a possible repository site. Once the base line of support requirements for the given area were established, the bolt performance portion of the literature search was begun. The rock bolt companies were helpful in providing information on their existing and newly developed rock bolts.

The latter part of the study involved actual field observations of emplaced bolt systems. Four sites were visited. New Mexico's Waste Isolation Pilot Plant (WIPP) for

Department of Defense nuclear waste, and the Kidd Creek, Detour Lake and Dome mines in Canada's Ontario Province. These underground operations were chosen because of their known stress induced rock displacements, the variety of bolts systems installed, the continuous monitoring of stopes and the wealth of geotechnical data assembled at each site. The rock types observed were rock salt at the WIPP site to metamorphosed volcanics at the three mines visited in Ontario. The depths of the operations ranged from 600m to 2000m. The in situ stresses at the WIPP site were hydrostatic while the horizontal stress are typically one and one half to two times the vertical stress in the mines visited in Ontario.

1.3 Research Direction

The Topapah Springs Tuff is approximately 60% saturated in the proposed storage areas, it is conceivable that water migration is possible. This migration could increase the corrosion potential of unprotected steel. Therefore the proposed bolts systems should be treated or constructed of corrosion resistant material, mainly either galvanized or preferably made from stainless steel. Most epoxy and polyester resins are suspected of creep under an elevated temperature environment, though some new types may perform better. The new fiberglass bolts, which are cemented with resin, require long term testing. The cement grouts, mainly the portland and calcium-aluminate based ones, become dehydrated at elevated temperatures, but maintain approximately 80% of their strength under the proposed conditions.

The proposed increased rock bolt stress, due to thermal loading, requires a need for more

flexibility in rock bolt systems. The additional rock bolt stress can be relieved by using a bolt system that yields, either by slowly shearing the steel with a specialized nut or by averaging the rock strains over most of the bolt length which reduces the effect of local high strains. It would be helpful if the Thermal Expansion Coefficients of the Topopah Springs tuff and rock bolt systems were similar. Another parameter that should be considered is increasing the length of the bolt system, so the strains can be accommodated over a larger portion of material. Given the foregoing requirements for suitable bolt systems, the research objective was in developing a data base with both theoretical and practical considerations.

The project was initiated by the U.S. government in 1977 to investigate the geologic and hydrogeologic conditions at the Nevada Test Site. A major objective was to determine the geologic and hydrogeologic conditions of the area. This was done by studying the geologic and hydrogeologic conditions of the area. The project was completed in 1987.

Yucca Mountain was designated by Congress as the only site to be studied for a geologic repository in 1987. The administration has certified since that time, it will be determine the suitability of Yucca Mountain as geologic repository.

2.2 Location

Yucca Mountain is located in Nye County, Nevada, about 100 miles northwest of Las Vegas (Figure 2.1). The locale is situated on the southeast corner of the Nevada Test Site, which is included within the Nevada Air Force Range.

CHAPTER II

A REVIEW OF YUCCA MOUNTAIN'S SITE CHARACTERIZATION

2.1 Introduction

Since the end of World War II, the Nevada Test Site has been withdrawn from public use for enhancement of nuclear power. Because of this, the U.S. government in 1977 contemplated placing a high level nuclear waste repository at the Nevada Test Site.

A major technical considerations was because of its arid environment, no surface water flow, deep groundwater levels, and long distances for groundwater to flow for discharge and the zeolite geochemical characteristics of the tuffs located in the area.

Yucca Mountain was designated by Congress as the only site to be studied for a geologic repository in 1987. Site characterization has continued since this time, in order to determine the suitability of Yucca Mountain as geologic repository.

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2.3 General Description

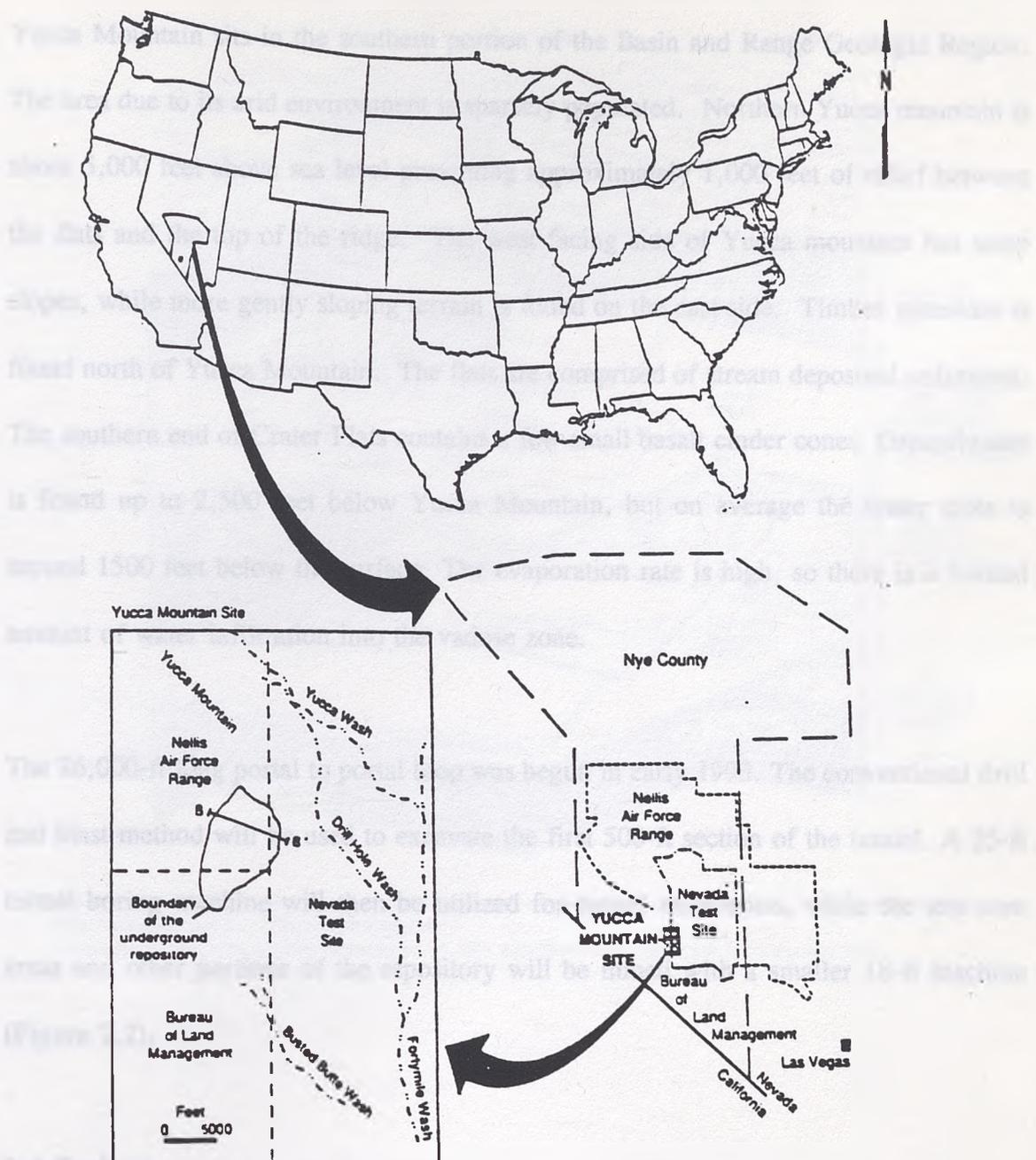


Figure 2.1 - Location of the Yucca Mountain in southern Nevada. The line labeled B-B' marks the location of the cross section in Figure 2.4 (DOE/RW-0198, 1988)

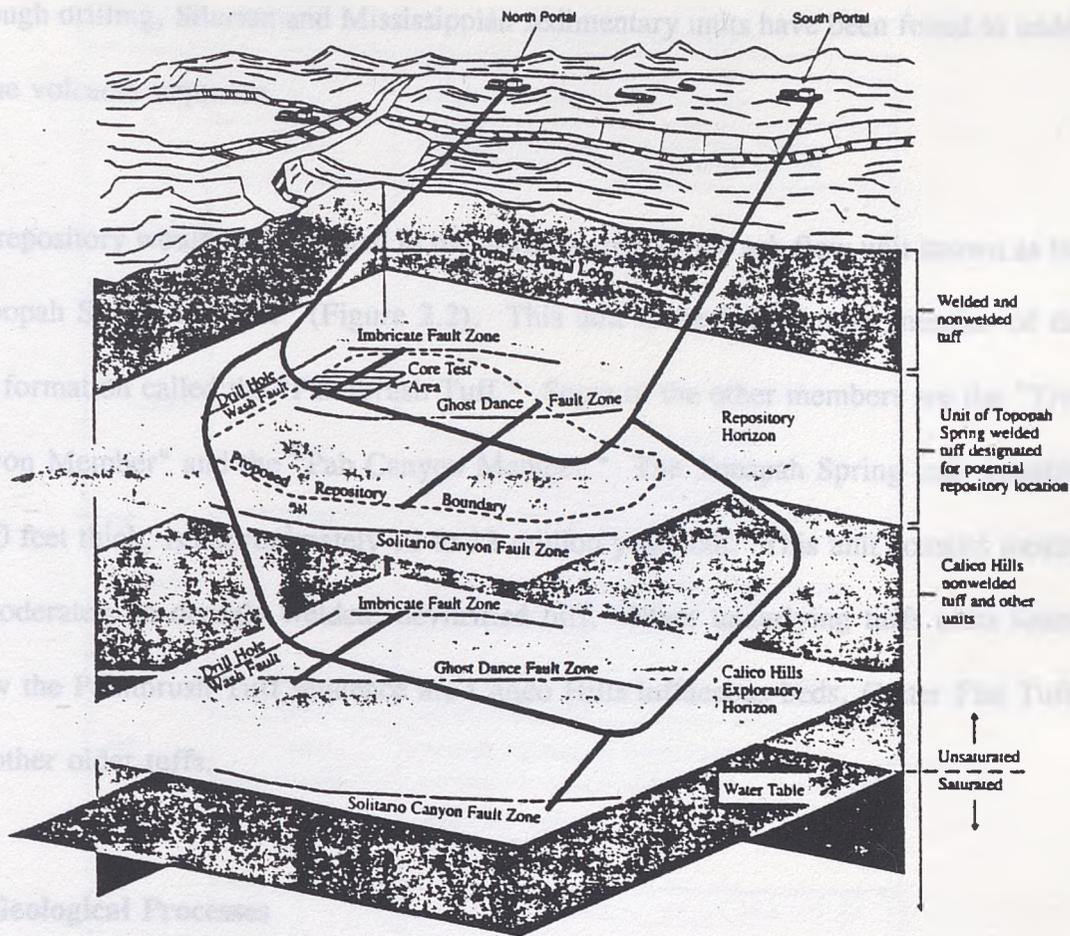
2.3 General Description

Yucca Mountain sits in the southern portion of the Basin and Range Geologic Region. The area due to its arid environment is sparsely populated. Northern Yucca mountain is about 5,000 feet above sea level presenting approximately 1,000 feet of relief between the flats and the top of the ridge. The west facing side of Yucca mountain has steep slopes, while more gently sloping terrain is found on the east side. Timber mountain is found north of Yucca Mountain. The flats are comprised of stream deposited sediments. The southern end of Crater Flats contains a few small basalt cinder cones. Groundwater is found up to 2,500 feet below Yucca Mountain, but on average the water table is around 1500 feet below the surface. The evaporation rate is high, so there is a limited amount of water infiltration into the vadose zone.

The 26,000-ft long portal to portal loop was begun in early 1993. The conventional drill and blast method will be used to excavate the first 500-ft section of the tunnel. A 25-ft tunnel boring machine will then be utilized for tunnel excavation, while the test core areas and other portions of the repository will be mined with a smaller 18-ft machine (Figure 2.2).

2.4 Geology

The majority of the rocks found in the Yucca Mountain region are silicic volcanic in nature. This volcanic sequence ranges from 3,000 to 10,000 feet thick, and are



Note: Configuration of fault zones at depth is inferred.

Source: Presentation to the NWTRB, September 18-19, 1991. RERGBV5 P.125. NWTRB/9-18/19-91 (RSN 1991).

Figure 2.2 - Presentation of the Yucca Mountain geologic block displaying currently proposed exploratory tunneling and the possible location of the repository (NWTRB, 1993)

mostly welded and non-welded ash flow and air fall tuffs (Figure 2.3). Some volcanic flows and breccias are found under the surface in the north Yucca Mountain area only. Through drilling, Silurian and Mississippian sedimentary units have been found to underlie the volcanic sequence.

The repository would be excavated in the lower portion of the ash flow unit known as the "Topopah Spring member" (Figure 2.2). This unit is the lowest most member of the rock formation called the "Paintbrush Tuff." Some of the other members are the "Tiva Canyon Member" and the "Pah Canyon Member." The Topopah Spring unit, roughly 1,100 feet thick, is approximately 12 to 13 million years old. This unit consists mostly of moderately to densely welded, devitrified tuff. Other underlying tuffs units found below the Paintbrush Tuff sequence are Calico Hills tuffaceous beds, Crater Flat Tuff, and other older tuffs.

2.5 Geological Processes

The site characterization addresses three major areas, volcanic activity, faulting and seismicity.

Volcanic Activity

In the southern Great Basin there has been volcanic activity since Cenozoic time (approximately 66 million years ago). The character of the volcanism ranged from

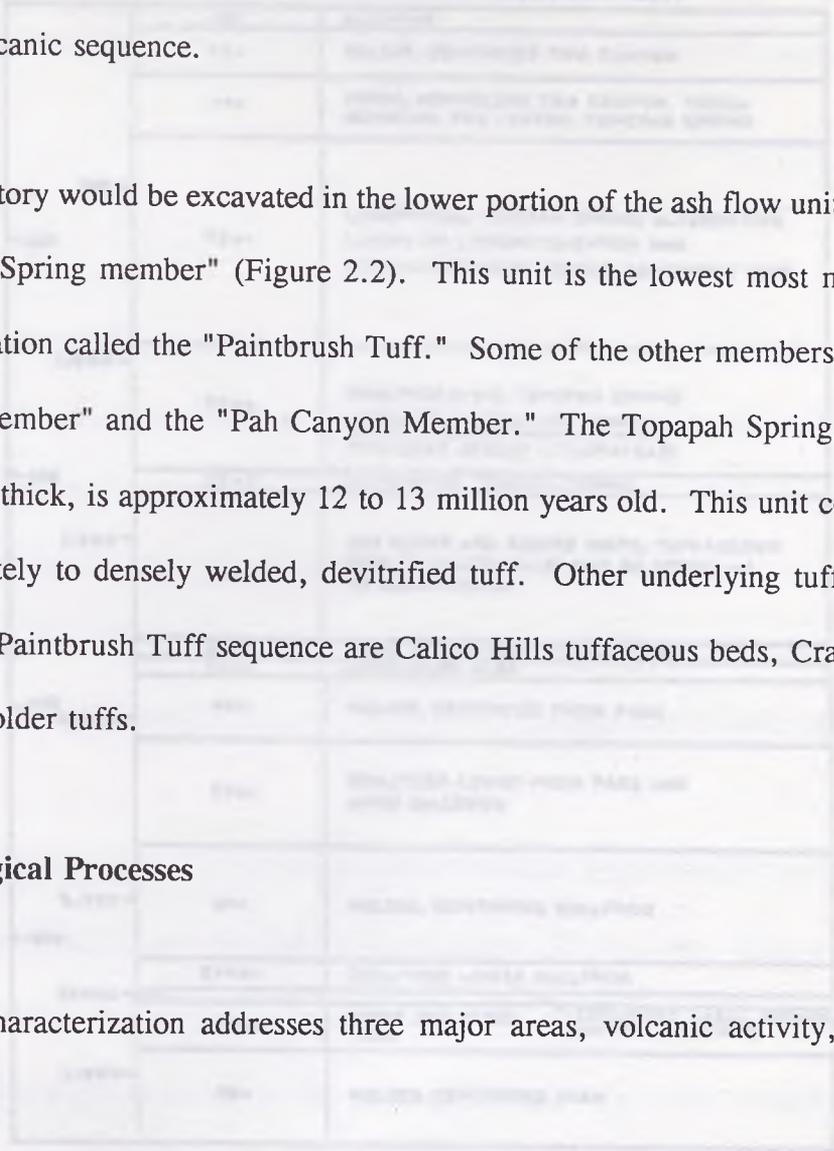


Figure 2.3 - Partial geological cross-section of Yucca Mountain (USGS W-4199, 1980)

DEPTH ft	THERMAL/ MECHANICAL UNIT	LITHOLOGIC EQUIVALENT
	UO	ALLUVIUM
	TCw	WELDED, DEVITRIFIED TIVA CANYON
	PTn	VITRIC, NONWELDED TIVA CANYON, YUCCA MOUNTAIN, PAH CANYON, TOPOPAH SPRING
500 -200	TSw1	LITHOPHYSAL TOPOPAH SPRING: ALTERNATING LAYERS OF LITHOPHYSAE-RICH AND LITHOPHYSAE-POOR WELDED, DEVITRIFIED TUFF
1,000 -400	TSw2	NONLITHOPHYSAL TOPOPAH SPRING POTENTIAL REPOSITORY HORIZON (CONTAINS SPARSE LITHOPHYSAE)
	TSw3	VITROPHYRE, TOPOPAH SPRING
1,500	CHn1	ASH FLOWS AND BEDDED UNITS, TUFFACEOUS BEDS OF CALICO HILLS: MAY BE VITRIC (v) OR ZEOLITIZED (z)
	CHn2	BASAL BEDDED UNIT OF CALICO HILLS
	CHn3	UPPER PROW PASS
2,000 -800	PPw	WELDED, DEVITRIFIED PROW PASS
	CFUn	ZEOLITIZED LOWER PROW PASS AND UPPER BULLFROG
2,500 -800	BFw	WELDED, DEVITRIFIED BULLFROG
	CFMn1	ZEOLITIZED LOWER BULLFROG
CFMn2	CFMn3	UPPER ZEOLITIZED TRAM ZEOLITIZED BASAL BEDDED UNIT OF BULLFROG
3,000	TRw	WELDED, DEVITRIFIED TRAM

Figure 2.3 - Thermal/mechanical stratigraphy found at Yucca Mountain (DOE/RW-0199, 1988)

older explosive silicic to more recent (8 to 9 million years to present) quiescent basaltic. Magma production seems to have tapered off since 4 million years ago with possibly the most recent activity being at the southern edge of Crater Flats. The Lathrop Wells Cinder Cones age is not certain but may have formed as little as 20,000 years ago (DOE, 1988).

Faulting

Faulting in the Great Basin can be categorized in overlapping phases during the last 11 million years. The first, occurring 7 to 11 million years ago, is extensional faulting which has been attributed to the more violent silicic volcanism.

Basin and Range faulting (7 million years to present) is the second, younger phase. This phase has been theorized to be from right lateral movement strike slip caused by the differences in motion of the Pacific Crustal Plate and the North American plate.

The majority of faults at Yucca Mountain are the high angle normal type, related to the formation of calderas and the basin and range type (Figure 2.4). Most of the lithologic units have been gently tilted to the east 5 to 10 degrees. After rotation, the north trending high angle faults offset the units and created large north trending blocks.

Most of the faults within the repository area have been mapped as 15 feet or less of

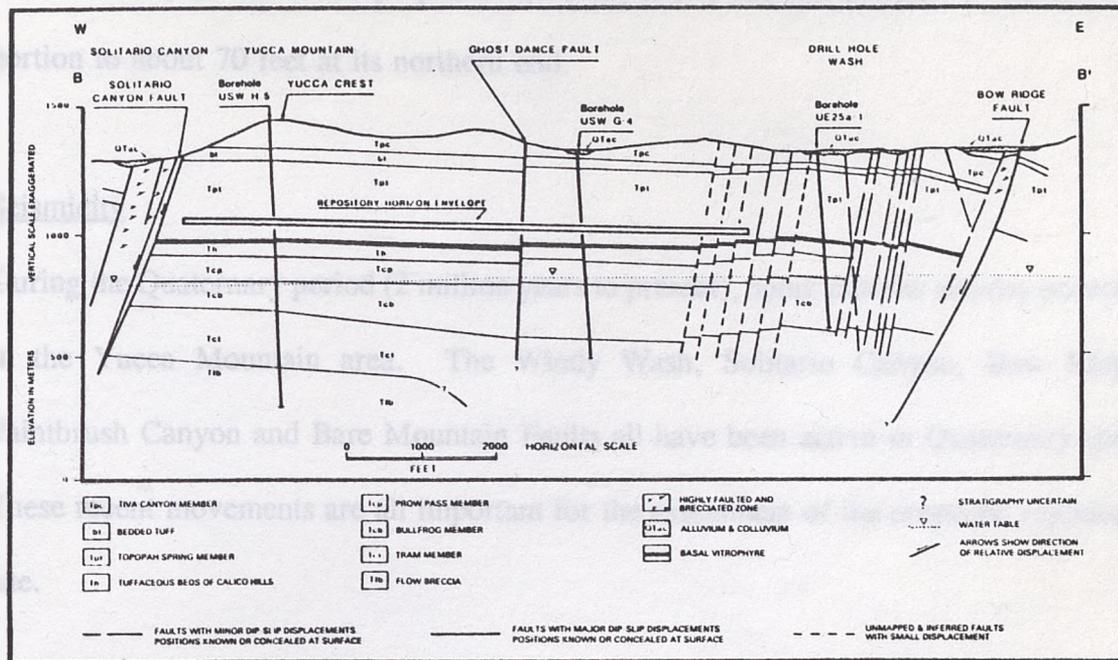


Figure 2.4 - East-west geologic cross section for the Yucca Mountain site. Figure 2.1 presents the location of B-B' (DOE/RW-0198, 1988)

vertical offset. The Ghost Dance Fault, found in the middle of the proposed repository site, on the other hand shows offset of approximately 125 feet. The Solitario Canyon Fault located just west of the proposed repository shows 700 feet of offset in the southern portion to about 70 feet at its northern end.

Seismicity

During the Quaternary period (2 million years to present), some tectonic activity occurred in the Yucca Mountain area. The Windy Wash, Solitario Canyon, Bow Ridge, Paintbrush Canyon and Bare Mountain Faults all have been active in Quaternary time. These recent movements are all important for the assessment of the proposed repository site.

The Nevada-California seismic belt lies 100 miles west of Yucca Mountain and the Intermountain seismic belt is 150 miles northeast of Yucca Mountain (Figures 2.5 and 2.6). Although the Yucca Mountain area has remained relatively seismically quiet since records have been kept, approximately 150 years, eight or nine earthquakes of 6.5 magnitude or larger have occurred within 250 miles. Six were in the Nevada-California seismic belt and the other two were on or near the San Andreas Fault. The nearest major recorded earthquake was 90 miles west of Yucca Mountain at Owens Valley in 1872 and was estimated at 8.25 in magnitude. In 1992, the Little Skull Mountain earthquake (Magnitude 5.4) caused an estimated 1 million dollars damage to buildings at the Nevada Test Site.

Although the Yucca Mountain area has been tectonically quiet in comparison to other portions of the Great Basin, the proximity to major seismic belts suggests a good chance of above average slip over the next 10,000 years. Obviously the repository if built will be designed for peak ground acceleration.



Figure 2.3- Location of earthquakes that occurred in the southwestern United States with a magnitude of 5 or more, between 1968 and 1975. The circles represent radii of about 60 and 250 miles which are centered around the Yucca Mountain site. The map displays the locations of major ($M = 4.5$ or more) historical earthquakes (DOE/BW-0010, 1983)

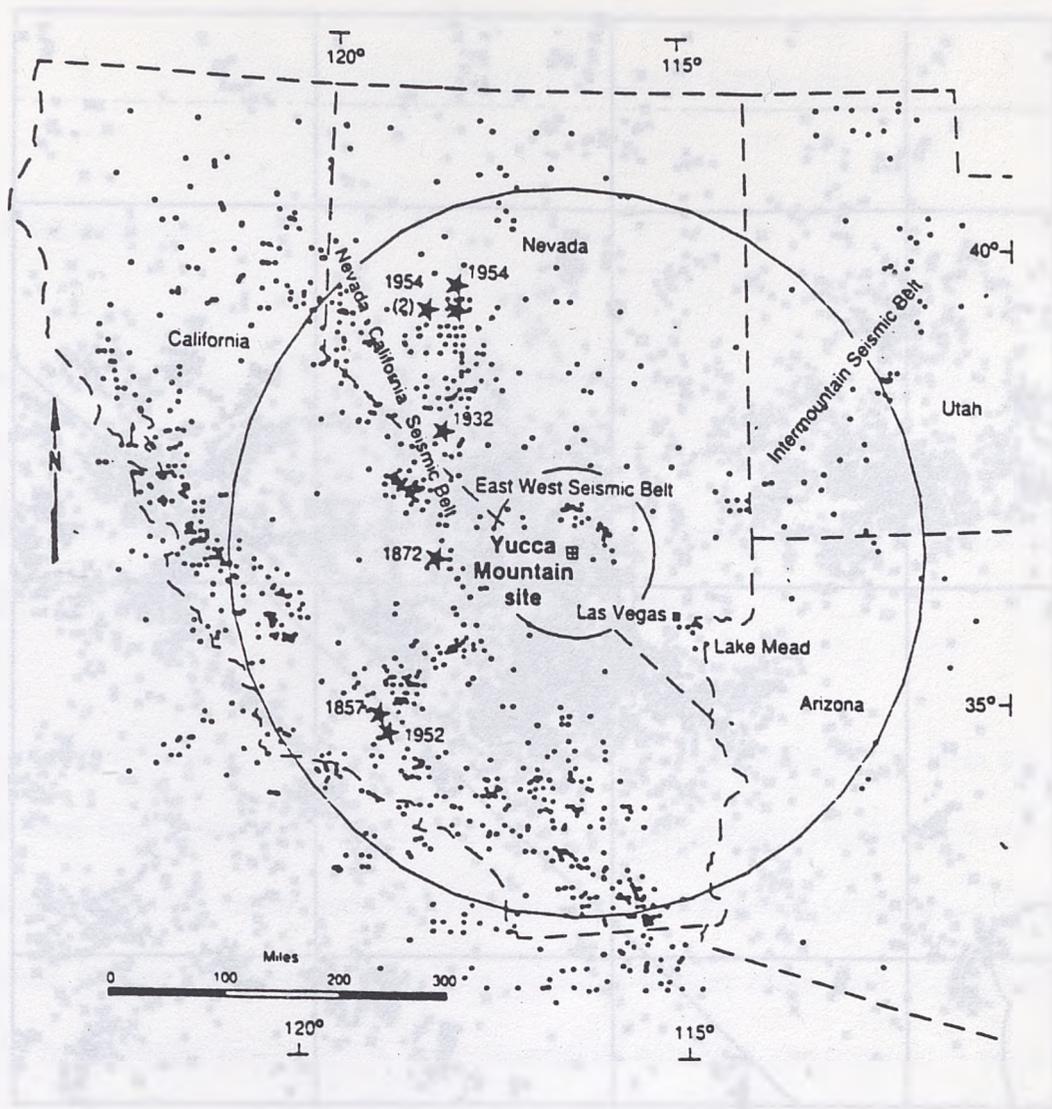


Figure 2.4 - An updated earthquake event location map for the southern great basin. The Nevada Test Site boundary outline is located in the middle of the figure (Siggerson, 1993)

Figure 2.5 - Location of earthquakes that occurred in the southwestern United States with a magnitude of 4 or more, between 1969 and 1978. The circles represent radii of about 60 and 250 miles which are centered around the Yucca Mountain site. The stars display the locations of major ($M = 6.5$ or more) historical earthquakes (DOE/RW-0198, 1988)

1978-1992 EVENTS IN THE SOUTHERN GREAT BASIN

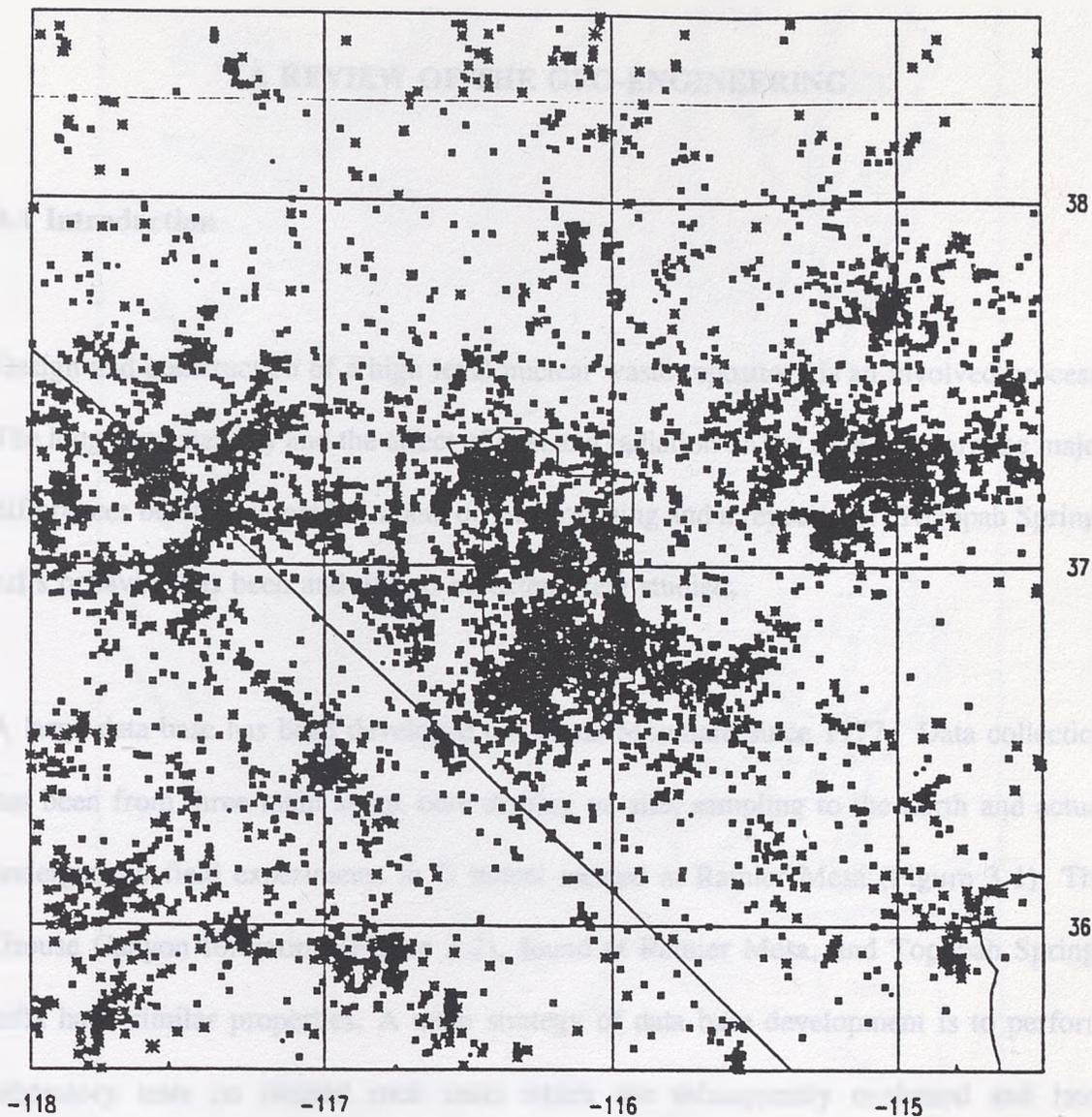


Figure 2.6 - An updated earthquake event location map for the southern great basin. The Nevada Test Site boundary outline is located in the middle of the figure (Seggern, 1993)

CHAPTER III

A REVIEW OF THE GEO-ENGINEERING

3.1 Introduction

Design and construction of a high level nuclear waste repository is an involved process. The long-time stability and the effect of heat and radiation on the rock mass are the major differences between a regular tunnel or mine opening and a repository. Topopah Springs tuffs behavior has been and will to be extensively studied.

