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A CASE HISTORY OF SUPPORT AT NACIMIENTO MINE

Ben L. Seegmiller

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A CASE HISTORY OF SUPPORT AT NACIMIENTO MINE

by

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written for the
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Contract Officer: R. Sage

This report has been prepared as part of the
PIT SLOPE PROJECT
of the
MINING RESEARCH LABORATORIES
CANADA CENTRE FOR MINERAL AND ENERGY TECHNOLOGY
DEPARTMENT OF ENERGY, MINES AND RESOURCES
OTTAWA

FOREWORD

The potential benefit of stabilizing mine slopes with rock anchors has been recognized by CANMET for some years. In 1969, field trials of support were held at the Hilton iron mine near Ottawa. Following this, the development of support was made a specific objective of the Pit Slope Project, which began in 1972. As part of this project, additional trials have been staged at various mines in Canada. As a result, a considerable body of information on the application of rock anchors and other methods of support in open pit mining has evolved.

However, there has as yet been no full scale installation of rock anchors to stabilize a pit wall in Canada. For this reason, the use of several hundred rock anchors to stabilize an operating slope at the Nacimiento mine in New Mexico was of particular interest. Accordingly, CANMET approached the Earth Resources Company and Seegmiller Associates for permission to publish the results of this support installation. This was granted, and Seegmiller Associates were commissioned to write the report.

The pioneering efforts of the Earth Resources Company in undertaking this, the first full scale application of rock anchors in open pit mining, and their candour in letting full details of the work be published, are greatly appreciated.

D.F. Coates,
Director-General

NACIMIENTO MINE - SUPPORT CASE HISTORY

Abstract

The Nacimiento copper mine has experienced stability problems that threatened to prevent further mining. The nature of the rock material and the type of slides that occurred indicated that it should be possible to stabilize the slope by means of rock anchors, and in 1974 a major support installation was begun. Several hundred rock anchors with capacities up to 200 tons were subsequently installed. These were effective in stabilizing the slope initially, but shortly after, a number of anchors failed through stress corrosion. Although precautions were taken to prevent further anchor failures, those which remained were inadequate to bear the entire burden and a substantial slide did occur. The case history does show that a slope was effectively stabilized by rock anchors, and that when the supporting force was reduced by corrosion the slope again became unstable.

L'HISTORIQUE DU SOUTENEMENT DE LA MINE NACIMIENTO

Résumé

Il y a eu, par le passé des problèmes de stabilité des pentes, à la mine de cuivre Nacimiento, qui auraient pu enfreindre la continuité de l'exploitation. La sorte de roche et le type d'éboulements produits indiquèrent qu'il y avait une possibilité de stabiliser les pentes par des ancrages; ainsi en 1974 une importante installation de soutènement débuta. On installa plusieurs centaines d'ancrages pour terrains rocheux dont les capacités étaient de 200 tonnes. Au début, ils réussirent à stabiliser les pentes, mais peu de temps après, un certain nombre de ces installations s'effondrèrent à cause de la corrosion sous tension. Bien que des mesures de précaution furent prises afin d'éviter d'autres écroulements des ancrages, ceux-ci n'étaient pas aptes à tenir toute la charge et par conséquent, un nouvel éboulement eut lieu. Par contre, l'historique démontra qu'effectivement des ancrages pour terrains rocheux réussirent à stabiliser une pente et que lorsque la force de soutènement était réduite par la corrosion, la pente devenait, encore une fois, instable.

ACKNOWLEDGEMENTS

Permission to use the Nacimiento Mine for this case history was given by the Earth Resources Company. Their cooperation and willingness in letting the candid details of this support project be published are an important contribution to the entire mining industry. Assistance was rendered by CANMET personnel in the editorial treatment and in various revisions of the case history. R. Sage made significant modifications and improvements in the text material. The author wishes to express his gratitude and thanks to both Earth Resources Company and CANMET.

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INTRODUCTION

The Nacimiento open pit copper mine (Fig. 1) has experienced slope stability problems since mining began in June 1971. In July 1973, a major slide occurred in the east wall after a geological contact between two formations was undercut. The mining plan called for continued excavation along the strike of the contact. This would mean undercutting the contact resulting in the probable instability of the entire east wall. To prevent instability during the planned life of six years, management investigated methods of stabilizing the wall. There were two possibilities: stripping away all material that might fail (i.e., cutting the east wall back to the contact) or stabilizing the slope with rock anchors.

After appraising the possibilities, management chose rock anchor support. This report is a case history of the investigations and analyses that led to this decision. The case history describes the general geology and the investigations leading to the installation of rock anchors. The difficulties encountered both during and after the installation of support are described, and the benefits of support, both to open pit mining in general and to the Nacimiento mine in particular, are discussed.

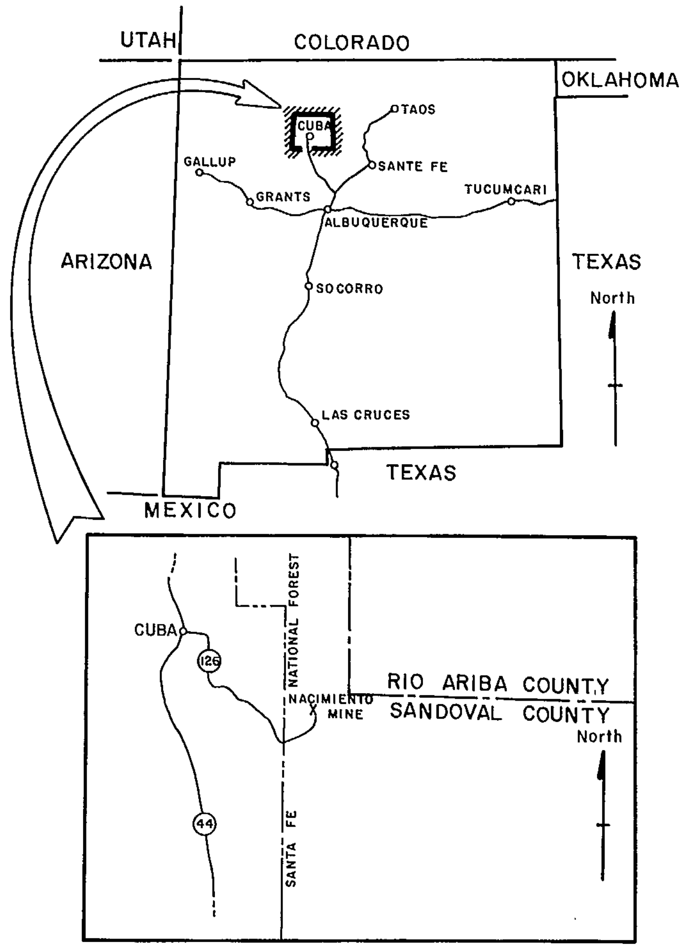


Fig. 1 - Location map for Nacimiento Mine, Cuba, New Mexico

GENERAL GEOLOGY

The Nacimiento mine is near the southeastern edge of the Colorado Plateau in northwestern New Mexico, USA. The ore deposit is typical of the "red bed" copper deposits of the southwestern USA (1). The general stratigraphy of the mine area is shown in Fig. 2. The Triassic Chinle Formation is composed of Agua Zarca Sandstone, Salitral Shale, Paleo Sandstone and Upper Shale. The Chinle Formation lies unconformably on the Permian Cutler Formation. The Agua Zarca Sandstone contains the major ore mineralization, a combination of copper sulphides and oxides. Much of the ore is replaced organic material (trash) and includes logs several feet long replaced by chalcocite.

The stratigraphic units of interest for slope stability purposes are the Salitral Shale, Agua Zarca Sandstone, and the Cutler Formation. These are described (2) as follows:

Salitral Shale: Calcareous shale, maroon-gray in color, variegated, contorted lamination, with red-brown limestone concretions. It includes a pink coarsely-crystalline limestone with pyrite grains and has lenses of buff colored very fine-grained sandstone and siltstone with minor amounts of carbon "trash", pyrite, and malachite. There is a gradational contact with underlying

Agua Zarca.

Agua Zarca Sandstone: The uppermost portion is a quartz-feldspathic sandstone which is white-buff-orange in color and is fine-to medium-grained. It is friable, thinly cross-bedded and has argillaceous cement. There are concentrations of carbon "trash" and copper mineralization. Beneath the sandstone is an inter-bedded lenticular gray shale and pebble-cobble conglomerate.

Cutler Formation: This formation is gradational from a red-orange, thinly cross-bedded siltstone to a shale-mudstone and coarsely crystalline limestone to an arkose. The shale-mudstone is calcareous, maroon in color and is laminated with lenses of brown-red, arkosic siltstone to very fine-grained sandstone. The arkose is gray in color and fine- to coarse-grained.

The major structural feature in the Nacimiento mine vicinity is the north-south striking Nacimiento fault. This fault is a high-angle reverse fault and forms the western boundary of the Nacimiento uplift. The mine lies to the east of the fault and in a graben structure formed by two east-west faults: the El Cajete to the north and the Blue Bird to the south.

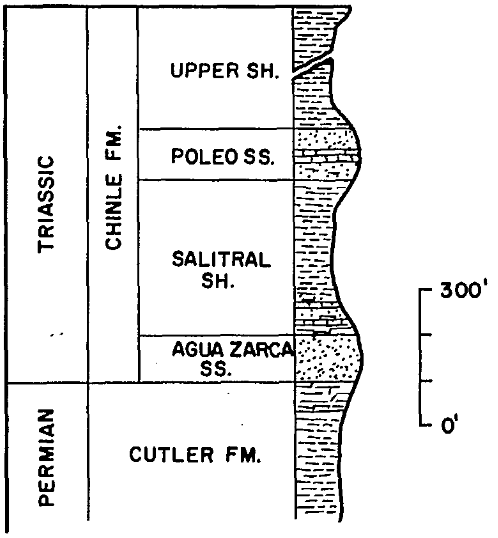


Fig. 2 - Generalized stratigraphic section

PRELIMINARY INVESTIGATION

The initial investigation of the stability of the east wall took place in October 1973. The existing data on structural geology, groundwater and mechanical properties were examined and some additional data collected. The results obtained are given below.

STRUCTURAL GEOLOGY

The instability of the rock slopes and obscuring of surface detail by the rippers used for mining meant that no detailed mapping could be done in the pit. Mapping was confined to the eastern perimeter, primarily in a series of drainage ditches east and southeast of the slope. Relatively few structures were mapped and these were confined to the Agua Zarca Sandstone which alone outcropped in the mine area. However, it is felt that the mapping adequately represented the structures important to stability, particularly the bedding planes.

Figure 3 is an idealized Schmidt plot of the mapping data. Four distinct sets of structures are evident. A summary of these in order of predominance is shown in Table 1. The bedding planes are the most prominent structures, composing about 40% of those mapped.

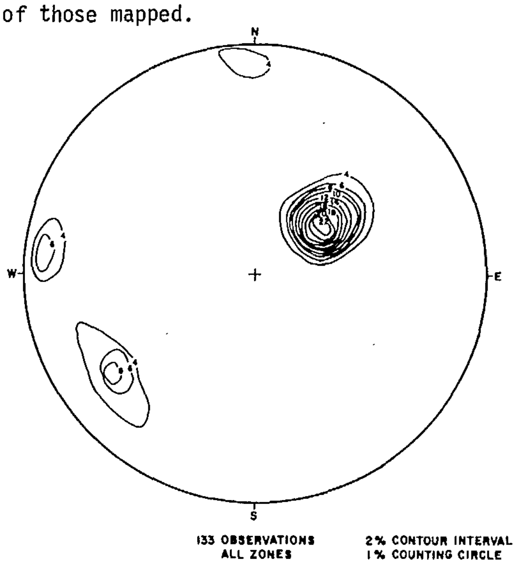


Fig. 3 - Lower hemisphere Schmidt net for bedding planes and joints in Agua Zarca sandstone

Table 1: Nacimiento geological structure system

Set	Mean dip direction	Mean dip	Structure type
1	219°	32°	bedding
2	56°	67°	joints
3	98°	79°	joints
4	177°	84°	joints

Investigation indicated that past failures had occurred on the Agua Zarca Sandstone - Cutler Formation contact. The approximate dip and dip direction of this contact are 29° and 252°.

An analysis of the structural data shows that the Agua Zarca bedding planes and the Agua Zarca - Cutler contact define a potential instability, as shown in the stereo plot of Fig. 4. The line of intersection of the contact and the bedding planes coincides with the direction of movement of the July 1973 slide (Fig. 5).

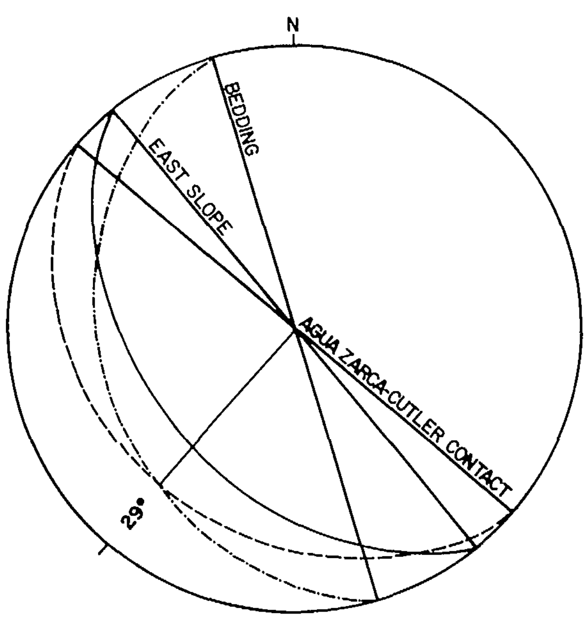


Fig. 4 - Stereo plot of potential instability

GROUNDWATER

The Nacimiento region is semi arid with an annual precipitation of about 14 inches. The period of greatest rainfall is late summer and the greatest snowfall usually occurs in December and January. The natural drainage in the mine area is southwest, into the pit. Drainage ditches were cut early during mining to divert surface runoff away from the mine.

The groundwater table was encountered almost immediately after stripping began in 1971. A single well, pumping approximately 40 gallons per minute, was installed to control groundwater; this maintained the groundwater level just below the pit floor. The only other source of direct hydrologic data was a single open hole piezometer. Mine staff felt that groundwater pressures did not contribute to instability of the east wall.

MECHANICAL PROPERTIES

Records of rock or soil tests were not available in October 1973 and therefore mechanical properties of the east wall rocks were estimated from a back-analysis of the July 1973 slide and by using engineering judgement. The back-analysis was based on observations and on a geologist's description of the actual slide. The instability mode for the July 1973 slide appeared to be planar and occurred after the Agua Zarca - Cutler contact had been undercut. The back-analysis was therefore a simple plane sliding analysis. It was assumed that:

a. the weight of the sliding mass can be approximated by a vector acting through its centre of gravity;
b. the strength characteristics of the failure surface can be represented by Coulomb's law.

A safety factor against sliding (the ratio of resisting forces to disturbing forces) is given by eq 1; the forces are shown in Fig. 6.

$$SF = \frac{W(\cos\alpha - U)\tan\phi + cA}{W \sin\alpha} \qquad \text{eq 1}$$

where W = weight of sliding mass
U = groundwater uplift force
A = area of surface of sliding

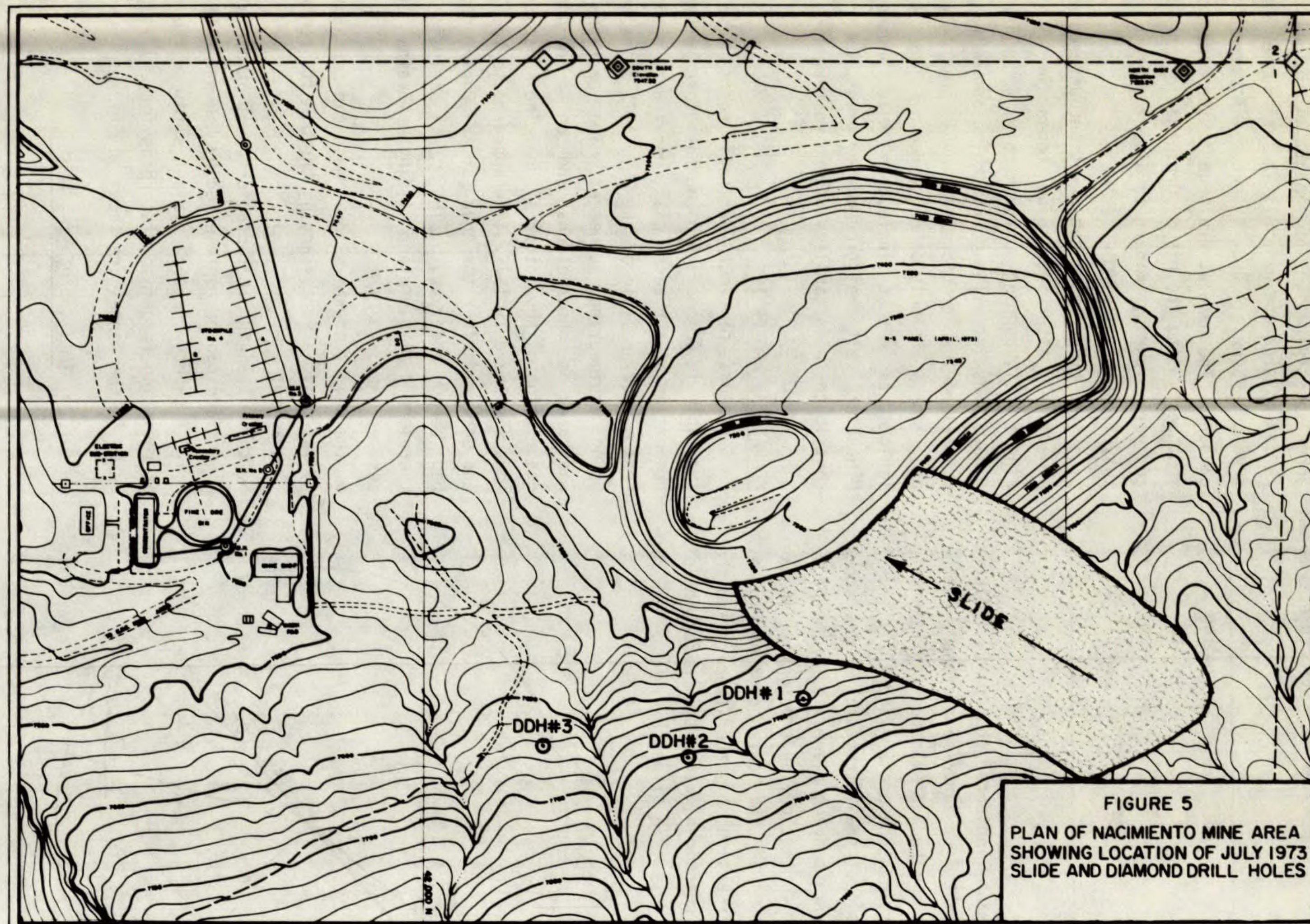


Fig. 5 - Plan of Nacimiento Mine area showing location of July 1973 slide and diamond drill holes.

c = cohesion

ϕ = friction angle along slide surface

α = sliding plane angle

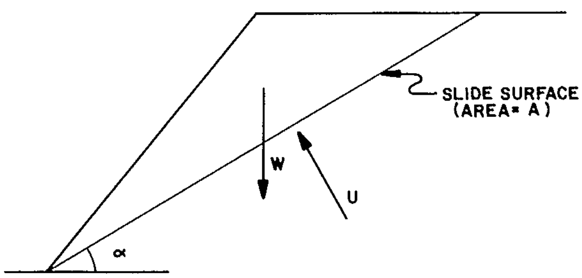


Fig. 6 - Forces affecting potential slide

At sliding, the safety factor is less than or equal to 1. Solving eq 1 for various safety factors gives a range of values for c and ϕ as shown in Fig. 7.