A large data base has been developed on Yucca Mountain since 1977. Data collection has been from three main areas: core drilling on site, sampling to the north and actual underground field experiments in G tunnel located at Rainier Mesa (Figure 3.1). The Grouse Canyon formation (Figure 3.2), found at Rainier Mesa, and Topopah Springs tuffs have similar properties. A main strategy of data base development is to perform laboratory tests on desired rock units which are subsequently evaluated and later confirmed with large scale field tests in the G-tunnel and in the future experimental tunnel itself. Core drilling and surface outcrop bulk samples are the two main sources of laboratory testing materials. The core drilling has occurred throughout the proposed site, and bulk samples of Topopah Springs tuff have been collected at Busted Butte which is just southeast of Yucca Mountain.

Figure 3.1 - Location map of the proposed Yucca Mountain, Rainier Mesa, Yucca Mountain and the G-tunnel. (Figure 3.1, p. 11, 1977)

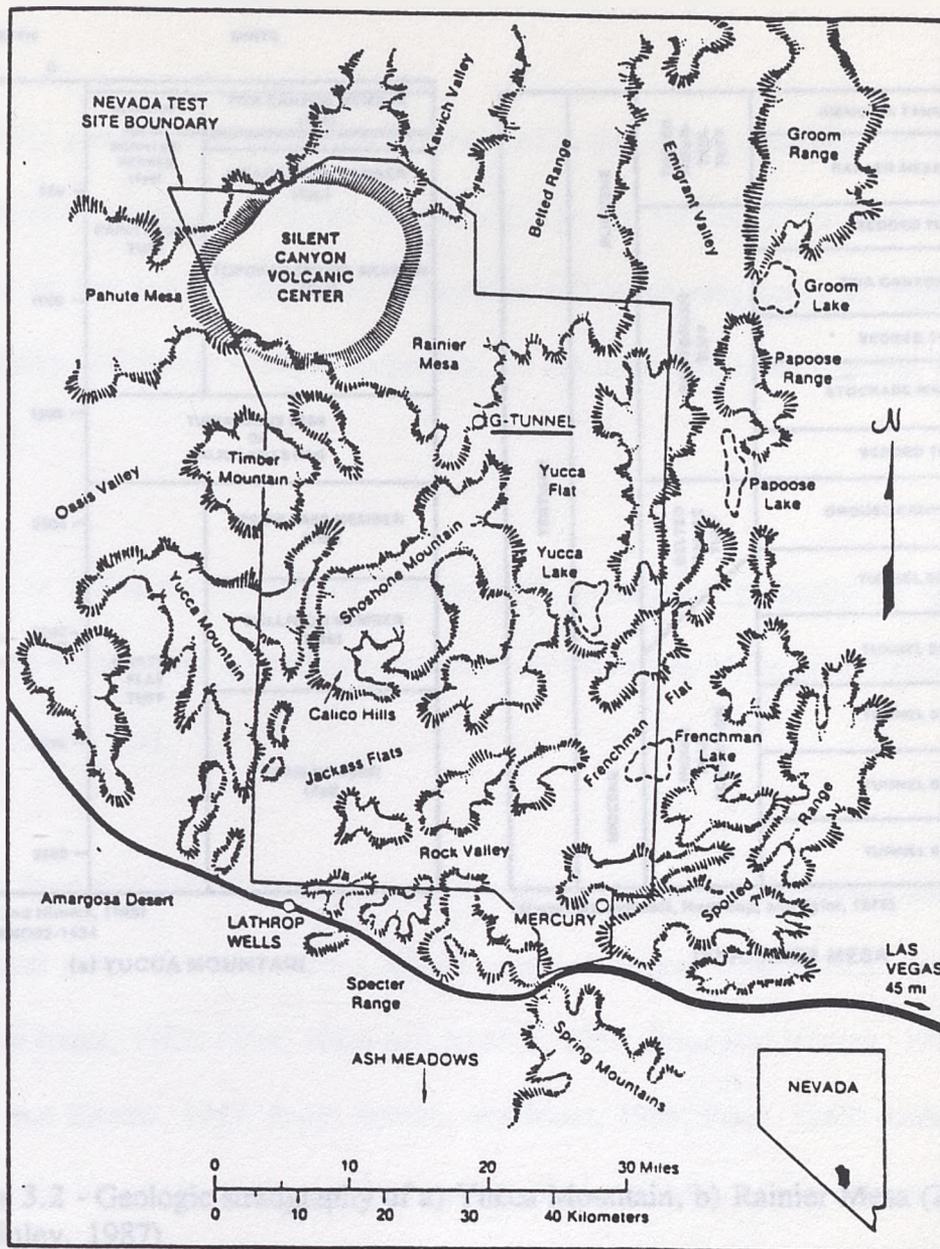


Figure 3.1 - Location Map for the Nevada Test Site, Rainier Mesa, Yucca Mountain and the G-tunnel (Snyder and Oliver, 1981)

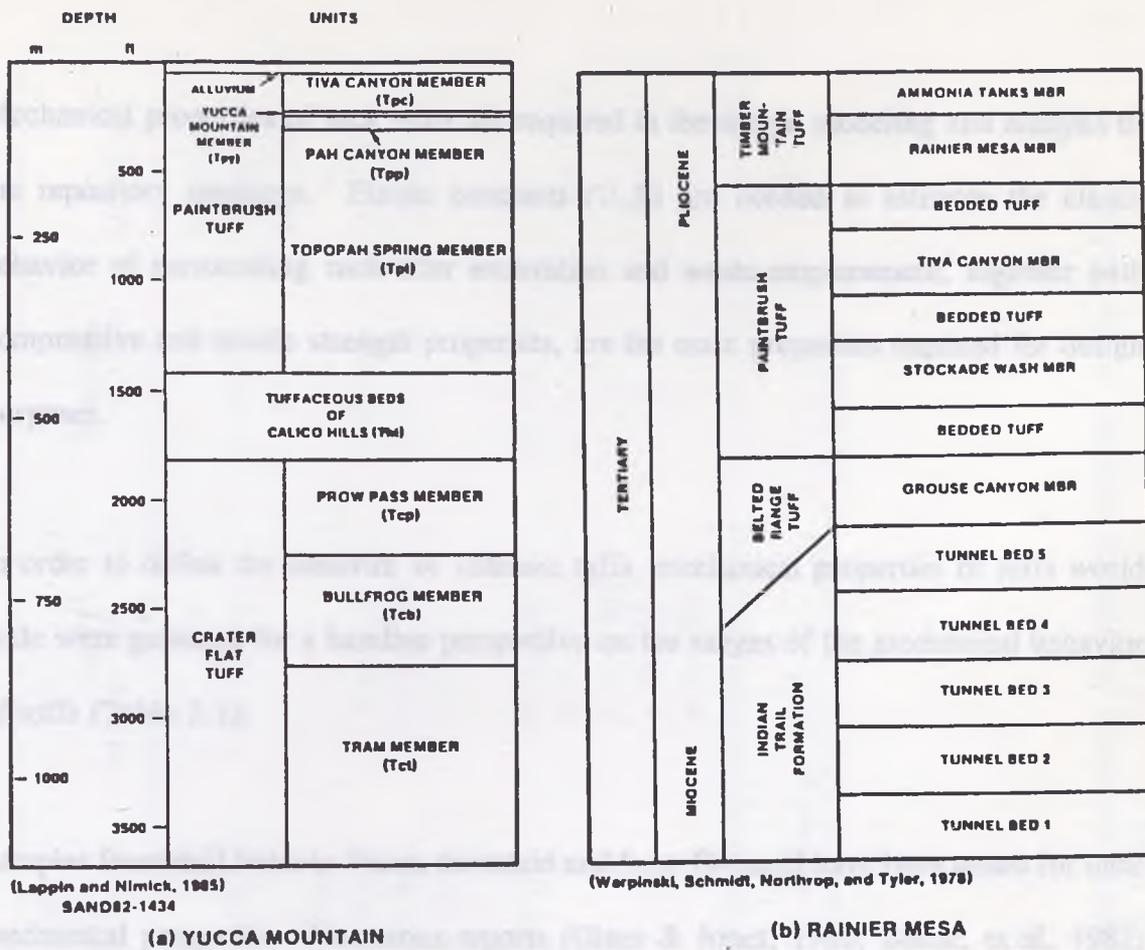


Figure 3.2 - Geologic stratigraphy of a) Yucca Mountain, b) Rainier Mesa (Zimmerman and Finley, 1987)

3.2 Mechanical Properties

Mechanical properties of rock units are required in the design modeling and analysis of the repository openings. Elastic constants (ν, E) are needed to estimate the elastic behavior of surrounding rock after excavation and waste emplacement, together with compressive and tensile strength properties, are the main properties required for design purposes.

In order to define the behavior of volcanic tuffs, mechanical properties of tuffs world wide were gathered for a baseline perspective on the ranges of the mechanical behavior of tuffs (Table 3.1).

Samples from drill holes in Yucca mountain and from G-tunnel have been tested for their mechanical properties. Numerous reports (Olsen & Jones, 1980; Blacic, et al, 1982; Price and Jones, 1982; Price, Jones and Nimick, 1982; Price and Nimick, 1982; Price, Nimick and Zirzow, 1982; Price, Spence, and Jones, 1984; Price, 1983) contain results of the majority of the testing. The average property value ranges for the Topopah Springs member can be observed on Tables 3.2 and 3.3. These results represent ambient temperature (23°C) and atmospheric conditions (unconfined).

Discontinuities of Yucca Mountain tuffs which include joints, faults, bedding planes and inhomogeneities (lithophysae and pumice inclusions) were tested for their strength characteristics. Laboratory analyses on simulated and natural joints reveal frictional

Location or tuff unit	Unconfined compressive strength (psi) ^b	Young's modulus (10 ⁶ psi) ^b	Poisson's ratio	Lithology ^c
Ohya tuff, Japan	0.804	-- ^d	--	Nonwelded
Rainier Mesa tuff units	1,350-5,125	0.45-2.26	0.09-0.38	Nonwelded
Tuff, E-Tunnel, NTS*	3,500	--	--	Nonwelded
Tuff, NTS	5,282-9,512	--	--	--
Oak Springs Formation, NTS	3,400-8,700	0.40-1.60	0.02-0.04	Bedded
Oak Springs Formation, NTS	6,800-29,100	0.86-1.75	0.05-0.15	Welded
Tuff, Oregon	3,141-4,999	0.85	--	--
Tuff, Red Hot Deep Well Experiment, NTS	1,560-4,910	0.33-0.95	0.13-0.49	--
Tuff and tuff breccia, USSR	--	3.23	0.13	--
Tuff, Japan	--	1.32-3.47	--	--
Tuff breccia, India	--	0.20-3.62	--	--
Tuff, locality unknown	--	0.99-2.92	--	--

*Source: Guzowski et al. (1983).

^bTo convert from psi to Pa, multiply the entries by 6,895.

^cLithologies have been assessed on the basis of original references when available.

^d-- = data not available.

*NTS = Nevada Test Site.

Table 3.1 - Summary of mechanical properties of tuffs not included in the Yucca Mountain Project (DOE/RW-0199, 1988)

Zone ^a	Young's modulus (GPa)	Poisson's ratio	Unconfined compressive strength (MPa)	Tensile strength (MPa)	Angle of internal friction (°)	Cohesion (MPa)
TCw	40.0	0.24 (TSw2) ^b	240	17.9	44.7	51
PTn	3.8	0.16 (CHnlz) ^b	19	1.0	8.5	8
TSw1	31.7±17.9 ^{c,d} 15.5±3.2 ^{d,e}	0.25±0.07 ^{c,d} 0.16±0.03 ^{d,e}	127±16 ^{c,d} 16±5 ^{d,e}	21.1±4.6 ^c 1.0 ^e	34.9 ^c 12.5 ^e	36 ^c 11 ^e
TSw2	30.4±6.3 ^d	0.24±0.06 ^d	166±65 ^d	15.2	23.5 ^d	34.5 ^d
TSw3	NA ^g	NA	NA	NA	NA	NA
CHnlz	7.1	0.16 (CHnlz) ^b	27	1.0	12.0	11
CHnlz ^f	7.1±2.1 ^d	0.16±0.08 ^d	27±9 ^d	1.0	7.6±2.6 ^d	10.9±1.6 ^d
CHn2	11.5	0.16 (CHnlz) ^b	40	2.6	16.4	15
CHn3	7.1	0.16 (CHnlz) ^b	27	1.0	12.0	11
PPw	16.3	0.13 (BFw) ^b	57	6.9	21.0	20
CFUn	7.6±3.8 ^d	0.16 (CHnlz) ^b	31±11 ^d	1.8	15.6	14
BFw ^f	10.8±4.7 ^d	0.13±0.02 ^d	42±14 ^d	6.9	21.0	20
CFMn1	15.2	0.16 (CHnlz) ^b	52	6.0	19.9	19
CFMn2	16.3	0.16 (CHnlz) ^b	57	6.9	21.0	20
CFMn3	13.2	0.16 (CHnlz) ^b	45	4.3	18.0	17
TRw ^f	17.6±3.8 ^d	0.13 (BFw) ^b	72±23 ^d	11.1	27.6	27

^aZone identifications, thicknesses, and relation to formal stratigraphy are shown in Figure 2-5.

^bValue assumed to be the same as mean value of thermal/mechanical unit listed in parentheses.

^cRepresentative of nonlithophysal zones within unit TSw1.

^dExperimental results for mechanical properties at baseline test conditions (see text); standard deviations are 1σ. All other mechanical data entries are calculated using porosity with empirical equations; no standard deviations are available for these entries.

^eRepresentative of lithophysal zones within unit TSw1.

^fZones previously considered for waste emplacement horizon.

^gNA = not available.

Table 3.2 - Mechanical properties of the intact tuffs found at Yucca Mountain (DOE/RW-0199, 1988)

Property	G-Tunnel Grouse Canyon Member	Yucca Mountain Topopah Spring Member
Matrix porosity (°°/0)	6-24 ^a	6-19 ^a
Grain density (g/cm ³)	2.57-2.63 ^b	2.51-2.58 ^b
Saturation	0.6-0.9 ^a	0.65 ^c
Saturated thermal conductivity (W/mK)	1.6-2.0 ^a	2.1-2.5 ^a
Dry thermal conductivity (W/mK)	1.0-1.6 ^b	1.5-2.1 ^b
Coefficient of linear thermal expansion (10 ⁻⁶ K ⁻¹)	7.8-10.6 ^a	7.3-14.1 ^a
Young's modulus (GPa)	22-28 ^a	24-38 ^a
Poisson's ratio	0.16-0.32 ^a	0.12-0.32 ^a
Unconfined compressive strength (MPa)	64-142 ^a	55-287 ^a

^aZimmerman et al. (1984b).

^bNimick and Lappin (1985).

^cMontazer and Wilson (1984).

Table 3.3 - Comparison of properties of Grouse Canyon and Topopah Springs tuffs (DOE/RW-0199, 1988)

properties (Table 3.4). Along with the cohesion and coefficient of friction, the orientation and spacing of joints help determine if slip can occur along discontinuities under different stress environments (thermal, in situ, excavation size). Joint behavior may dominate the response of the excavation to the expected mechanical and thermal loadings. This is due mainly to the heavily fractured nature of the Topopah Spring Member [up to 42 fractures per m^3 in bore hole USW Gu-3 (Scott and Castellanos, 1984)].

3.3 Thermal Properties

The internal generation of heat expected from the emplaced nuclear waste canisters will raise the proposed geologic host in the vicinity by as much as $200^{\circ}C$ at the edge of the wall of the emplacement hole (Peters, 1983). Figure 3.3 shows the "defined" field designation areas surrounding the emplaced canisters. The host rock mass could expand or contract excessively due to the thermal gradients induced stress fields, causing rock fractures or displacement along discontinuities. This could affect room stability during operational, storage and retrieval periods. Therefore, the supporting system chosen must be permanent and deformable or debonding. It will have to allow the rock to expand and/or contract but still maintain its ability to support the rock.

The thermal properties of the Topopah Springs member and other similar units have been studied in order to understand the reaction of the rock units and their discontinuities after waste emplacement (Table 3.3). This will help predict thermal

Unit ^b	Intact rock properties ^a					Rock-mass deformation modulus (GPa) ^c	Fracture properties		
	Unconfined compressive strength (MPa)	Young's modulus (GPa)	Poisson's ratio	Tensile strength (MPa)	Angle of internal friction (°)		Cohesion (MPa)	Cohesion (MPa)	Coefficient of friction
TCw	240	40.0	0.24	17.9	44.7	51	20.0	0.2	0.54
PTn	19	3.8	0.16	1.0	8.5	8	1.9	0.2	0.59
TSw1	127±16 16±5 ^e	31.7±17.9 15.5±3.2 ^e	0.25±0.07 ^d 0.16±0.03 ^e	12±4.6 ^d 1.0 ^e	34.9 ^d 12.5 ^e	36 ^d 11 ^e	15.9 ^d 7.8 ^e	0.2 ^d 0.2 ^e	0.54 ^d 0.54 ^e
TSw2	166±65	30.4±6.3	0.24±0.06	15.2	23.5	34.5	15.2	0.2	0.54
TSw3	NA ^f	NA	NA	NA	NA	NA	NA	0.2	0.54
CHnlv	27	7.1	0.16	1.0	12.0	11	3.6	0.2	0.59
CHnlz	27±9	7.1±2.1	0.16±0.08	1.0	7.6±2.6	10.9±1.6	3.6	0.2	0.59
CHn2	40	11.5	0.16	2.6	16.4	15	5.8	0.2	0.59
CHn3	27	7.1	0.16	1.0	12.0	11	3.6	0.2	0.59
PPw	57	16.3	0.13	6.9	21.0	20	8.2	0.7	0.59
CFUn	31±11	7.6±3.8	0.16	1.8	15.6	14	3.8	0.7	0.64
BFw	42±14	10.8±4.7	0.13±0.02	6.9	21.0	20	5.4	0.7	0.59
CFMn1	52	15.2	0.16	6.0	19.9	19	7.6	0.7	0.64
CFMn2	57	16.3	0.16	6.9	21.0	20	8.2	0.7	0.64
CFMn3	45	13.2	0.16	4.3	18.0	17	6.6	0.7	0.64
TRw	72±23	17.6±3.8	0.13	11.1	27.6	27	8.8	0.7	0.59

^aData from Table 2-7.

^bSee Figure 2-5 for definition of thermal/mechanical units.

^cTaken as 50 percent of values in Table 2-7, as discussed in text.

^dNonlithophysal layers within unit TSw1.

^eLithophysal layers within unit TSw1.

^fNA = not available.

Table 3.4 - Values for intact rock and rock-mass mechanical properties and fracture properties (DOE/RW-0199, 1988)

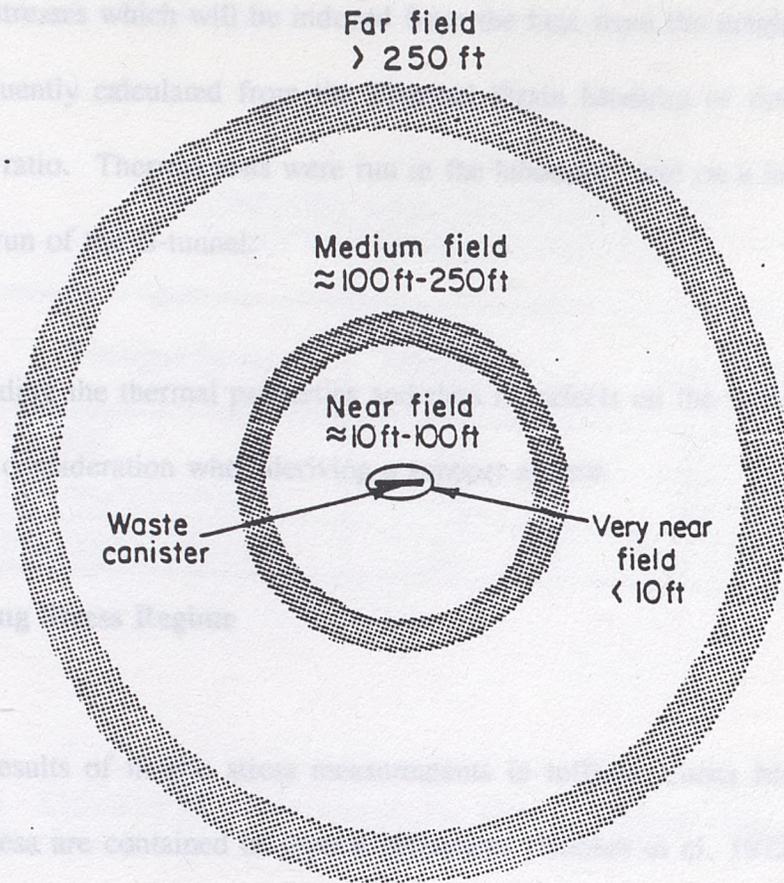


Figure 3.3 - Field designations for regions near waste emplacement areas (USBM, 1987)

gradients and its effects on differential thermal stresses. Knowledge of the thermal expansion behavior of the desired rock will reveal the thermal strain of the rock. Thermal stresses which will be induced from the heat from the emplaced canisters can be subsequently calculated from the Thermal Strain Modules of deformation and the Poisson's ratio. Thermal tests were run in the laboratory and on a larger scale with in situ tests run of the G-tunnel.

Understanding the thermal properties and then its effects on the surrounding rocks are important consideration when deriving a support system.

3.4 Existing Stress Regime

Detailed results of in situ stress measurements in tuffs at Yucca Mountain and/or at Rainier Mesa are contained in several references (Hooker et al, 1972; Haimson et al, 1974; Ellis and Ege, 1975; Tyler and Vollendorf, 1975; Ellis and Magner, 1980; Warpinsk et al, 1981; Zimmerman and Vollendorf, 1982; Soback et al, 1987; and Stock et al, 1984). These contain details and limitations of testing techniques. In situ principal stress orientations and magnitudes are mainly due to lithostatic loading and are contained on Table 3.5. Table 3.6 includes the predicted thermal induced stress changes over time for the midpanel area, for a temperature change of 50 degrees celsius. Greater stress changes will be induced near the actual waste.

Principal Stress	Stress Magnitude	Orientation
Vertical	20	NA
Maximum Horizontal	42	N30E
Minimum Horizontal	32	N17W

Waste (yr)	Vertical	Dist. from Waste	Dist. from Waste	Dist. from Waste
0	0.0	0.0	0.0	0.0
30	-3.05	4.54	4.75	8.0
50	-4.18	7.86	3.17	33.0
100	-2.57	10.36	3.34	50.0

x is perpendicular to dist. axis; y is parallel to dist. axis

Table 3.5 - Stress regime for Yucca Mountain (Hardy and Bauer, 1991)

MAGNITUDE AND ORIENTATION OF
IN SITU PRINCIPAL STRESSES

Principal Stress	Stress Magnitude (MPa)	Orientation
Vertical	7.0	NA
Maximum Horizontal	4.2	N32°E
Minimum Horizontal	3.5	N57°W

Table 3.6 - Estimated stress changes for the Midpanel Access Drift due to temperature changes (Hardy and Bauer, 1991)

THERMAL STRESS CHANGE USED FOR MIDPANEL ACCESS DRIFT

Time After Waste Emplacement (yrs)	Thermal Stress Change (MPa)			Temperature Change (°C)
	Vertical σ_x	Horizontal Drift Plane σ_x	Horizontal Drift Axis σ_y	
0	0.0	0.0	0.0	0.0
10	0.0	0.0	0.0	0.0
35	-3.60	4.84	0.96	8.0
50	-4.16	7.89	3.17	23.0
100	-2.53	10.86	7.24	50.0

x is perpendicular to drift axis; y is parallel to drift axis

3.5 Excavation Characteristics

The expected excavation characteristics of the repository were assessed with rock mass ratings from bore hole core and a large test excavation in a similar tuff. Both rock mass classification systems [Barton (NGI) and Bienowski (RMR)] have been used to characterize the Yucca Mountain Horizons considered for waste emplacement.

The rock mass classifications NGI and RMR ratings from the Topopah Springs member are located on Table 3.7. Both classification systems recommend minimum to moderate support under ambient temperatures which would be untensioned grouted rock bolts with shotcrete (Hardy and Bauer, 1991) (Figure 3.4).

The G-tunnel experiment was completed a few years ago. Similarities of the (Grouse Canyon Member) G-tunnel tuffs and Yucca Mountain tuffs can be seen on Table 3.3. The excavation of the G-tunnel and the testing performed by Sandia Labs has helped to define the expected properties of Yucca Mountain tuff as it is being excavated.

Excavation of the portal to portal loop is underway, hence underground mapping of the rock and its discontinuities has been an on going process.

CLASSIFICATION PARAMETERS AND ESTIMATED RANGE OF VALUE

Classification System	Parameter	Description	Range*
NGI-Q system	RQD	Rock quality designation	35.0 to 80.0
	J _N	Joint set number	6.0 to 12.0
	J _R	Joint roughness number	4.0 to 2.0
	J _A	Joint alteration number	1.0 to 4.0
	J _w	Joint water reduction factor	1.0
	SRF	Stress reduction factor	1.0
	Q	Rock mass quality	1.46 to 53.3
RMR system	C	Intact core strength rating	12.0
	RQD	Rock quality designation rating	8.0 to 17.0
	JF	Spacing of joint, rating	10.0 to 20.0
	JC	Joint condition rating	20.0 to 25.0
	JW	Groundwater rating	10.0
	AJO	Adjustment for joint orientation	0.0 to -12.0
	RMR	Rock mass rating	48.0 to 84.0

* Langkopf and Gnirk (1986)

Table 3.7 - Estimated range of values for both the NGI and Q Rock Mass Classification Systems (Hardy and Bauer, 1991)

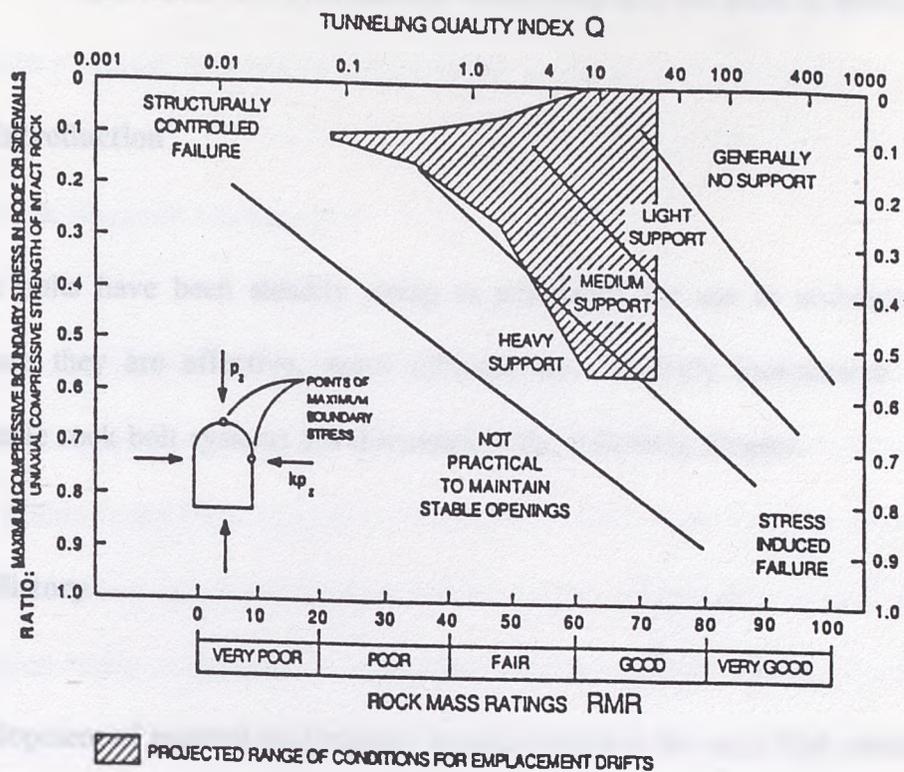


Figure 3.4 - Hoek's (1980) proposed support guidelines for underground excavations with the range of expected conditions for emplacement drifts at the Yucca Mountain Project (Hardy and Bauer, 1991)

CHAPTER IV

REVIEW OF PHYSICAL PROPERTIES OF ROCK BOLTS

4.1 Introduction

4.2 Rock Support Background

Rock bolts have been steadily rising in popularity for use in underground support, because they are effective, space efficient and relatively inexpensive. Most of the available rock bolt systems are discussed in the following chapter.

4.2 History

Development of internal roof support systems began in the early 20th century. By 1936, split wedge rock bolts were providing reinforcement to mine roofs at the St Joseph Lead company's Leadwood Lead mine located in Missouri (Weigel, 1943). In the mid 1940's the expansion anchor bolt was developed, and by 1949, fifty mines were systematically using the bolts for ground support. Panek, from the Bureau of Mines, described the support characteristics of bolt systems, as an increase of the coefficient of friction between bolted layers by 1956 (USBM, 1987). Also in 1956, the American Standard Specifications for rock bolting were approved by the by the American Mining Congress and American Standards Association. These ASTM Standard Specifications, currently referred to ASTM F432-83, were revised in 1983. Advancements in grouting, especially the development of resin based grout in the 1960's and 1970's, helped to popularize the

grouted rock bolts. By the mid 1970's, the friction rock stabilizers, which are based on deformable steel tubes, had been developed. During the 1980's, over 100 million rock bolts were used in the American mining industry. Today, a multitude of bolts systems and secondary support accessories can be found in use all over the world.

4.3 Rock Support Background

Rock reinforcement with dowels, bolts and anchors can provide passive or active support to an underground opening. In a passive support no tension is added during installation. This system is activated with rock deformation, and should therefore be emplaced soon after excavation of the opening. Active support, describes a system that has been tensioned during installation, thus providing a compressional force against the rock mass.