MONITORING

A program of monitoring displacement across tension cracks was begun shortly after the July 1973 slide. By mid-October 1973, displacement rates had decreased to an average of about 0.004 in./day.

PRELIMINARY SUPPORT DESIGN

Because further slides on the east wall would severely affect operations, the need for stabilizing measures led management to appraise the feasibility of providing mechanical support with rock anchors.

To make a preliminary estimate of the support required before detailed site investigations were completed, the following assumptions were made.

a. Future slides would occur on the contact between the Agua Zarca Sandstone and the Cutler Formation and would be simple plane shear instabilities.

b. Location and inclination of the entire potential sliding plane would be as hypothesized by mine engineering personnel. Geology data from drill holes used in the ore body delineation had been studied for several years by the mining company. This information, combined with the interpretation of the local geology by mine geologists was considered the best available indication of the potential sliding plane location behind the pit wall.

c. Mining of the east wall would be as laid out on the current mining plan.

d. The shear strength on the contact (the potential sliding surface) is defined by a cohesion (c) of 0, and a friction angle (ϕ) of 29°. Reasoning was that the July 1973 slide took place on a plane estimated to dip 29° into the pit. Before sliding, the slope had been stable for more than six months; it was then felt it had been in limiting equilibrium and the strength parameters would correspond to the above.

e. Rock above elevation 76B0 and below elevation 7320 (Fig. 5) would be self-supporting because the sliding plane dips at an angle less than the assumed angle of friction.

f. Rock anchors would be installed at 5° below horizontal. The optimum angle is given by: $\delta = \phi - \alpha$, where $\alpha$is the dip of the surface of sliding, ϕ is the angle of friction on that surface, and σ is the angle below horizontal of the rock anchors. With the assumption that $\phi = \alpha = 29°$, δ would be zero. However, an inclined hole is preferable for anchor installation and grouting, and the overall support from anchors has only minor sensitivity to anchor inclination.

g. There would be no disruptive forces from blasting or earthquakes acting on the slope. At the time the preliminary design was undertaken, no blasting except small secondary blasts on boulders was done within the pit area, nor was any blasting planned during future mining in the cable anchoring area. All mining was carried out using rippers, scrapers and front-end loaders. No earthquakes were known to have been recorded or to have occurred in the Nacimiento or surrounding area in the past 20 or more years.

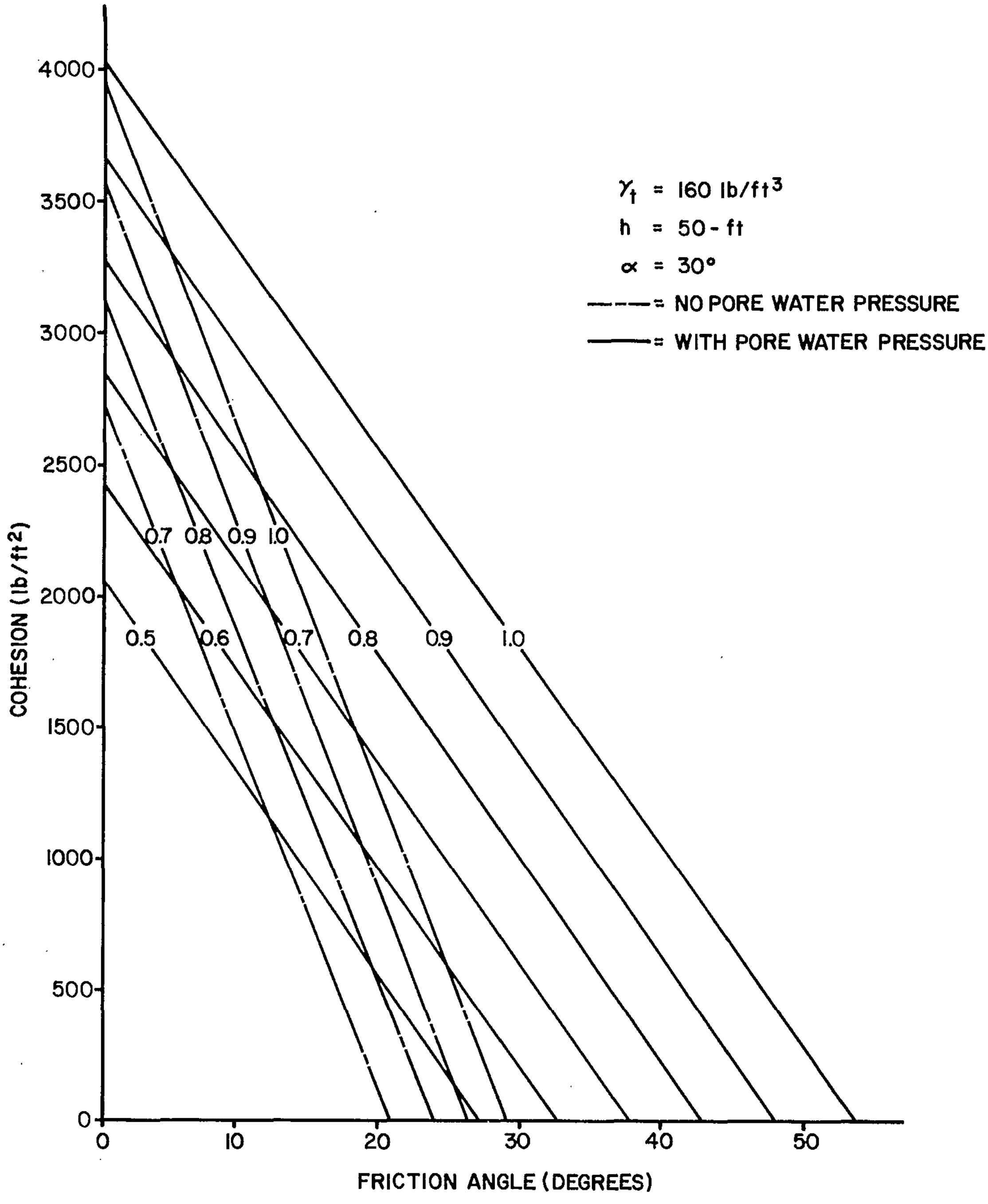


Fig. 7 - Strength parameters from back - analysis

h. Hydrostatic or groundwater forces would not act along the potential sliding plane. According to mine personnel, water was not observed in ore delineation boreholes in the cable anchoring area, nor had water been observed along or behind the sliding plane of the July 1973 slope failure. Surface ditches had been dug behind the slope in the cable anchoring area and had been effective in minimizing surface runoff into the pit.

The principles of the analysis were identical with those described below under Investigation and Design of Support. Results are presented in Fig. 8 and show that between 360 and 600 anchors would be required to achieve acceptable stability.

RESULTS OF PRELIMINARY INVESTIGATION

It was concluded that the July 1973 slide resulted from the unfavourable orientation of intersecting structures, and removal of Agua Zarca Sandstone which had provided lateral restraint in the toe area of the slope. The continuation of these unfavourably oriented structures throughout the east wall indicated that slope stability along the Agua Zarca - Cutler contact would be a problem for future mining. Rock anchor support might provide stability, but additional investigation would be required before engineering and economic analyses could be made.

It was also concluded that the probable mode of instability affecting future east wall mining would be plane sliding on the Agua Zarca - Cutler contact. The potential sliding plane would contain saturated materials within the existing groundwater table. The effects of earthquakes, surcharge load, and blast forces would be negligible.

RECOMMENDATIONS FROM PRELIMINARY INVESTIGATIONS

As a result of the preliminary investigation, the following recommendations were made to mine management:

a. Three vertical diamond drill holes should be drilled in the southeast slope and logged in detail for structural and engineering data. All three holes should penetrate the Agua Zarca - Cutler contact.
b. Tests should be made to determine the following properties. For the rock substance (intact rock): unconfined compressive strength, tensile strength, unit weight, and deformation modulus. For the clay infilling on the contact: cohesion and angle of sliding friction.
c. Several large blocks of Cutler Formation rock should be collected and tested to determine bonding strength between grout and rock for rock anchor design.
d. Hydrologic data should be gathered by using two of the three diamond drill holes as piezometer holes.
e. A shear strip should be placed in the third diamond drill hole and monitored at regular intervals.
f. A continuing and more detailed displacement monitoring program should be initiated.
g. A detailed analysis of rock anchor support should be made as soon as sufficient rock mechanics data is collected.

INVESTIGATION AND DESIGN OF SUPPORT

Following results of the initial investigation, mine management decided to install rock anchor support on the east wall. The report on the preliminary support design emphasized that the estimates were only approximate and that additional work would be necessary before a final design could be established. However, a decision was made to proceed with support before a detailed site investigation could be completed because mining the east wall could not be delayed without jeopardizing ore supply to the mill. Because further slides could not be risked there was no alternative but to proceed with stabilization. Management was prepared for the possibility that the final design might require increased expenditure for rock anchors.

The target for starting support installations was March 1st, 1974. The actual

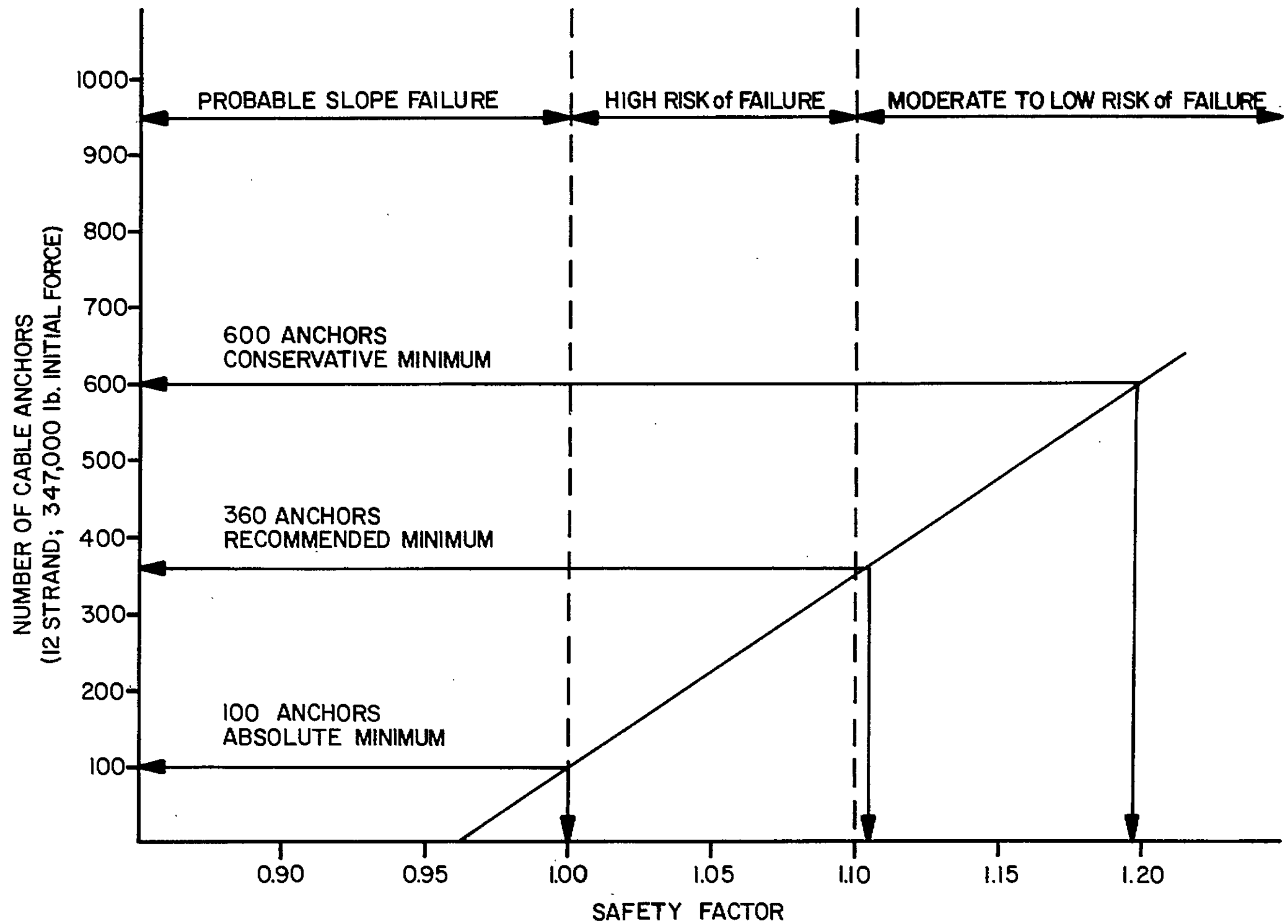


Fig. 8 - Safety factor vs. number of anchors for preliminary design

start was delayed a month, but installation still began well before the support design was completed in May.

A program of collecting and testing samples began in November, 1973 and drilling of the three recommended holes began soon after. A target of February 1, 1974 was set to complete sample collecting, testing and analyzing, and this work proceeded approximately as scheduled. However, problems were encountered during drilling and work fell several weeks behind schedule. Drilling was not completed until February and logging, testing and data analysis could not be completed until late March. A report on the core logging and testing for mechanical properties was submitted to the mine management on April 22.

CORE LOGGING

Core from the three diamond-drill holes was logged for both engineering properties and structural geology. Location of the holes is shown in Fig. 5. The engineering properties logged were core recovery, rock quality designation (RQD), average size of pieces, rock type, grain size, estimated compressive strength, rock substance deformation moduli and rock mass deformation moduli. Structural geology data recorded were frequency of discontinuities, relative roughness, estimated cohesive strength, fracture spacing, fracture intersection angles, and characteristics of fault zones and infilling materials. A summary of these logs follows:

Hole 1 - The Agua Zarca Sandstone, while having low strength and modulus values, has reasonably consistent engineering properties. The contact between Agua Zarca and Cutler rock was quite evident and the Cutler rock appeared to have 75 - 100% better rock quality than the sandstone. The frequency of discontinuities was low but three distinct fracture sets were found. Interbedded clays were frequently recorded.

Hole 2 - The log in general indicated the same characteristics as hole 1. In addition, some core discing was noted.

Hole 3 - The log for this hole essentially confirmed the characteristics found in holes 1 and 2. Discing was also noted in this hole and the Agua Zarca - Cutler contact was gradational and not obvious.

MECHANICAL PROPERTIES

Direct Shear Tests

Thirty direct shear tests were performed on clay joint infill material from drill holes 1 and 2. Composite shear-normal plots for residual and peak values are shown respectively in Fig. 9 and 10. The clay samples taken from hole 1 averaged 10 - 12% moisture. Lithic fragments found in most of the clays may have caused the relatively high values of cohesion and friction angle. Samples from hole 2 contained 15 - 16% moisture and were almost entirely free of lithic fragments; they were also of relatively low shear strength.

Samples collected from the actual formation contact, which was the potential sliding plane, were wet at more than 16% moisture content and had some of the lowest shear strengths. The Agua Zarca - Cutler contact is known to be quite variable in shear strength. In some zones the contact undulates and has no clay infilling while in other zones the contact is relatively planar and contains wet clay infilling. Because of this variation, the selection of a minimum residual failure envelope as a basis of design would have been conservative. The chosen design curve, shown in Fig. 10, is felt to be a reasonable engineering choice; 82% of the test results lie above this curve. The equation for this curve is

$$T = \sigma_n \tan 20 + 1000 \text{ psf} \qquad \text{eq 2}$$

Substitution of the strength parameters in eq 2 gives FS of 0.84, assuming no groundwater effect. This indicates the chosen shear strength parameters are still conservative.

Tensile Strength Tests

Brazilian tensile tests were performed on 202 samples from the three diamond-drill holes.

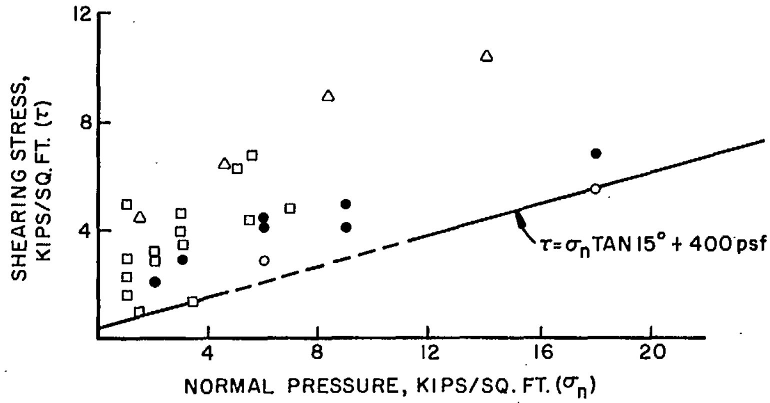


Fig. 9 - Direct shear tests: residual values

LEGEND

- □ CLAYS DDH#1 AGUA ZARCA
- △ CLAYS DDH#1 CUTLER (NEAR CONTACT)
- ● CLAYS DDH#2 AGUA ZARCA
- ○ CLAYS DDH#2 CUTLER CONTACT
- ——— DESIGN CURVE (RECOMMENDED)
- – – – DESIGN CURVE (WORST PROBABLE CASE)

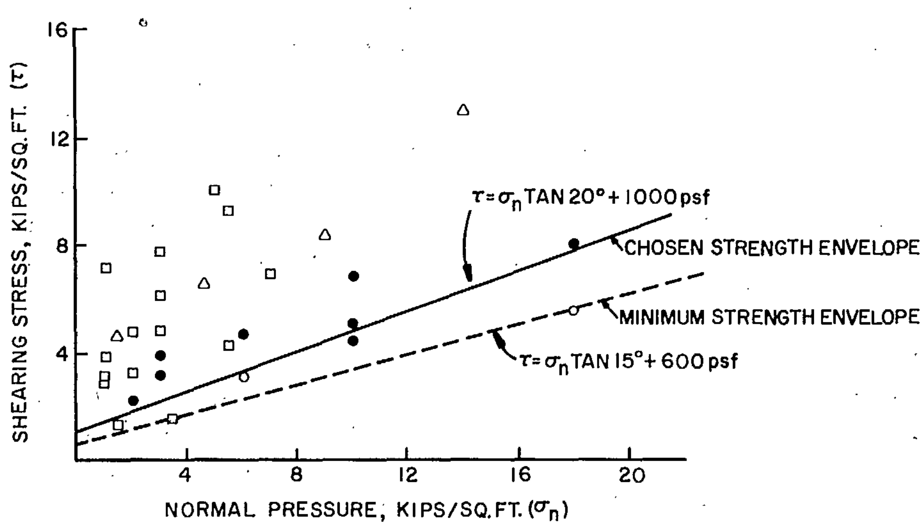


Fig. 10 - Direct shear tests: peak values

Twenty-one of these samples were from the Cutler Formation and 181 from the Agua Zarca Sandstone. A summary of tensile strengths by rock formation and drill hole is given in Table 2.

Compressive Strength Tests

Twenty-seven uniaxial compression tests were performed on samples from the three drill holes. Twenty-three of the samples were from the Agua Zarca Sandstone and four were from the Cutler Formation. A summary of compressive strength test results by rock formation and drill hole is given in Table 3.