Rock bolts help the rock support itself by providing support either active or passive. In the past, rock bolt users realized there was a stress free zone in the roof of underground excavations, so they applied tensioned bolts in order to create an artificial vault (Figure 4.1). If the length to spacing ratio is not at least equal to 2, no uniform compression ring can develop and tension zones around bolt heads may cause crumbling (Chouquet, 1991). Today, most of the rock bolt literature, including the New Austrian Tunneling Method (NATM), suggest approaching rock reinforcement in a different way. Their strategy is to let the rock support itself with only the minimum

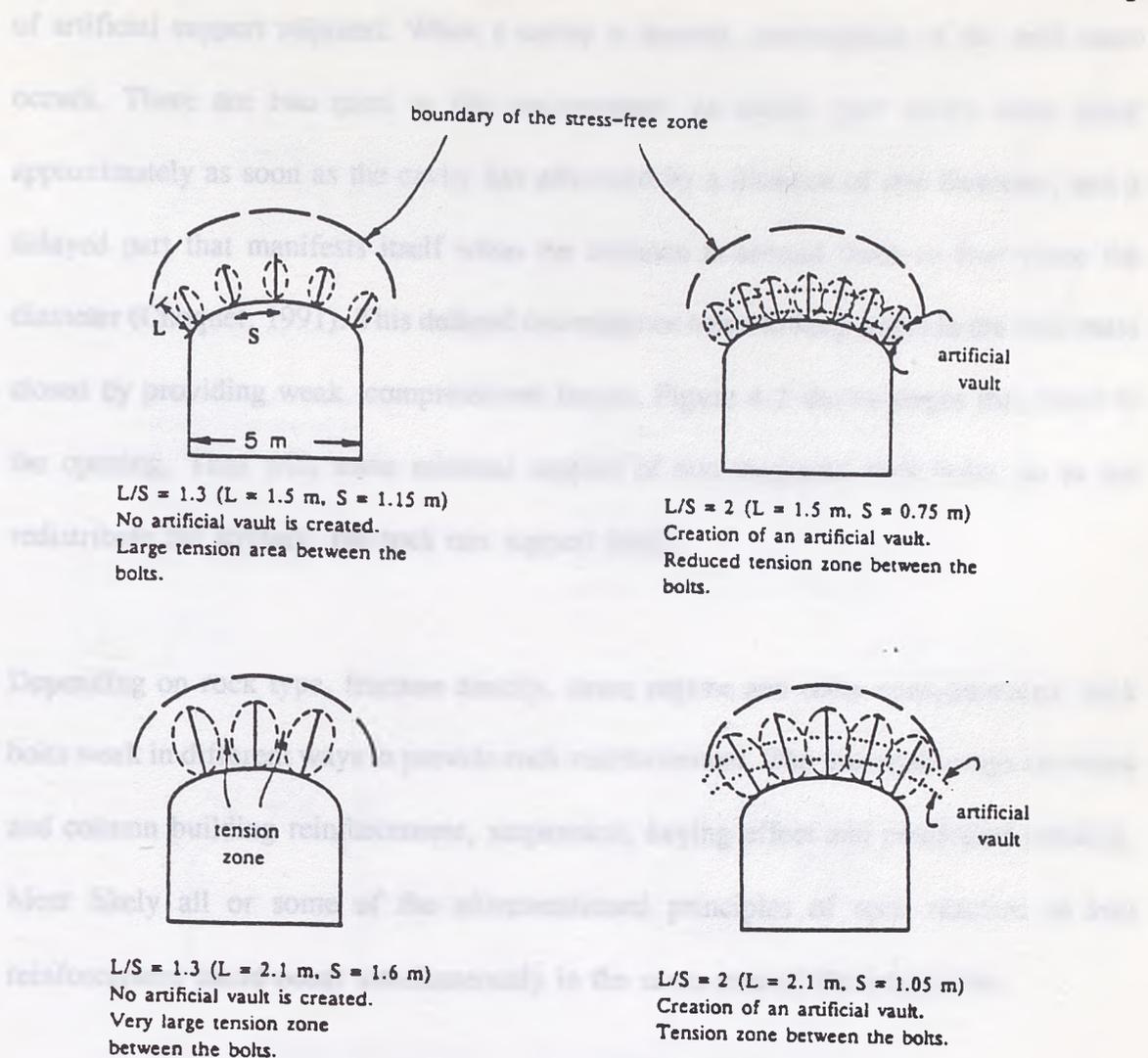


Figure 4.1 - An artificial rock vault can be developed with tensioned bolts, if the length and spacing of the bolts are suitable (Choquet, 1991)

of artificial support required. When a cavity is opened, convergence of the rock mass occurs. There are two parts to the convergence: an elastic part which takes place approximately as soon as the cavity has advanced by a distance of one diameter; and a delayed part that manifests itself when the advance is around three to four times the diameter (Choquet, 1991). This delayed convergence helps to keep joints in the rock mass closed by providing weak compressional forces. Figure 4.2 shows stages that occur to the opening. Thus with some minimal support of non-tensioned rock bolts, so as not to redistribute the stresses, the rock can support itself.

Depending on rock type, fracture density, stress regime and other considerations, rock bolts work in different ways to provide rock reinforcement. The four main ways are beam and column building reinforcement, suspension, keying effect and controlled yielding. Most likely all or some of the aforementioned principles of rock reaction to bolt reinforcement could occur simultaneously in the same area of the excavation.

When the beam and column building reinforcement concept is implemented, rock bolts are utilized in an area of thinly bedded units of rock which cannot support themselves. The bolts are installed through a number of these units which attaches the thin beds together, therefore providing a beam for high vertical stresses or a column for high horizontal stresses (Figures 4.3 and 4.4).

Suspension reinforcement involves using rock bolts to support blocks or bedded units which have been separated from the rock mass. The rock bolts are emplaced through

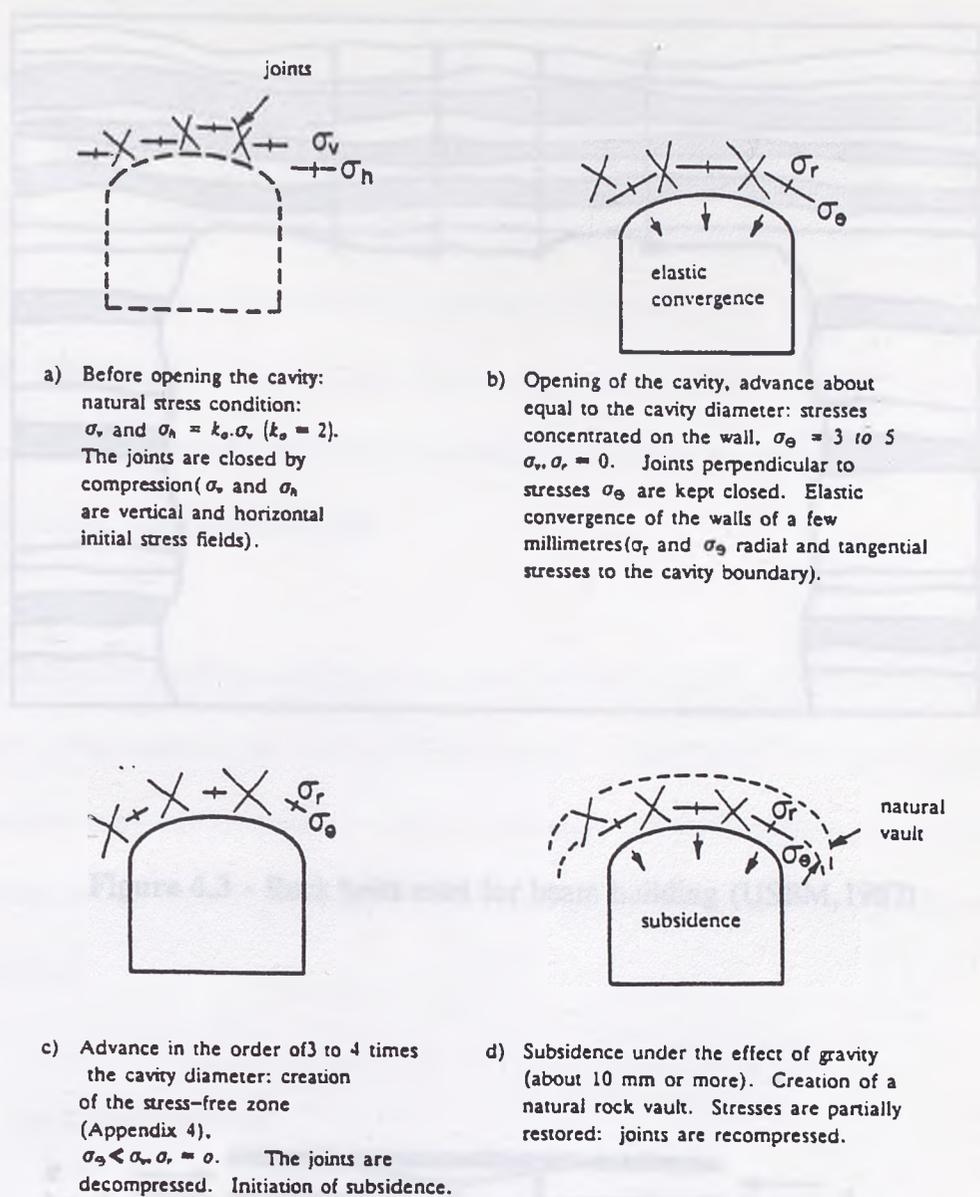


Figure 4.2 - The development of a natural rock vault in the roof of an underground excavation (Choquet, 1991)

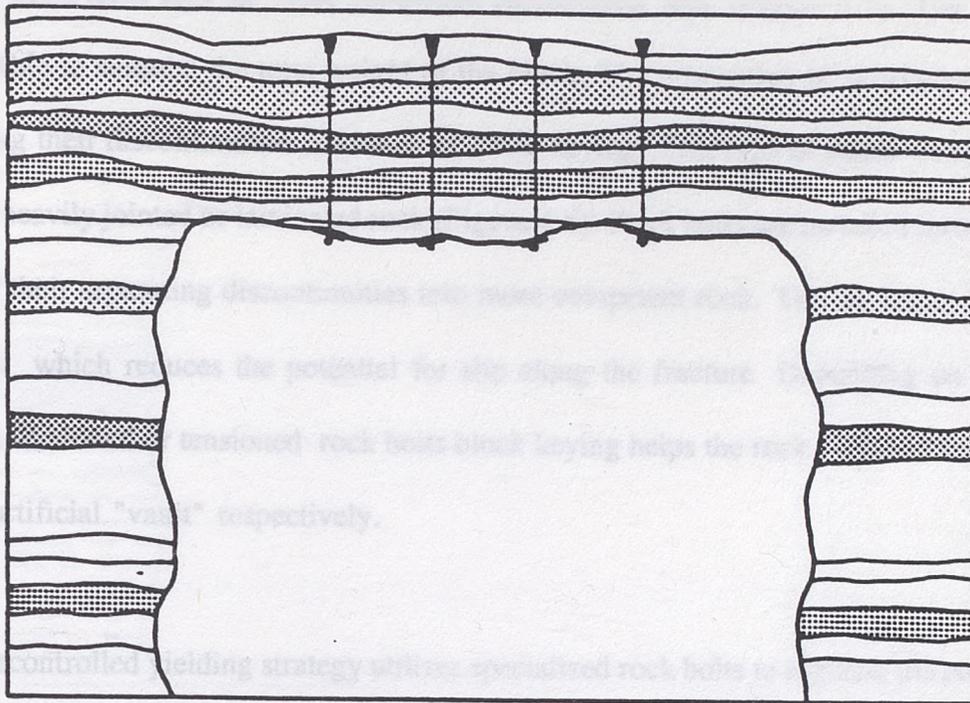


Figure 4.3 - Rock bolts used for beam building (USBM, 1987)

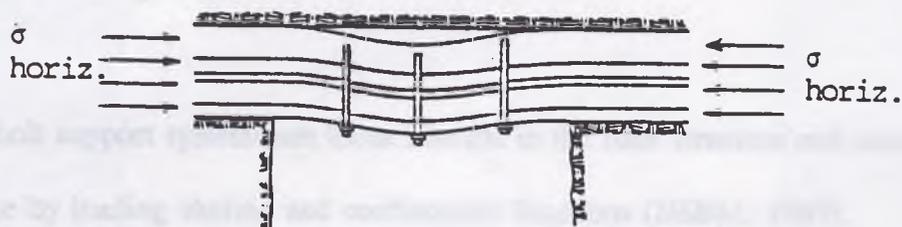


Figure 4.4 - Rock bolts used for column building (Choquet, 1991)

the weak units into the more competent surrounding rock (Figure 4.5). The bolts must be able to suspend the total weight of the block. The prevention of movement of blocks along their discontinuities is known as block keying. This type of reinforcement is used for heavily jointed or laminated rock (Figure 4.6). Rock bolts are installed through blocks and their separating discontinuities into more competent rock. This increases the normal load which reduces the potential for slip along the fracture. Depending on the use of non-tensioned or tensioned rock bolts block keying helps the rock mass to form a natural or artificial "vault" respectively.

The controlled yielding strategy utilizes specialized rock bolts to regulate the convergence of the rock mass in an underground opening. In this case, the opening undergoes excessive convergence because of or a combination of a low deformation modulus, high stresses or heat induced wall convergence. Yieldable bolt systems are often used in these situations.

4.4 Rock Bolt Systems

Rock bolt support systems are those internal to the rock structure and accomplish their purpose by load sharing and confinement functions (USBM, 1987).

The main rock bolt systems used today are mechanically anchored, resin and portland cement based grout anchored, friction stabilizers and/or combinations of some systems. Many variations of these systems exist. Although most rock bolts considered

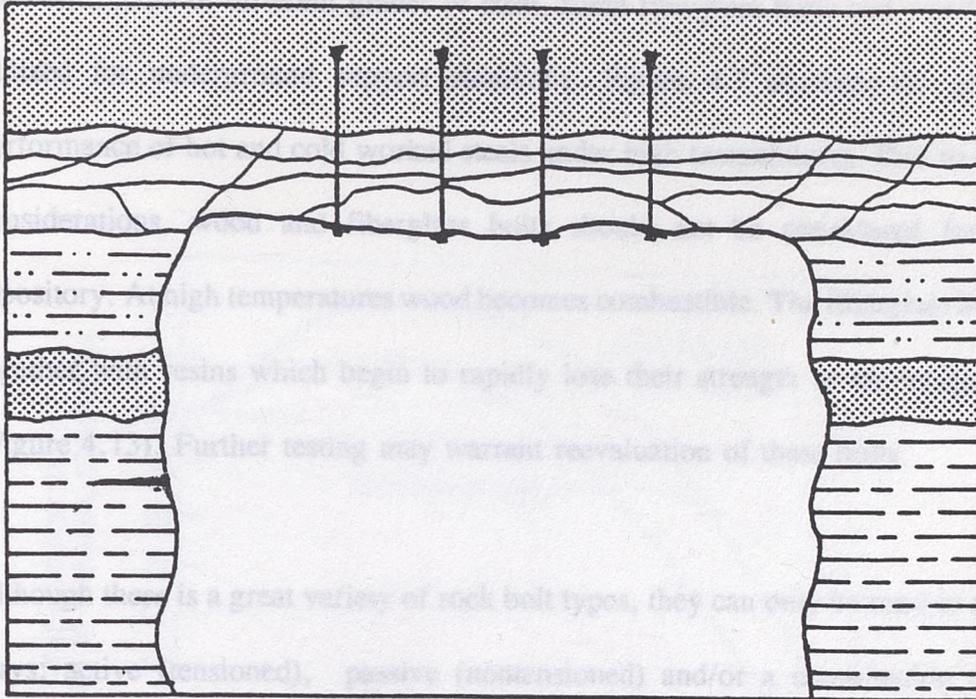


Figure 4.5 - Rock bolts used in the suspension of the roof (USBM, 1987)

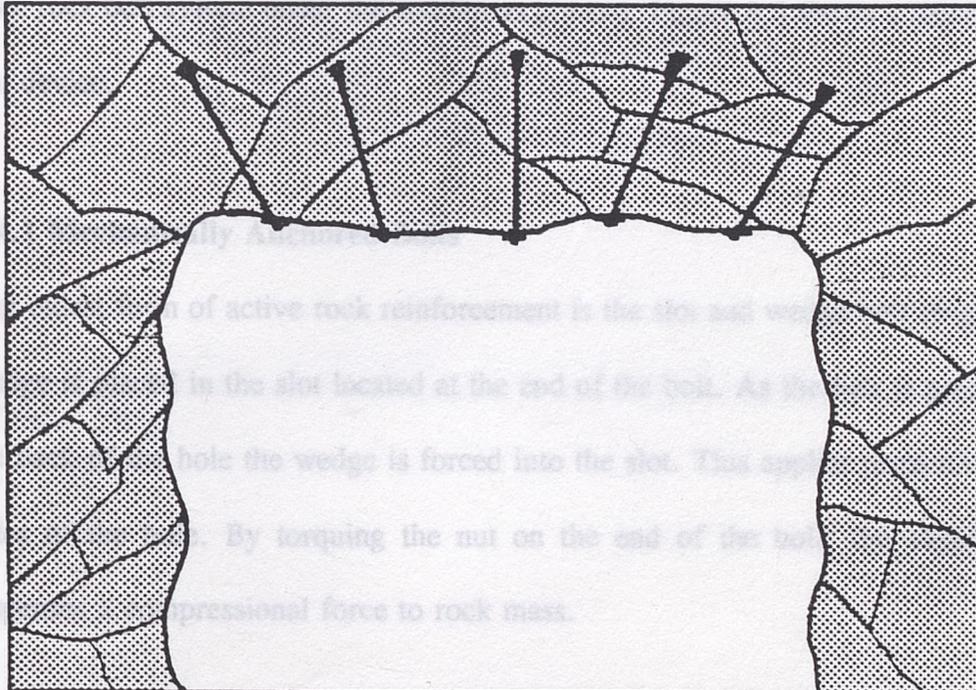


Figure 4.6 - Rock bolts used for block keying (USBM, 1987)

are constructed of different grades of steel, some fiberglass bolts and wood dowels are utilized for underground support situations. Figure 4.7 displays the general yield performance of hot and cold worked steels under high temperatures. Due to temperature considerations, wood and fiberglass bolts should not be considered for the waste repository. At high temperatures wood becomes combustible. The fiberglass bolts are held together with resins which begin to rapidly lose their strength as the temperature rises (Figure 4.13). Further testing may warrant reevaluation of these bolts.

Although there is a great variety of rock bolt types, they can only be used in one of three ways; active (tensioned), passive (nontensioned) and/or a combination of the two. Geologic conditions and man's expectations of the excavation will warrant which systems should be used for rock support. Pull tests should always be performed in order to test each rock bolt system under the conditions at hand. Figure 4.8 shows a typical bolt pull-out device.

4.4.1 Mechanically Anchored Bolts

The oldest form of active rock reinforcement is the slot and wedge bolt (Figure 4.9). A wedge is placed in the slot located at the end of the bolt. As the bolt is pressed against the back of the hole the wedge is forced into the slot. This applies pressure against the sides of the hole. By torquing the nut on the end of the bolt, the supplied tension dispenses a compressional force to rock mass.

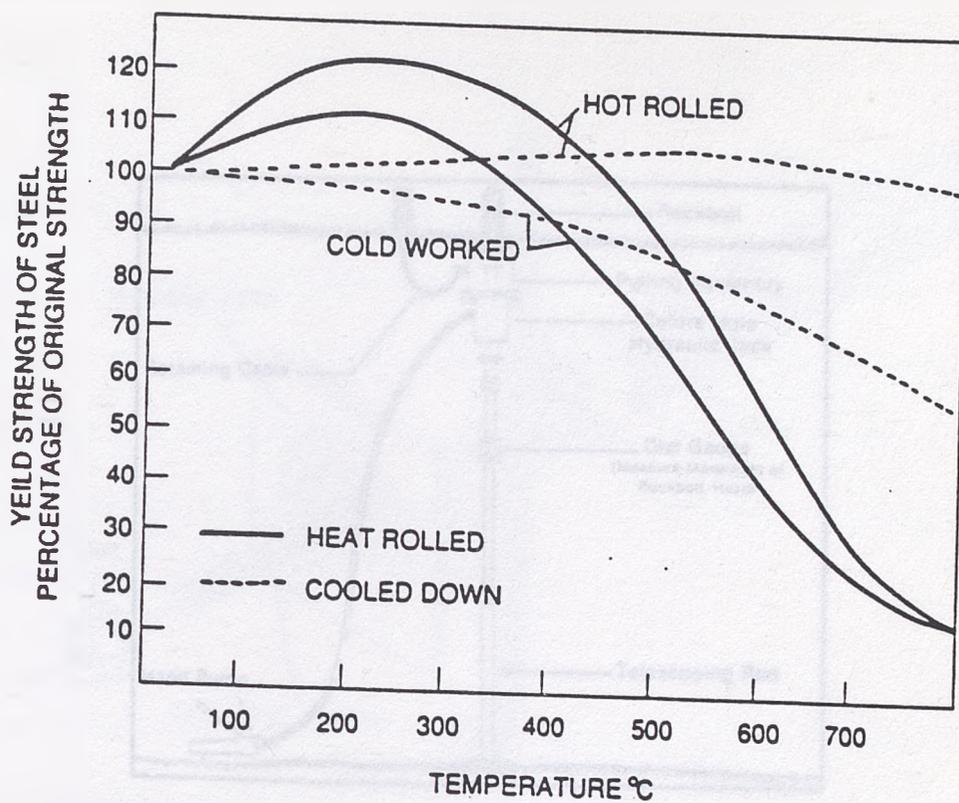


Figure 4.7 - Temperature vs. the general yield strength of both cold and hot worked steel (USBM, 1987)

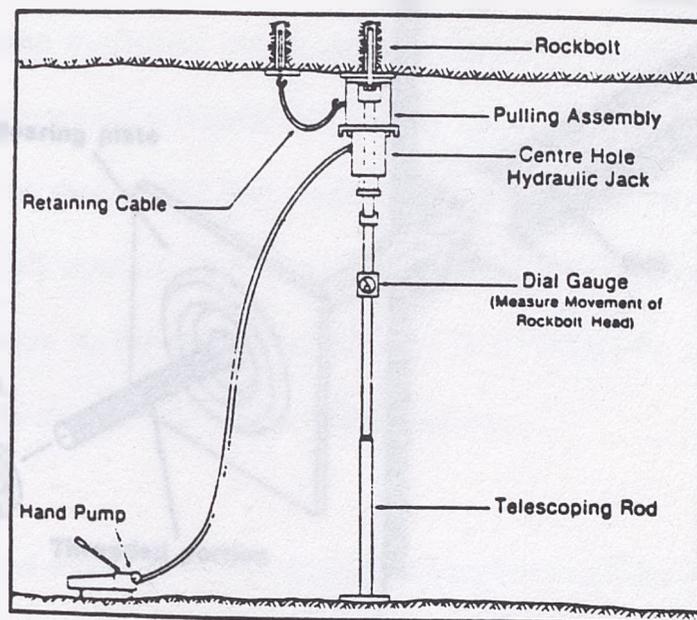
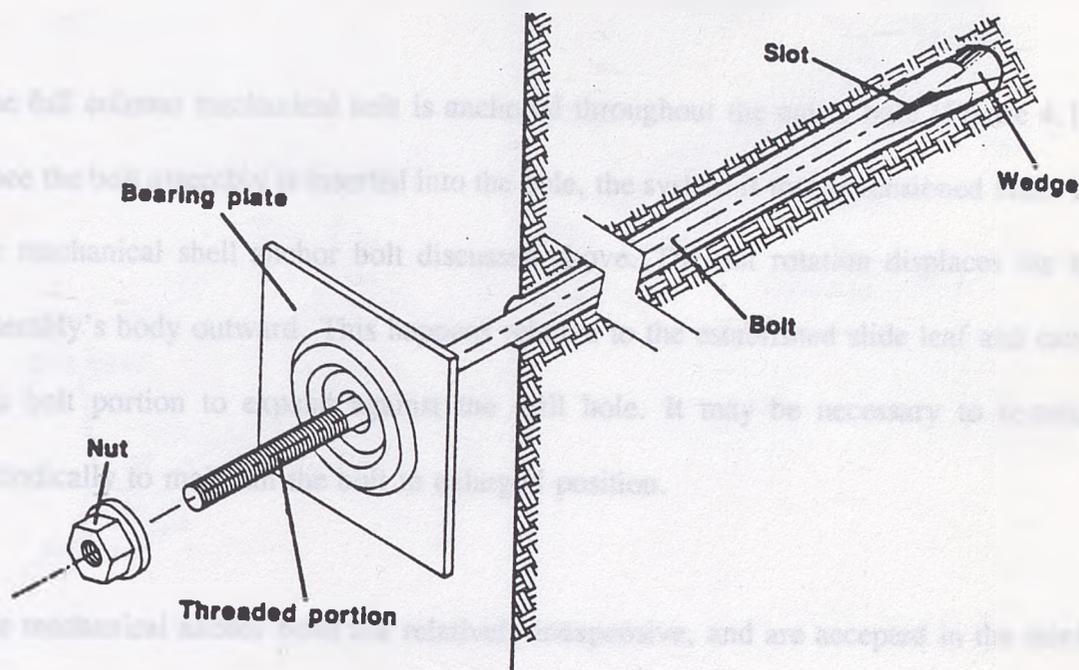


Figure 4.8 - A typical bolt pull-out device (Choquet, 1991)

The mechanical anchor bolt is inserted into the hole and the nut is tightened. The mechanical shell or expansion shell is expanded by wedge action at the head of the bolt. With tension, the shell's lowest aspect to the hole is stretched outward. This causes pressure to build up at the hole, thus wedging the shell to the hole. The hole is filled with concrete, making a compressed bond against the rock.

The full expansion mechanical bolt is anchored throughout the hole. Once the bolt is inserted into the hole, the expansion shell displaces the bolt assembly's body outward. The expansion shell is displaced by the bolt's body outward. The bolt's body is displaced by the expansion shell. The bolt's body is displaced by the expansion shell. The bolt's body is displaced by the expansion shell.



The mechanical anchor bolt is inserted into the hole and the nut is tightened. The mechanical shell or expansion shell is expanded by wedge action at the head of the bolt. With tension, the shell's lowest aspect to the hole is stretched outward. This causes pressure to build up at the hole, thus wedging the shell to the hole. The hole is filled with concrete, making a compressed bond against the rock.

The hole diameter is critical, because if the diameter is too large, the bolt will not grip the rock. The hole diameter is critical, because if the diameter is too large, the bolt will not grip the rock. The hole diameter is critical, because if the diameter is too large, the bolt will not grip the rock.

Figure 4.9 - Slot and wedge anchor bolt system (USBM, 1987)

The mechanical anchor bolt (Figure 4.10) is an improvement of the slot and wedge bolt. The mechanical anchor or expansion shell is activated by torque applied to the head of the bolt. With rotation, the shell's leaves expand as the bolt is threaded through. This causes pressure to walls of the hole, thus anchoring the end of the bolt. The bolt is tensioned with more rotation, causing a compressional force against the rock.

The full column mechanical bolt is anchored throughout the entire hole (Figure 4.11). Once the bolt assembly is inserted into the hole, the system is torque tensioned much like the mechanical shell anchor bolt discussed above. The nut rotation displaces the bolt assembly's body outward. This happens relative to the established slide leaf and causes this bolt portion to expand against the drill hole. It may be necessary to re-torque periodically to maintain the bolt in enlarged position.

The mechanical anchor bolts are relatively inexpensive, and are accepted in the mining and civil engineering industries because they have been around for many years. This bolt system works well in strong rock, but cannot be torque tensioned to its maximum load in weaker rock. These rock bolts are mostly considered as temporary support, because over time there is a tendency for the anchors to slip progressively with time (Hoek and Wood, 1992). Re-torquing bolts is possible unless corrosion has destroyed the threads. The hole diameter is critical, because if it is too big the expansion shell will not come in contact with the wall rock.

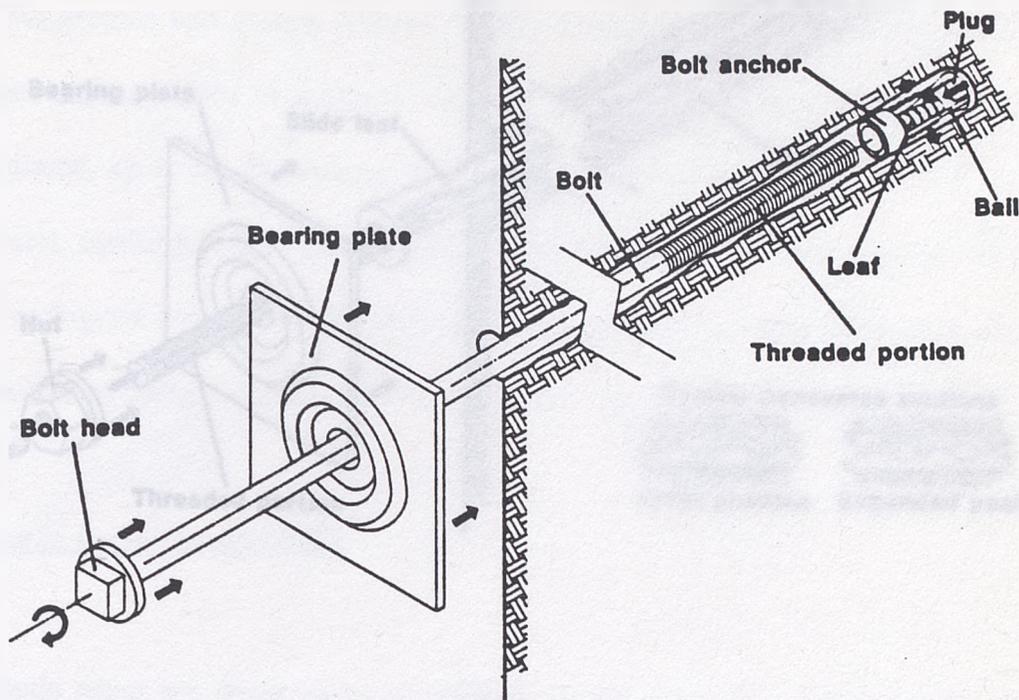


Figure 4.10 - Mechanical anchor bolt system (USBM, 1987)

4.4.2 Grout Anchored Bolt

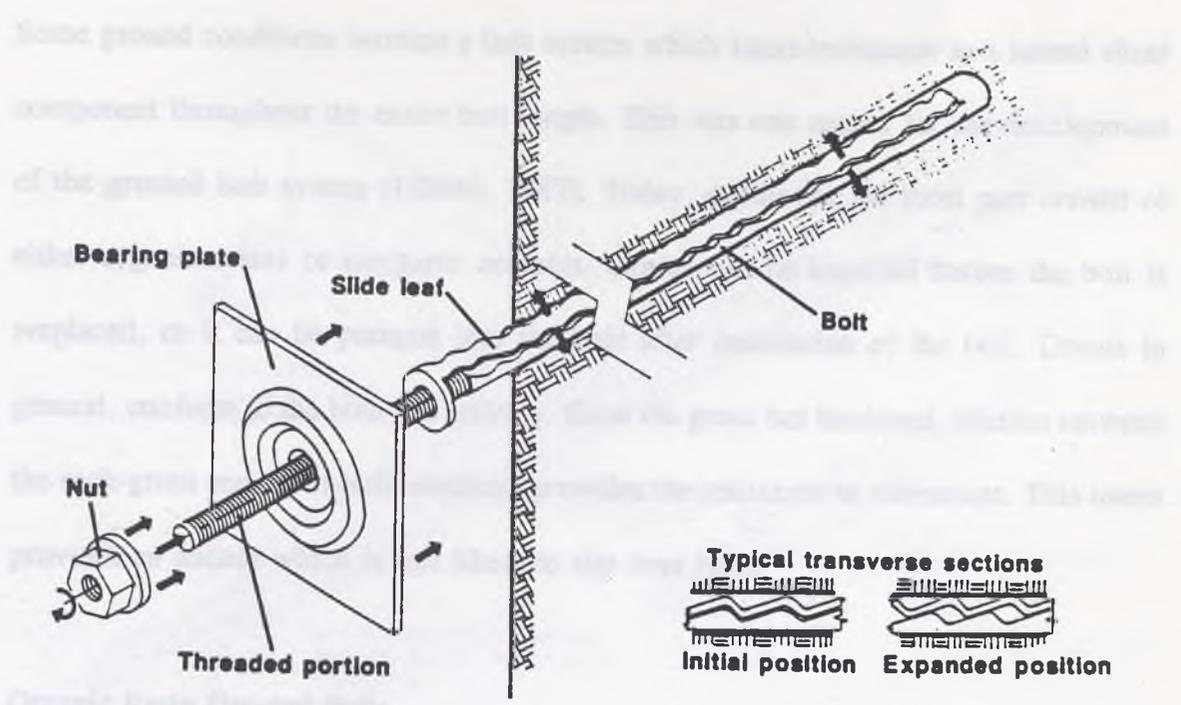


Figure 4.11 - Full-length mechanical anchor bolt system (USBM, 1987)

4.4.2 Grout Anchored Rock Bolts

Some ground conditions warrant a bolt system which bears resistance to a lateral shear component throughout the entire bolt length. This was one reason for the development of the grouted bolt system (USBM, 1987). Today, grouts for the most part consist of either organic resins or inorganic cements. Grouts can be installed before the bolt is emplaced, or it can be pumped into the hole after installation of the bolt. Grouts in general, conform to the hole and solidify. Once the grout has hardened, friction between the rock-grout and grout-bolt interfaces provides the resistance to movement. This in turn provides an anchor which is less likely to slip over time.