Table 2: Brazilian tensile test results

Sample source	Number of tests	Mean strength, psi†
hole 1 (Agua Zarca)	28	215 ± 25
hole 1 (Cutler)	5	282 ± 226
hole 2 (Agua Zarca)	83	315 ± 87
hole 2 (Cutler)	16	696 ± 17
hole 3 (Agua Zarca)	70	178 ± 85
total (Agua Zarca)	181	247 ± 102
total (Cutler)	21	489 ± 273
total (all tests)	202	265 ± 135

† mean strength ± one standard deviation

Table 3: Uniaxial compression test results

Sample source	Number of tests	Mean strength, psi†
hole 1 (Agua Zarca)	4	3578 ± 496
hole 1 (Cutler)	2	6413 ± 512
hole 2 (Agua Zarca)	7	5256 ± 1528
hole 2 (Cutler)	2	6811 ± 2716
hole 3 (Agua Zarca)	12	3840 ± 1331
total (Agua Zarca)	23	4226 ± 1432
total (Cutler)	4	6612 ± 1612
total (all tests)	27	4579 ± 1668

† mean strength ± one standard deviation

Bonding Tests

Six tests were performed to measure bonding strength between the grout and rock, which would influence a possible support installation using rock anchors. Approximately 1-ft cube blocks of Cutler Formation rock were used. A 3-in. hole was drilled through each block and a grout plug poured in and left to cure for two weeks. The grout mixture and curing time represented actual field installation conditions - rock anchors are normally inserted in the holes and are grouted and cured for approximately two weeks before tensioning. After curing, the grout plugs were forced out of the hole and bonding strengths determined. The results of these tests are given in Table 4.

Table 4: Bonding strength test results

Sample	Mean strength, psi
1	144
2	244
3	239
4	269
5	440
6	587

DESIGN ASSUMPTIONS

Following the more detailed investigations, assumptions made during the preliminary support design were revised and amplified. These were:

The Agua Zarca - Cutler contact was assumed to be the deepest plane along which potential failure might occur. A series of cross sections was prepared by the mining company's geological and engineering personnel indicating approximate location of the contact.

The major geological structures defined by the mine geologists are shown in Fig. 11. It was assumed there were no other major structures which might have affected slope stability.

All rock above the first mining bench (at

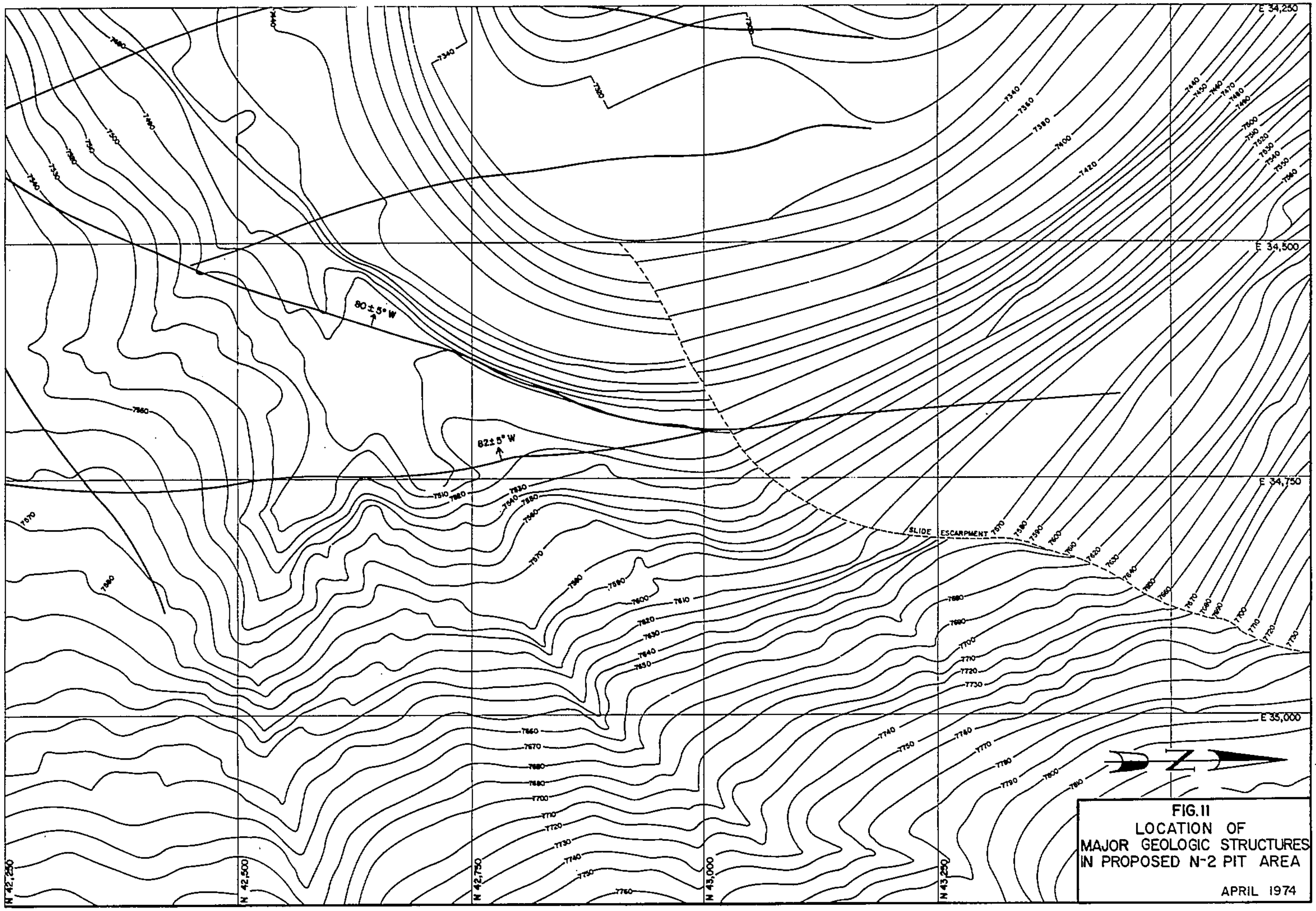


Fig. 11 - Location of major geologic structures in proposed N-2 pit area

7560 elevation) was assumed to be either self supporting or supported by the cable anchors on the three upper non-mining benches at elevations of 7600, 7640, and 7680 feet. Rock above 7720 elevation was assumed to be self-supporting provided the three non-mining benches remained stable.

Values of cohesion and friction angle for the Agua Zarca - Cutler contact were assumed to be:

friction angle, $\phi = 20°$
cohesion, $c = 1000$ psf

as determined from direct shear tests.

The unit weight was established by the mine staff as 148 lb/ft^3. The bonding strength between the grout and Cutler Formation rock, as indicated by testing, was assumed to be 250 psi.

Although water does occur in isolated pockets in and around the pit area, it was assumed for design purposes that pore water pressure was small or close to zero. This was based on data supplied by the mining company from open hole piezometer measurements and experience in past mining operations. Shear testing had been performed at in situ conditions, which for the weaker materials was wet, and it was felt this would account for any direct influence of water on material strength. It was strongly recommended, however, that the mine staff be ready to immediately implement a dewatering program using horizontal drains, should water problems manifest themselves.

The forces resulting from blasting, earthquakes and surcharge loads were assumed to be negligible.

A working load of 60% of ultimate cable strength was assumed for the rock anchor calculations. With an initial load of 70% of ultimate, this allows for 10% creep loss. An angle of 8° below horizontal was assumed for anchor inclination. Although an uphole would provide more efficient support, a downhole is much less expensive and easier to use. In the event that tendon insertion should prove too difficult at 8°, it was recommended this angle be increased and the total number of bolts be increased to maintain the designed factor of safety.

CONCLUSIONS

Conclusions drawn from tests on the mechanical properties were:

a. The Agua Zarca Sandstone as a unit has fairly consistent engineering properties typical of a weak sandstone. Numerous clay seams reduce the rock mass strength and instabilities would probably be controlled by the location and orientation of these seams. The rock substance uniaxial compressive strength of over 4000 psi is adequate for surface anchorage of a rock anchor support system.
b. The Cutler Formation is 50 to 100% stronger than the Agua Zarca Sandstone. It is a weak to moderately strong sandstone and anchorage for a rock anchor should present no difficulty.
c. The contact between the Agua Zarca Sandstone and the Cutler Formation, which is considered to be a potential sliding surface, has the following shear strength parameters:

   angle of friction, $\phi = 20°$
   cohesion, $c = 1000$ psf

DESIGN ZONES

The southeast slope was divided into three design zones, as shown in Fig. 12, according to the significance of slope stability to mining operations. A separate anchoring system was designed for each zone. Stability was considered most significant in Zone I and least significant in Zone III.

TIME CONSTRAINTS

Mine management stipulated that all mining and associated anchor installations for the east wall be completed in a seven-month period. The specified life of the slope, and therefore of the anchors, was a maximum of six years.

ANALYSIS

The cable anchoring design technique was based on the concept of balanced forces - forces

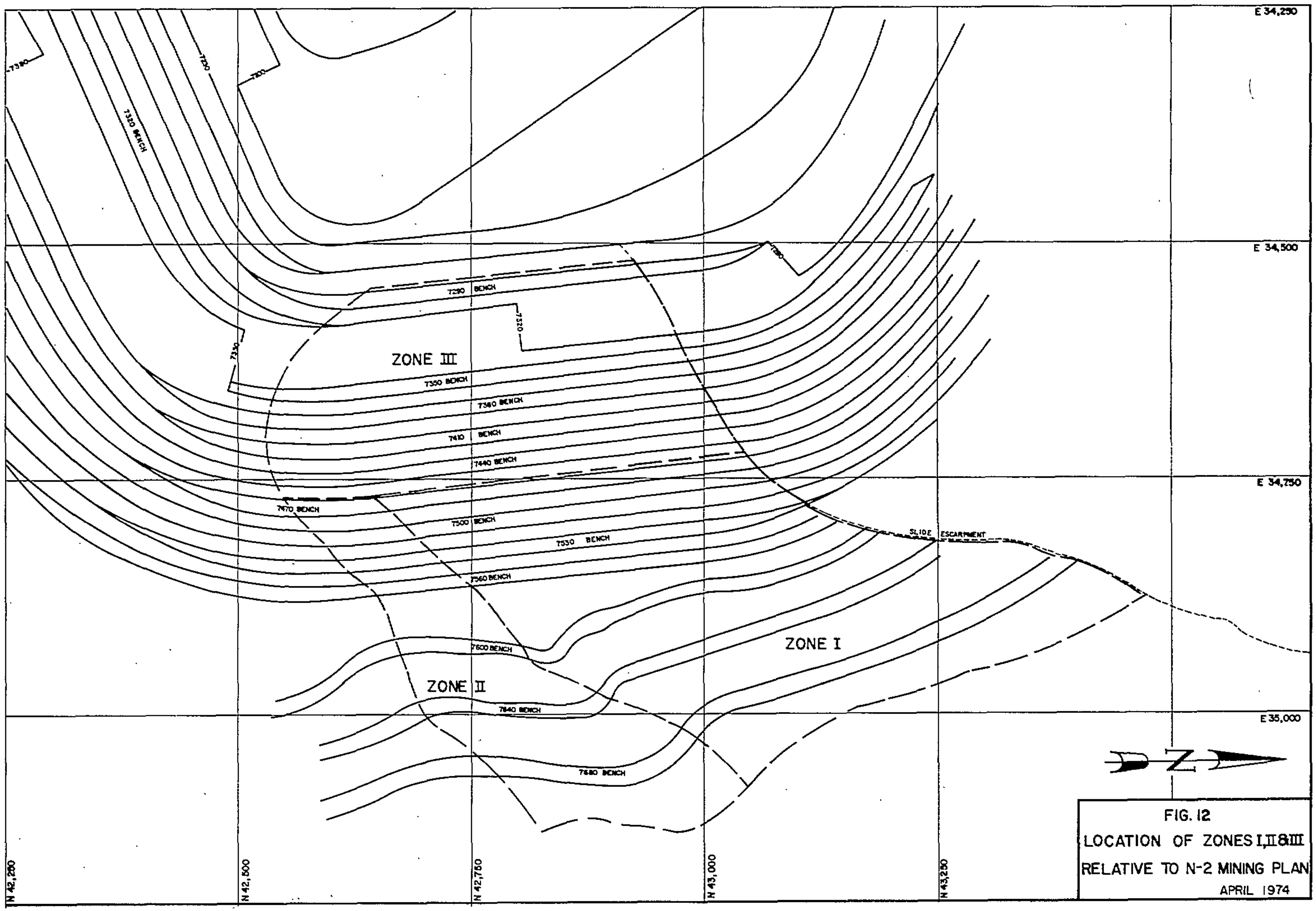


Fig. 12 - Location of Zones I, II and III relative to N-2 mining plan

tending to resist slope movement being determined and summed, and forces tending to cause failure also being determined and summed. The ratio of resisting to disrupting forces is defined as the safety factor against failure (eq 3). A safety factor of 1 indicates the state of limiting equilibrium, less than 1 suggests failure, and greater than 1 suggests stability.

$$SF = \frac{\text{shear strength}}{\text{shear force}} \qquad \text{eq 3}$$

Shear strength is assumed to be given by Coulomb's equation,

$$s = c + \sigma_n \tan\phi \qquad \text{eq 4}$$

where s = shear strength
c = cohesion
ϕ = angle of friction
σ_n = normal stress

Equation 4 may be rewritten in terms of forces and substituted into eq 3 to produce the following safety factor:

$$SF = \frac{cA + F_n \tan\phi}{\text{shear force}} \qquad \text{eq 5}$$

where A is the area of the sliding surface. In this case

$$F_n = \sigma_n A$$

The forces involved are shown in Fig. 13; the resolution of these forces into components is shown in Fig. 14.

Resolving normal to the plane of sliding gives

$$F_n = W\cos\alpha - U - V\sin\alpha + T\sin(\alpha + \delta) \qquad \text{eq 6}$$

where W = weight of sliding mass
U = water uplift force
V = horizontal water force
α = dip of sliding surface
δ = angle between anchor and horizontal (negative if above horizontal, positive if below horizontal)

Resolving parallel to the plane of sliding gives:

$$SF = W\sin\alpha + V\cos\alpha - T\cos(\alpha + \delta) \qquad \text{eq 7}$$

Substituting eq 6 and 7 into 5 gives

$$SF = \frac{W\cos\alpha - U - V\sin\alpha + T\sin(\alpha + \delta)\tan\phi + cA}{W\sin\alpha + V\cos\alpha - T\cos(\alpha + \delta)} \qquad \text{eq 8}$$

and eq 8 is the basic analysis equation.

Note that the term $T\cos(\alpha + \delta)$ is an active force, i.e., positively applied. For this reason it is included in the denominator of eq 8, as reducing the disturbing (shearing) forces.

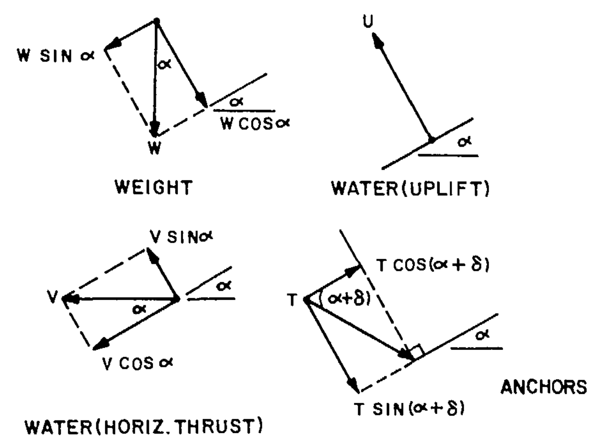


Fig. 13 - Geometry of a slope section showing forces

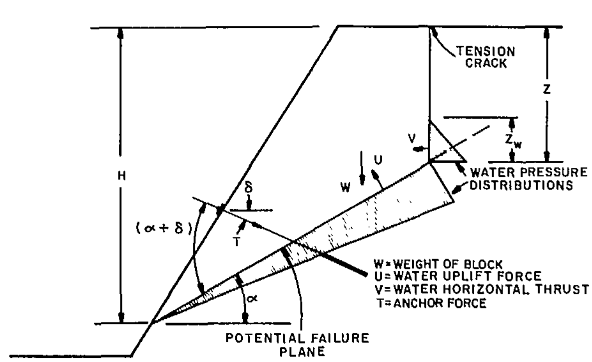


Fig. 14 - Resolution of forces in slope

SELECTION OF SAFETY FACTOR

Figure 15 shows the relationship between the total number of anchors and the safety factor, using anchors with a working load of 391 kips. A safety factor of 1 would require 240 anchors. The recommended minimum number of anchors was 360, with a corresponding low safety factor of 1.033. Mine management had made a prior decision, based on the preliminary estimate, to use about 360 anchors. Conservative estimates had been used in evaluating the design variables, e.g., sliding plane angle, volume of rock, and shear strength. It was therefore decided that the safety factor of 1.033, though lower than normally acceptable, would be a realistic value to adopt. It was stipulated, however, that close monitoring of the installation using load cells as described below, would be necessary and that additional anchors be installed if monitoring of the initial anchors indicated overloading. Conversely, if early monitoring indicated that fewer anchors were required, a reduction in total numbers might then be possible.

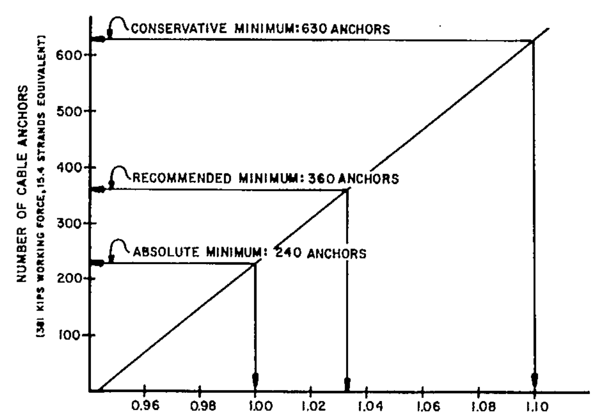


Fig. 15 - Safety factor vs. number of anchors

ANCHOR LAYOUT

The final anchor design included 19-strand (470 kips working load), 16-strand (396 kips working load) and 8-strand (210 kips working load) anchors, laid out as shown in Fig. 16. The 8-strand units were used as monitoring anchors, and included a 350-kip capacity load cell between the anchor head and bearing plate. Table 5 gives the anchor sizes and spacings for the various design zones.

Load cells were installed on approximately every sixth anchor. The load cells were included to monitor both short-and long-term changes in anchor load, and to assess the degree of conservatism of the original design. Rate of change of load during the installation would indicate the need for either more or fewer anchors. Rapid increase in load would indicate the need for more anchors, and no change might indicate the need for fewer anchors.

SUPPORT SPECIFICATIONS

Cable anchors consist essentially of the components shown in Fig. 17. The operations involved, include hole drilling, anchor fabrication and emplacement, primary grouting, blockout construction, stressing, and secondary grouting. Basic recommended specifications are described below.

HOLES

The drill hole size required depends on the number of cable strands to be used, anchor angle, diameter of the anchorage section where the spreader rings are located, and overall length. The number of strands varied between 8 and 19 and the anchor angle was initially set at 8° below horizontal. The maximum diameter of the anchorage section was set at 4 in. and the maximum length was calculated to be between 250 and 275 ft, depending on the location of the Agua Zarca - Cutler contact. Based on these variables, a 4-1/2-in. hole size was recommended for 19- and 16-strand units and a 3-5/8-in. hole size for 8-strand monitor units.