Organic Resin Grouted Bolts

Organic resins are either epoxy or polyester based. Most resin grouted bolts are of the polyester type. Polyester resin is a thermosetting, viscous liquid which has a promoter that solidifies by a cross-linking polymerization when blended with a heat generating catalyst (USBM, 1987). The heat prompts the resin to solidify and rapidly approach its ultimate strength quickly. Most resin gel times range from 1 to 30 minutes. Some resins can be injected, but more commonly they come in "sausage like" containers which are placed in the hole before bolt insertion (Figure 4.12). The unit contains both resin and the isolated catalyst. The rebar, usually steel or fiberglass, is rotated and pushed into the hole simultaneously usually with the drilling machine. This procedure should follow manufacturers suggested spin time and rotation rate for the

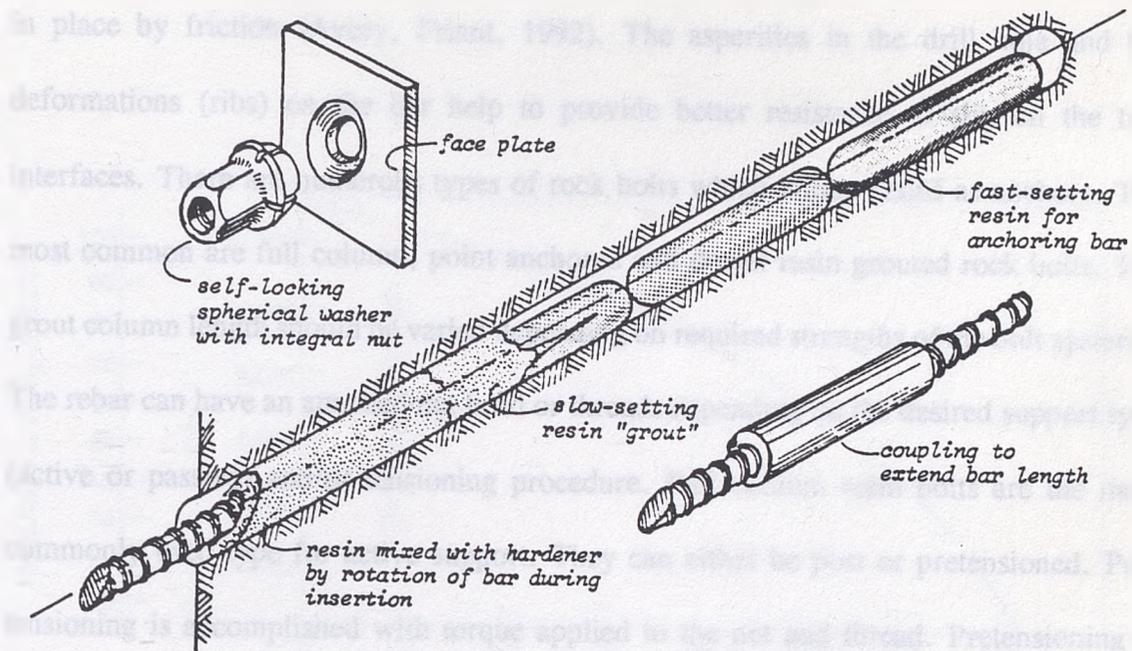


Figure 4.12 - Tensioned resin grout anchor bolt system (Hoek and Brown, 1980)

particular resin. An extra 15% of the resin should be allowed for oversized asperities and fractures in the hole. Once the resin has solidified, it provides a good anchor for the bolt. Contrary to popular belief polyester resins are not an adhesive glue, but hold a rock bolt in place by friction (Avery, Friant, 1992). The asperities in the drill hole and the deformations (ribs) on the bar help to provide better resistance to slip on the two interfaces. There are numerous types of rock bolts which utilize resins as anchors. The most common are full column, point anchored and dowel resin grouted rock bolts. The grout column length should be varied depending on required strengths of the bolt systems. The rebar can have an attached nut head or threads depending on the desired support type (active or passive) and/or tensioning procedure. Full column resin bolts are the most commonly used type for active support. They can either be post or pretensioned. Post tensioning is accomplished with torque applied to the nut and thread. Pretensioning is established by using a fast curing resin at the back of the hole followed by a slower setting resin. The forged nut on the rebar is torqued with a machine drill while the slower setting one is still solidifying. Resins help provide corrosion resistance to the encapsulated areas of the steel rebar. One disadvantage is the creep potential of resin grouts when introduced to high heat environments (Figure 4.13).

Inorganic Based Grout Anchored Rock Bolts

The most common inorganic grouting agent is Portland cement. Other similar grouting agents are Gypsum and Calcium Aluminate based cements. Usually cement grouts are

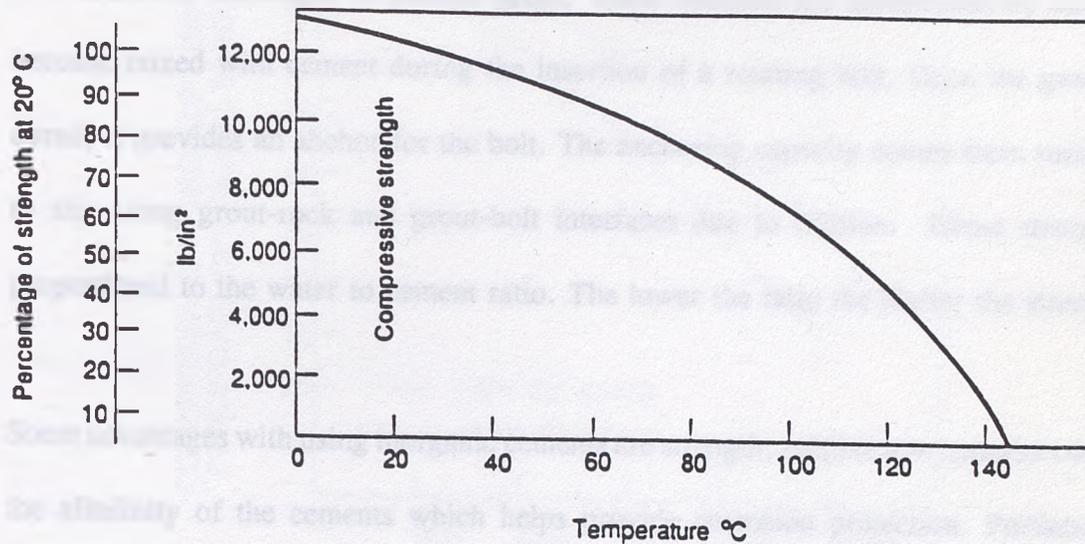


Figure 4.13 - Temperature vs. the strength of polymer resin (USBM, 1987)

installed by pumping a thick (.3:1-.5:1 water to cement ratios) grout medium into drill holes prior to bolt insertion. Grout can be pumped into the hole after installation by utilizing grout and breather tubes. Another way to introduce grout into the system is by using porous paper cartridges of grout which have been soaked in water. These cartridges are placed in the hole before the bolt. Some advancement has been made with resin like self contained cartridges of cement grout. Water droplets are surrounded by wax and become mixed with cement during the insertion of a rotating bolt. Once the grout has cured, it provides an anchor for the bolt. The anchoring capacity comes from resistance to slip along grout-rock and grout-bolt interfaces due to friction. Grout strength is proportional to the water to cement ratio. The lower the ratio the higher the strength.

Some advantages with using inorganic cements are strength, relative low material cost and the alkalinity of the cements which helps provide corrosion protection. Portland and Calcium Aluminate based cements retain most of their strength under high temperatures (USBM). Figure 4.14 displays the compressive strength of concrete under high temperatures. Grouts based on portland cement in general have similar properties to monolithic concrete (USBM, 1987).

The main drawbacks to inorganic grouts are slow cure time and sophisticated grout pumps are needed to pump the viscous cements. Another disadvantage to consider is that since these grouts are fluid like, they are likely to flow into any open joints or fractures. This may increase the volume of grout used.

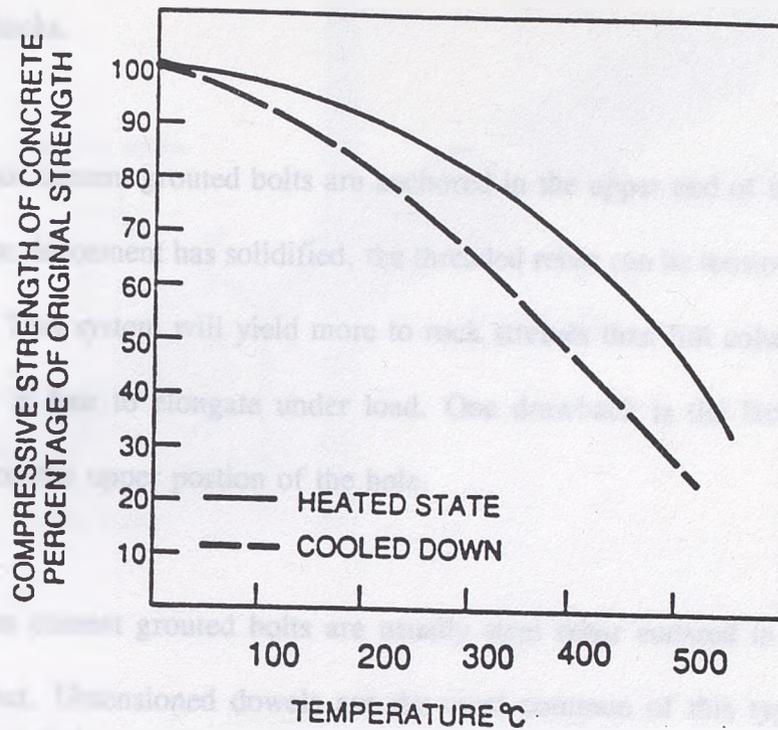


Figure 4.14 - Temperature vs. compressive strength of concrete (USBM, 1987)

Many variations of cement grouted bolts exist. They are either point anchor or full column based bolts with the grout acting as the anchor. Fiberglass and steel rebar and steel wire (cable) can be used in both of these ways. These bolt systems can be used either as active or passive support. The active systems can be tensioned by torque or by hydraulic jacks.

Point anchor cement grouted bolts are anchored in the upper end of the hole by cement grout. Once the cement has solidified, the threaded rebar can be tensioned if it is deemed necessary. This system will yield more to rock stresses than full column bolts, because more steel is free to elongate under load. One drawback is the lack of lateral shear resistance on the upper portion of the hole.

Full column cement grouted bolts are usually steel rebar encased in a full column of cement grout. Untensioned dowels are the most common of this type (Figure 4.15). Dowels can have a forged head or be plain rebar. These systems are activated by rock convergence. In certain cases, the bolt ends are threaded for post tensioning (Figure 4.16). Under certain high stress conditions, full column grout bolts may be too rigid. Only small portions of the bolt will be allowed to elongate, so the high strain rates will result in failure. A "debonded" bolt, which has smooth portion or a sleeve on the bolt, will allow more displacement capacity (Figure 4.17).

Mechanically anchored and grouted bolts utilize a mechanical anchor for pretensioning.

A specially adapted mechanical anchor bolt is installed into a drill hole

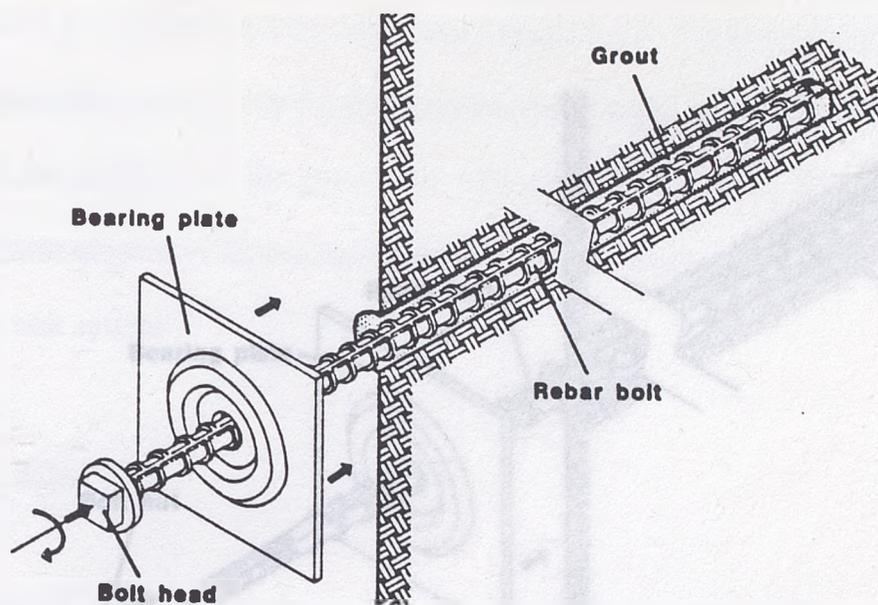


Figure 4.15 - Full column cement grout anchored untensioned bolt system (USBM, 1987)

Figure 4.17 - A spherical full column cement grout anchored bolt system (Douglas and Aron, 1983)

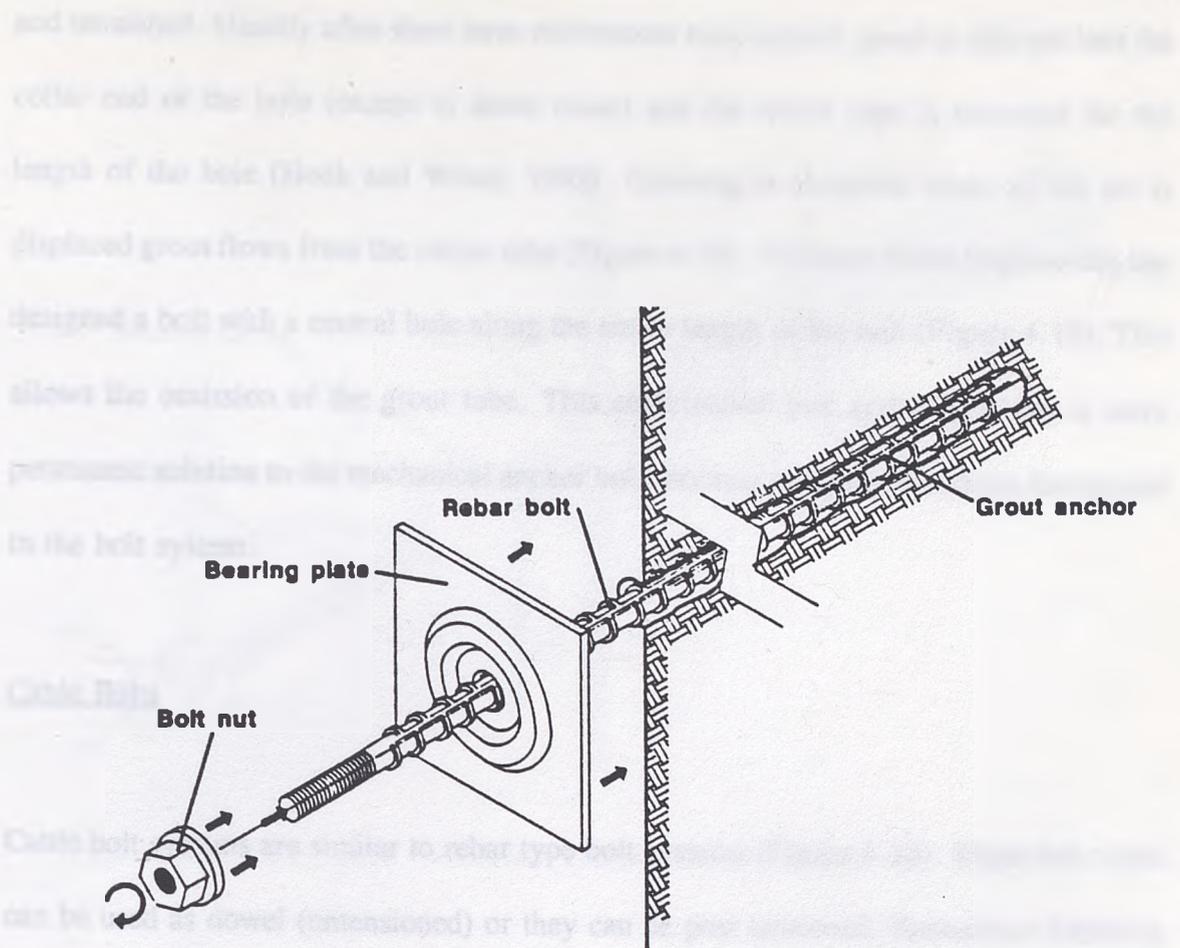


Figure 4.16 - Tensioned cement grout anchored bolt system (full column or point anchor type) (USBM, 1987)

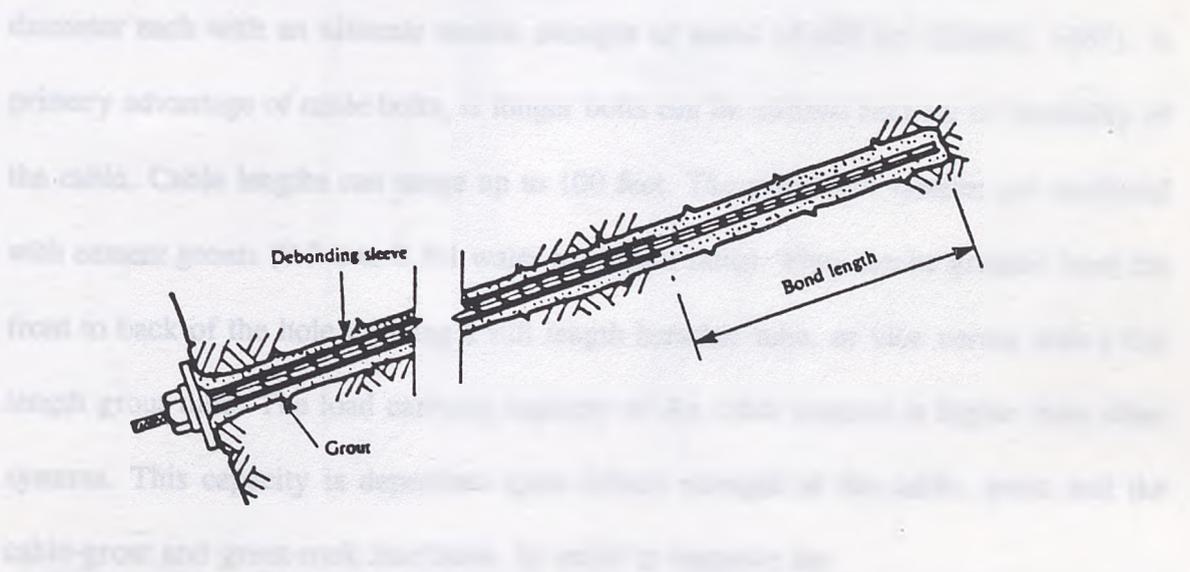


Figure 4.17 - A debonded full column cement grout anchor bolt system (Douglas and Arthur, 1983)

and tensioned. Usually after short term movements have ceased, grout is injected into the collar end of the hole (except in down holes) and the return pipe is extended for the length of the hole (Hoek and Wood, 1992). Grouting is complete when all the air is displaced grout flows from the return tube (Figure 4.18). Williams Form Engineering has designed a bolt with a central hole along the entire length of the bolt (Figure 4.19). This allows the omission of the grout tube. This combination bolt system provides a more permanent solution to the mechanical anchor bolt, because the grout maintains the tension in the bolt system.

Cable Bolts

Cable bolt systems are similar to rebar type bolt systems (Figure 4.20). These bolt types can be used as dowel (untensioned) or they can be post tensioned. Sometimes fastening devices such as the one on Figure 4.21 are used to attach plates to the front of the bolt system. Cable bolts are constructed of 7 steel wires which are 0.6 to 0.625 inches in diameter each with an ultimate tensile strength of about 58,000 lbf (USBM, 1987). A primary advantage of cable bolts, is longer bolts can be utilized because of flexibility of the cable. Cable lengths can range up to 100 feet. The cable bolt systems are anchored with cement grouts (0.3:1 to 0.5:1 water to cement ratio). They can be grouted from the front to back of the hole utilizing a full length breather tube, or vice versus with a full length grout tube. The load carrying capacity of the cable systems is higher than rebar systems. This capacity is dependent upon failure strength of the cable, grout and the cable-grout and grout-rock interfaces. In order to improve the

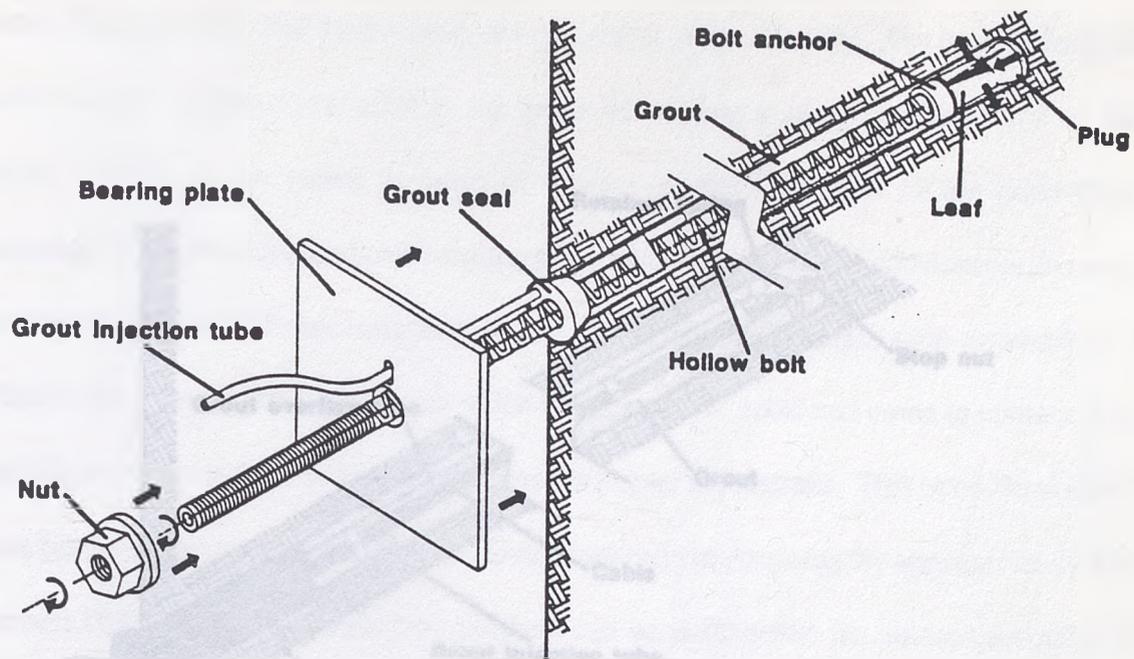


Figure 4.18 - Tensioned mechanical anchor bolt with cement grout (USBM, 1987)

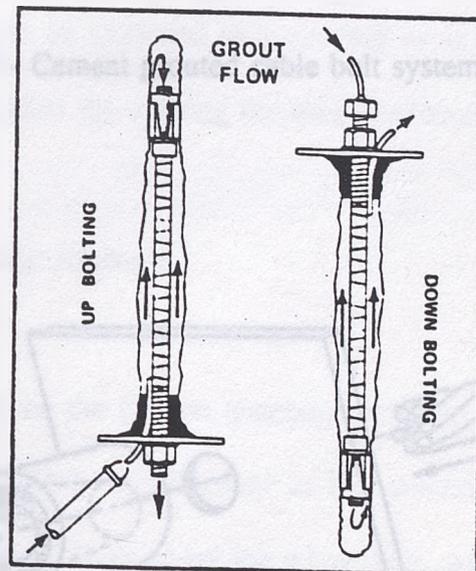


Figure 4.19 - A Williams Hollow-Core anchor can be grouted in up and down bolting applications (Williams Form Engineering Corp., 1992)

cable-grout interface, three friction inducing modifications have been introduced. They are steel buttons, grit coated resin coatings and bird cage configurations in the wires of cable (Figure 4.23). The steel buttons are placed at given intervals. The buttons help the load transfer process from cable to the grout by causing a compressional force on the grout in front of the button is sized of

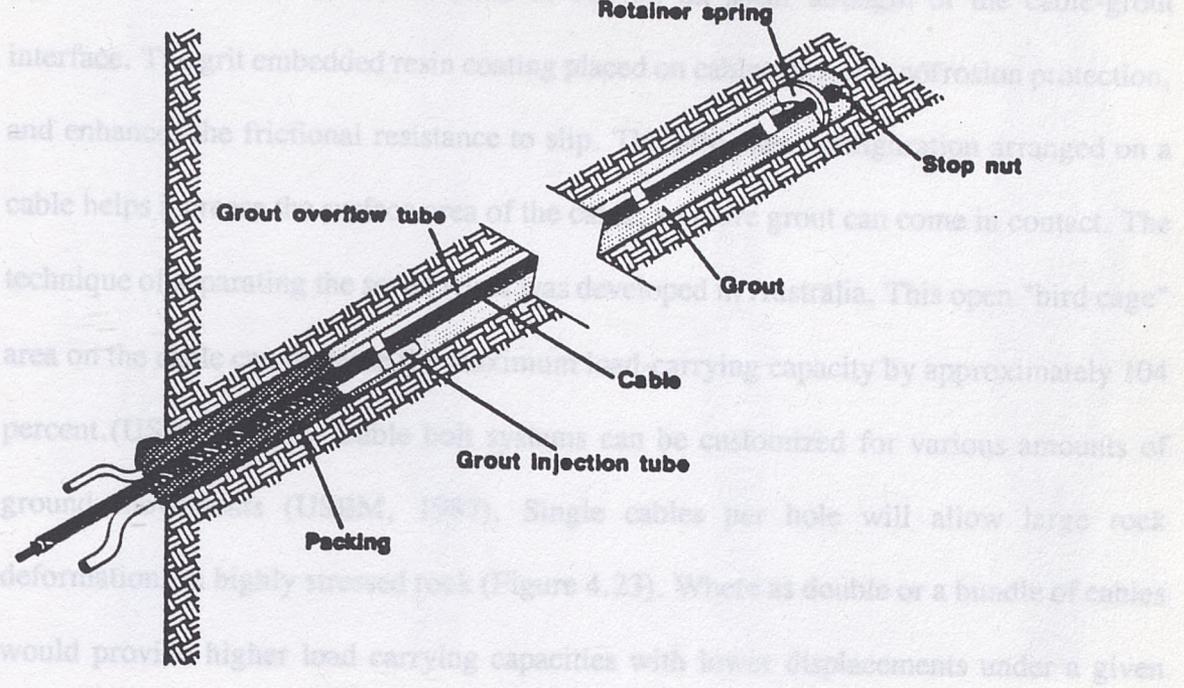


Figure 4.20 - Cement grouted cable bolt system (USBM, 1987)

what is required to support the existing the rock conditions.

4.4.3 Friction Stabilizer Anchors

Friction stabilizers rely on the friction generated between the cable and rock to provide its anchorage. This "friction" as it is sometimes referred to as, is caused by creating a wedge along the whole bolt, which pushes the bolt against

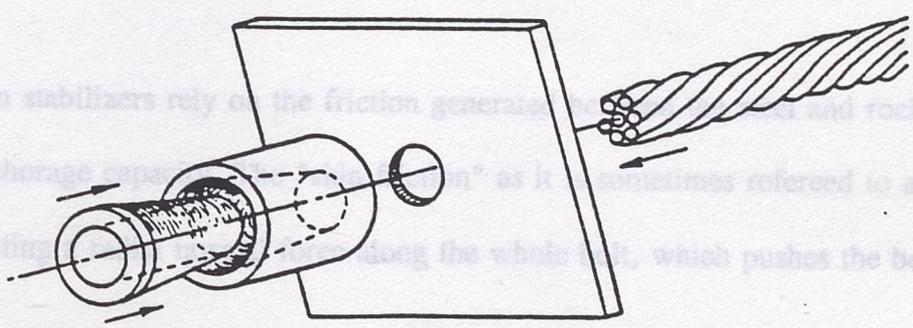


Figure 4.21 - Fastening device used to connect a bearing plate to a cable bolt (Choquet, 1991)

cable-grout interface, three friction inducing modifications have been introduced. They are steel buttons, grit bedded resin coatings and bird cage configurations in the wires of cable (Figure 4.22). The steel buttons are pressed at given intervals. The buttons help the load transfer process from cable to the grout by causing a compressional force on the grout in front of the button in stead of relying on shear strength of the cable-grout interface. The grit embedded resin coating placed on cables provides corrosion protection, and enhances the frictional resistance to slip. The bird cage configuration arranged on a cable helps increase the surface area of the cable, so more grout can come in contact. The technique of separating the seven wires was developed in Australia. This open "bird cage" area on the cable can increase the maximum load-carrying capacity by approximately 104 percent.(USBM, 1987). Cable bolt systems can be customized for various amounts of ground movements (USBM, 1987). Single cables per hole will allow large rock deformations in highly stressed rock (Figure 4.23). Where as double or a bundle of cables would provide higher load carrying capacities with lower displacements under a given stresses. Cable bolts can be exploited as a yielding or as a rigid systems depending on what is required to support the existing the rock conditions.