All holes were to be thoroughly cleaned prior to tendon insertion. Clay or other similar materials which remained on the sides of the hole were to be removed for ease of tendon insertion and to maintain maximum bonding between grout and rock. Holes were to have a reasonably water-tight anchorage section to prevent grout loss and

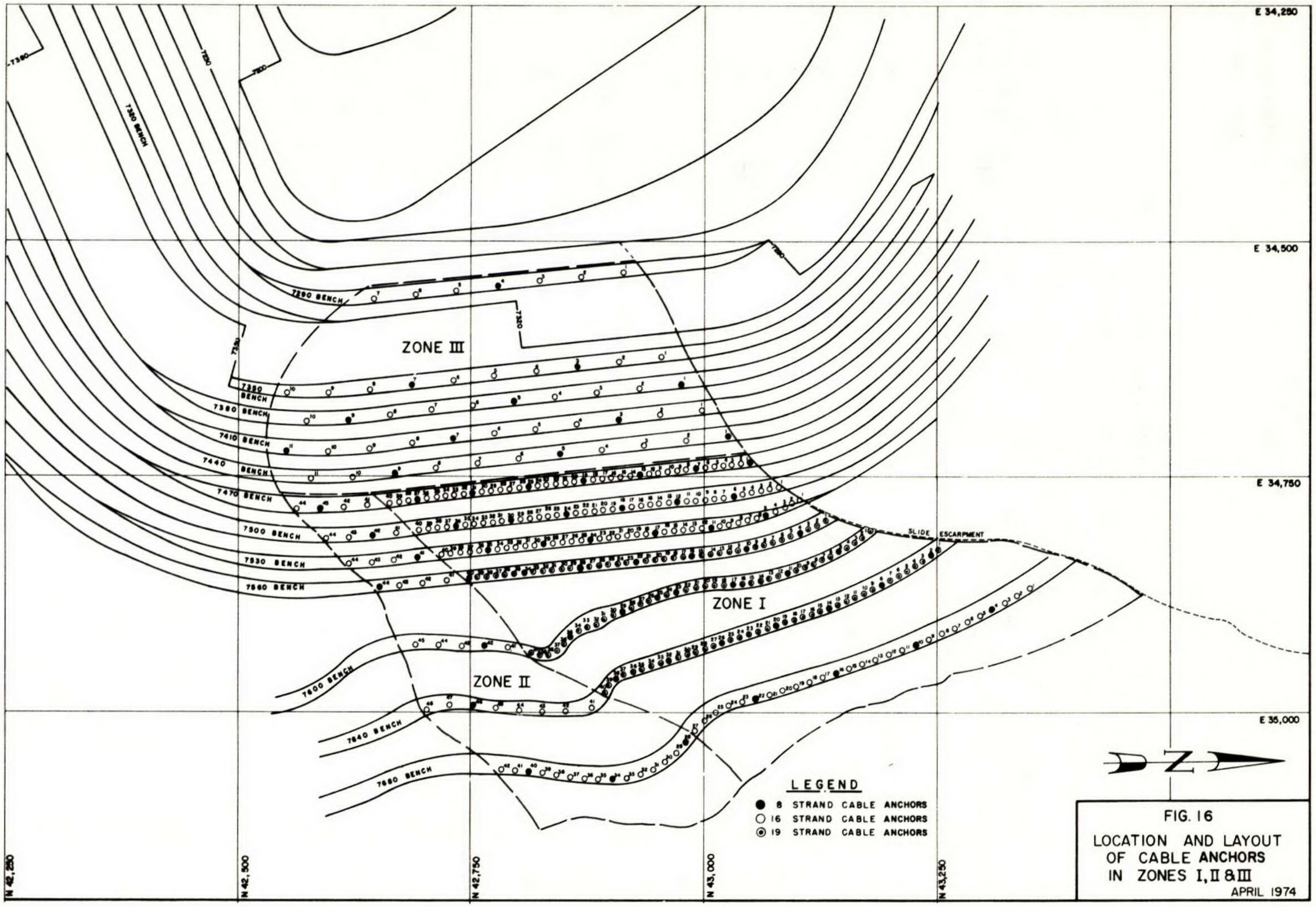


Fig. 16 - Location and layout of cable anchors in Zones I, II and III

Table 5: Summary of anchor sizes and spacing on each bench of east N-2 pit slope

	Number of anchors by strand size											
	Zone I			Zone II			Zone III			Horizontal spacing, ft.		
Bench	8	16	19	8	16	19	8	16	19	Zone 1	Zone 2	Zone 3
7680	4	23	0	3	12	0	0	0	0	15	15	-
7640	7	0	33	1	7	0	0	0	0	10	25	-
7600	6	0	34	1	4	0	0	0	0	10	25	-
7560	7	0	33	1	3	0	0	0	0	10	25	-
7530	6	34	0	1	4	0	0	0	0	10	25	-
7500	6	34	0	1	3	0	0	0	0	10	25	-
7470	7	33	0	1	3	0	0	0	0	10	25	-
7440	0	0	0	0	0	0	3	8	0	-	-	45
7410	0	0	0	0	0	0	3	8	0	-	-	45
7380	0	0	0	0	0	0	3	7	0	-	-	45
7350	0	0	0	0	0	0	2	8	0	-	-	45
7290	0	0	0	0	0	0	1	6	0	-	-	45
Totals												
8	43			9			12			64		
16		124			35			37		196		
19			100			0			0	100		
										360	Grand total	

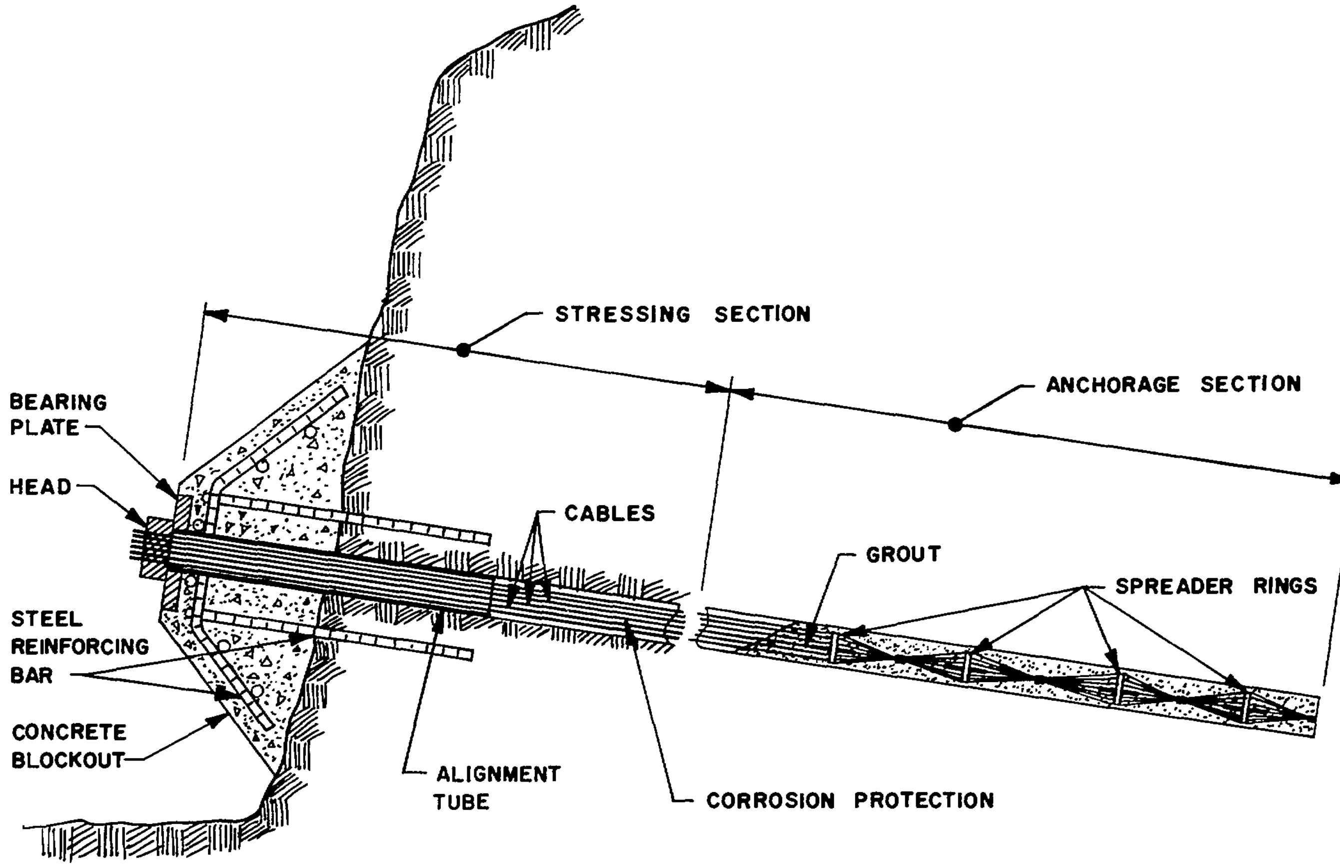


Fig. 17 - Basic components of a finished cable anchor

possible anchorage slip. Holes with shattered or broken anchorage sections were to have been pressure grouted and redrilled. If difficulties were encountered in cleaning the holes or inserting the anchors, larger holes would have to be drilled.

CABLE ANCHORS

The cable used to construct anchors was to satisfy the requirements of ASTM A 416 and 1/2-in. diameter, 7-wire strand. The physical properties were to be

Guaranteed Ultimate Strength	- 41,300 lb
Yield Strength at 1% elongation)	- 35,100 lb
Approximate Modulus of Elasticity	- 27,000,000 psi
Minimum elongation at rupture	- 3.5% in 24 in.

Rust coated or pitted cable was to be rejected because it might have had less than the required ultimate strength, or its life might have been less than required by the mining company. Care was to be taken during tendon construction to keep the cable clean and rust free. Each tendon was to have four spreader rings in the anchorage section. They were to be located approximately at 2, 7, 12 and 17 ft from the tendon end. The suggested design for the spreader rings is shown in Fig. 18 and 19.

The length of the anchorage section depends on the maximum tension applied to the anchor, the diameter of the hole, and bonding strength between rock and grout. The bonding strength between grout and steel tendon is known from both laboratory and field testing in civil applications to exceed the tendon tensile strength and therefore need not be considered.

The length of the stressing section is governed by distance from the hole collar to the potential sliding plane. This distance along bench 7680 in places exceeded 200 ft. Toward the bottom of the pit, the stressing length was as short as 30 ft, which is the recommended minimum. Short anchors do not elongate as much as long ones during tensioning. Consequently, the loss of force during seating of the wedges is proportionately much greater and may result in tension forces less than the design value. The average stressing length for all anchors was estimated to be 90 ft with the average weighted anchorage length being 38 ft. For 360 anchors of an average length of 128 ft, approximately 46,000 ft of drilling was required.

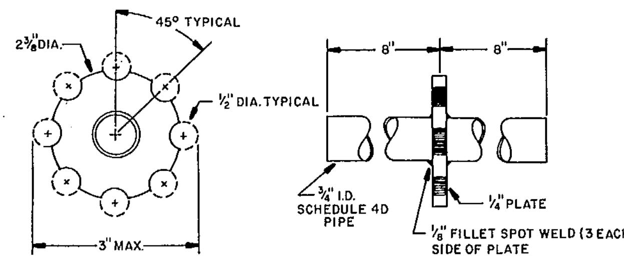


Fig. 18 - Spreader ring design for 8 strand anchor

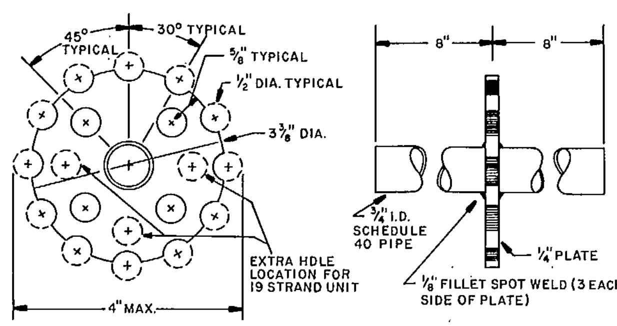


Fig. 19 - Spreader ring design for 16/19 strand anchors

BLOCKOUTS

The blockout is a steel reinforced concrete structure used to transmit the stressing forces from the tendon strands to the surrounding rock. In addition, the blockout provides the directional alignment of the stressing forces from the anchor head to the hole through a steel alignment tube and bearing plate. It was recommended that one blockout size be used for all anchors to simplify design and construction.

A steel reinforcing set was placed in each blockout to increase strength and dissipate stress concentration in the concrete. The suggested design of a reinforcing set is shown in Fig. 20.

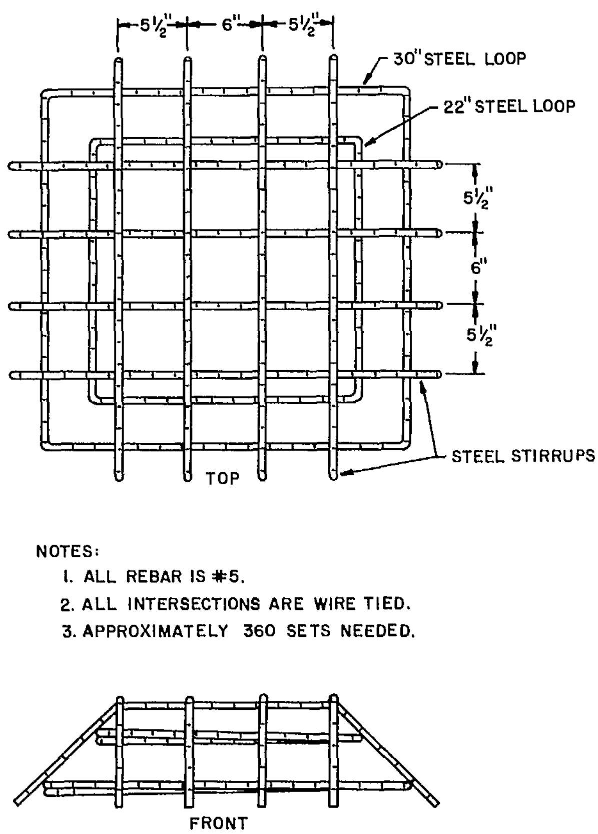


Fig. 20 - Reinforcing set design for blockouts

For convenience, these sets were to be fabricated prior to drilling. Four No. 10 steel reinforcing bar dowels approximately 30 in. long were to be used in each blockout. These dowels were placed in holes drilled into the rock using a standard air-powered jackhammer. The suggested design for alignment tubes and bearing plates is shown in Fig. 21. A quick setting high strength cement, such as Portland Type 2 or Type 3 was to be used in the blockouts; a minimum strength of 4500 psi at 14 days was required. The recommended shape and arrangement of reinforcing bar, bearing-plate alignment tube and concrete relative to a typical borehole is shown in Fig. 22. Each blockout site was to be faced in such a way as to yield a relatively flat surface perpendicular to the borehole. Bearing surfaces inclined to the borehole may result in shearing and offsetting of the blockout relative to the borehole during post-tensioning. All blockouts were to be constructed to meet or exceed the design requirement described above. Standard test cylinders of concrete were to be taken during blockout construction to ensure that design requirements for concrete strength were being met.

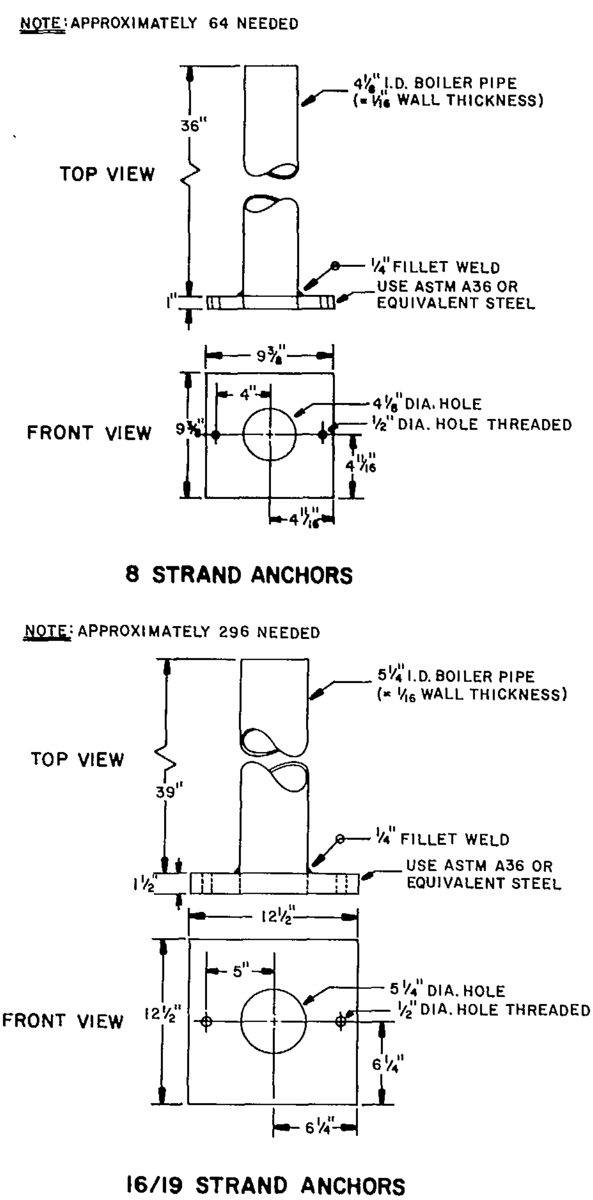
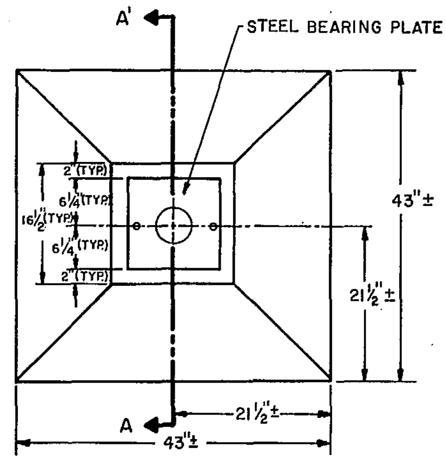
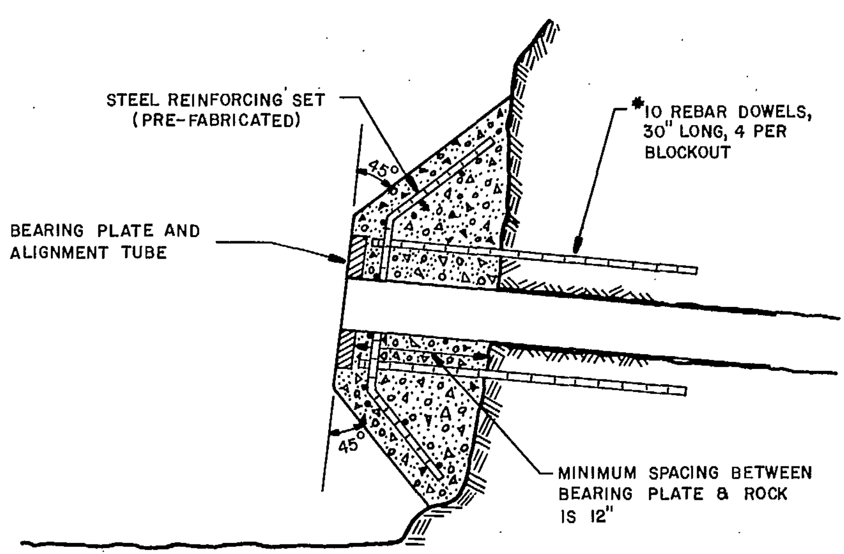


Fig. 21 - Alignment tube design for 8 and 16/19 strand anchors



NOTE:

① MINIMUM CONCRETE–ROCK CONTACT AREA: 1000 SQ. IN.
② TOP BOTTOM SIDES SLOPE OUT AT A MINIMUM OF 45°
③ CONCRETE SHOULD BE 4500 P.S.I. (14 DAYS)



A–A'

Fig. 22 - Blockout design

PRIMARY GROUTING

Anchorage grouting was to be conducted as soon as possible following tendon emplacement and was to be a mix of portland type 2 cement, water and Intraplast N, which increases grout fluidity and results in expansion of the grout before hardening. The recommended mix was:

Type 2	one 94-lb bag
Water	5 gallons (U.S.)
Intraplast N	1/2 - 3/4% by weight of cement

The amount of grout pumped into the anchorage section was not to exceed 1-1/2 bags and 3 bags for the 8-strand and 16/19-strand units respectively. After the grout had fully set, a check on the length of the anchorage section was to be made. This was to be done by measuring the length of a single strand forced down the hole to the anchorage section. If the anchorage section length was less than the design value, additional grout was to be added. The above 1-1/2-bag mix would yield an approximate anchorage length of 28.6 ft in a 3-5/8-in. diameter hole containing 8 strands. The 3-bag mix would yield 38.7- and 29.6-ft anchorage lengths in a 4.5-in. diameter hole containing 16 and 19 strands, respectively. These anchorage lengths were to be adequate to ensure that rock-grout bonding failure did not take place. The calculated safety factors against bonding failure of such anchorage sections of 8, 16, and 19 strands were 2.9, 2.4 and 2.1, respectively.

STRESSING

Post-tensioning of the cable anchors was to take place not less than 14 days following the later of primary grouting or blockout construction. The maximum stressing force was not to exceed 85% of ultimate strand strength and the initial working force was not to be less than 70% of ultimate strand strength. A record of the tendon elongation during stressing was to be kept to the nearest 0.10 in. Such a record allows an additional check to be made on the anchor loads and the stressing length. During stressing of the 8-strand units with load cells, an additional record of loading was to be kept to check the agreement between load cells and stressing equipment.

After the initial stressing of the tendons, the excess strand was not to be cut off as is sometimes done. This excess strand was to be greased using non-oxide grease. Following stressing of all anchors in the slope, a re-stressing of the units was to be made if deemed practical and feasible. Creep losses in the strand, grout, and blockout could cause load losses resulting in a working force of approximately 60% of ultimate strand strength or less. A re-stressing of the tendon could allow as much as a 5% overall gain in the tendon load. This increase in tendon load could increase the effectiveness of 360 to the equivalent of 378 anchors at very little added cost.

PROTECTION AGAINST CORROSION

Protection against corrosion was originally to be achieved by first greasing and then covering each individual cable strand with polyethylene pipe. The cost of such protection was estimated to increase the total installation cost by 10%. Discussion were held by project participants as to the need for corrosion protection, the alternate methods available, and their costs. One of the major unknown factors was the time element during which serious corrosion problems could develop. The mining plan called for the anchored slope to be mined in a seven-month period; consequently, actual anchoring was to be completed in that period. This meant that if corrosion protection were delayed until anchoring were completed, the first anchors placed in the slope would have to withstand the effects of corrosion for up to seven months. If there were no corrosion protection at all, the anchors would have to survive unprotected for the desired life of up to six years. Strong recommendations were made to mine management to include corrosion protection, if not as the project proceeded, at least near the end of the project. This would mean that the first anchors placed would have to last approximately six to seven months unprotected. The recommended corrosion protection was as follows:

a. Secondary grout all 16- and 19-strand units using a thinner but similar grout to that used in primary grouting.
b. After completion of the anchoring, remove the load cells from half the 8-strand units and secondary grout. If slope displacement had not taken place by that time, the slope should remain stable unless the overall average working force dropped below 60%. The excess load cells could be saved for future anchors.
c. The remaining 8-strand anchors with load cells were to serve as load monitors for the long term - up to 6 years. They were to be protected against corrosion by pumping a heated non-oxide grease down the holes.

MONITORING

Load cell reading was recommended at two-or three-week intervals during the anchor installation period. The purpose of such The purpose of such monitoring was to note any adverse load increases indicative of potential slope movements, and to take appropriate remedial action. Similarly, if no load increases were observed, the number of anchors in the lower benches might be reduced.

Long term monitoring with monthly readings was recommended for at least one year following mining of the supported wall. Subsequently, bi-monthly or quarterly readings should continue as long as mining was conducted in the area of the anchored slope. Load changes were to be plotted on graphs as a function of time so that trends could be predicted.

DESIGN MODIFICATIONS

During implementation of the support system, new data concerning the basic design assumptions would become available and modifications might be warranted. The most likely areas of new data would probably concern the potential sliding plane location, and hydrology. As hole drilling continued for the anchors, the location, and in particular, the angle, of the Agua Zarca - Cutler contact could be determined more exactly. Knowing its location would permit better estimates of the volume of rock involved in a possible slide. If the angle and/or volume were greater than assumed, the number of anchors would theoretically have to be increased to maintain the required safety factor. Likewise, if the angle and/or volume were less than assumed, a reduction in the number of anchors could be made. It had been assumed that groundwater would not be detrimental to stability.

If water were encountered in more than about 10% of the holes, adverse pressures could exist and the number of anchors would have to be increased or the water would have to be drained.

Other factors such as anchor angle, major geological structures, mechanical properties of the slope materials and outside forces could vary from those assumed. If data became available that indicated these factors to be more adverse than anticipated, an increase in the number of anchors could be necessary.

ANCHOR INSTALLATION

Anchor installation at Nacimiento began during the first week in April 1974. There were five phases: drilling; fabrication and emplacement; primary grouting; stressing; and secondary grouting. Monitoring of the load cells took place throughout the project but this was not considered part of the installation work. Drilling was done by a contractor. Anchor fabrication, emplacement, primary grouting and stressing were done by a contractor specializing in rock anchors. Blockout construction and general labor were provided by the mining company. Project engineering and specification supervision was provided by a consultant.

SITE ACCESS

Site access for the upper three non-mining benches was from the south side of the pit and was maintained by the mining company. Use of these benches was, therefore, unrestricted with the exception of periods of heavy rainfall. Site access to lower benches was through the pit and was occasionally restricted for short periods of time. During periods of heavy rainfall mining operations were frequently halted.

DRILLING

All holes were drilled using a Model 125 Aardvark track mounted drilling machine. This is capable of drilling horizontal, vertical or angled holes using a standard tri-cone bit or a down-the-hole hammer. The majority of the Nacimiento cable anchor holes were drilled with the latter.

The first hole was completed on the 7680 Bench on March 30, and was 4.5 in. in diameter drilled at 12° below horizontal. The orginal design called for all holes at minus 8° at 4-1/2 in. diameter for regular anchors and 3-5/8 in. diameter for monitor anchors. Holes were drilled at minus 12° until the anchoring contractor verified by successful emplacement that minus 8° holes were feasible. On April 20 a change was made to minus 8°. Holes were drilled in the recommended sizes until about May 20 when the driller changed to a 4-1/2 in. diameter throughout because he felt it would be easier and less expensive to standardize.

The location of the assumed sliding plane along the contact of Agua Zarca Sandstone and the Cutler Formation was determined by the color of the drill cuttings. Sandstone cuttings were off-white but as soon as the Cutler Formation was reached they immediately turned dark brownish-red.

The drilling rate for many of the holes was about 40 ft per hour. In some cases where water circulation problems were encountered, the rate fell to 25 ft per hour. During one 11-day period in June, 1974, an average of over 400 ft per day was maintained, including set-up time. Average set-up times were in the order of 15 to 20 minutes.

Standard drill logs were kept by the drilling contractor, including hole angle, hole length, bits used, problems encountered, circulation characteristics and location of clay zones. The soft clay zones were for the most part identified by penetration rate. However, if the clay particles were large enough to see and did not become covered with fine sand they were identified by the cuttings. A summary of the drilling compiled from the records is shown in Table 6. A comparison of these actual figures and the original estimated drilling requirements is included. Variations between the two are due in part to the variation in location of the failure plane and in part to a change in spacing during the final phases of the installation.

FABRICATION

Anchors were fabricated after each hole was completed. This allowed the contractor to make up each one to the exact length needed for a particular hole. Anchors were usually fabricated outside the pit area because considerable space was required. The required number of strands were drawn out of a cable rack and cut to length. Spreader rings were used to form the anchorage end and a plastic tube placed in the centre for pumping grout. The leading end of the anchor had the individual cable ends offset slightly from each other. They were then tightly taped with duct sealing tape to form a tapered end to facilitate insertion.

Blockouts were formed and poured as soon after drilling as possible and either before or after cables were placed in the hole. Construction was as described previously, using an alignment tube, a steel reinforcing rod, and concrete.

Standard test cylinders were taken periodically to determine whether or not the concrete strength was equal to or exceeded design specifications. A summary of test results and design recommendations for blockout concrete strength is presented in Table 7. These results indicated that the 8- and 16-strand anchors met strength specifications.

Emplacement of the cable anchor was usually done as soon as possible after drilling and anchor fabrication were completed. It was accomplished in a few cases by manually pushing the anchor down the hole. A mechanical tugger was used for the

Table 6: Summary of estimated drilling requirements and actual drilling for Nacimiento cable anchoring project

	Recommended no. of holes	Actual holes	Estimated total length	Drilled	Estimated average length	Actual average length*	Actual range
7680	42	37	7896	7103	188	192.0 ± 25.8	148 - 260
7640	48	38	8544	7591	178	199.8 ± 24.8	161 - 259
7600	45	42	7560	7632	168	181.7 ± 30.4	136 - 250
7560	44	43	6072	7345	138	170.8 ± 33.7	115 - 220
7530	44	57	5192	7857	118	137.8 ± 38.9	73 - 205
7500	44	46	4312	6141	98	133.5 ± 32.6	70 - 197
7490	-	16	-	2080	-	130.0 ± 34.9	70 - 180
7480	-	15	-	1845	-	123.0 ± 35.4	70 - 170
7470	44	15	2992	1976	68	131.7 ± 39.1	75 - 201
7460	-	7	-	686	-	98.0 ± 21.6	70 - 128
7440	11	-	858	-	78	-	-
7410	11	-	748	-	68	-	-
7380	10	-	680	-	68	-	-
7350	10	-	680	-	68	-	-
7290	7	-	476	-	68	-	-
Total	360	316	46,010	50,256	128	159	

* mean ± 1 standard deviation

Table 7: Summary of blockout concrete strength

Anchor strands	Recommended concrete strength (14 day), psi	Tested strength, †psi	Number of tests
8	4500	4453* ± 402	5
16	4500	4798 ± 310	14
19	4500	4106	1

† mean ± one standard deviation

* because the 8-strand unit load is only one-half the 16-strand load this value was quite acceptable.

majority of the units, as shown in Fig. 23. Four labourers were required for anchor emplacement in either case.

Grouting of the anchorage section is illustrated in Fig. 24. Such grouting was usually done after several anchors had been placed. This enabled the contractor to spend one day grouting several units and another day tensioning the same units after the 14-day curing time.

Fig. 23 - A mechanical tugger to aid anchor insertion

Fig. 24 - Grouting of an anchorage section

STRESSING

Post-tensioning of anchors was done a minimum of 14 days after both secondary grouting and blockout construction had been completed. The equipment used by the contractor for stressing is shown in Fig. 25 and 26. Stressing was done using a variety of different jacks during the course of the project, ranging from a small hand-held single strand jack to a 500- ton jack for 16- and 19-strand anchors. Anchors were stressed to 80% of ultimate strand strength. Cable elongation was recorded and used to determine the depth to anchorage.

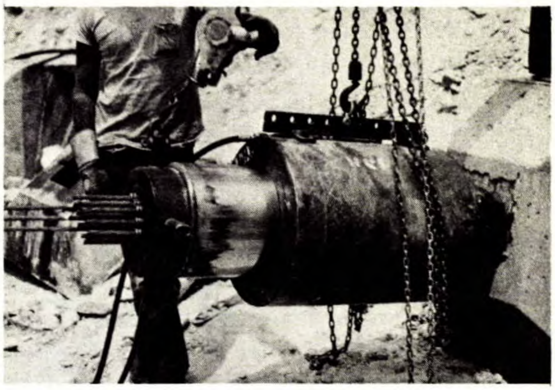


Fig. 25 - Stressing a regular anchor with a 500 ton jack

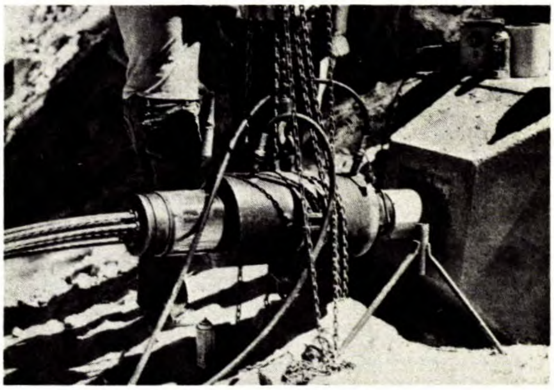


Fig. 26 - Stressing a monitor anchor

SECONDARY GROUTING

Actual grouting of the cable anchors for corrosion protection did not begin until late December. Original plans had called for the project to be completed in seven months, or by November 1974, including secondary grouting. Owing to delays, particularly in cable anchor installation and stressing, secondary grouting was slow in beginning and continued to receive a lower priority than installation and stressing. This was in part due to the need for rapid mining to maintain the supply of ore for the mill. Anchor installation could not be allowed to fall too far behind mining. If it did, installation became difficult because of cramped space once a bench was mined out. The anchor contractor thus placed his entire crew on installation and stressing to keep pace with mining. After secondary grouting began, winter weather resulted in access difficulties. At times several weeks passed when no secondary grouting took place, particularly in January and February 1975. Cable stress corrosion problems, described below, become strongly evident in early 1975; these further delayed secondary grouting, because the mining company did not want to proceed with remedial action until they were convinced they had the best possible solution to the problem. Shortly after this, the cable anchoring contractor finished installing and stressing the remaining units. The mining company then elected to finish the secondary grouting themselves. Further secondary grouting did not begin properly until late April and early May 1975 and was only completed by late June or early July. There is some question as to whether or not all anchors were secondary-grouted.

LOAD CELLS

Load cells for monitoring were purchased from the cable anchoring contractor and were of the strain-gauge type. Four SR-4 strain gauges were mounted in each unit at radial positions 90° apart in a hollow steel tube approximately 6 in. in length with 5-1/4 in. inside diameter and 6-1/2 in. outside diameter. A balancing bridge readout unit was used in conjunction with the load cells. A typical monitor anchor with load cell and readout unit is shown in Fig. 27. Approximately every sixth anchor was an 8-strand unit with such a load cell. Installation of these devices was made just prior to stressing by placing the load cell between the bearing plate and anchor head. The load was continuously monitored during post-tensioning to check both the loading and elongation of the cable. After stressing, the monitor anchors were read weekly to determine if adverse loading was occurring during installation. If the load were to decrease slightly, it was presumed due to creep in the system. If the load were to decrease rapidly it was presumed the anchorage section had failed or slipped. If the load increased, it was presumed that the slope was moving.

Fig. 27 - Typical load cell with read-out unit

ANCHOR EFFECTIVENESS

Data on cable elongation and load were collected for both the regular and monitor anchors. The stressing length for each anchor was determined by a back-calculation using the elastic modulus of the steel cable, cross sectional area, cable elongation and the change in stress for the corresponding elongation.

Elasticity theory relates stress and strain in the cable as follows:

$$E = \frac{\sigma}{\varepsilon} \qquad \text{eq 9}$$

where E = elastic modulus
σ = stress
ε = strain

A change in stress is effectively given by

$$\Delta\sigma = \frac{\Delta F}{A} \qquad \text{eq 10}$$

where ΔF = change in stressing force
A = cable cross-sectional area

and the strain change in the cable is given by

$$\Delta\varepsilon = \frac{\Delta L}{L} \qquad \text{eq 11}$$

where ΔL = elongation due to change in stressing force

L = total stressing length

Substituting eq 10 and 11 in eq 9 and rearranging gives:

$$L = \frac{\Delta L \ A \ E}{\Delta F} \qquad \text{eq 12}$$

This equation was used to calculate the stressing length for each anchor. The length of the anchorage was then determined by subtracting this value from the total length of the anchor.

A stressing analysis was made for each regular anchor to assess effectiveness. A cable whose anchorage section projects above the potential failure plane is not completely effective in applying the anchor force across this plane. The basis upon which the effectiveness of a particular anchor was assessed was as follows:

a. An anchorage section from beyond the failure plane to 10 ft above the failure plane was assumed 100% effective.
b. An anchorage section from 10 to 20 ft above the failure plane was assumed to be 75% effective.
c. An anchorage section from 20 to 30 ft above the failure plane was assumed to be 50% effective.
d. An anchorage section 30 ft or more above the failure plane was assumed to be only 25% effective.

While this system of assigning anchor effectiveness was arbitrary, it was believed to be realistic. A PVC grout tube runs to the bottom of the hole. For grout to rise above the anticipated sliding plane, it must first be pumped out of the PVC tube and then flow back up the hole. There must therefore be at least some grout from the top of the anchorage section to the bottom of the hole. This grout results in bond between the rock and tendon throughout the anchorage section. If movement occurs across the anticipated sliding plane, the tendon, in the worst case, would provide at least some passive resistance to sliding. That is, a small movement would cause part of the anchorage section to give some resistance to movement. To disregard the passive effect of these anchorage sections in resisting movement is to be overly conservative. However, quantitative values for this passive effect cannot be determined. Clearly, the closer the top of the anchorage section is to the anticipated failure plane, the greater is the active resistance to movement. The farther away the top of the anchorage section is, the more necessary it will be to have slope movement across the anticipated sliding plane to tension the passive units before these can resist sliding. The decision was therefore made to proportion the anchor effectiveness as a function of distance from the top of the anchorage section to the potential sliding plane. In the event of movement across the anticipated sliding plane, some breakage of the grout in the anchorage section would probably take place. For distances up to 10 ft above the plane, it is felt there would be enough stress in the tendon - the stressed portion above the anchorage section - that very little movement along the potential sliding plane would immediately cause the tendon to resist movement. For distances beyond 30 ft, probably the only resistance to movement would be the passive effect resulting from the actual slope movement. Because of the unknown nature of the anchorage bonding in the vicinity of the potential sliding plane, it was estimated that no more than 25% effective resistance should be counted on if movement were to take place. For intermediate distances of 10 to 20 ft and 20 to 30 ft, more resistance could probably be expected and therefore 75% and 50% effectiveness were respectively assigned to these anchors.

Figure 28 is a schematic drawing showing calculated anchor locations and the assumed anchor effectiveness on the 7600 Bench. A summary by mining bench of the number of stressed anchors and their mean anchorage lengths is presented in Table 8. Mean anchor lengths for regular anchors for various groupings of hole angles and strand numbers are presented in Table 9. The importance of hole angle in achieving effective anchorage sections is shown by the data in Table 9.