4.4.3 Friction Stabilizer Anchors

Friction stabilizers rely on the friction generated between the steel and rock to provide it's anchorage capacity. The "skin friction" as it is sometimes refereed to as, is caused by creating a radial normal force along the whole bolt, which pushes the bolt against

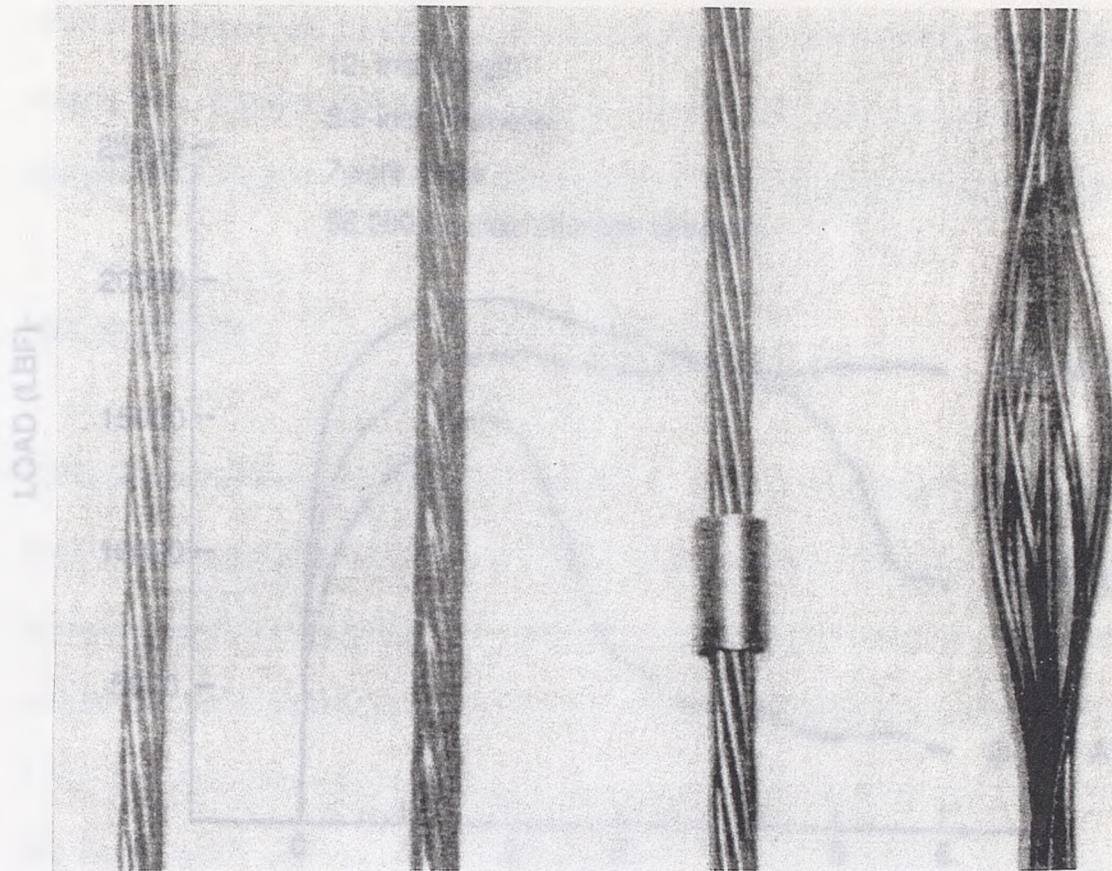


Figure 4.22 - Cable bolt variations arranged from left to right, conventional, epoxy-coated, conventional with steel button and birdcage (Goris, 1991)

Figure 4.23 - Diagrammatic of cable bolt variations (USFIM, 1987)

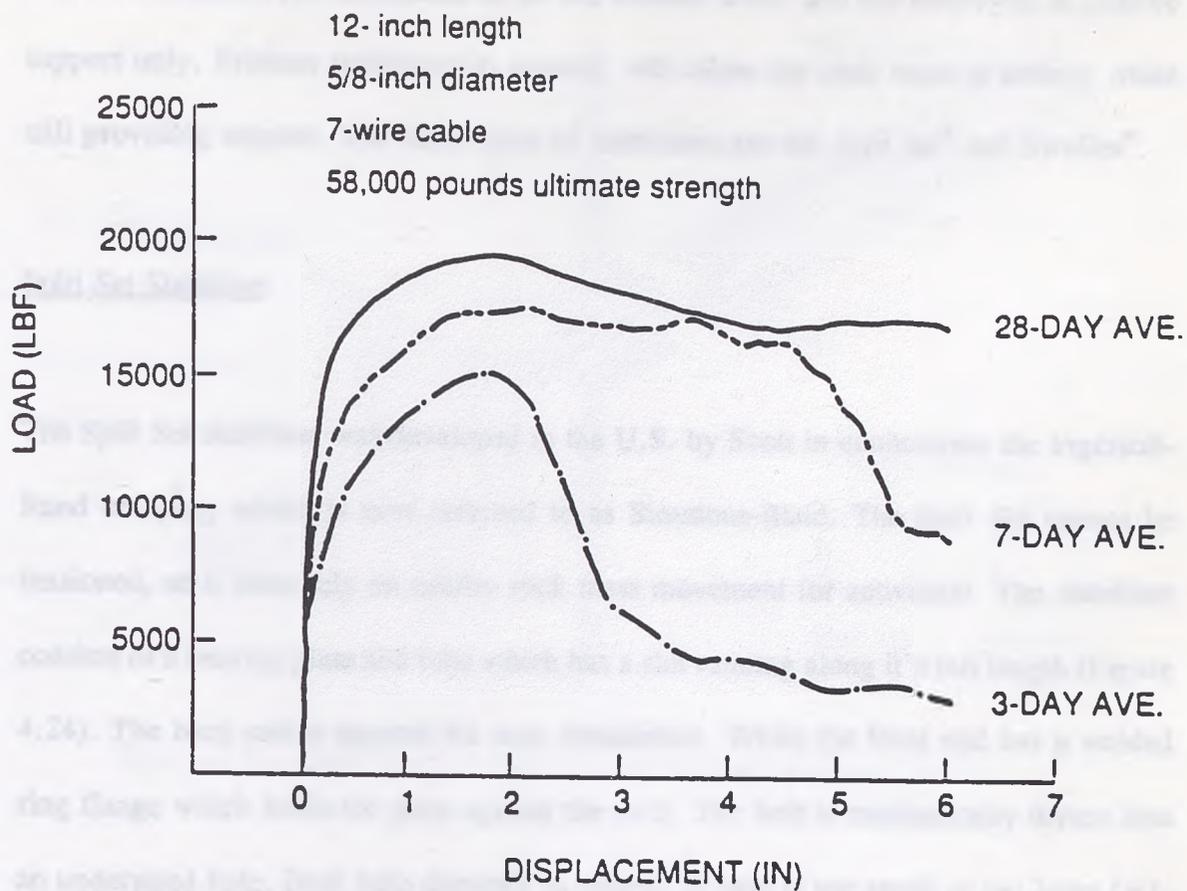


Figure 4.23 - Displacement vs. load for cable bolt systems (USBM, 1987)

the rock. The normal force is generated by driving an oversized tube into a drill hole, or by expanding a preformed undersized tube into the hole with internal hydraulic pressure. Friction stabilizers are considered to be full column bolts, and are employed as passive support only. Friction stabilizers in general, will allow the rock mass to deform while still providing support. The main types of stabilizers are the Split Set[®] and Swellex[®].

Split Set Stabilizer

The Split Set stabilizer was developed in the U.S. by Scott in conjunction the Ingersoll-Rand company which is now refereed to as Simmons-Rand. The Split Set cannot be tensioned, so it must rely on nearby rock mass movement for activation. The stabilizer consists of a bearing plate and tube which has a slot running along it's full length (Figure 4.24). The back end is tapered for easy installation. While the front end has a welded ring flange which holds the plate against the rock. The bolt is mechanically driven into an undersized hole. Drill hole diameter is critical. If hole is too small or too large (+/- 4mm) anchorage capacities decrease. The spring action of the compressed tube applies a radial normal force against the drill hole. This is a simple and quick process. (Hoek and Wood, 1992) stated this system worked well in moving or bursting ground. Conforms to lateral ground shifts which causes bolt to grip more tightly (Simmons-Rand, 1992). One drawback to the Split Set is that in corrosive conditions, rusting occurs very rapidly, because of the large surface area that is exposed to environment. Galvanized and stainless steel versions are available for

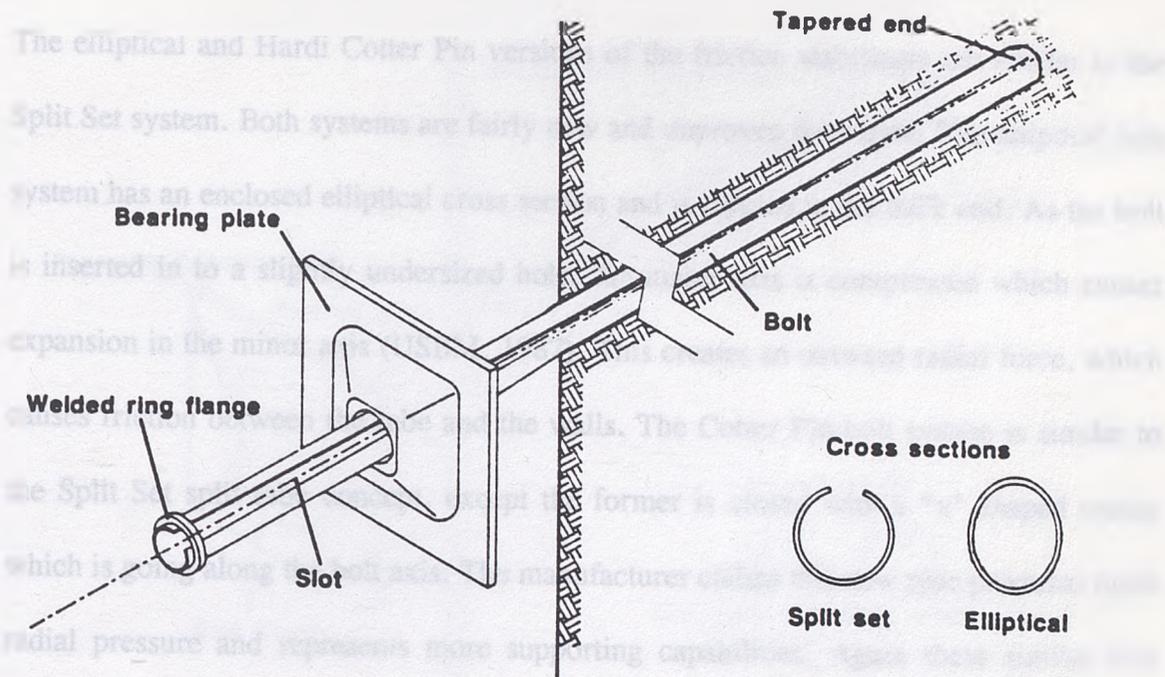


Figure 4.24 - Split Set and elliptical bolt systems (USBM, 1987)

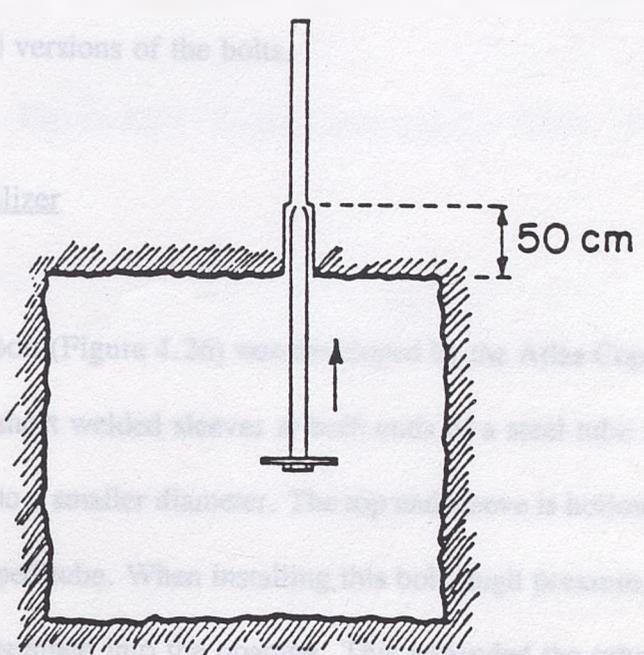


Figure 4.25 - Insertion of long Split Set bolts may require this technique (Choquet, 1991)

additional costs. The use of longer bolts which may bend unless the front of the hole is drilled with a an oversized drill bit is another drawback (Figure 4.25).

The elliptical and Hardi Cotter Pin versions of the friction stabilizers are similar to the Split Set system. Both systems are fairly new and unproven over time. The elliptical bolt system has an enclosed elliptical cross section and is tapered at the back end. As the bolt is inserted in to a slightly undersized hole, the major axis is compressed which causes expansion in the minor axis (USBM, 1987). This creates an outward radial force, which causes friction between the tube and the walls. The Cotter Pin bolt system is similar to the Split Set split tube concept, except the former is closed with a "v" shaped crease which is going along the bolt axis. The manufacturer claims this new type produces more radial pressure and represents more supporting capabilities. Again these similar bolt systems are susceptible to quality control, with respect to drill hole diameter (+/- 4mm undersized), and corrosion. Corrosion can be addressed with the use of galvanized and stainless steel versions of the bolts.

Swellex Stabilizer

The Swellex bolt (Figure 4.26) was developed by the Atlas Copco Company. The system consists of a short welded sleeves at both ends of a steel tube which has been collapsed and reshaped to a smaller diameter. The top end sleeve is hollow. This opening continues into the reshaped tube. When installing this bolt, high pressure, up to 29.6 mpa or 4300 psi, water is pumped into the opening. This expanded the tube

(Figure 4.26). The high hydraulic pressure causes the bolt to conform to any irregularities in the drill hole, thus interlocking the bolt to the rock mass (Figure 4.27). Expansion of the reshaped tube leads to some reduction in axial length, resulting in a slight tensioning of the bolt. Once the desired pressure is achieved, the pump automatically shuts off, so the water pressure can be released. A small amount of water leaves the tube after the inflation device is removed. This process is simple and quick and high rates of placement can be attained. With the flexibility of the inflatable bolt, longer bolts can be placed in smaller headings (Figure 4.28.) Lengths of this type of bolt range from 0.6 to 12.0 meters for the Standard Swellex bolts. For heavy duty and yielding support requirements, Atlas Copco has designed the "Super" and "Yielding" Swellex bolts, respectively. They have also devised coated and stainless steel versions to provide long term support in corrosive environments. One possible drawback to the use of high pressure in the inflation process is that under certain weak rock conditions, the surrounding rock may be fractured from the high pressure. This can weaken the bolt substantially or cause spalling near collar of the hole. A sleeve placed around the upper part of the hole may help solve this problem (Figure 4.29).

4.4.4 Yielding Rock Bolts (Sliding Nut and Spring Type)

Practical experience in mines has shown for mine roof stability under certain applications with roof bolts these systems are still rigid and more yieldable bolts types were required (Herbst, 1990). Yielding rock bolts are designed for use in highly

Figure 4.28 - Insertion of long section bolts (Goldberg, 1982)

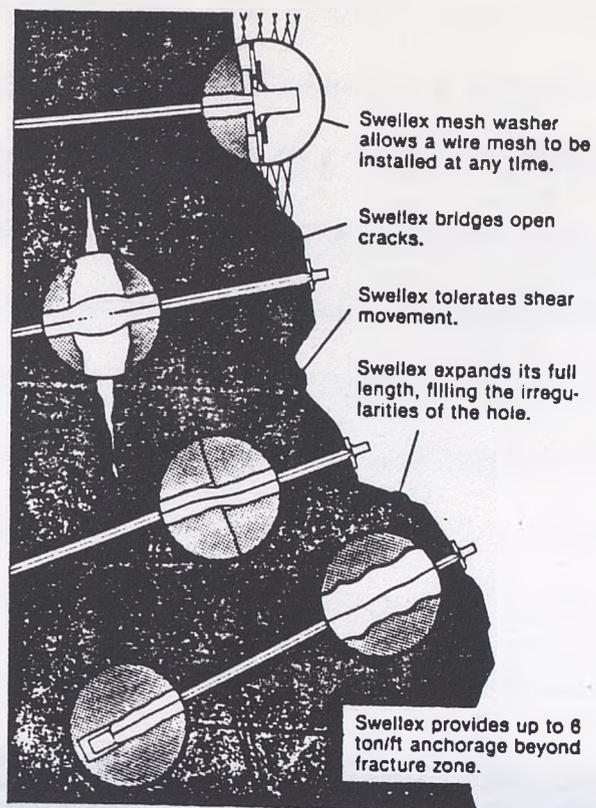


Figure 4.27 - Some basic attributes of the Swellex system (Stillborg, 1986)

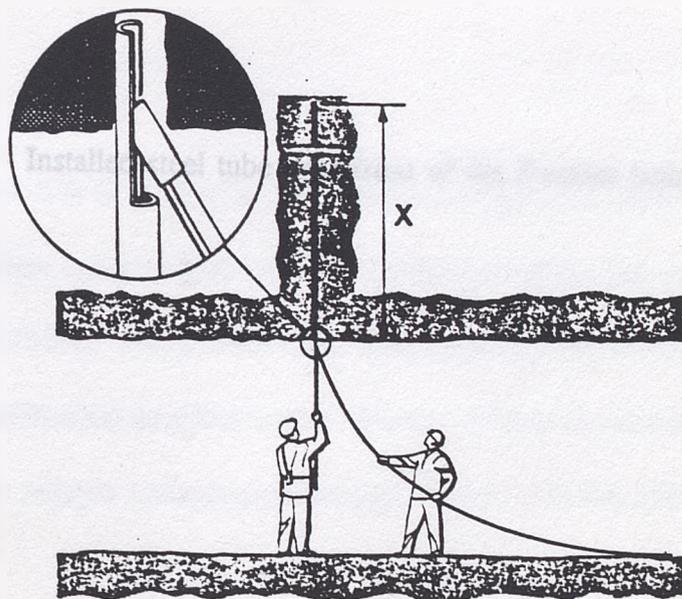
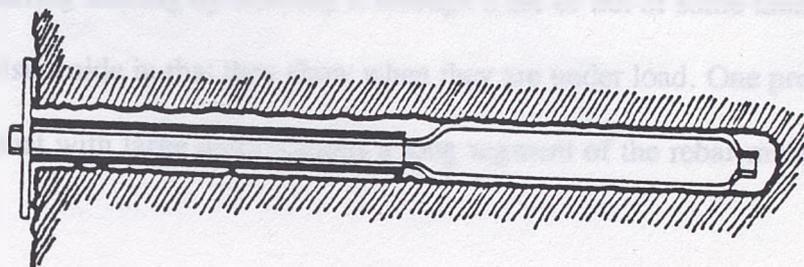


Figure 4.28 - Insertion of long Swellex bolts (Stillborg, 1986)

stressed ground where the bolts must maintain structural integrity while under great deformation. These bolts are also used in rock masses that require to stay loose, because they can resist dynamic loads without failure. Tensioning bolts can be put into various categories. They are the sliding nut, spring, wire and friction anchors. Although placed in other bolt categories some wire and friction anchors systems should be considered as pretensioning systems. The sliding nut and spring type will be discussed later. Several designs have been used but most have been made only used in the U.S. because of their cost (USM, 1971). Although for the various bolt types, the maximum yielding bolt systems generally are the same general. That of a bolt is continuously deformed during loading by drawing it through a hole or into a hole. These systems provide a yield point that is constant under load. One problem with these systems is that they are not as strong as the rock mass that they are installed in.



Helical Bolt

Figure 4.29 - Installed steel tube near front of the Swellex bolt (Choquet, 1991)

The Helical rock bolt (Figure 4.29) is a type of bolt that can be used where constant load must be maintained within the rock over a large range of deformations (USM, 1971). This bolt, which appears like a long spring, is anchored with either a mechanical or grout anchor. Further testing remains before this bolt can be established as a reliable underground support method (USM, 1972).

stressed ground where the bolts must maintain structural integrity while under going deformation. These bolts are also used in excavations susceptible to rock bursts, because they can resist dynamic loads without failure. Yielding rock bolts can be put into separate categories. They are the sliding nut, spring, cable and friction stabilizers. Although placed in other bolt categories some cable bolt and friction stabilizer systems should be considered as yielding systems. The sliding nut and spring type will be discussed further. Several designs have evolved, but none have been widely used in the U.S. because of their costs (USBM, 1987). Except for the Helical bolt (spring type), the remaining yielding bolt systems generally use the same principal. Part of a bolt is permanently deformed during loading by drawing it through a die or nut of some kind. These systems provide a visual aide in that they show when they are under load. One problem with these systems is that with large deformations a long segment of the rebar must stick out of the hole.

Helical Bolt

The Helical rock bolt (Figure 4.30) was designed at the U.S. Bureau of Mines. This bolt can be used where constant load must be maintained within the rock over a large range deformations (USBM, 1987). This bolt, which appears like a long spring, is anchored with either a mechanical or grout anchor. Further testing remains before this bolt can be established as a reliable underground support method (USBM, 1987).

Convey and Orificia Bolt

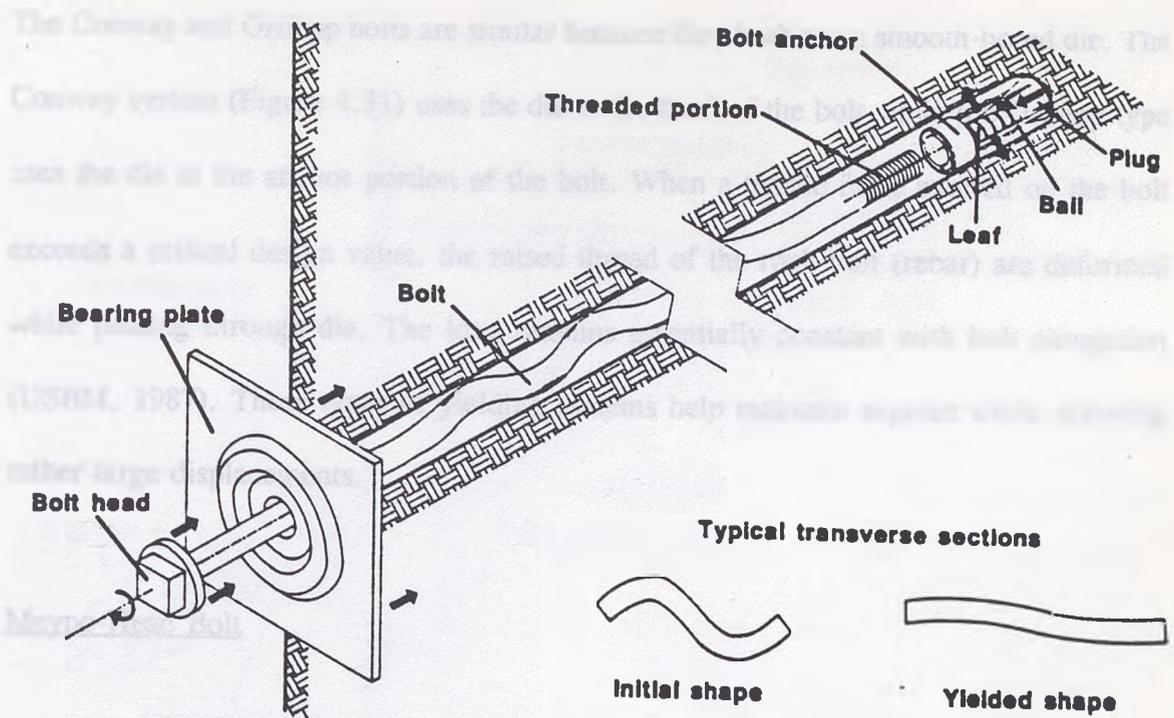


Figure 4.30 - Helical bolt system (USBM, 1987)

The Meयो Head system utilizes an ordinary steel bolt (Figure 4.31). The back end of the bolt is either pointed and/or mechanically or with grooves. The latter usually provides a foot or two out of the hole to allow for large deformations. The front portion of the bolt is fitted with a special anchor head (Meयो-head). This type of die has a number of hardened steel cylinders. These cylinders strip grooves along the rebar as the rock mass deforms. This helps maintain a constant load, while allowing the opening to converge. The Meयो-head can be customized for any desired extensional load by altering the number, angle and geometry of the cylinders (USBM, 1987).

Conway and Ortlepp Bolts

The Conway and Ortlepp bolts are similar because they both use a smooth-bored die. The Conway system (Figure 4.31) uses the die at the head of the bolt while the Ortlepp type uses the die at the anchor portion of the bolt. When a tensile force exerted on the bolt exceeds a critical design value, the raised thread of the rock bolt (rebar) are deformed while passing through die. The load remains essentially constant with bolt elongation (USBM, 1987). These types of yielding systems help maintain support while allowing rather large displacements.

Meypo-Head Bolt

The Meypo-Head system utilizes an ordinary steel rebar (Figure 4.32). The back end of the bolt is either pointed anchored mechanically or with grout. The rebar usually protrudes a foot or two out of the hole to allow for large deformations. The front portion of the bolt is fitted with a special anchor head (Meypo-head). This type of die has a number of hardened steel cylinders. These cylinders strip grooves along the rebar as the rock mass deforms. This helps maintain a constant load, while allowing the opening to converge. The Meypo-head can be customized for any desired extensional load by altering the number, angle and protuberation of the cylinders (USBM, 1987).

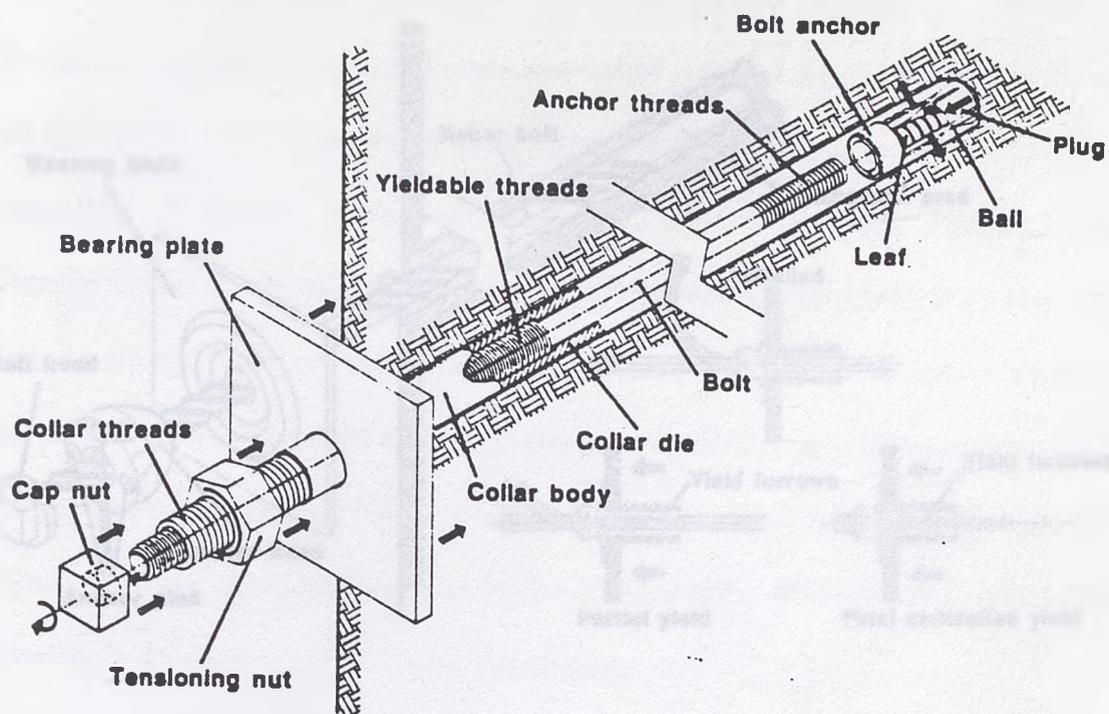


Figure 4.31 - The Bureau of Mine's Conway Yielding bolt system (USBM, 1987)

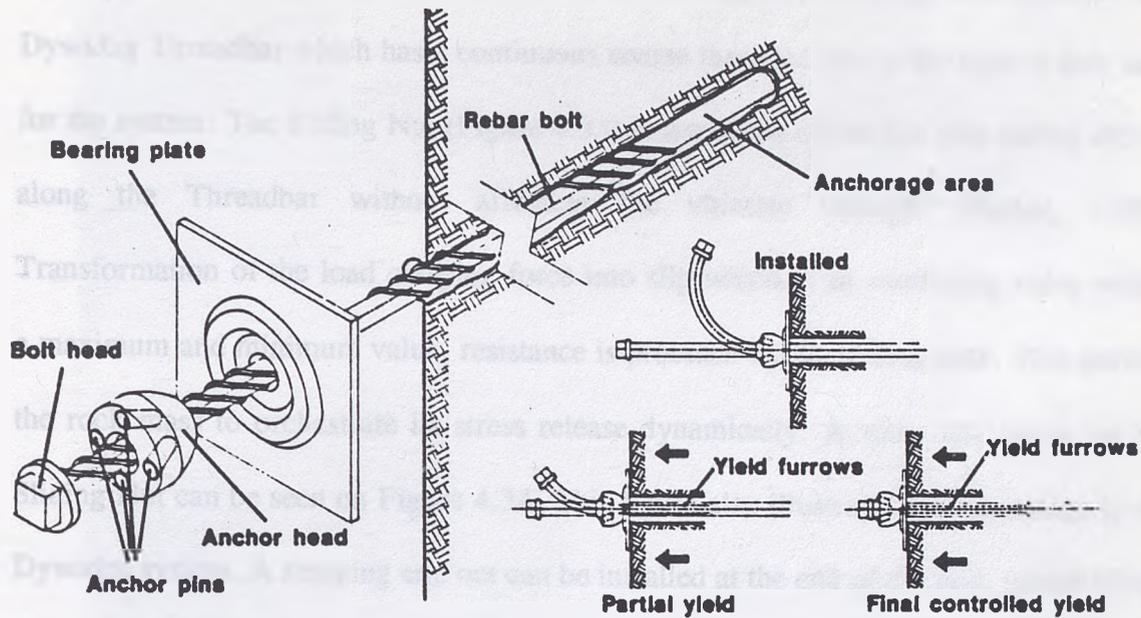


Figure 4.32 - The Meypo-Head Yielding bolt system (USBM, 1987)

4.4.3 Secondary Bolt Anchorage

Secondary Spacing

Wire mesh is often used in the roof of underground excavations to prevent rock spalling and sloughing. They are kept in place by rock bolts. The mesh can be made of various sizes. The two main types of mesh are #9 gauge (1/2 inch by 2 inch) welded wire mesh and #9 gauge (2 inch by 2 inch) expanded metal. The welded wire mesh has a tensile strength of 3100 and 3200 lbs. respectively (Crawford, 1981).

Dywidag's Sliding Nut

Dywidag's Sliding Nut is another version of this type of yielding bolt system. The Dywidag Threadbar which has a continuous coarse threaded ribs is the type of bolt used for the system. The Sliding Nut (Figure 4.33) deforms the coarse bar ribs during sliding along the Threadbar without affecting the ultimate strength (Herbst, 1990). Transformation of the load carrying force into slip which is an oscillating value within a maximum and minimum value, resistance is provided by the sliding path. This permits the rock mass to orchestrate its stress release dynamically. A static test curve for the Sliding Nut can be seen on Figure 4.34. This figure also illustrates one advantage to the Dywidag system. A stopping end nut can be installed at the end of the bolt, which brings Dywidag's Threadbar to its ultimate bearing capacity.

4.4.5 Secondary Bolt Accessories

Secondary Support

Wire mesh is often used in the roof of underground excavations to help prevent spalling and sloughing. They are kept in place by rock bolts. The mesh can conform to uneven spaces. The two main types of mesh are #4 gage (4 inch by 4 inch) welded wire mesh and #9 gage (2 inch by 2 inch) chain-link mesh. The welded wire and chain-link meshes have load-bearing capacities of 3100 and 6300 lbs, respectively (Choquet, 1991).

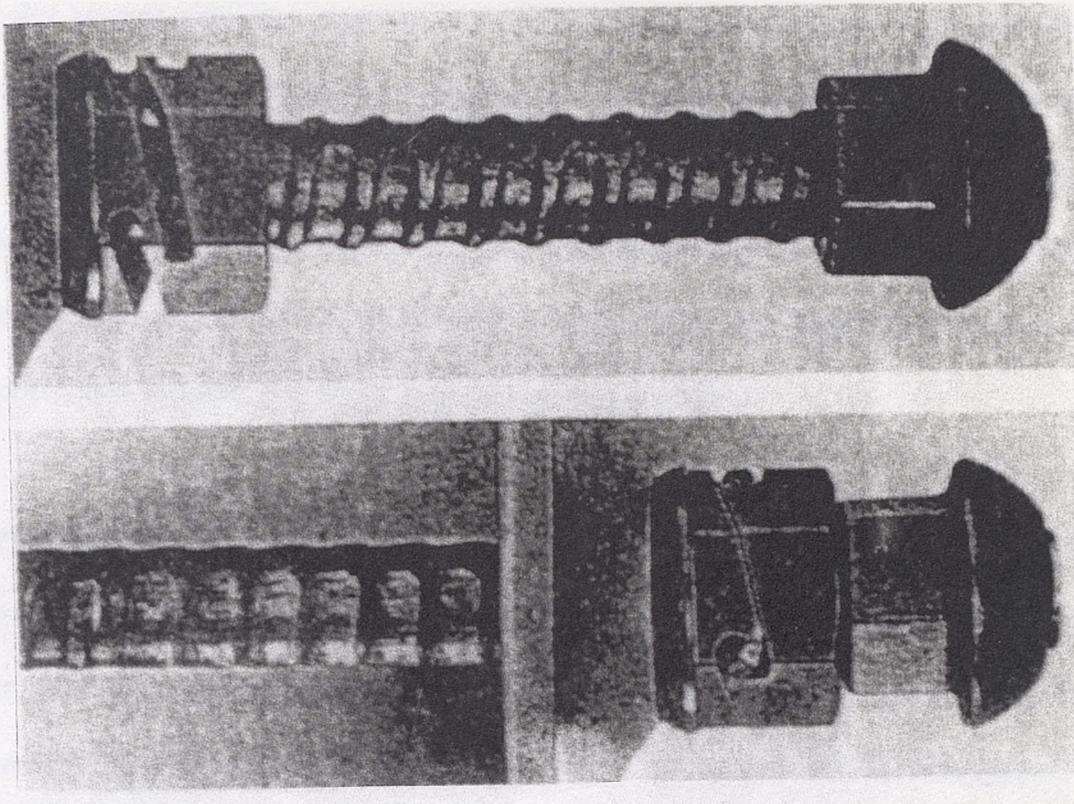


Figure 4.33 - Dywidag's Sliding Nut placed on threadbar: before and after sliding (Herbst, 1990)

Figure 4.33 - Dywidag's Sliding Nut placed on threadbar: before and after sliding (Herbst, 1990)

Bearing Plates

Usually bearing and head plates are constructed of steel, but wood is still sometimes used. The plates when affixed to the rock bolts will allow the surrounding rock to bear on the plate. This action makes the bolts more efficient, because it securely supports more rock. The plates help maintain tension in mechanically anchored rock bolts, and also help distribute the load on bolts when installed in angle holes. The installation of secondary support, such as rock web strips requires the use of plates.

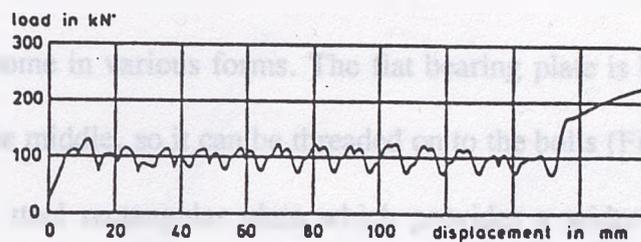


Figure 4.34 - Displacement vs. load for Dywidag's Sliding Nut (Herbst, 1990)

Bearing Plates

Usually bearing and head plates are constructed of steel, but wood is still sometimes used. The plates when affixed to the rock bolts will allow the surrounding rock to bear on the plate. This action makes the bolt more efficient, because it actually supports more rock. The plates help maintain tension in mechanically anchored rock bolts, and also help distribute the load on bolts when installed in angle holes. The installation of secondary support, such as mesh and straps requires the use of plates.

Bearing plates come in various forms. The flat bearing plate is basically a square plate with a hole in the middle, so it can be threaded on to the bolts (Figure 4.35). The header plate is a large steel rectangular plate which provides a wider load distribution. The domed or dished bearing plate (Figure 4.35) resists deflection under load, because of its convex downward shape (USBM, 1987). The donut bearing plate is also convex downward, but is partially recessed in the center. This plate acts more like a spring allowing slight deformations in the system without loading the bolt. Header boards are rectangular wood planks that range in thickness. They are used in conjunction with bearing plates. In areas of rock movements they are deformed by the bolt, thus permitting the bolt system to deform and yet maintain a constant load. Disfigured wood helps to indicate the system is enduring rock movement.

Figure 4.35 - The domed and rectangular plates (Chapman, 1991)

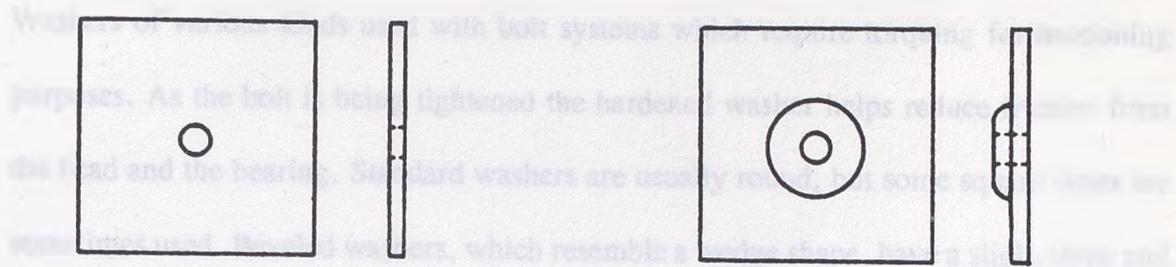


Figure 4.35 - The flat and dome bearing plates (Choquet, 1991)

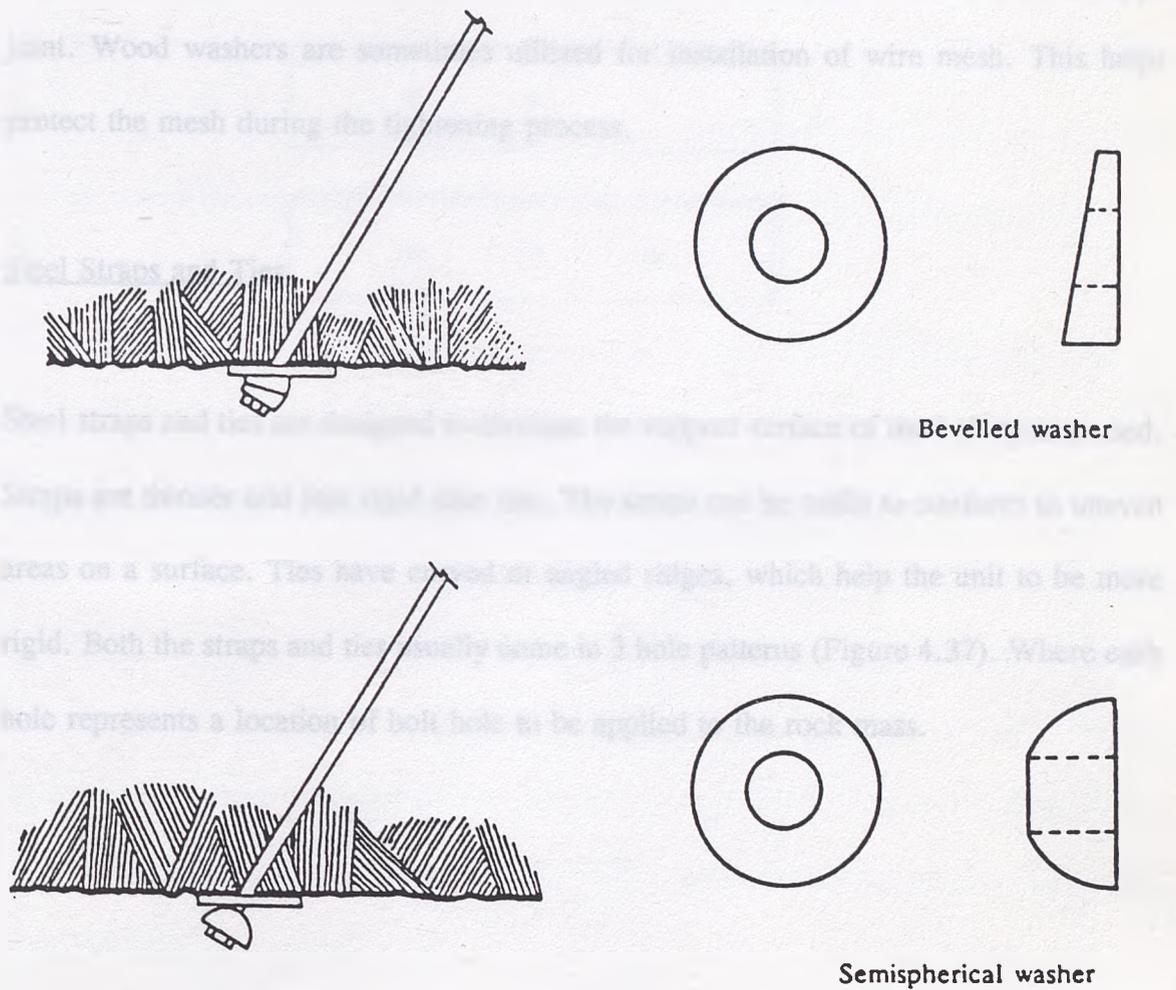


Figure 4.36 - The bevelled and semispherical washers (Choquet, 1991)

Washers

Washers of various kinds used with bolt systems which require torquing for tensioning purposes. As the bolt is being tightened the hardened washer helps reduce friction from the head and the bearing. Standard washers are usually round, but some square types are sometimes used. Beveled washers, which resemble a wedge shape, have a slight slope and are used for rock bolts holes with axis not at right angles to rock surface (Figure 4.36). The spherical face on the spherical washer (Figure 4.36) can accommodate a greater angle between the rock surface and bolt hole. It works much like a ball and socket type joint. Wood washers are sometimes utilized for installation of wire mesh. This helps protect the mesh during the tightening process.

Steel Straps and Ties

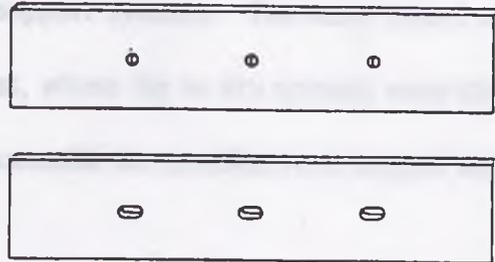
Steel straps and ties are designed to increase the support surface of the bolt system used. Straps are thinner and less rigid than ties. The straps can be made to conform to uneven areas on a surface. Ties have curved or angled ridges, which help the unit to be more rigid. Both the straps and ties usually come in 3 hole patterns (Figure 4.37). Where each hole represents a location of bolt hole to be applied to the rock mass.

GROUND CONTROL SYSTEMS

5.1 Introduction

A crucial aspect of this research program was determining the circumstances under which rock support is being used. Four high-stress underground projects were visited. The purpose of this visit to observe different rock conditions was to assess the effectiveness of the implemented rock

deforming rock mass, when the rock is in a state of high stress, and how to



assess the effectiveness of the implemented information.

Four sites were visited, the Yucca Mountain Test Plant (YMP), Lake Umbagog, Dorset Lake and Dorset Mine. The YMP site is located in western part of Ontario, near

Figure 4.37 - Two common types of steel straps (Choquet, 1991)

Tennant, Ontario. Each of these sites has an active high-stress, high-stress underground ground control program. The objective of this visit was to assess the effectiveness of the implemented rock support systems for the different stress levels and deformation anticipated at Yucca Mountain's main development area. Table 5.1 provides a characteristic overview of each site visited and includes the main systems used for high stress areas.

CHAPTER V

UNDERGROUND SITE INVESTIGATIONS

5.1 Introduction

A crucial aspect of this research project was investigating underground facilities where rock support is being used. Four major underground projects were visited. The purpose of this was to observe different rock conditions and assess the effectiveness of the implemented rock support systems. The main extent of selected location sites was a deforming rock mass, where the in situ stresses were such that rock was converging or failing, and how successful the installed rock support was in controlling deformation.

Four sites were visited, the Waste Isolation Pilot Plant (WIPP), Kidd Creek, Detour Lake and Dome Mines. The WIPP site is located 30 minutes west of Carlsbad, New Mexico. The Kidd Creek, Detour Lake and Dome Mines are located in the vicinity of Timmons, Ontario. Each of these four sites has active rock mechanics, and an aggressive ground control program. The instability at these sites might mimic and possibly shed some light on applicable systems for the thermal stress induced rock deformations anticipated at Yucca Mountain's waste containment areas. Table 5.1 displays an characteristic overview of each site visited and includes the bolts systems used at each mine for high stress areas.

Mine Site	Rock Types	In Situ Stresses ($K = \sigma H / \sigma V$)	Average Rock Strengths Unconfined (MPa, psi)	Difficult Ground Conditions (High Stress, Rock Failure) *	Rock Bolts Utilized (Effectiveness) A=Working; B=Not Working; C=No Results
WIPP	Rock Salt	$K = 1.0$	27.6; 4000	yes	Dywidag thread bars with Sliding nuts (A); Super Swellex bolts (C)
Kidd Creek Mine	Meta-volcanics (Rhyolite)	$K = 1.5$	160; 23,200	yes	Cable Bolts (A); Super Swellex (A)
Detour Lake Mine	Meta-volcanics (Silicified mafics)	$K = 2.0$	166; 24,080	yes	Cable Bolts (A); Super Swellex (A)
Dome Mine	Meta-sediments and Meta volcanics	$K = 1.4$	114; 16,530	yes	Super Swellex (A)

* May be similar to the probable thermal induced rock deformations and fault crossings expected at Yucca Mountain's waste emplacement areas.

Table 5.1 Comparisons of Utilized Rock Bolts for Visited Sites

5.2 Waste Isolation Pilot Plant Site

Introduction

The Waste Isolation Pilot Plant or WIPP site is located in SE New Mexico, 26 east miles of Carlsbad, New Mexico (Figure 5.1). The WIPP site is a research and development facility of the US Department of Energy. This facility, operated by the Westinghouse Waste Isolation Division, was developed to demonstrate and later provide safe disposal of low level defense generated radioactive waste.

Geology

The WIPP site is developed in a 3000 foot thick salt formation which is approximately 225 million years old. The large expanse of uninterrupted salt beds provides a predictable environment. It is in a stable region so seismic activity should be low. Specifically the underground site is located in the Salado Formation which is approximately 1500 feet thick (Figure 5.2). The quality of the rock salt is poor and has no commercial value.

Mining Methods

By late 1990, over 10 miles of underground excavation headings had been mined, including 4 shafts (Figure 5.3). The only working level is located 2150 feet below

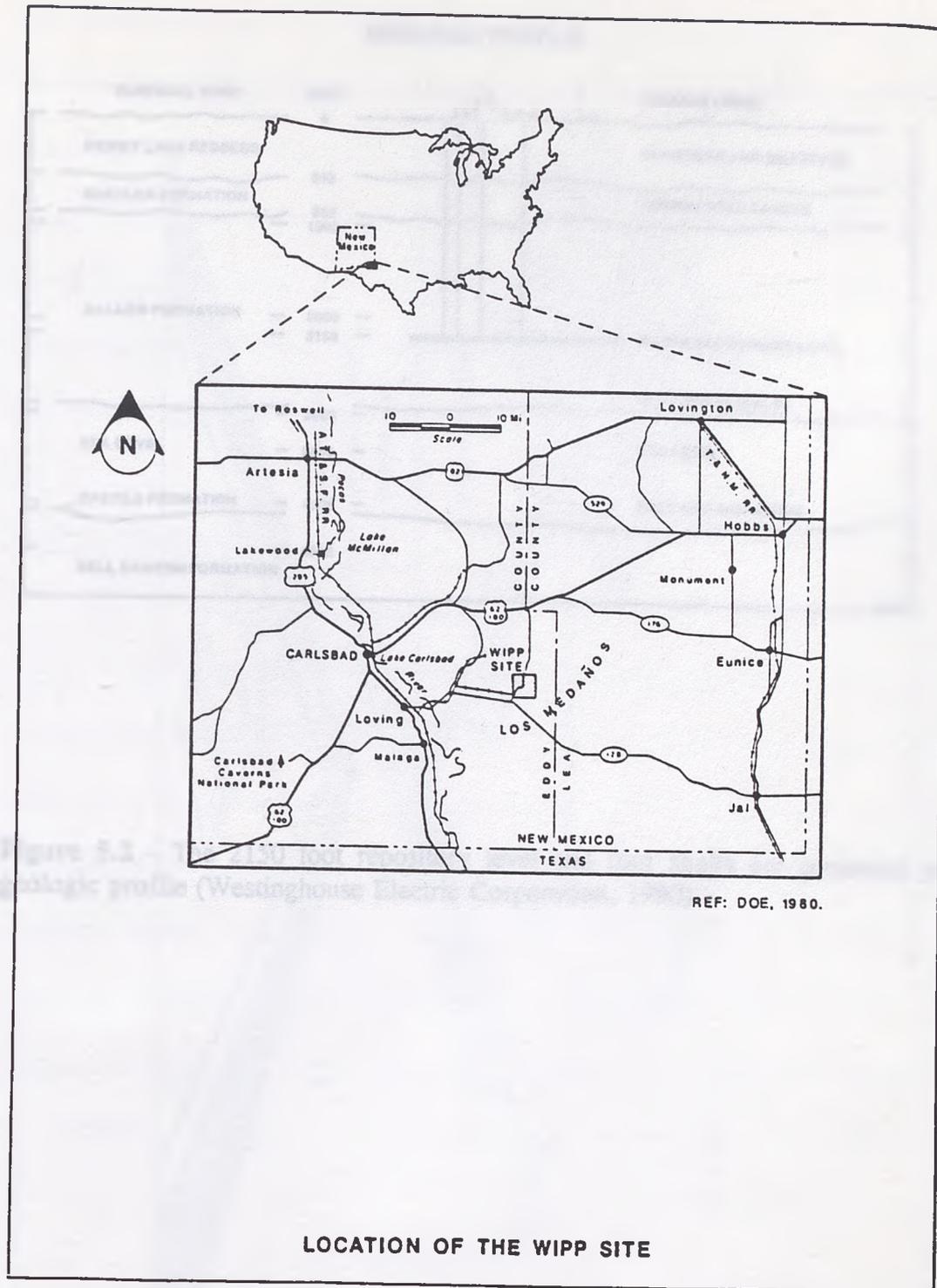


Figure 5.1 - Location of the WIPP site (DOE/EIS-0026-DS, 1989)

GEOLOGIC PROFILE

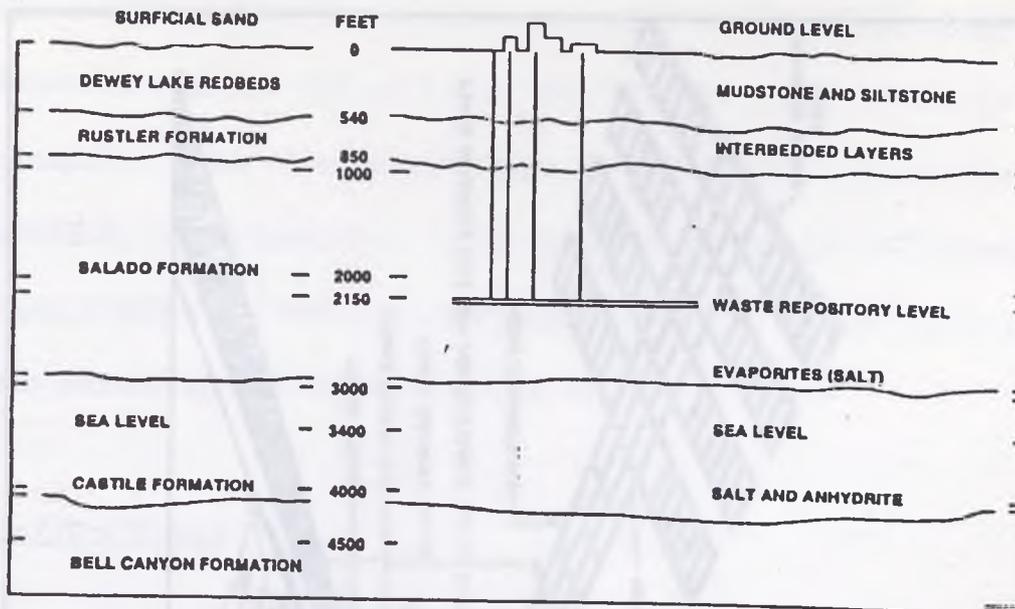


Figure 5.2 - The 2150 foot repository level and four shafts are presented with the geologic profile (Westinghouse Electric Corporation, 1990)

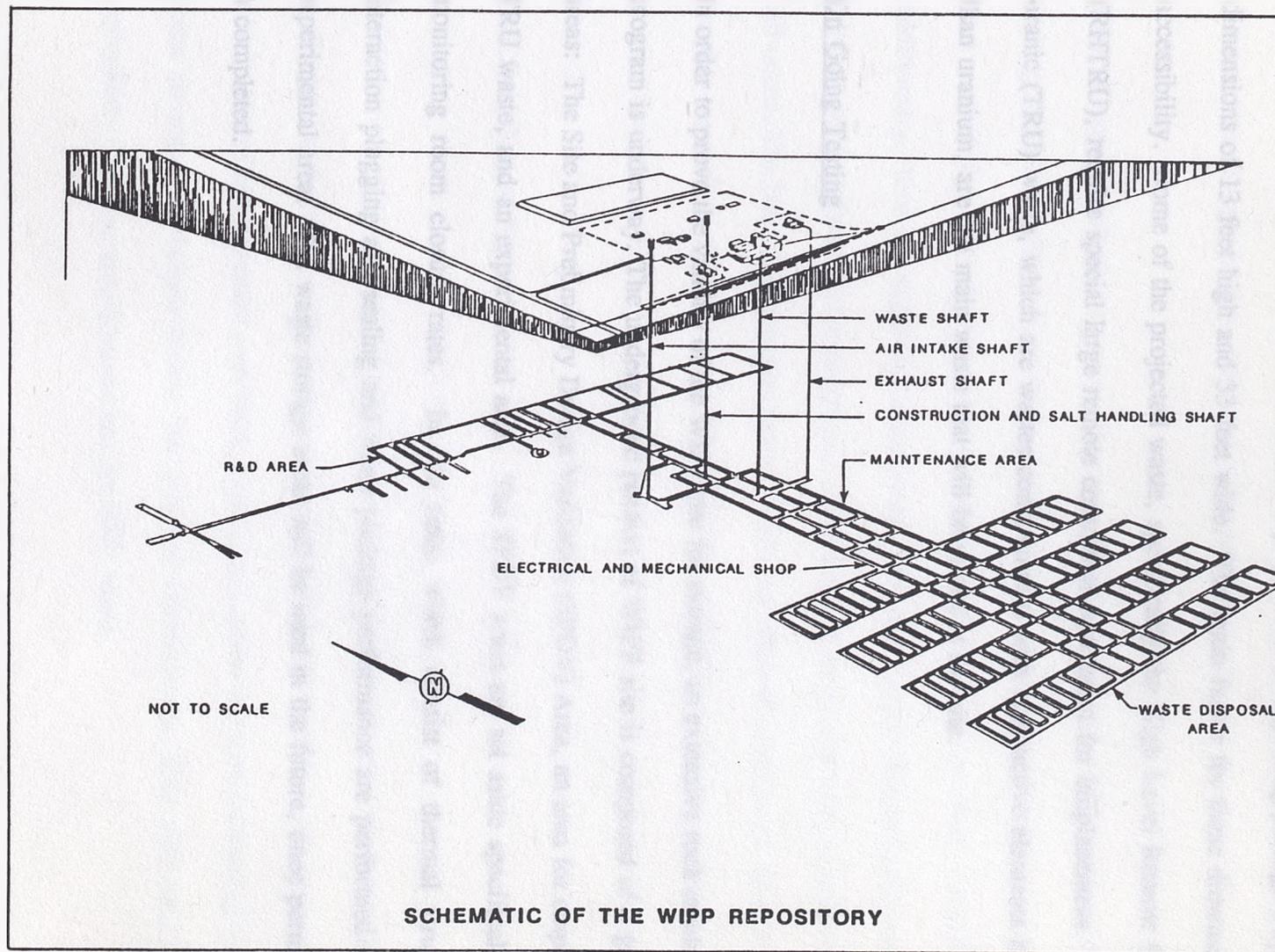


Figure 5.3 - Schematic of the WIPP repository (DOE/EIS-0026-DS,1989)

the surface. Mine crews using continuous miners have excavated approximately half, or 800,000 tons rock salt so far. Proposed waste storage locations are arranged in eight sections or panels, with several rooms in each panel. Room-adjoining passage ways have dimensions of 13 feet high and 33 feet wide. The main factor for these dimensions is accessibility. Some of the projected waste, specifically the High Level Remote Handle (RHTRU), require special large remote controlled equipment for emplacement. Trans-uranic (TRU) waste, which are wastes contaminated with radioactive elements heavier than uranium, are the main waste that will be stored at the site.

On Going Testing

In order to prove the validity of the WIPP site for storage, an extensive rock mechanical program is underway. The underground portion of WIPP site is composed of 3 general areas: The Site and Preliminary Design Validation (SPDV) Area, an area for emplacing TRU waste, and an experimental area. The SPDV areas are set aside specifically for monitoring room closure rates. In situ tests, which consist of thermal structural interaction plugging and sealing and waste package performance are performed in the experimental area. The waste storage areas will be used in the future, once permitting is completed.

In Situ Stresses

The in situ stresses found at the WIPP site are hydrostatic. The original presumption that these salt beds were under hydrostatic stresses, were confirmed by hydraulic fracturing. These stress measurements were completed in the experimental area. The actual compressive strength of the rock salt is 27.6 MPa or 4000 psi.

Rock salt exhibits plastic behavior, allowing a characteristic creep. This poses a problem in designing a bolt system that will provide adequate support. Since these openings have been around for at least eight years, supporting the ground is more complicated. Additionally, there is a small clay seam located between 4 and 5 feet above the roof. If bolts are not installed into the component salt formation above this clay, a wedge failure will occur. The bolts help create columns which help resist the horizontal shear stresses found to occur from the roof corners up to the small clay layer. In some areas the floor has also heaved.

Rock Bolt Support Systems

The main rock support used at the WIPP site are rock bolts. Most of the bolts used are mechanical anchors, as well as some resin anchored and Swellex bolt systems. A combination of resin grouted rock bolts, beams, and cables are used in some of the test rooms in order to reduce inches per year of convergence. This will enable the continuation of a 5 yr emplacement test for TRU waste.

The mechanical shell anchor bolts are the most common bolt utilized at WIPP. These

lengths range from 5 to 8 feet. The 8 foot lengths are more common, because of the clay layer found 4.5 feet above the roof in certain areas of the mine. The anchor bolt systems don't reach their yield point because the shell easily moves along the interior of the drill hole due to the low strength and creep characteristics of the rock salt.

In 1991, a panel of technical experts recommended the use of resin anchored rock bolts. They also specified the addition of flexibility into the support system. A yielding system would be able to adjust to the convergence, while still providing support to the roof. The WIPP site chose the company Dywidag and Celtite to supply their tendons and resin respectively. No. 7 and No. 8 Dywidag rebars are anchors with resin in the bottom of the hole. Usually the bolt hole lengths are 10 feet in length. A special yieldable nut the Sliding Nut was designed by Dywidag to work with its continuously threaded rebar systems. The yieldable nut deforms the coarse bar ribs bar during sliding along the bar. The sliding resistance gives a controlled yield to the system.

This year (1993) another yielding system was tested. Atlas Copco has designed a yielding Swellex bolt. The bolt is referred to as the Ex1 Swellex bolt. This bolt has been specially annealed to allow the bolt to elongate under deformation of surrounding rock. This system in effect yields with the converging rock while still providing support. So far the tests procedures for the Ex1 Swellex at the WIPP site have proven the system has possibilities, but more testing is necessary.