A stressing analysis similar to that for the

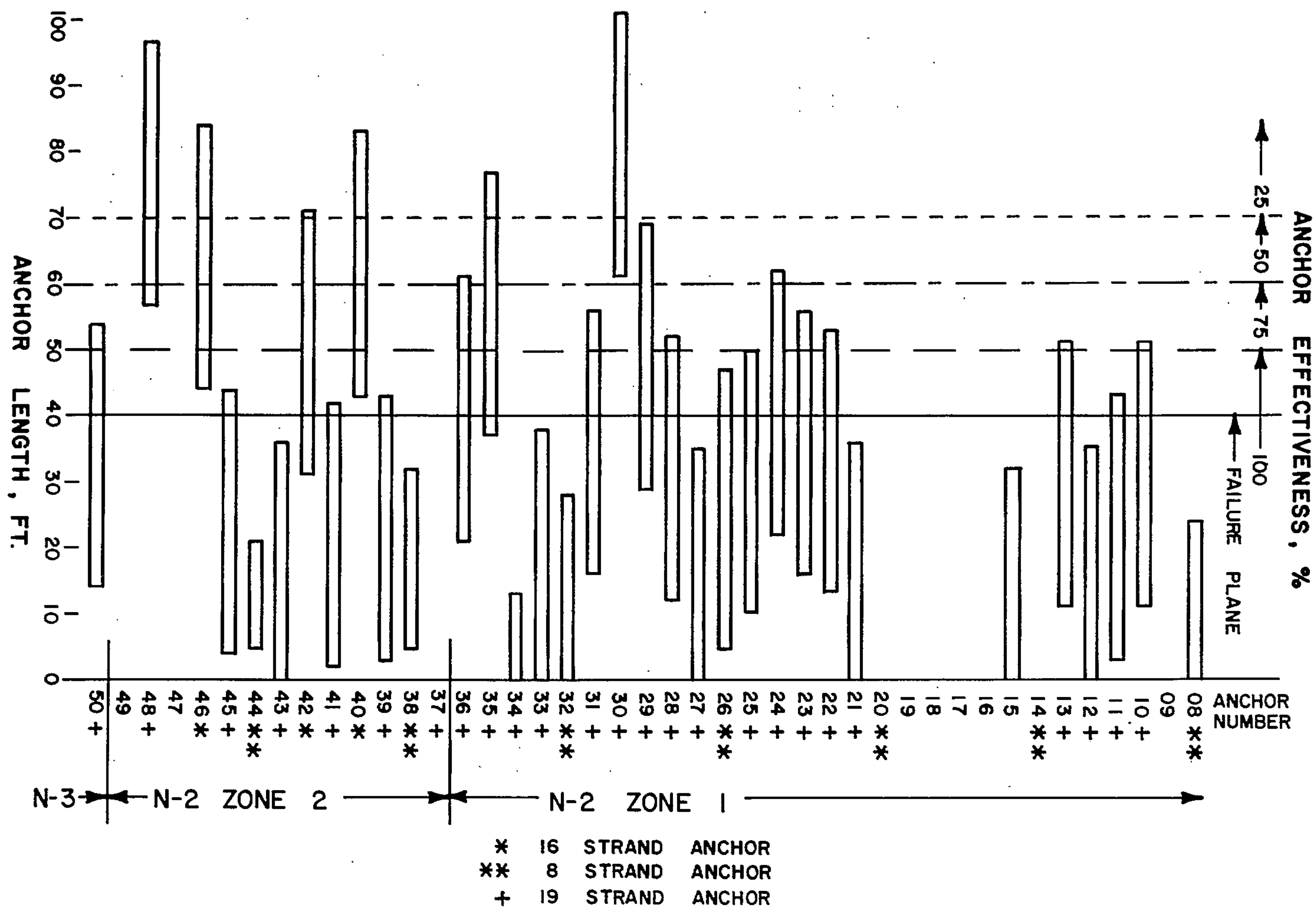


Fig. 28 - Schematic of 7600 Bench showing calculated anchor locations and assumed anchor effectiveness

Table 8: Summary of anchorage lengths by bench for regular anchors

Bench number	Number of anchors	Mean anchorage length, ft.	Standard deviation of anchorage length, ft.
7680	27	35.2	12.9
7640	28	40.4	9.3
7600	28	54.5	20.4
7560	29	41.0	21.0
7530	16	31.4	12.4
7500	18	23.9	6.4
7490	13	24.6	5.4
7480	12	32.9	13.3
7470	8	24.7	8.3

Table 9: Anchorage lengths

Parameter	Number of anchors	Mean anchorage length, feet	Standard deviation of anchorage length, feet
All regular anchors	179	37.2	17.2
All anchors at 8°	89	46.9	17.4
All anchors at 12°	90	27.5	10.2
19 strand anchors	74	44.0	18.2
16 strand anchors	105	32.4	14.7
19 strand anchors at 8°	65	47.6	16.5
19 strand anchors at 12°	9	18.7	4.2
16 strand anchors at 8°	24	46.0	19.7
16 strand anchors at 12°	81	28.5	10.2

regular anchors was also done for each monitor anchor. In addition, the depth to grout was measured directly before stressing in several of the units for comparison with the calculated stressing lengths. In all cases, the measured stressing length differed only one or two feet from that calculated. The mean and standard deviation for anchor lengths of all monitor anchors was as follows:

$$n = 24$$
$$\bar{x} = 35.2 \text{ ft}$$
$$\text{S.D.} = 11.8 \text{ ft}$$

LOCATION OF SLIDING PLANE

The angle of the potential sliding plane was continually re-evaluated by constructing sections through the slope and using drill hole data to locate the Agua Zarca - Cutler contact. For the original design, the potential sliding plane in Zone I was assumed to be inclined at 28°. Seven sections normal to the slope were evaluated, bench by bench, to determine more precisely the potential sliding plane angle. Successive evaluations during the first eight months of the project resulted in the following values for benches above 7500.

	Mean sliding plane angle	Standard deviation
Initial design	28°	-
First evaluation	25.1°	6.7°
Second evaluation	23.2°	2.2°
Third evaluation	22.6°	1.1°
Fourth evaluation	23.0°	0.9°

It was apparent by the second evaluation that the Agua Zarca - Cutler contact dipped at less than 28°, and therefore that the original design was conservative, with a safety factor higher than expected.

MONITORING

ANCHOR LOAD

A careful record of all load cells was maintained to assess seating and creep losses in the 8-strand units. This data was then used to assess the actual working force applied to the potential failure plane. It was assumed that the results of the 8-strand load cell units could be extrapolated to estimate the seating and creep losses and working forces in the regular 16- and 19-strand anchors. Load cells installed during May and June were read several times during the first few days to determine the approximate magnitude and duration of load losses. These initial readings, on approximately 13 load cells on the 7680 and 7640 Benches, indicated that most load loss occurred in the 24 hours after loading. The load loss due to seating and creep for one load cell is given in Table 10 and is typical of that observed in most units.

Table 10: Seating and initial creep losses for load cell no. 7 on anchor 68-40

Date	Time	Readout reading	Magnitude of force, kips
5/30/74	10:49 a.m.†	528.6	264.3
5/30/74	10:50 a.m.	507.0	253.5
5/30/74	11:00 a.m.	505.5	252.7
5/30/74	11:15 a.m.	503.9	251.9
5/30/74	11:30 a.m.	503.2	251.6
5/30-74	2:35 p.m.	502.2	251.1
5/30/74	4:45 p.m.	500.3	250.1
5/31/74	8:00 a.m.	494.1	247.0

† at 10:49 a.m. the hydraulic jack was still pressurized; at 10:50 a.m. the jack was released

It was originally assumed that all units would be stressed initially to 80% of ultimate strand strength and would lose approximately 10% of ultimate strength in seating losses and 10% in creep losses. This would result in a working force of approximately 60% of ultimate strength, which was the working force assumed in the original support design. Initial indications, after completing the 7680 and 7640 Benches, were that working forces would be 8 to 10% higher than the assumed 60% of ultimate strength. The average seating loss in 13 monitor units was calculated to be only 6.5% of ultimate strength (initial working force 73.5%). After two months (through July 31, 1974) these same units had a working force of 70.6% of ultimate strand strength, indicating a two-month creep loss of only 2.9%.

Calculations after approximately four months, through September 30, 1974, for 22 load cell units tended to verify the values for seating and creep losses. The overall average working force remained about the same, decreasing slightly to 70.4%. The average seating loss for 22 units was 5.2% and average creep loss increased to 4.4%.

A summary of seating and creep losses, and working force for all load cells installed through November 30, 1974 is presented in Table 11. This table indicates that the original assumptions of a 10% loss for seating and a 10% loss for creep were conservative. Excluding five anchors which had strand failure and three which had anchor slippage, as described below, the following creep losses and working forces, as a percentage of ultimate strength, were obtained:

Creep loss:

mean	= 4.1%
standard deviation	= 1.7%

Working force:

mean	= 70.6%
standard deviation	= 2.7%

Anchorage slippage or individual strand bonding failures in the anchorage sections took place in about one third of the units shown in Table 11. These failures were presumed due to insufficient grout in the anchorage section. Sufficient grout had been pumped down the hole in all cases. The grout, therefore, probably did not sufficiently cover the tendon or an individual strand due to sand or clay material in the hole, or was lost in a crack.

Load cells were initially read weekly, or occasionally twice weekly, and plots of load against time were drawn for each cell. Typical plots for two monitor units are illustrated in Fig. 29. The initial drop in load was due to seating of the anchor wedges and the further decrease reflected creep loss.

With the exception of two load cells on the 7680 Bench, no load cells had shown any significant increase in load to September 1974. The two units which increased reflected a small surficial movement on the 7680 and 7640 Benches; the increase in load was approximately 10 kips. Both units maintained constant load after this initial increase.

In late September 1974 one load cell on the 7680 Bench began to lose load and during October and November five more load cells on the upper two benches lost a portion of their load. There was no apparent surface movement and the remaining cells showed a constant load. The problem was initially believed to be slippage of the anchor section. Liftoff tests, in which the anchor is restressed using the stressing jack, were performed to determine whether or not the anchor could be restressed to its initial load without causing the anchorage section to slip. If the anchorages slipped during the liftoff tests, the problem would be confirmed and a change in anchorage design could be made. However, the tests showed that individual strands had failed with no increase in load. The strands had not failed in the anchorage section, but at various points in the stressing section of the anchor. The majority of the load cell units on the upper benches had shown some drop in load by the end of December 1974. The load loss as it typically occurred in these units is illustrated in Fig. 30. In September 1975, load cells were still being monitored approximately monthly, and although several had failed partially or completely, none had shown a significant increase in load.

GROUNDWATER

The original support design assumed that adverse groundwater forces would not be encountered. It was recommended, however, that if they were, a larger number of anchors should be installed and/or horizontal drains should be placed

Table 11: Summary of seating losses, creep losses and working forces through November 31, 1974

Load cell	Ultimate load kips	80% of ultimate, kips	Initial force, kips	Working force, kips	Working force %†	Seating loss, %†	Creep loss, %†
1††	330.4	264.3	244.5	122.0	36.9	6.0	37.1
2††	330.4	264.3	247.5	176.0	53.3	5.1	21.6
3††	330.4	264.3	251.5	82.7	25.0	3.9	51.1
4*	289.1	231.3	215.8	200.0	69.2	5.4	5.5
5	330.4	264.3	248.0	231.3	70.0	4.9	5.1
6††	330.4	264.3	253.0	200.8	60.8	3.4	15.8
7	330.4	264.3	253.5	236.1	71.5	3.3	5.3
8	330.4	264.3	243.2	229.7	69.5	6.4	4.1
9	330.4	264.3	229.6	216.7	65.6	10.5	3.9
10††	330.4	264.3	252.5	203.0	61.4	3.6	15.0
12	330.4	264.3	253.1	226.8	68.6	3.4	8.0
13	330.4	264.3	243.1	224.8	68.0	6.4	5.5
14	330.4	264.3	244.0	221.1	66.9	6.1	6.9
15	330.4	264.3	245.0	236.3	71.5	5.8	2.6
16	330.4	264.3	242.1	228.5	69.2	6.7	4.1
17	330.4	264.3	250.2	235.1	71.2	4.3	4.6
18*	247.8	198.2	199.5	192.9	77.8	-	2.7
19	330.4	264.3	248.8	235.5	71.3	4.7	4.0
21	330.4	264.3	248.0	235.8	71.4	4.9	3.7
22	330.4	264.3	246.3	234.5	71.0	5.4	3.6
23	330.4	264.3	246.0	233.8	70.8	5.5	3.7
24	330.4	264.3	250.1	244.5	74.0	4.3	1.7
26	330.4	264.3	242.1	236.2	71.5	6.7	1.8
27	330.4	264.3	244.9	240.9	72.9	5.9	1.2
28**	330.4	264.3	129.4	127.8	38.7	-	0.5
29**	330.4	264.3	188.2	180.8	54.7	-	2.2
30**	330.4	264.3	163.2	159.7	48.3	-	1.1
mean			234.2	207.1	63.7	5.3	8.2
standard deviation			30.2	41.5	12.9	1.6	11.6
standard error			5.8	8.0	2.5	0.3	2.2

* strands failed during tensioning

** anchorage failed during tensioning

† per cent of ultimate strength

†† apparent strand failure

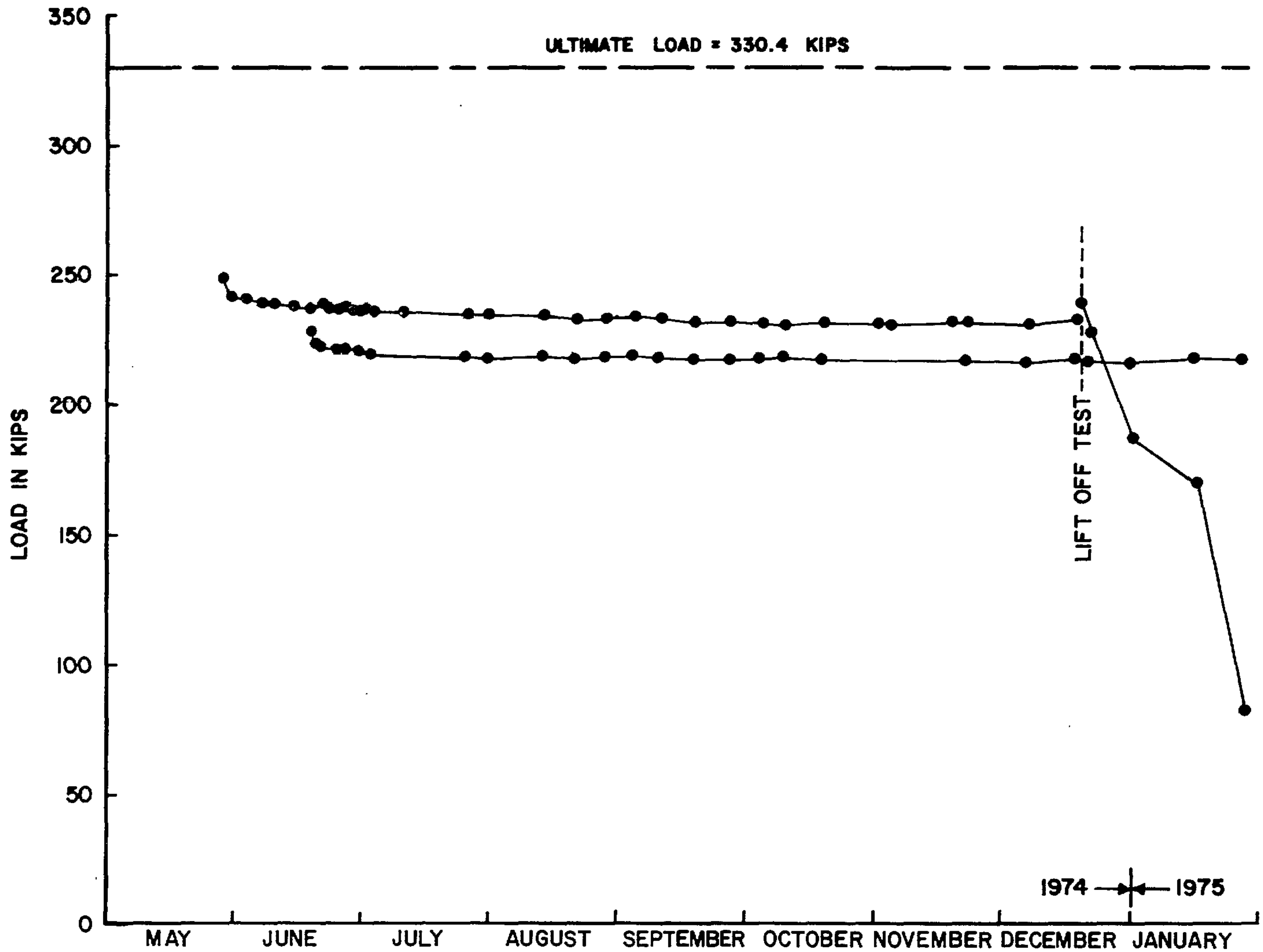


Fig. 29 - Typical load cell plot for monitor anchors on 7680 Bench

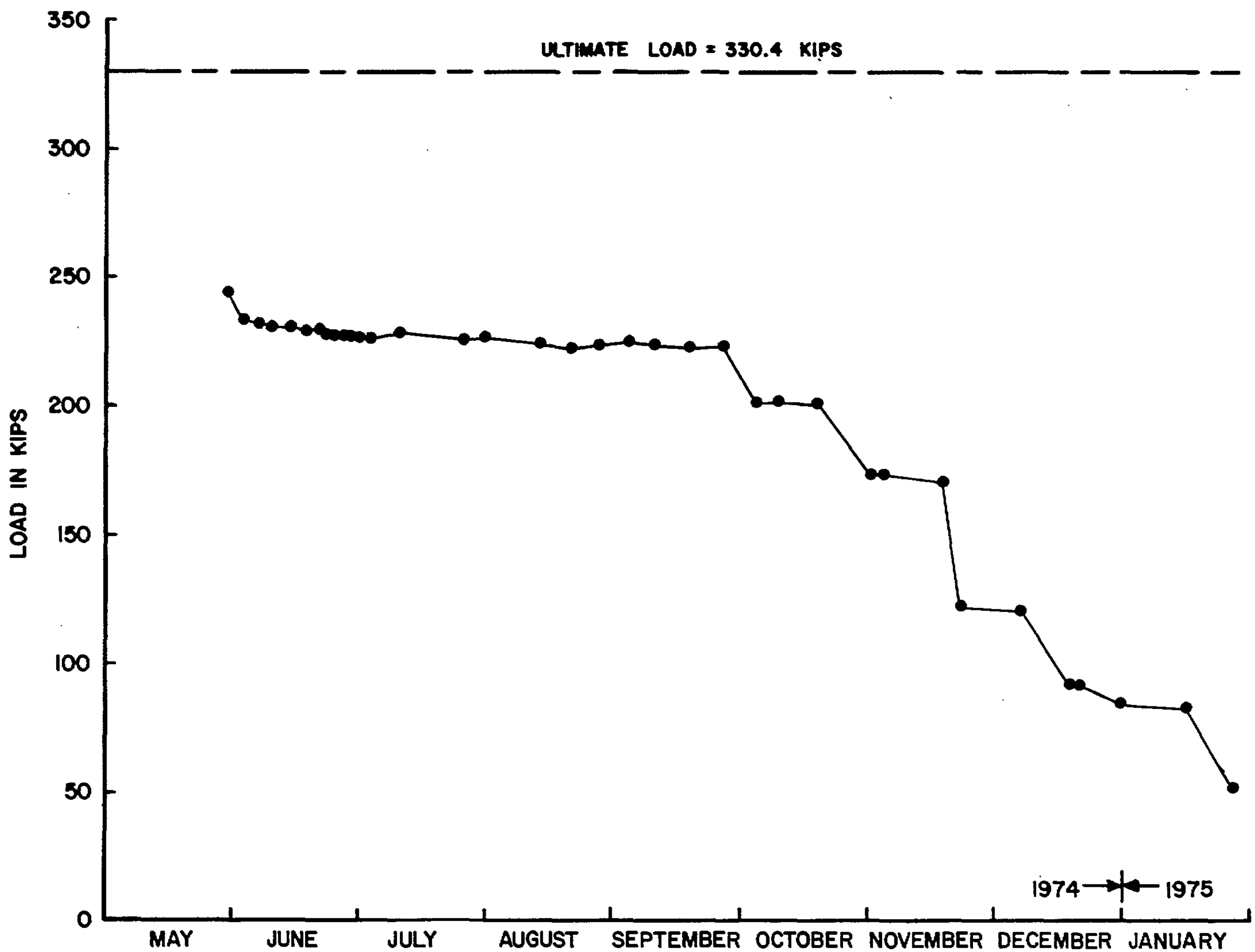


Fig. 30 - Load cell plot showing typical load loss

in the slope to maintain the required stability. During summer rains in 1974, large amounts of water were noted at various locations within the pit and some water seeps were observed on the 7600 Bench. In September, after continuous seeps were observed over a two-to three-month period, a recommendation was made to drill at least two horizontal drains. At that time, the drilling program indicated a wide variation in the amount of water in the holes. A single drain hole was drilled late in September on the 7500 Bench at an incline of +3%. It was located in the zone of highest potential groundwater level based on anchor drilling, but no water was encountered.

When the Agua Zarca - Cutler contact was undercut at the north end of the 7470 Bench water flowed out along the contact at approximately 50 gpm and persisted for several days. As each successive level was stripped and more of the contact exposed, water continued to drain. The undercut contact on the north end of the anchor installation is shown in Fig. 31. Two additional drain holes were drilled on the south end of the 7470 Bench, but these were also dry. When the 7460 Bench was drilled, water flowed from several of the anchor holes and continued for several days. It was concluded that large adverse groundwater pressures had not been present, but that isolated zones of groundwater did exist with localized effect only.

Fig. 31 - Undercut Agua Zarca-Cutler formation showing draining water

DISPLACEMENT

The support design included a recommendation for displacement monitoring, should this be required during the project. Two methods suggested were borehole extensometers and a survey net using a precision distance-measuring instrument. The recommendation to implement this monitoring was made in July 1974, following several small surface failures. It called for monitoring from the west side of the pit with a light ranging instrument, and the installation of two multiple position borehole extensometers.

A distance-measuring program was begun in September 1974 to monitor any possible deep seated movement of the cable bolted slope. A light ranging instrument was used to measure distances from a base station on the west side across the pit to the anchored slope. Reflector pins were placed on load cell blockouts and distances measured monthly.

The instrument was rented by the mining company and readings were made over a 9-month period until April 1975. During that time a variation in readings was observed and it was difficult to draw precise conclusions except that the slope was probably not suffering overall failure. The mining company abandoned the use of a distance meter after April 1975. It is not known if the variation was the result of a faulty instrument, incorrect operational procedures, or improper environmental correction.

On July 13, 1974 it was recommended that two multiple position borehole extensometers (MPBX) be installed on the 7500 Bench when mining reached that level. The extensometers were ordered in September and the first unit was installed on the 7500 bench in mid-October. The second unit could not be installed due to poor surface conditions at the planned location. An alternate location was recommended on the 7600 Bench but the unit was never installed. The existing extensometer was read initially once a week, but at the time of writing is read approximately once a month. Displacement of the extensometer anchors is plotted against time in Fig. 32. The depth from the collar of the hole to each anchor is given in

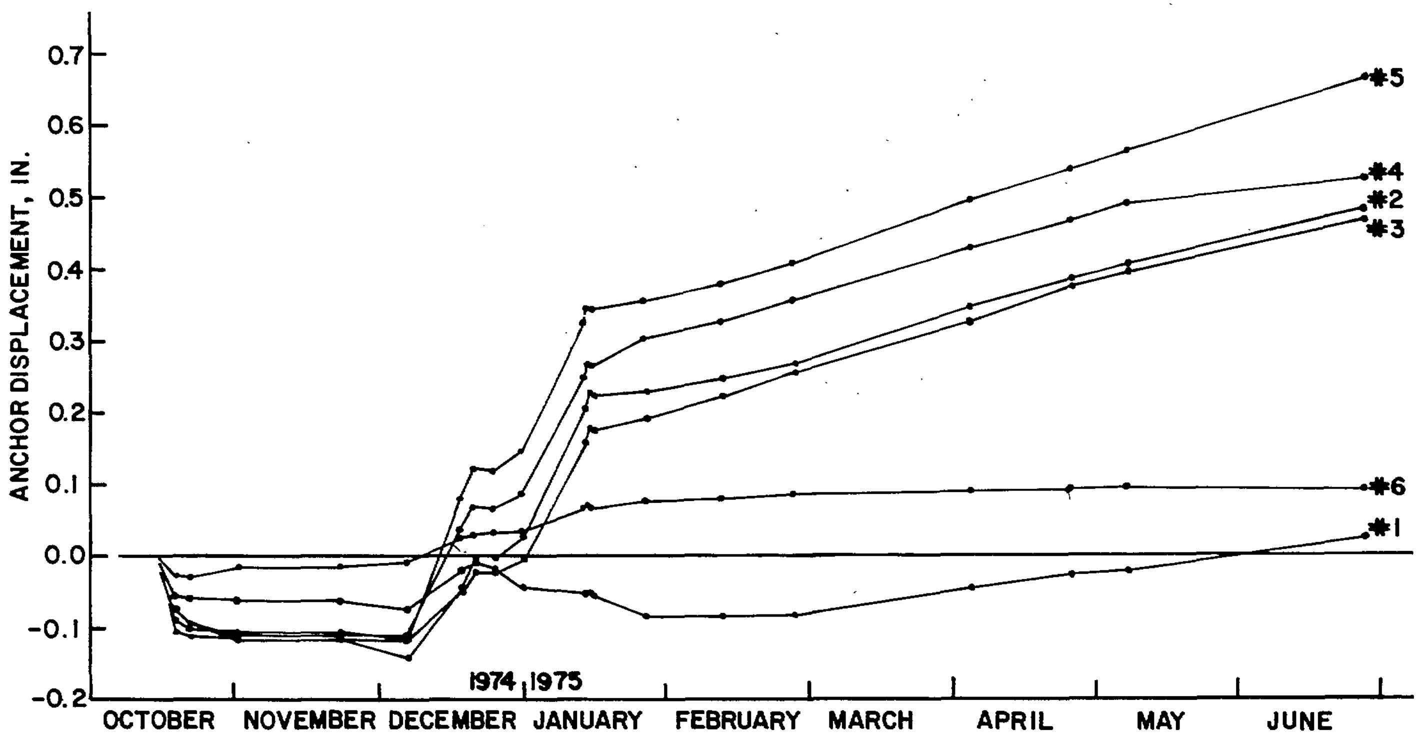


Fig. 32 - Displacement plots for extensometer anchors

Table 12. As shown in Fig. 32, anchors 1 and 6 had moved only slightly. Anchors 2 to 5 showed movement greater than 0.5 in. Any actual movement should have been shown by the deepest anchor, no. 1. It appears therefore that anchor 1 was no longer functioning and should have been disregarded. In this case, Fig. 32 indicates that a movement of approximately 0.5 in. had occurred between anchors 5 and 6 and appeared to be continuing at a slow but steady rate. This possible movement is relatively shallow and occurred between 30 and 100 ft from the collar. The Aqua Zarca - Cutler contact in the extensometer hole was located at a depth of 214 ft from the collar.

Table 12: Extensometer anchor depths

Anchor number	Depth from collar, ft.
1	350
2	300
3	250
4	200
5	100
6	30

REDESIGN

A re-evaluation and redesign was undertaken in September, 1974 at the request of the mining company. An optional mining plan was being considered and they wished to have the redesign completed for both the original and the revised mining plans. The fundamental purpose of the design re-evaluation was to determine if updated design data would indicate fewer anchors being required while retaining the same degree of safety. The report was submitted to the company late that November.

ASSUMPTIONS

Several of the original design assumptions were re-evaluated and updated using the most recent installation data. The most notable changes were in the potential sliding plane angle, the mining plan and effectiveness of the anchors.

The chief potential sliding plane was still assumed to be the Agua Zarca - Cutler contact. However, many clay seams had been encountered during drilling which could act as secondary slip planes if slope movement occurred. Small bench slides had already occurred in some areas of the anchored slope along clay seams.

No new geological structures of major importance had been found between the time of the original design and the redesign, and therefore the original assumptions were retained.

The shear strength properties of the potential failure plane, which were assumed for the original design, were still considered valid. A variation of strength properties was known to exist along the sliding plane. However, no new test work was performed and therefore the same properties were assumed for the redesign calculations. These shear strength properties were:

Sliding friction angle, $\phi = 20°$
Cohesion, $c = 1000$ psf

The original design assumed groundwater forces were not a major factor in the Nacimiento east slope. However, water is known to exist in some areas of the slope as indicated by several seeps in the slope face and by water encountered in anchor drill holes. Groundwater effects were therefore considered in the redesign.

Anchor forces of 60% of ultimate strand strength were assumed for the original anchor design. Although load cell data indicated working forces may have been as high as 70% of ultimate strand strength, the long-term effects of creep were not known and therefore 60% was used for redesign calculations.

During the early portions of the anchor installation, hole angle declinations of 5°, 8°, 10°, 12°, and 15° were drilled to determine the optimum angle for ease of anchor emplacement. The original design calculations assumed an inclination of 8° from horizontal. After placing several units at 8°, difficulties with emplacement suggested a change to 12°, which was thereafter used for new anchors in the redesign calculations.

For the original design it was assumed that 100% of the anchor working force would be distributed across the potential failure plane. Experience with the upper four benches indicated that less than 100% of anchor force was being applied across the potential failure plane. Following evaluation of anchor effectiveness through direct measurement and stressing analysis, the values shown in Table 13 were used for the redesign calculations.

Earthquakes and blasting forces were assumed to have no effect on stability for the initial design and the same assumptions were followed for the redesign calculations.

It was assumed, for purposes of redesign, that the anchors on a particular bench were installed and stressed as mining took place. In practice, mining was usually 1 to 2 benches in advance of anchor completion. This was not believed to present a stability problem but careful monitoring was recommended during the time that mining would undercut the assumed sliding plane below the 7500 Bench.

MINING OPTIONS

Redesign calculations were made for both the original mining plan and for the alternate plan provided by mine management. The basic difference between these plans was in mining of the lower benches directly above the ore zone. For design purposes the basic difference between mining plans was in choosing design zones for Options I and II.

Option I was essentially the same plan as the original design. Shown in Fig. 33 is the plan with the revised design zones. Installation data indicated the potential sliding plane angles for some of the various design zones were probably less than the 28° used for the original design. Following drill hole evaluation of the sliding plane angle, the angles shown in Table 14 were used for the respective design zones shown in Fig. 33.

Option II differed from Option I in that it basically removed the potentially unstable material of Zones III and III X as shown in Fig. 34. Consequently, most of the anchors for Zones III and III X would not be required. All other assumptions for Options I and II were the same and the basic method of analysis was also the same. The design zones were modified to conform with the revised mining plan as shown in Fig. 34.

RESULTS OF REDESIGN

The results of redesign, using the same analysis as the original, are summarized for option I in Table 15. If all anchors were 100% effective, only 312 units would have resulted in the same safety factors. If all 355 anchors were 100% effective, the safety factor would be approximately 1.162. Zones II A, II B, and III X were designed with a safety factor of 1.0 because it was felt that if Zones I and III remained stable the other zones could not fail. Zone III X has a calculated safety factor of 1.042; however this was decreased to 1.0 to account for potential loading from Zones II A and II B.

For Zones I, a hydrostatic head of approximately 7.8 ft must act on the entire sliding plane to result in a calculated safety factor of 1.0. It was considered unlikely that such a head would occur; however, isolated areas could have a hydrostatic head exceeding 7.8 ft and localized failures could take place.

Minor changes were recommended for anchor spacing based on the variation of design zones. Mining of Zone I was essentially completed, making changes impractical. As a result of lower-than-desired effectiveness for 19-strand units, no additional 19-strand anchors were recommended. This size of unit would have to be placed in a larger and steeper hole, free of debris, if its effectiveness were to be increased. It did not seem practical to attempt these modifications for the small increase in anchor force which a 19-strand unit could give compared with a 16-strand unit. Because the largest anchor size would be 16 strands, no increase in hole size above the previously used 4-1/2 in. diameter was made. However, a minimum overall anchor length of 70 ft (30 ft stressing and 40 ft anchorage) was recommended.

Table 13: Anchor effectiveness: upper benches

Anchor size	Number of anchors	Number of equivalent 100% effective	Average effectiveness
8 strand	21	19.75	94%
16 strand	37	32.50	88%
19 strand	65	54.25	83%

Table 14: Potential sliding plane angles

Design zone	Assumed sliding plane angle
I	26°
II A	24°
II B	29°
III X	22°
III	33°

Table 15: Number of anchors and safety factors for mining option I

Design Zone	Number of anchors	Safety factor
I	228	1.031
II A	25	1.000
II B	46	1.000
III X	22	1.000
III	34	1.042
Entire slope	355	1.021

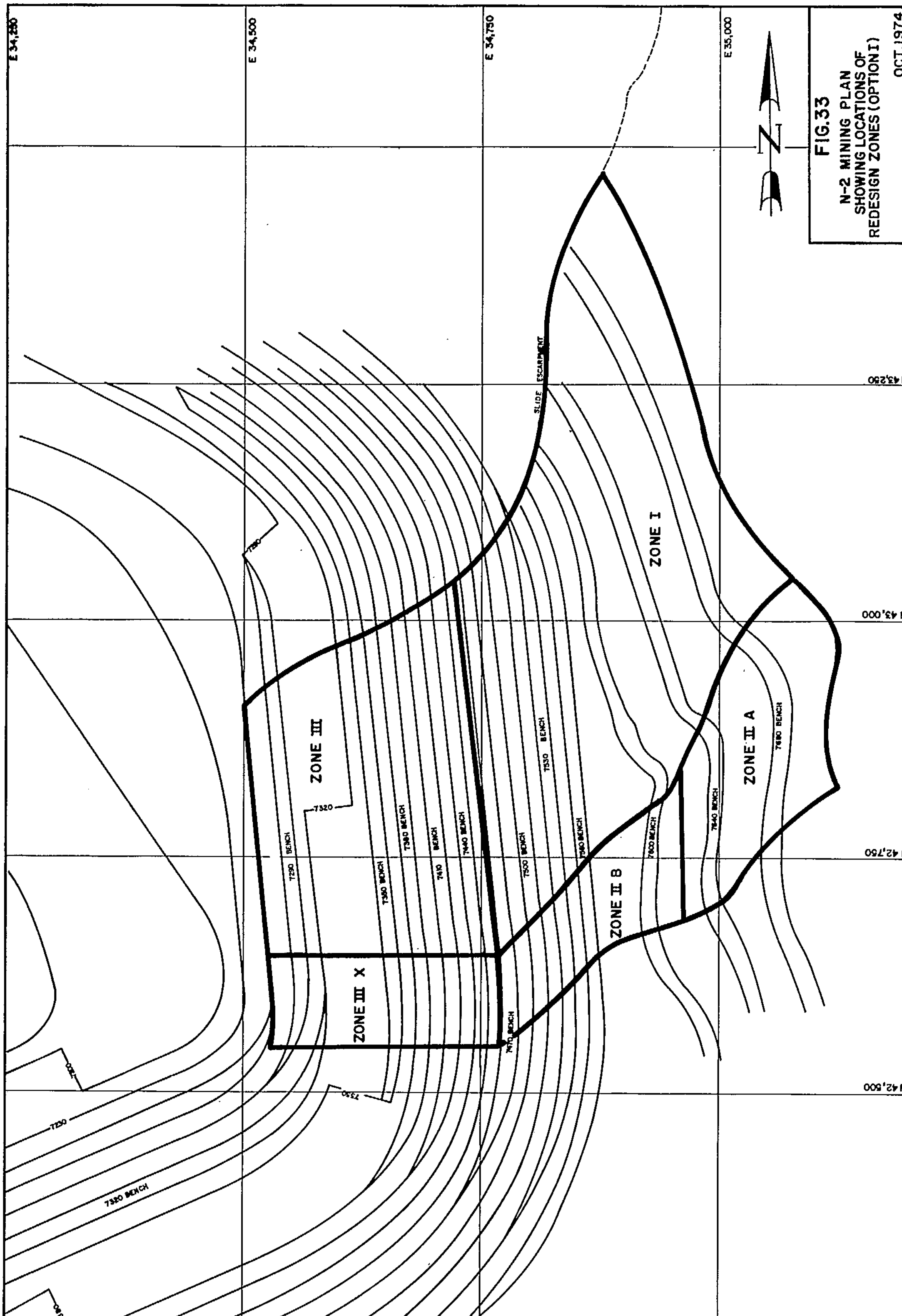


Fig. 33 - N-2 mining plan showing locations of redesign Zones (option I)

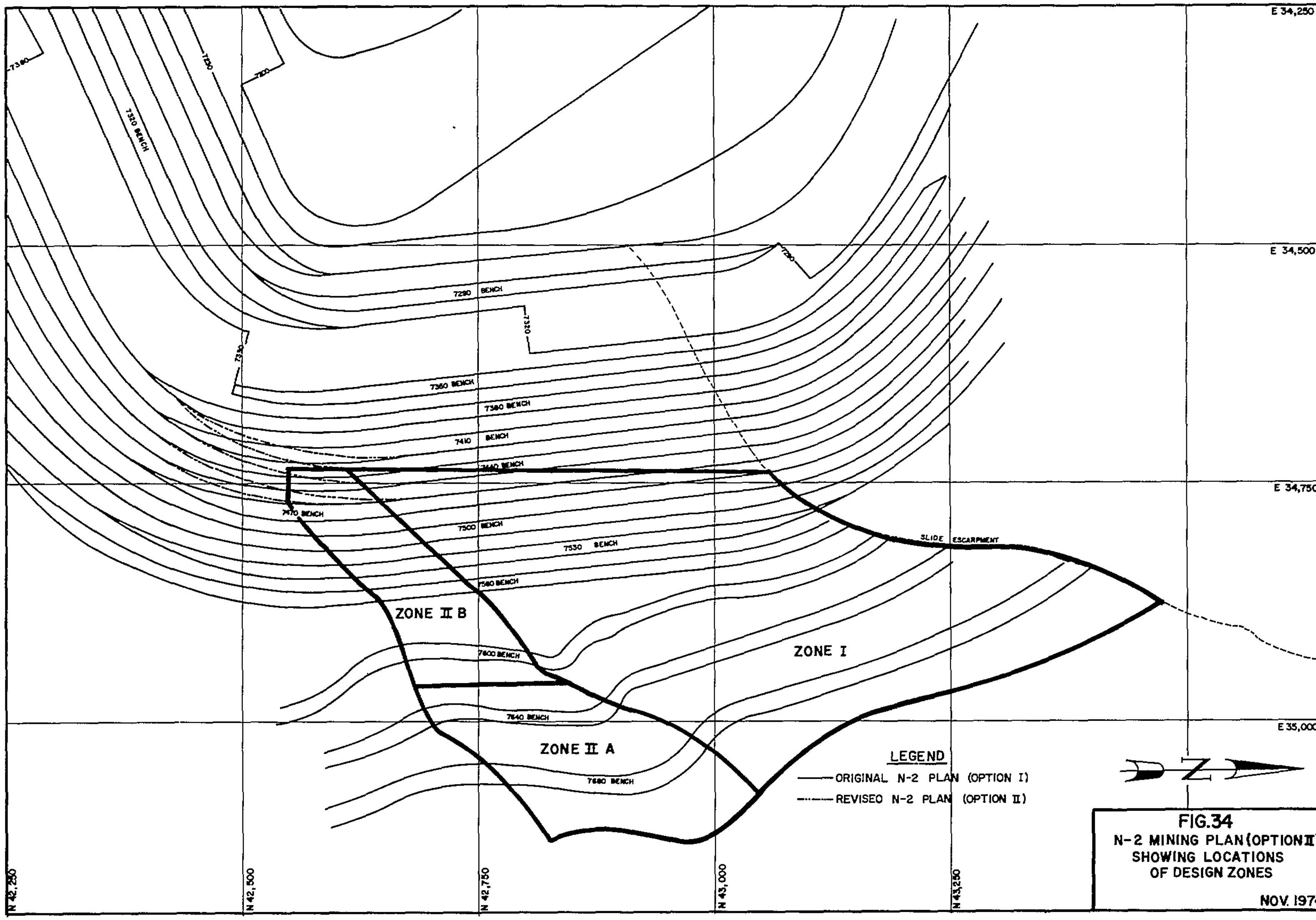


Fig. 34 - N-2 mining plan (option II) showing locations of design Zones

In Option II, the removal of Zones III and III X created a situation whereby Zones I, II A and II B had to support themselves. In Option I, Zones II A and II B both had an excess driving force indicating that individually each would probably have slipped. However, safety factors for the remaining zones were such that loading from II A and II B would still have resulted in an overall safety factor greater than 1.0. By removing material from Zones III and III X for Option II, Zone I would have had to carry all of the excess driving force from Zones II A and II B for the entire slope to have remained stable. However, Zone I could not have accepted all of the loading because it did not buttress the lower-most portion of Zone II B and, therefore, the lower portion of Zone II B must have had to support itself. Using the same sliding plane angles and other basic assumptions from Option I resulted in the safety factor for Zone I being reduced to 1.002. This was based on a conservative 29° sliding plane angle for Zone II B. If the angle had been 27°, a safety factor of 1.020 would have resulted for Zone I with no addition of anchors. However, assuming a 27° sliding plane angle and the addition of ten 16-strand anchors yielded a safety factor of 1.025 for Zone I.