5.3 The Kidd Creek Mine

Introduction

The Kidd Creek mine is located in the Kidd Township forming part of the Timmons Mining District. The mine is approximately 27 kilometers North of Timmons, Ontario, Canada and is approximately 700 kilometers (420 miles) north of Toronto (Figures 5.4 and 5.5).

Geology

The orebody was discovered in 1963 during a follow up drilling program. Extensive Airborne Magnetometer surveys conducted over the previous 6 years helped delineate the program. This copper-zinc-silver massive sulphide deposit lies at the west end of the Archean age Abitibi Greenstone Belt. The ore body's host rocks are a sequence of rhyolite volcano clastics. These rocks have been through pre and post ore deformations. The ore occurs in two main deposits. The south and north ore bodies, separated by a major east-striking normal fault. The south orebody exhibits poor metal zonation, while north orebody has more defined metal zones.

Figure 5.4 - Location of the Kidd Creek Mine (Ontario Minerals Map 2004)

Mining Methods

Mining at Kidd Creek has been done in stages. The first stage was an open pit to a



Figure 5.4 - Location of the Ontario Province (Ontario Minerals Map 2024)

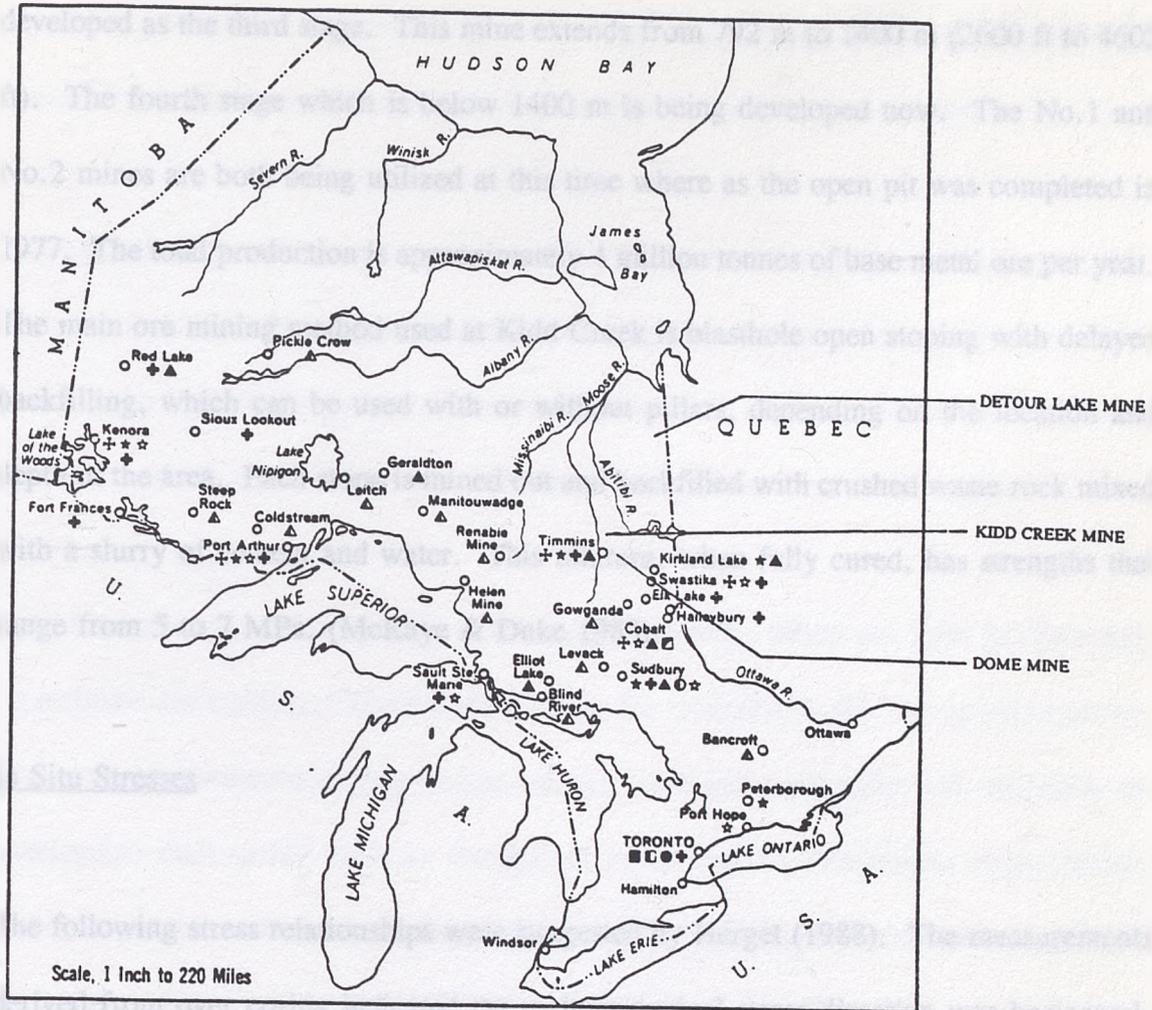


Figure 5.5 - Location of the visited mine sites (Ontario Minerals Map 1957A)

depth of 220 meters (722 feet). The second stage was an underground mine known as No.1 mine, which extends from 244 m to 792 m (800 ft to 2600 ft). The No.2 mine was developed as the third stage. This mine extends from 792 m to 1400 m (2600 ft to 4600 ft). The fourth stage which is below 1400 m is being developed now. The No.1 and No.2 mines are both being utilized at this time where as the open pit was completed in 1977. The total production is approximately 4 million tonnes of base metal ore per year. The main ore mining method used at Kidd Creek is blasthole open stoping with delayed backfilling, which can be used with or without pillars, depending on the location and depth of the area. Each stope is mined out and backfilled with crushed waste rock mixed with a slurry of cement and water. This mixture, when fully cured, has strengths that range from 5 to 7 MPa. (McKaye & Duke 1983).

In Situ Stresses

The following stress relationships were suggested by Herget (1988). The measurements derived from over coring indicated the major principal stress direction was horizontal. The values ranged from 1.1 to 1.46 times the vertical stress ($\text{Sig V} = 0.027D$, $D = \text{Depth}$). The average unconfined compressive strength of the rock is 160 MPa (Yu, Henning, Counter, 1984).

Rock Bolt Support Systems

Many types of bolt systems are utilized at the mine site. They are mechanical shell anchor bolts, Split Sets, resin anchored bolts, Swellex bolts and cable bolts. The ground control engineer uses experience of near ground support and localized rock mechanic character sites to determine a correct bolt system or combination of them.

Mechanical shell anchors are mostly used in little to moderate ground movement areas, and also in rock deforming areas with wood squeeze blocks. The blocks help define visually the load on the system. The shell anchor bolt is susceptible to blast and other vibrations which cause the system to become untensioned.

Split Set bolts are used in the ribs of the excavation only. Mesh and Split Set bolts help to maintain rib stability. Their limited use is due probably to low corrosion resistance and poor performance during testing. Resin anchored rock bolts are replacing the mechanical shell anchor bolts for the light to medium ground movement areas, because the resin provides a better anchoring capacity. The 1.5 to 3 meter rebars are supplied by Dywidag and the resin is supplied by Celtite.

Atlas copco Swellex and Super Swellex bolts are used in the light to medium and medium to large ground movement areas respectively. The Swellex bolts are used also for hanging mesh. The Super Swellex bolts were emplaced in highly stressed areas of the mine, and were successful in limiting the deformations in these areas. The Swellex bolts are considered the "primary" support for converging ground.

Cable bolt systems are used at Kidd Creek. One or two smooth cables are grouted in a hole up to 10 m long. Their main uses are for large ground movement areas especially near faults and for the strengthening of pillars. The birdcage and or buttons will be tested in the future. Cable bolts are being installed in the roofs of suspected rock burst areas in order to minimize damages. This aspect is becoming more important at the mine site.

5.4 The Detour Lake Mine

Introduction

Placer Dome's Detour Lake Mine (DLM) is situated 300 km northeast of Timmons, Ontario (Figure 5.5). The mine site certainly qualifies as remote, being in northern Ontario and 100 km from James Bay. There are two ways to access the mine. The first is by plane and the second is a 3 hr journey by vehicle from Timmons. The last hour and half is along a maintained dirt road.

Production commenced with an open pit in 1984 which was developed to the 120 meter level. In 1987, the underground mine was developed and includes a timbered 612 meter shaft and a 20% grade decline. The decline reaches the 100 meter level and is connected to ramp system, which spirals in the hanging wall from the 160 to 460 meter levels. The ramp provides equipment access to the orebody.

Geology

The Detour Lake Mine gold deposit is located at the northern part of the Abitibi Orogenic Belt (Figure 5.6). A complex history of intermittent faulting and extrusive activity is reflected by the mines ore depositional history. Silicified mafic flows with chert and talc-chlorite are the primary host rocks. The steeply dipping lenticular ore zones are associated with east trending low angle structures. Three separate ore environments can be found in the deposit, they are the Main, Talc and Quartz zones.

Mining Methods

Mining the Detour Lake orebody is done with Mechanized Cut and Fill, Longhole and Captive Cut and fill methods. Basically, the ore zones are mined at spans of 20 meters with post pillars for added support, although some spans up to 30 meters have been mined with additional cable bolt supports. Once the spans are mined, the lower portion is filled with crushed waste mixed with a slurry of cement and water.

In Situ Stresses

In situ stresses were derived from strain recovery measurements. The Door Stopper method with four gage biaxial strain cells were used in three sites. Elastic strain recoveries were measured in the footwall, hanging wall and ore zone of the fine-grained rock strata. The results reveal that the maximum principal stress lies in the

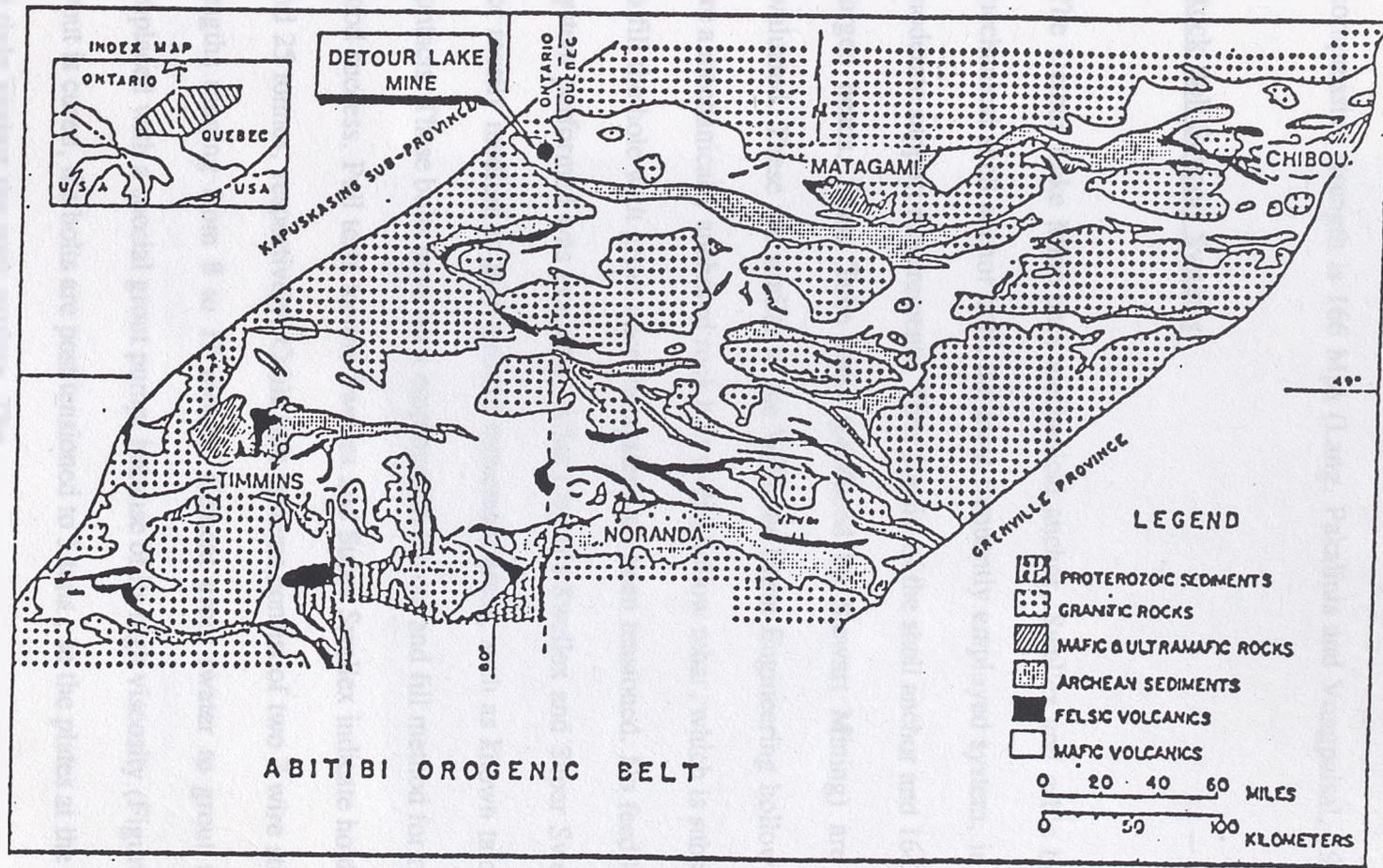


Figure 5.6 - The regional geology around the Detour Lake Mine area (Arjang and Herget, 1985)

horizontal plane. The average horizontal stress is approximately twice the measured vertical stress component which is comparable with calculated overburden pressures [Sig V = 0.0260-0.0324 Mpa/m (overburden load) (Herget, 1987)]. The average unconfined compressive strength is 166 Mpa (Lang, Pakalinis and Vongpaisal, 1992).

Rock Bolt Support Systems

The Detour Lake Mine uses mechanical anchor, Swellex, and cable bolt systems. The mechanical shell anchor bolt, the most frequently employed system, is used for light to moderate support requirements. They consist of the shell anchor and 16-20mm rebar with forged heads. Some Tube bolts (produced by Stewart Mining) are now just being evaluated. These are similar to the Williams Form Engineering hollow rock bolt. They are a mechanically anchored rock bolt with a hollow rebar, which is subsequently utilized to fill the hole with grout once the system has been tensioned. No feedback on how this system performed was available. Atlas Copco's Swellex and Super Swellex are utilized for mostly medium to high ground movement areas, such as known talc zones and fault contacts. These bolts have been employed in the cut and fill method for roof support with good success. Pull tests for the Swellex and Super Swellex indicate holding forces of 12 and 25 tonnes, respectively. Cable bolt systems consist of two 7 wire strand cables with lengths ranging from 8 to 17 meters. Cement grout (water to grout ratio, 0.3:1) is emplaced with a special grout pump, because of its high viscosity (Figure 5.7). Once the grout is cured, the bolts are post tensioned to 5 tons, so the plates at the front of the bolt fit tight against the rock surface. The

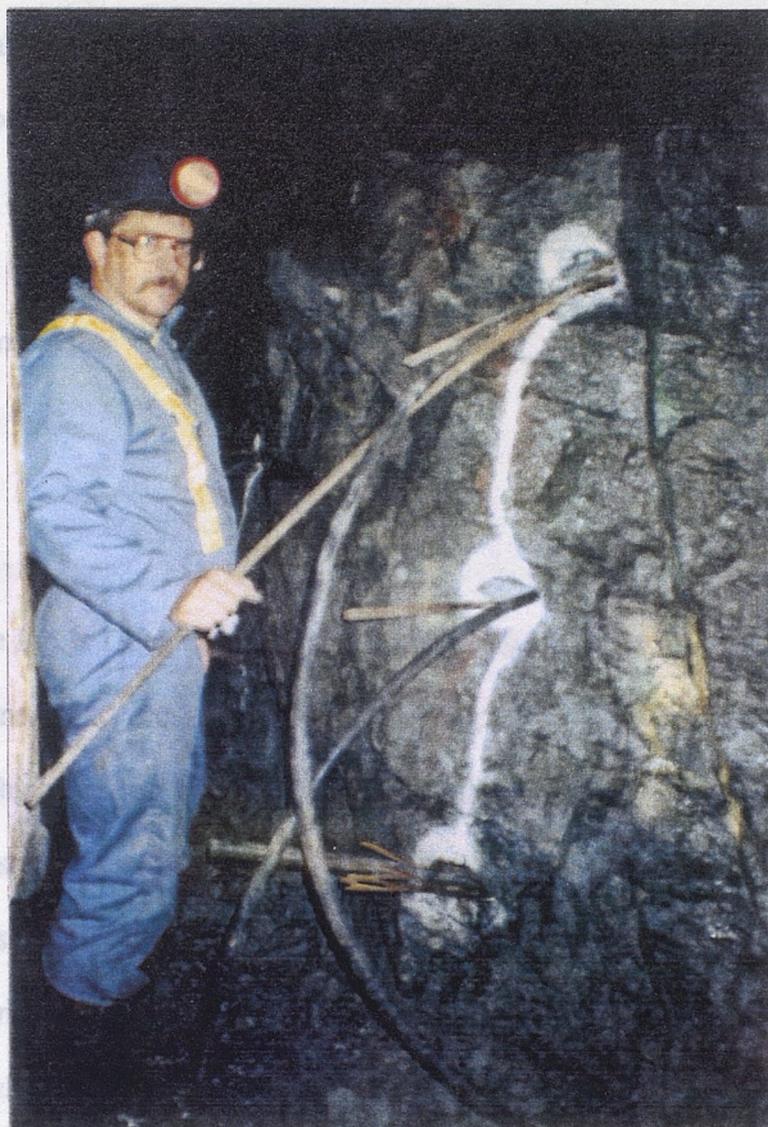


Figure 5.7 - Three double cable bolts prepared for grouting at the Detour Lake Mine

main use of cable bolts is pillar enhancement and rib and roof support near high ground movements areas, such as faults. In certain areas mine post pillars were replaced by additional cable bolting in the roof of the stope. Shotcrete was employed in areas of faults in order to prevent moisture losses to fault gouge. This prevention helps to stabilize the area.

5.5 The Dome Mine

Mining Methods

Introduction

Placer Dome Inc.'s Dome mine (Figure 5.5) is located in the Timmons Mining District (original name was Porcupine Mining District). The actual location is next to South Porcupine which lies just east of Timmons, Ontario. A party of prospectors, headed by John S. Wilson (Placer Dome Inc, 1983) discovered the Dome mine in 1909. Large quartz domes laden with gold were found by the prospectors. After, 84 years, an area of 2 miles long by 1 mile wide and 1 mile deep has been developed for gold production. The mine, which is the "granddaddy" of Placer Dome, has produced over 12 million ounces of gold.

Geology

A combination of sediments, meta-sediments, dacite and andesite flows, felsic pyroclastic rocks and two different porphyries reveal a complex geological picture (Figure 5.8). The

structural settings, including the mines location on the south limb of the Porcupine syncline, have contributed to the over all pod type orebody. There are four main ore classifications. Type 1 consists of long narrow veins of Ankerite or Quartz Tourmaline contained in Schist. Type 2 deposits are lenticular or irregular en-echelon or stockwork veins in massive rock. The type 3 is a gold associated pyrite dissimination in rock, and silicified Greenstones are considered to be the type 4 deposit.

Mining Methods

The 5400 foot (1646m) #8 shaft is the main production shaft. The 7th through 26th levels are active, with over 36 stopes in operation. Underground mining methods, mainly Cut and Fill and Longhole, make up 77% of the total gold production. The remaining 23% of production comes from a soon to be expanded open pits. Stope sizes range from small (2 meters) to large (10 meters) depending on ore types. Once a stope is completed, the level is filled with either waste rock, hydraulic fill or dewatered tails.

In Situ Stresses

As in the other mines visited in the Timmons area, the major and intermediate stresses are aligned horizontally. The horizontal stresses are on average 1.4 times larger than verticle stresses ($\text{Sig V} = 0.027D$, $D = \text{Depth}$). CanMet calculated these in situ

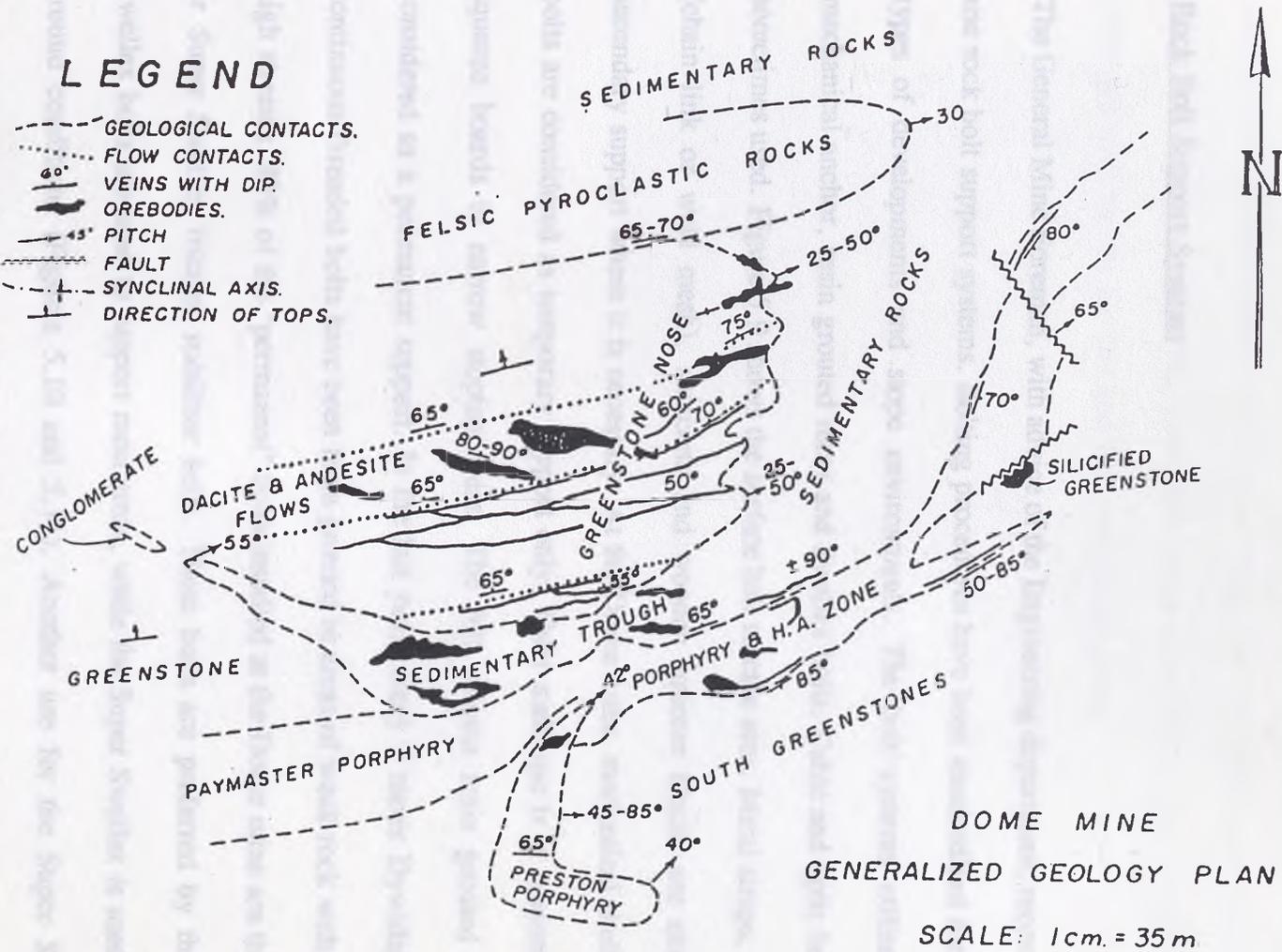


Figure 5.8 - Generalized geology map of the Dome mine (Placer Dome Inc., 1989)

stresses from evaluations of Young's Modulus and Poisson's ratio, from uniaxial tests and investigated stress levels and their orientations. The rocks at Dome Mine have an overall unconfined compressive strengths of 114 Mpa (Placer Dome Inc., 1993).

Rock Bolt Support Systems

The General Mine Foreman, with advice of the Engineering department, recommends the use rock bolt support systems. Bolting procedures have been standardized for all known types of developments and stope environments. The bolt systems utilized are the mechanical anchor, resin grouted rebar and Swellex bolts. Cable and Split Set bolts are sometimes used. Figure 5.9 shows the surface bolt storage area. Metal straps, wire mesh (chain link or weld mesh), shotcrete and wooden squeeze blocks are employed as secondary support where it is necessary. At the Dome mine, mechanical shell anchored bolts are considered as temporary support only. Their main use is in conjunction with squeeze boards in narrow stoping areas. The full column resin grouted rebar is considered as a permanent support. In the last year, many 2 meter Dywidag's 19mm continuous threaded bolts have been resin grouted in areas of weak rock with relatively high stresses. 80% of the "permanent" bolts installed at the Dome mine are the Swellex or Super Swellex friction stabilizer bolts. These bolts are preferred by the miners. Swellex bolts are used to support most areas, while the Super Swellex is used for poor ground conditions (Figures 5.10 and 5.11). Another use for the Super Swellex is prepinning or reinforcement of the next level to be blasted. The bolt length goes beyond the next cut, so when blasting is completed the roof remains supported. Pull tests

performed on the regular type of bolt resulted in holding forces up to 18 tons. Split Set friction stabilizers are only used on the ribs of the openings. A new type of friction stabilizer has seen some use at the Dome mine. The Hardi Cotter Pin bolt, which is like the Split Set, has only recently come out on the market. No results were available on how they performed at the mine yet. Cable bolts systems are also used on a limited basis only. They are used mainly for Cut and Fill roof reinforcement.



Figure 5.9 - Surface bolt storage area at the Dome Mine



Figure 5.10 - Super Swellex bolts installed for added support near mechanical anchor bolts and straps. Other bolts installed in the rock to help the mechanical anchor bolts and mesh maintain stability.



Figure 5.11 - Swellex bolts installed in the rib to help the mechanical anchor bolts and mesh maintain stability

The Tropicana Springs salt's expansion environment is discussed in the Technical Report (DOE/RW 2001) (1999). The rock chemistry, volume of gas, and the amount of gas being sampled, shows a low concentration of gas. The gas concentration is on the order of 1 ppm. This low concentration of gas is in equilibrium with the surrounding environment. The gas concentration is low because the dissolved oxygen content of the gas is low. The gas concentration is low because the dissolved oxygen content of the gas is low. The gas concentration is low because the dissolved oxygen content of the gas is low.

Chapter VI

**APPLICABLE BOLT SYSTEMS
FOR THE YUCCA MOUNTAIN REPOSITORY****6.1 Introduction**

Previous work, including (Zimmerman 1987), (USBM 1987) and (Hardy & Bauer, 1991), indicates the necessity of specially designed support systems for portions of the Yucca Mountain Project. The internally generated heat from the emplaced waste canisters will disperse and affect the near and mid field areas the most. The elevated temperatures which are approaching 200°C, could impair the performance of certain bolt systems by inducing corrosion, weakening anchorage systems, by overloading a rigid bolt system with thermal induced convergence of the opening, or cause new fracture system to develop and may create more support problems.

The Topopah Springs tuff's repository environment is discussed in D.O.E. Reports (DOE/RW 0073 1986). The water chemistry, assumed to be similar to results of drill hole sampling, shows a low water flux and a near neutral pH. Chloride ion concentrations are on the order of 6 ppm (very low) and the dissolved oxygen content is at equilibrium with air at ambient temperatures under higher temperatures, which would reduce the dissolved oxygen content and subsequently lower the rate of corrosion. Under these conditions, uniform corrosion would be suspected as the major factor in strength

reduction of plain carbon and HSLA (High Strength Low Alloy) steels (USBM 1987). Often low chloride ion (Cl^-) concentrations indicate a low possibility of pitting or crevice corrosion. The strength of resins at high temperatures is suspect (Figure 4.13). The reduction in strength of the grout anchor would reduce the effectiveness of the bolt system. Because of this characteristic, the epoxy and polyester resin grouts and resin cemented fiber bolts will not be recommended for possible use in the near and mid field areas until further long-term testing has been undertaken.

The heated rock mass will probably undergo thermal expansion around openings. The expansion will be more evident under ambient temperatures (Zimmerman 1987). Inorganic grouts such as portland and calcium aluminate based cements should be considered for further testing for the repository. The calcium aluminate based grout has tolerance to both high temperatures and a wide range of pH, but due to the cost, information is scarce on its use in the underground support industry.

Portland based cement has similar characteristics to concrete. Exposure to high temperatures ($75\text{-}200^\circ\text{C}$) causes dehydration which may weaken the strength of the grout, but exposure for expected temperatures in the mid field should only reduce the strength by 20%. Both these cement grouts need to be tested further under repository conditions.

Special rock bolt design will most likely be required to accommodate the thermal expansion and yield of the rock mass, specifically more flexibility may need to be added to the rock bolt systems to accommodate the rock deformations over time (Hardy &

Bauer 1991). Rock bolt stresses may be relieved by using yielding bolts, soft bolts with spring components, or other systems with large displacement capacities. Increasing the length of the rock bolt system will add more elastic flexibility. The displacement of stiff rock bolt systems is a linear function of bolt length, therefore, a microinch displacement places 10 times more load on a 5 ft bolt than on a 50 ft bolt (MacGregor 1950). The thermal coefficients of thermal expansion should be matched as closely as possible to tuff ($8.8 \times 10^{-6} \text{m/m } ^\circ\text{K}$ or $5.0 \times 10^{-6} \text{in/in } ^\circ\text{F}$). Zimmerman et al. (1987) speculated that, under ambient temperatures found at the G tunnel, and using an 80 year drift operational period, the estimated maximum convergence of the rock mass would be around 32mm. This matches well with what Pell (1974) calculated (38mm). Within these estimates, it is apparent that the grouted rock bolts used for most support jobs at the Nevada test site would be adequate for short term applications. Thermal induced stresses as expected in the waste emplacement areas will further complicate the situation.

For battling corrosion, coatings or special steels with protective films seem to be the best options. Coatings, most commonly zinc or cadmium can be an effective means of corrosion control. They work in one of two ways: (1) to act as a barrier against the environment; (2) to function as an anode (sacrificially) in order to protect the substrate from corrosion. Galvanic coatings are the most popular in rock bolt design. The most common specialized steel is stainless steel. This type has a protective film of ($\text{Cr}_2 \text{O}_3$), which provides good resistance in many corrosive environments. Portland and calcium-aluminate based cements, due to their alkalinity, alter the near bolt Ph, which helps to inhibit corrosion. These cements do provide corrosion protection, but are susceptible to

fracturing which could allow some of the steel to be exposed to the environment.

Thermal Effects

Thermal pulses may effect bolt anchorage because of expansion and contraction of the drill hole, or by degradation of some of the grouting material. Under the right conditions drill hole expansion or contraction could occur, resulting in either enhancing or lowering of the friction between anchors (grout or metal). There may also be some rubbilization of certain grouts. These possible cases can be avoided by keeping the thermal expansion coefficients as near equal as possible for the tuff, grout and steel. A general solution for rock to bolt differential thermal expansion was calculated by the U.S. Bureau of Mines for their 1987 report. Appendix I contains a review of their calculations and conclusions. Degradation of most organic grouts, which results in creep, has been demonstrated under higher temperatures (Figure 4.13). These grouting agents will not be considered for use at the Yucca Mountain Project, unless further long term testing indicates otherwise.

6.2 Applicable Bolt Systems

The following bolt systems appear to have promise and need to be investigated further.

- a). Cable Bolts Systems
- b). Yielding Bolt Systems

c). Point Anchor Bolt Systems

d). "Debonded" Full Column Grout Anchored Bolt Systems

Cable Bolt Systems

Since their development in the 1970's, the cable bolt system has been increasingly utilized for large rock mass deformation areas. The high strength cables can be used in unison, or doubled up where needed. The buttons and birdcage systems also help with the development of higher anchor strengths. Cable bolts lengths can vary up to 30 meters. Increased elastic flexibility and emplacement of anchoring grout further away from the pulse are some of the advantages to a longer system. The large mass of the cable system also helps increase thermal storage capacity and the cable's ability to dissipate heat. This system may work well in expanding and contracting thermal pulsed ground, providing that the grout anchor maintains its integrity. Cable bolts should be protected from corrosion by shielding the wires with coatings or plastic sleeves.

Yielding Bolt Systems

Yielding bolt systems can be broken in to two main categories. The first are yielding die or nut types and the second is the friction stabilizer type. Another type, which is unsubstantiated at this point, is the flexible "spring" like rock bolt. This system known as the Helical Bolt was designed by the Bureau of Mines. Further testing remains before this bolt is validated as a credible method as rock support (Babcock, 1978).

There are at least four yielding die or nut types. They are the Conway, Ortlepp, Meypo-Head and Dywidag's Sliding Nut systems. These systems utilize an anchored rebar with a special hardened die or nut. Depending on design, the die or nut is placed on either the end or the beginning portion of the bolt system. When the tensile force exerted on the bolt exceeds a certain load, the rebar is forced through these hardened dies or nuts. This allows the load to remain essentially constant with bolt displacement.

Friction stabilizers are full column supported bolts, which develop their support capacities from friction between the steel tubes and rock. The Split Set and Swellex bolts are the main types used in underground rock support. The Split Set will slip along the steel and bolt hole contact as the bolt is overloaded. During slip, the bolt system will still provides some support to rock mass. Atlas Copco on the other hand, has developed the Swellex EXL system. The system will deform constantly under a certain load because of the specially annealed steel used for the bolt design. Due to the high susceptibility to corrosion for both systems, they should be protected with coatings or preferably constructed of stainless steel.

Point Anchor Bolt Systems

Point anchored bolts reduce the local high strains by averaging the deformation over the full bolt length excluding the anchor. These systems utilize the grout or mechanical anchor type. The grouted anchor type has more potential in the repository environment. This is because the mechanical anchor is more likely to slip over time.

Debonded Full Column Grouted Bolt System

The "debonded" bolt under loads will perform superior to the regular full column bolt under the expected conditions in the emplacement areas. The strain from the load is averaged over a longer length. Debonding is attained by separating a portion of the rebar from the grout, which allows for more steel elongation or contraction. The full column of grout will provide some corrosion protection.

A data base of both the T-type threaded bar and the debonded bolt system and their analysis was obtained in all cases. The analysis was undertaken. The selected and proposed bolt system was analyzed under high stress. A survey of the field and design work was conducted. The field work was achieved. All information for the design was obtained in the field. Applicable rock bolt systems for design were determined. The design was implemented in the emplacement areas.

The available thermal conductivity and rock properties for the design were obtained from the U.S. Bureau of Mines, Department of the Interior, and other sources and computer modeling. The design was completed and is described. There will be unanswered questions, some how to use the proposed bolt system will be answered.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The scope of this study was to determine which available rock bolt systems have potential for the nuclear waste emplacement drifts and surrounding areas at the Yucca Mountain site. The analysis was complicated, because the proposed repository is still in the early construction stage. Therefore, there is very limited data on potential effects of a thermal pulse on the stability of underground openings. Additionally, little information is available on rock bolt systems emplaced in a high heat environment ($> 65^{\circ}\text{C}$ or 150°F).

A data base of both the Yucca Mountain Site Characterization and available rock bolt systems and their attributes were collected. A site specific investigation was then undertaken. The selected underground facilities had ground movements induced by high stress. A survey of the utilized rock support, mainly rock bolts, for these high stress areas was achieved. All information was then summarized and employed to help suggest applicable rock bolt systems for testing under the conditions expected in the waste emplacements areas.

The available thermally calibrated rock and rock mass properties for the Topopah Springs Horizon is from the G-Tunnel Mining Experiment, small scale laboratory testing of cores and computer modeling. Until the actual rock mass is characterized, there will be unanswered questions, about how the rock mass and support systems will interact under

high temperatures.

The project considered a wide range of rock bolt systems. Conventional support systems may be inadequate for the waste emplacement environment. The imposed unusual conditions of the elevated temperature environment on the waste emplacement, midpanel emplacement and to a lesser extent tuff main areas creates ground support component design difficulties. Compatibility of the support systems and its materials with the rock and the potential long term temperature activated physical and chemical interactions signify an uncertainty in long term performance. The rock bolt systems should be designed for corrosion resistance, and for a yielding rock mass. The thermal expansion coefficients for rock bolts need to be similar to that of tuff, and the rock bolt systems need to have large displacement capacities in order to allow bolt elongation during thermal stresses induced rock deformation. Utilization of longer rock bolt systems will increase displacement capacities and possibly limit the effect of differential thermal expansion on the anchor.

7.2 Recommendations

The following recommendations emphasize the need to investigate rock support system materials, the geologic rock mass, and their interacting relationships. To develop a reliable ground support design for the medium and near-field environments, and thermal mechanical characterization of both the rock mass and proposed support systems should be undertaken.

A thorough knowledge of the thermal effects on the strength and competence of the tuff is necessary for the characterization of the repository. Research needs to be directed toward understanding rock mass responses to heating and cooling. This can be accomplished with actual measured displacements, temperatures, acoustic emission activity and differential stresses. Fracture creations, magnitudes and orientations around thermal expansion areas should also be quantified for the rock mass.

Rock bolt anchorage responses for thermally pulsed ground should be quantified. The effects of increased load and temperature on the strength of the bolt anchor and host rocks should be defined. A pressurized triaxial tester might be used to evaluate the thermal mechanical parameters of the both materials together or alone. Developing design criteria for rock bolt support material in a high temperature is essential. Pull-out tests will help establish this design criteria, by further defining the interactions of each rock bolt and the rock mass. Load cells could also be used to help clarify the interaction of the rock bolt and its surroundings under these conditions.

A data base for pull-out tests performed on the suggested rock bolts under ambient conditions needs to be undertaken. This would provide a baseline for pull-out tests that will be performed on these rock bolts under the high temperature conditions. These pull-out tests could be conducted on rock bolts installed in large samples of Topopah Springs tuff and later in the proposed test core area located at Yucca Mountain.

For the high temperature testing, the large rock sample should be directionally heated to

200°C and loaded with approximate stress encountered at repository depths. At times, one of the faces of the sample must be available for the measurements of the effects of blast cooling on the emplaced rock support. This might mimic the ventilation of the emplacement drifts after being sealed off for awhile. Pull tests should be conducted on the installed bolt systems before, during and after the heating phase. The pull apparatus must be able to exert a variable extracting force that is at least twice the yield strength of any of the proposed bolts. The mechanism should record the force continuously, and measure any movement of the rock bolt system being tested.

Load cells could be placed on rock bolts which are undergoing an elevated temperature tests. The load cells would indicate the induced tension placed upon the bolt stem by differential thermal expansion. The same large tuff type sample could be used for this evaluation. Load cells could later be operated in the performance confirmation area, in order to characterize the actual differential thermal expansion effect on the emplaced rock bolts and its surrounding rock mass. An in situ heater test would provide the heat source.

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APPENDIX I

SIMPLIFIED DIFFERENTIAL THERMAL EXPANSION CALCULATIONS

Assumptions

Assumptions used for the calculation of tensioning force exerted on an installed rock bolt by differential thermal expansion, are:

- Rigid anchor
- The confinement of the rock mass is infinite except in directions of drift
- Physical properties of rock and bolt material were chosen from end of the range in order to impose the failure mode
- Simplified characterization of geology and thermal pulse
- Simplified theory for loading applications
- Rock thermal expansion > steel thermal expansion
- The intersecting joints show no expansion absorption
- The bolt and rock temperatures changes are equal
- The column length of rock is equal to the bearing plate and bolt anchor length together

The definition terms and symbols are:

α = thermal coefficient of expansion

E = deformation modulus = stress/strain

ϕ = diameter

$\sigma_1, \sigma_2, \sigma_3$ = principal stress

σ_N = Normal stress

τ = Shear stress

ΔT = Temperature difference

F = Force = stress x area

ϵ = strain = in/in deformation

σ_v = vertical stress

R = rock

G = grout

B = bolt

$^{\circ}\text{C}$ = temperature in Celsius

L = load in psi

F = force in lbs.

A = cross-sectional area

L_o = original length

δ = change in length

Differential Thermal Expansion. General Solution:

The linear expansion of the rock mass parallel to the bolt will be greater than the bolt expansion per degree of temperature change, if it is unrestrained. However, the bolt rigidity of the rock bolt will constrain the rock column parallel to the bolt, which results

in less expansion of the rock mass and elongation of the bolt beyond its untempered length. This constraint represents a force acting over the bolt stem cross-sectional area, thus establishing an induced stress (tension).

The elongation of an unconstrained bolt due to thermal expansion is $\alpha_B \Delta T_B L_0$ (Faupel, 1964).

The elongation of bolt while constrained with a bearing plate is

$$\frac{F'}{A_B E_B} L_0$$

The change in length of an unrestrained rock mass due to thermal expansion is $\alpha_R \Delta T_R L_0$.

The constraint force on the rock column, which is a result of applied bearing plate pressure, acts over the faceplate area. The change in length is

$$-\frac{F'}{A_R E_R} L_0$$

The rock mass and bolt maintain their lengths during heating, therefore:

$$\epsilon_B = \epsilon_R$$

$$\epsilon_B = \alpha_B \Delta T + \frac{F'}{A_B} \frac{1}{E_B} L_o$$

$$\epsilon_R = \alpha_R \Delta T - \frac{F'}{A_R} \frac{1}{E_R} L_o$$

Solving for F produces:

$$F' = \frac{(\alpha_R - \alpha_B) (\Delta T) (A_B) (E_B)}{1 + \frac{A_B E_B}{A_R E_R}}$$

The constraint force from the differential expansion is the induced bolt tension. The bolt will fail if the induced force is large enough.

Case 1: $\Delta T_{\text{Rock}} = 75^\circ\text{C}$; $\Delta T_{\text{Bolt}} = 25^\circ\text{C}$: This condition may possibly occur for main and sub-main entries if bolts are ungrouted and air exchanges in the bolt hole and at the bearing plate help keep the temperature of the bolt at 50°C (Figure 1.1).

The first case (Case 1) represents an applied tension to the rock and bolt. They expand together, although slightly different between 25 and 50 degrees Celsius. However, from 50 to 100 degrees Celsius, the rock alone expands causing elongation of the bolt. Hence,

$$F' = \frac{(\alpha_R - \alpha_B) (\Delta T) (A_B) (E_B)}{1 + \frac{A_B E_B}{A_R E_R}}$$

Manufacturing of the permanent anchor is necessary for rock temperatures above 300°F because of dimensional changes. The anchor manufacturer is advised:

Figure 1.1 - Differential Thermal Expansion of a point anchor bolt (USBM, 1987)

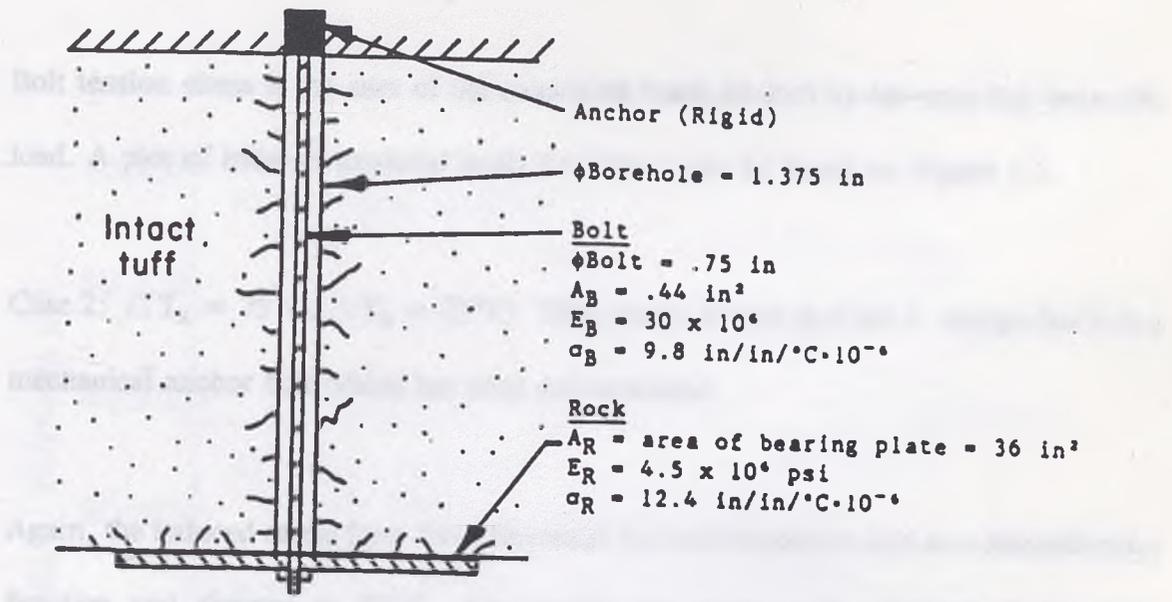


Figure 1.1 - Differential Thermal Expansion of a point anchor bolt (USBM, 1987)

Since stress induced by rock expansion exceeds the differential thermal expansion, the investigated temperature range is increasing. Case 2 is identical to Case 1 except it is adjusted against 12,000 psi of the amount of temperature rise shown in Figure 1.2.

Refactoring of the temperature term is necessary for rock temperatures above 50°C because of dissimilar heating. The new equation is as follows,

$$F_{25 \rightarrow 50} \text{ } ^\circ\text{C} = \frac{(\alpha_R \Delta T_R - \alpha_B \Delta T_B) (A_B) (E_B)}{1 + \frac{A_B E_B}{A_R E_R}}$$

Bolt tension stress is the sum of the calculated loads divided by the area that bears the load. A plot of induced tensional loads for Case 1 can be found on Figure 1.2.

Case 2: $\Delta T_R = 75^\circ\text{C}$; $\Delta T_B = 25^\circ\text{C}$: This case is similar to Case 1, except that it is a mechanical anchor bolt which has been pre-tensioned.

Again, the induced stress from the differential thermal expansion acts as a discontinuous function and disjoins at 50°C. Additionally the mechanical anchor bolt has been mechanically tensioned to 12,000 psi during installation. This initial tension is superimposed upon by the thermally induced tension. Unloading of the initial tension will occur, if the tensioned portion of the bolt returns to its pre-tensioned length without any more loads applied. In this case, bolt unload will occur when $\Delta T = 32^\circ\text{C}$ [Unload = 12,000 psi = $(9.8 \text{ in/in}/^\circ\text{C} \times 10^{-6}) (\Delta T) (30 \times 10^6 \text{ psi}) = 32^\circ\text{C}$].

Since stress induced by rock expansion exists from the differential thermal expansion, and investigated temperature range is increasing, Case 2 is identical to Case 1 except it is adjusted upward 12,000 psi on the tension vs. temperature plot shown on Figure 1.2.

Case 3: $\Delta T_{rock} = \Delta T_{steel} = 75^{\circ}\text{C}$ A possible condition predicted for grouted bolts where heat transfer for rock is used is efficient and the heat loss of the steel is limited to the bearing plate zone heat transfer (Figure 1.2). Also see equation.

Grout helps minimize difference heat transfer from rock to the steel where:

$$\Delta T_1 = \Delta T_2 = \Delta T_3$$

Grout has minimal tensile strength so the bolt carries the reduced load

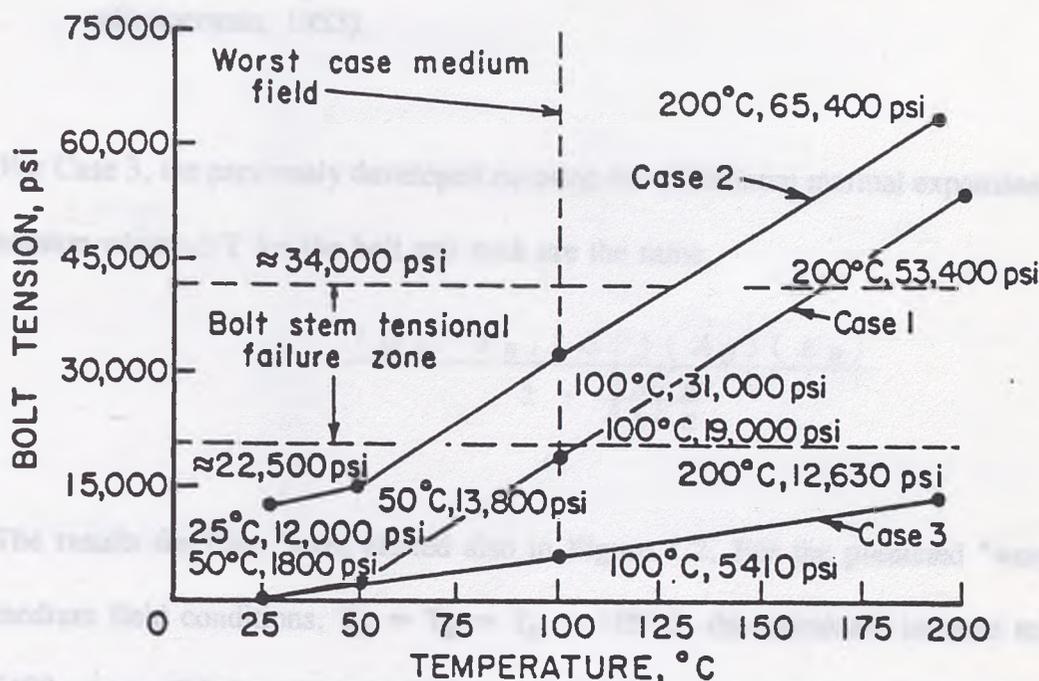


Figure 1.2 - Rock bolt Differential Thermal Expansion loadings (USBM, 1987)

Case 3: $\Delta T_{\text{Rock}} = \Delta T_{\text{Bolt}} = 75^\circ\text{C}$: A possible condition predicted for grouted bolts where heat transfer for rock to steel is efficient and the heat loss of the steel is limited to the bearing plate zone heat transfer (Figure 1.3). Not pre-tensioned.

- Grout helps maintain efficient heat transfer from rock to the steel where:

$$\Delta T_R = \Delta T_B = \Delta T_G$$

- Grout has minimal tensile strength so the bolt bears the induced load

(Gonnerman, 1953).

For Case 3, the previously developed equation for differential thermal expansion induced tension where ΔT for the bolt and rock are the same,

$$F = \frac{(\alpha_R - \alpha_B)(\Delta T)(A_B)(E_B)}{1 + \frac{A_B E_B}{A_R E_R}}$$

The results for Case 3 are plotted also in Figure 1.2. For the predicted "worst case" medium field conditions, $T_R = T_B = T_G = 100^\circ\text{C}$, the calculated induced tension of 5400 psi is well below the bolt yield stress. For every degree of temperature change ($\Delta T = 1^\circ\text{C}$), the force exerted on the bolt is 31.7 lb/°C.

Nonetheless, the grout-rock and grout-bolt interfaces are critical for grout anchored rock bolts. In a thermal environment anchorage will fail if, the shear strengths of the bolt, grout or rock materials are exceeded by thermal induced stresses. Thermo-chemical deterioration or thermo-mechanical failures are other ways grout interfaces can be broken

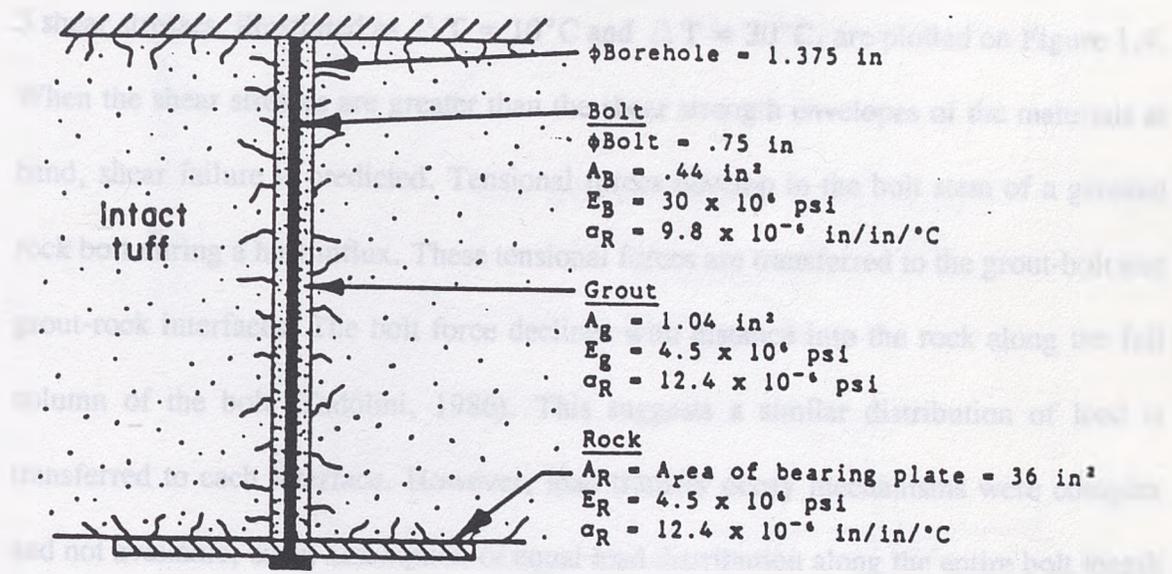


Figure 1.3 - Differential Thermal Expansion of a full column grouted bolt (USBM, 1987)

in a thermal environment. The anchors of the full column grout bolts fail by either exceeding the frictional resistance between the interfaces or by internal shear of grout or rock. Since the analysis of bond strengths was beyond the scope of the U.S. Bureau of Mines simplified calculations for differential thermal expansion failure, a simplified shear failure evaluation was deemed reasonable. The Mohr-Coulomb shear stress failure envelopes for tuff and grout were generalized, and are shown in Figure 1.4. The Case 3 shear stresses, illustrated as $\Delta T = 10^\circ\text{C}$ and $\Delta T = 30^\circ\text{C}$, are plotted on Figure 1.4. When the shear stresses are greater than the shear strength envelopes of the materials at hand, shear failure is predicted. Tensional forces develop in the bolt stem of a grouted rock bolt during a heat influx. These tensional forces are transferred to the grout-bolt and grout-rock interfaces. The bolt force declines with distance into the rock along the full column of the bolt (Tadolini, 1986). This suggests a similar distribution of load is transferred to each interface. However, load transfer decay mechanisms were complex and not available, so an assumption of equal load distribution along the entire bolt length was made. Another assumption for this analysis was that there be confinement from the sides of the drill hole (Figure 1.5). The internal stress induced by the confined thermal expansion is alluded to as the normal stress (σ_N) in these calculations.

For a change in temperature of one degree ($\Delta T = 1^\circ\text{C}$), the normal stress for both interfaces are,

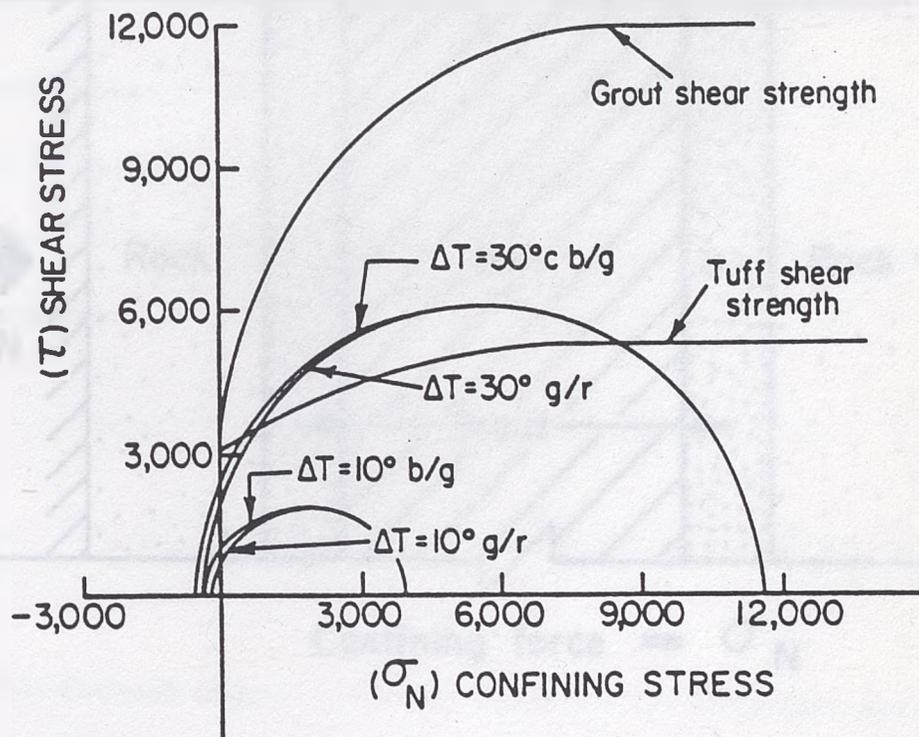


Figure 1.4 - Mohr-Coulomb Failure Envelopes and Base Case Shear from calculations (USBM, 1987)

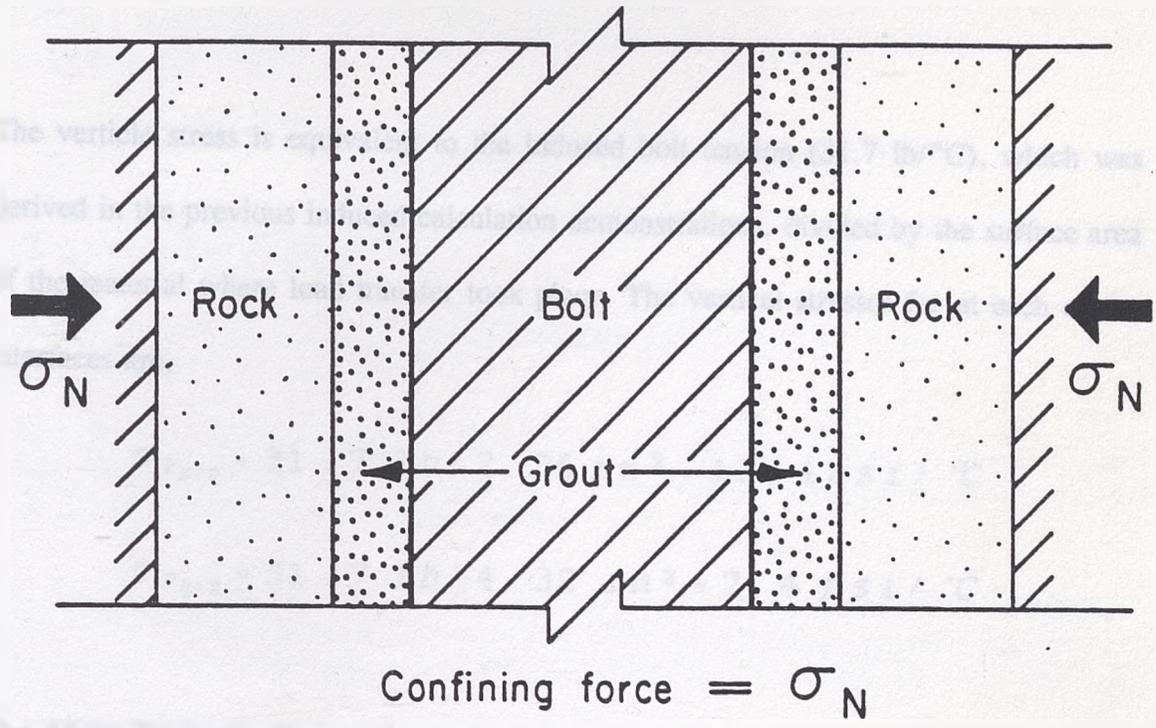


Figure 1.5 - Lateral Confining Pressure from Thermal Rock Expansion (USBM, 1987)

Each anchor type and its interaction with the surrounding rock, under the assumed thermal expansion, needs to be further investigated. This investigation should help further define some of these parameters.

$$\sigma_{N_{B/G}} = \sigma_{N_{G/R}} = \alpha_R \Delta T E_R + \alpha_B \Delta T E_B + \alpha_G \Delta T E_G$$

$$\sigma_N = (12.4)(1)(4.5) + (9.8)(1)(30) + (10)(1)(4.5)$$

$$\sigma_N = 394.8 \text{ psi} / ^\circ\text{C}$$

The vertical stress is equivalent to the induced bolt tension (31.7 lb/°C), which was derived in the previous induced calculation demonstrations, divided by the surface area of the material where load transfer took place. The vertical stresses for at each of the interfaces are,

$$\sigma_{V_{B/G}} = 31.7 \text{ lb} / 2.36 \text{ in}^2 = 13.4 \text{ psi} / ^\circ\text{C}$$

$$\sigma_{V_{G/R}} = 31.7 \text{ lb} / 4.32 \text{ in}^2 = 7.4 \text{ psi} / ^\circ\text{C}$$

The Mohr-Coulomb diagram located on Figure 1.4 has temperature induced failure envelopes plotted on it. For the assumed tuff and grout strengths, the tuff-grout and grout-bolt interfaces experience failure at $\Delta T = 20\text{-}30^\circ\text{C}$ and 60°C , respectively. Where the tuff shears on the rock-grout interface, and the grout shears on the bolt-grout interface. Tests have shown once shear failure occurs in either case, the fragments may mechanically interlock. This interlocking may help maintain anchorage (USBM, 1987).

In a very simplified manner, the scoping calculations of the Bureau of Mines illustrate the possible effects of differential thermal expansion on some commonly used rock bolts.

Each anchor type and its interaction with the surrounding rock, under the differential thermal expansion, needs to be further evaluated. Pull-out tests under these conditions should help further define some of these parameters.

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