The total number of anchors recommended for Option II was 334. Spacing and number of anchors in each zone is outlined in Table 16.

REDESIGN RECOMMENDATIONS

Due to cracks in many of the blockouts, recommendations were made for additional steel reinforcement and to change from portland type 2 to type 3 cement. In addition, it was recommended that greater care be taken in selecting drill hole locations to avoid blockouts bearing on clay or other weak material. Some blockouts had failed during tensioning because they were bearing on both solid rock and clay material.

To provide a double check on anchor effectiveness it was recommended that the length of all anchors be measured directly before stressing. Even though the stressing analysis data had proved to be quite accurate, it was felt the low projected effectiveness of some anchors justified the double check. The recommendation was re-emphasized that each anchor head should be coated with a non-oxide grease as soon as possible after stressing to prevent corrosion of the anchor head and wedges. In addition to greasing, each anchor was to have a cap placed over the head and wedges. This was a safety precaution against anchor failure and release of energy at the head in the form of flying wedges and cable.

The initial design report recommended re-stressing of tendons after the completion of all anchors and prior to secondary grouting. This process could possibly add as much as 5% to the effective anchor forces. However, after the redesign was completed it was felt the added cost of re-stressing and the difficulty of doing this in the field were not justified. The recommendation for re-stressing was withdrawn.

At the time of the redesign, displacement monitoring was being carried out with three independent systems: load cells, distance meter monitoring, and a 6-point borehole extensometer. This monitoring was believed adequate if done at the intervals specified and no additional recommendations were therefore made.

As a result of water seeps in the face and water encountered in drill holes, a recommendation was made to closely observe and record groundwater conditions. The possibility of a dewatering program with horizontal drains was re-emphasized.

PROJECT DIFFICULTIES

Several unexpected problems occurred at various stages of the project. These arose in part from the inevitable unpredictability of geotechnical materials, and also in part from the relatively new concepts involved in what was the first major open pit support installation. Difficulties were also caused by the need to design and install support in haste to meet mining exigencies.

Due to the urgency of the need for slope stability, the design had to be completed with whatever data was available. The installation of

Table 16: Number and spacing of cable anchors in each zone for mining option II

		Number of Units		
Bench	Spacing, feet	Zone I	Zone II A	Zone II B
7680	15/30	27	9	-
7640	10/12.5	27	16	-
7600	10/12.5	33	-	11
7560	10	29	-	9
7530	10	38	-	10
7500	10	34	-	7
7490	30	14	-	3
7480	30	13	-	3
7470	30	13	-	3
7460	30	8	-	3
7450	30	7	-	2
7440	30	5	-	-
7430-7440	Variable	-	-	10
	Total	248	25	61

anchors had to be started as soon as possible to keep the mining and milling operations going continuously. Therefore, the initial design had to allow for re-evaluation of the various design parameters as the project proceeded and data became available. Apart from the consequent design difficulties, some problems of note occurred during and after anchor installation.

INSTALLATION DIFFICULTIES

Access to the work site was occasionally interrupted, but seldom for more than an hour. The major access problem encoutered was due primarily to adverse weather conditions. Rain and snow made it impossible at various times to reach the job site. This was largely due to the extreme clayey nature of the various geologic formations for which no remedial action was possible.

During drilling, the major problem was the loss of drilling fluid due to open cracks and hydraulic connections to adjacent holes. Either open cracks existed between some holes or the holes actually intersected. In some cases drill cuttings or drilling fluids were observed coming out of an adjacent hole. Circulation problems were solved to a large degree by using a detergent with the down-the-hole hammer drill. Water and detergent were mixed and pumped into the drill rods. The air was then turned on forcing the mixture through the hammer. The return at the hole collar was a combination of air, water, cuttings and copious amounts of foam. Cleaning holes which intersected clay zones and open cracks was extremely difficult, and in many instances almost impossible.

Many problems were encountered during emplacement. In some instances anchors could not be inserted and holes had to be redrilled or flushed. In other cases up to 15 ft had to be cut off the ends of anchors because they could not be forced further into the hole. Remedial action included steepening the anchor holes, inserting units as soon after drilling as possible and eliminating 19-strand units. Emplacement problems on the upper benches were probably also due in part to the length of the anchor.

Relatively few problems were encountered during primary grouting. The standardization of hole size at 4.5 in. resulted in a change in the amount of grout used for 8-strand anchors. In some instances anchorage grout was lost through cracks and these units required re-grouting to obtain the desired anchor length.

During winter, blockout construction was made difficult by freezing temperatures. Gas heaters kept temperatures above freezing until the blockout concrete had adequately cured.

Many of the problems encountered during the installation occurred during tensioning. These were generally associated with a small or confined working area, blockout failure during tensioning, or anchorage sections slipping. The steepness of the natural slope resulted in limited working space on upper benches. Limited working space on lower benches was caused by proximity of the mining operation, which was only one or two benches ahead of anchor installation. In a few cases limited working space was the result of partial bench slides. Blockout failure was usually caused by the blockout being placed partially in or across clay seams which had much less bearing capacity than the Agua Zarca Sandstone. This was largely solved by taking greater care in locating hole collars and by adding more rebar to the blockout in areas of excess clay seams. The cause of anchorage slipping was not completely determined. It was believed due either to dirty holes or to loss of grout through cracks during primary grouting. Remedial action usually involved re-grouting and tensioning at a later date. In some instances, additional holes were drilled and anchors placed.

A minor problem was encountered during secondary grouting. Several anchors on the upper benches required more than 50 bags of grout to fill the hole. Much of this grout passed out of the hole into cracks in the surrounding rock. This problem was solved on the lower benches by using a thicker grout.

MONITORING DIFFICULTIES

Problems with the displacement monitoring program arose primarily with the distance measuring device. The variation in readings from one month to another was excessive and unacceptable; the mining company abandoned this program before an adequate solution was found. The only other major problem with the monitoring program was the loss of load on many of the load cells. It was believed initially that the loss was due to anchorage deterioration or slippage, but was later found to be due to strand failure from stress corrosion.

STRESS CORROSION

The problem of anchor load loss was first observed in late September 1974. Initial examination of anchors indicated that the units were maintaining a reduced load. However, after further load losses, individual strands became loose in the anchor head. Examination of the failed strands indicated that failure was of a brittle nature and only in a relatively few instances was a typical necking and tensile break observed.

Strength tests with similar but new and unused cable were conducted by an independent metallurgical laboratory. These tests indicated that the unused cable had a strength of approximately 42,000 lb and met the ASTM specifications for A 416 grade cable. No evidence of excessive inclusions, wire damage, carburization, decarburization of undesirable structures were found. The same metallurgical laboratory also examined various corroded cable strands in actual anchors which had failed. These strands showed evidence of transverse and longitudinal surface cracking in areas where corrosion was present. It was concluded initially that the failures were the result of stress corrosion. Further study of the failed strands was made by another metallurgical consultant who corroborated the initial findings. In part, his study included the following:

"Brittle failure of prestressing steel due either to hydrogen embrittlement or to stress corrosion has been of concern to prestressed concrete technologists and corrosion engineers for several years. In laboratory situations stress corrosion cracking has been demonstrated with prestressing steel in nitrates and chlorides. Hydrogen embrittlement has similarly been observed with wires exposed to H_2S solutions and where prestressing steel has been galvanically coupled to a more active metal (e.g. aluminum) in field installations. Certain ions (i.e., sulphide and arsenide) act as surface catalysts or poisons and cause rapid penetration of hydrogen into the steel. The reported failure of prestressing steel in chloride solutions is thought by some investigators to be the result of minute amounts of sulphides in the corrosive environment acting to cause cathodic charging of hydrogen into the steel during the chloride corrosion process.

"Failure of prestressing steel by stress corrosion or by hydrogen embrittlement will occur over a wide range of applied stresses. Although there may be a threshold stress level below which failure will not occur, this threshold is a function of the concentration of the specific ions of the corrodent. Consequently, if conditions exist in the environment around the cable bolts which will promote stress corrosion or hydrogen embrittlement, failure would be expected to occur at any significant applied stress level.

"Protection of prestressing steel against corrosion, stress corrosion, and some occurrences of hydrogen embrittlement is easily achieved by surrounding the steel with a high pH environment such as that of a saturated $Ca(OH)_2$ solution, portland-cement grout, or portland-cement concrete. In an environment of pH greater than 10.0, steel is passive and no corrosion occurs. Also, with environments greater than pH 8.0, hydrogen embrittlement from H_2S exposure is precluded. It should be emphasized, however, that hydrogen embrittlement has occurred in prestressing steel encased in concrete or cement-grout when the steel was in contact with aluminum. Complete corrosion protection is afforded by the high pH environment if no such galvanic couples occur.

"Corrosion inhibitors could be introduced to the drilled holes in which the rock anchors are placed. However, the inhibitor would have to be introduced in aqueous solution. The best inhibitor for this situation is probably a lime water solution [sat. $Ca(OH)_2$]. Pumping lime water around the rock anchors would result in penetration of the solution into all possible cracks, pits, crevices, and spaces between wires and cables. Passivation of the steel would result and the only possible condition of continued corrosion would be at the bottom of pits and in cracks where the corrosion products would present a barrier to the lime water. Additional corrosion inhibitors such as $NaNO_2$ could be added to the lime water, but they would not result in additional corrosion protection in those areas just mentioned if they too were prevented from reaching the metal surface by the corrosion products.

"Corrosion of steel in a neutral or alkaline medium is controlled by the cathodic reaction: $1/2\ O_2+H_2O+2e^- \rightarrow 2(OH)^-$. If oxygen can be excluded, then corrosion will cease. Grouting of the cable bolts with a portland-cement grout will therefore result in passivation of the steel (pH = 12 or greater), and, also, will prevent oxygen from reaching the steel surface.

"It is concluded that the wire failures which have occurred in the rock anchors are attributable to stress corrosion, corrosion assisted stress cracking, or hydrogen embrittlement. The specific species of the environment which is most directly responsible for the brittle cracking failures is not known. However, the fact that aggressive corrosion is occurring (as evidenced by surface pitting of the wires) indicates that the conditions exist for any of these corrosion mechanisms.

"Immediate action to eliminate the corrosion attack is therefore necessary. The recommended procedure to achieve the maximum protection possible is to first flush the installed cable bolts with saturated lime water or lime slurry. Immediately after the flushing, the cable bolts should be grouted. Any additives to the Portland-cement grout should be controlled on the basis of their effect on pH of the grout mixture (pH > 10 must be maintained, with pH > 12 desirable)."

The mining company proceeded with a program to prevent further stress corrosion in mid-May 1975. This program consisted of flushing all anchors not already grouted with a 1% lime solution. The regular anchors were then secondary grouted. A thick lime paste was pumped down the holes of the monitor anchors. The success of these actions cannot be assessed at the time of writing, except to say that no further anchor detensioning or anchor failure due to stress corrosion has been observed.

SHALLOW SURFACE INSTABILITIES

Several shallow surface instabilities occurred during the course of the anchor installation. Some were minor and presented no major problems. Others were disruptive and resulted in loss of several incompleted anchors.

The first troublesome displacement occurred in June 1974 and involved the upper three benches. Displacement was up to about 15 ft, and sliding occurred along a 2- to 3-ft thick clay seam which had been undercut on the 7560 Bench. Several of the units on the 7600 Bench where the instability occurred had not been tensioned.

Small surface movements continued throughout the project but no major problems occurred again until mid-April 1975. At that time, a series of small movements involved approximately the entire anchor installation. This instability was controlled in part by the stress corrosion of many anchors in the upper three benches, and also involved the same area that had slipped in June 1974. In September 1975, there were still several anchors at the top and bottom of the unstable zone which appeared to be maintaining load. Field examination of the installation in mid-August 1975 indicated that the slip plane of these near surface instabilities was at a depth of between 30 and 50 ft. Although the extensometer was not in the unstable zone, it was directly adjacent and indicated slight movement at approximately the same depth as the slip plane. Inspection also indicated that the remaining anchors were maintaining load.

CONCLUSIONS

This case history has presented the steps taken in a pit slope stabilization project and given the reasoning behind those steps. Many lessons were learned as the project progressed and it is believed that the state of the art has been advanced. A summary of pertinent conclusions concerning the project and its problems is as follows:

a. It is essential from a design standpoint to have reliable input data. Although an ongoing re-evaluation of the original design is desirable, if not essential, a complete re-design might be avoided if the time were taken at the outset to obtain reliable input data. Specific details where emphasis should be placed include: location of a potential sliding plane, strength properties along a potential sliding plane, and groundwater conditions affecting a potential sliding plane.
b. The importance of timing of the various phases and close attention to the sequence of the phases cannot be overemphasized. There must be complete and close coordination between all parties involved in any such stabilization project. In general, it would be best to have a single contractor for all phases of the project, including drilling.
c. Installation of anchors as mining progresses can provide a better and safer working area for drilling and stressing personnel. Depending on the particular mining method, it might also provide an opportunity to vary the vertical spacing of anchors and thus possibly eliminate or reduce the problem of small surface instabilities. Immediate insertion of the anchor upon completion of drilling, followed by primary grouting, is strongly recommended to improve overall anchor effectiveness. The optimum hole angle, which is dictated by ease of placement, should be determined in the field by actual installation trials. After the angle is determined, a re-evaluation of the number of anchors required to provide the desired stability may be necessary.
d. As demonstrated by the rapidly occurring stress corrosion encountered in this project, it is of paramount importance to protect the cable. Perhaps the first step should be to have a detailed study made of the chemical environment to which the tendons will be subjected. From such a study it may be possible to ascertain potential stress corrosion or other harmful effects on the cable. In any case steps should be taken to avoid dragging the cables through mud, soil or water prior to installation. The use of high pH solutions such as lime water may be useful during drilling and hole cleaning. Secondary grouting of the exposed cables should be given high priority and performed, if at all possible, immediately after stressing. The use of greased or plastic-wrapped cable should be considered for all monitor anchors, and possibly even for regular anchors.

The ultimate success of the Nacimiento Project cannot be appraised at this time. At a future date when mining of the supported area is finished, a complete appraisal from both stability and economic standpoints may be made. However, certain aspects of the project and its field performance are now evident. These may be summarized as follows:

a. Mining has in part undercut the potential sliding plane and no movement has yet been observed across the plane. In other words, the project may have been successful in preventing overall slope instability. Other possibilities are that there may still remain enough buttressing from the unmined material to keep the slope stable and that the slope would have remained stable even if no anchors had been placed. The latter is believed unlikely.
b. Prevention of near surface instabilities due to slope undercutting was unsuccessful in the early stages of the project because it was not anticipated that such instabilities would occur

as they did, and because delay in stressing anchors allowed such movement to take place. Preventing such instabilities was more successful in later stages of the project first, because they were anticipated, second, because a re-arrangement of anchor spacing was undertaken, and third, because anchor stressing was given a higher priority. Nevertheless, some near surface instabilities could not be prevented in later stages of the project because stress corrosion had reduced the effectiveness of the anchors to such an extent that the force available for stabilization was insufficient.

c. The stress corrosion potential on the cables was originally underestimated. Remedial action to combat the problem has been at least partially successful in that no stress corrosion failures have subsequently been observed.

The results of the Project have contributed significantly to the promise of successful future stabilization projects at Nacimiento and other open pit mines. Any application of new technology under field conditions may be expected to encounter unforeseen difficulties. The solution of these problems lays the groundwork for the success of future undertakings. As the state of the art improves, future open pit support project can be expected to become commonplace in achieving greater mining efficiency.

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APPENDIX A
CHRONOLOGICAL SUMMARY

June 1971	Mining began.
Early 1972	First slope instability occurred.
Mid-January 1973	Tension cracks observed above slope.
April 1973	Tension cracks continued to widen.
July 17, 1973	Large tension crack noticed.
July 29, 1973	Second major slide occurred (approximately 400,000 cubic yards of waste)
August 1973	Displacement monitoring program began.
October 4, 1973	Authorization given to begin initial stability assessment.
October 8-28, 1973	Field studies for initial assessment of cable anchoring began.
October 12, 1973	Displacement of July 29 slide had slowed to about 0.004 in. per day.
November 2, 1973	Authorization given to proceed with testing of mechanical properties.
November 9, 1973	Initial assessment report submitted.
November 1973	Sample collection and testing for mechanical properties began.
Early December 1973	Preliminary estimate made of number of anchors to stabilize slope.
Late December 1973	Decision made to proceed with anchoring program.
February 1, 1973	Initial deadline for mechanical properties report.
February 1974	Drilling completed for mechancial properties testing.
March 1, 1974	Tentative starting date for anchor installation.
Late March 1974	Mechanical properties testing completed.
March 30, 1974	First drillhole for anchor installation completed.
Early April 1974	Anchor installation began.
April 22, 1974	Mechanical properties report submitted.
May 15, 1974	Anchor design report submitted.
June 1974	First bi-monthly engineering report submitted.
June 1974	First troublesome surface displacement in anchored slope.
July 31, 1974	Recommendation made for additional monitoring techniques.
August 10, 1974	Second bi-monthly project engineering report.
September 1974	Recommendation for two horizontal drains on the 7500 Bench.
September 1974	Redesign began.
Late September 1974	Initial load loss on load cell on 7680 Bench (first indication of stress corrosion problem).
October 11, 1974	Third bi-monthly project engineering report submitted.
Mid-October 1974	Installation of extensometer on the 7500 Bench.
November 26, 1974	Redesign report submitted.
Early December 1974	Slip plane undercut on 7470 Bench.
Mid-December 1974	Slight movement indicated by extensometer.
December 1974	Lift off tests made to determine extent of load losses and realization of stress corrosion problem.
December 1974	Fourth bi-monthly project engineering report submitted.

January 13, 1975	Preliminary evaluation made of stress corrosion problem.
February 1, 1975	Mine closed due to depressed copper market.
April 10, 1975	Detailed report with recommended remedial action for stress corrosion problem submitted.
Mid-April 1975	Surface failure of one third to one half of the anchored slope.
April 25, 1975	Update of displacement monitoring data.
Mid-May 1975	Anchor holes flushed with 1% lime solution to stop or slow stress corrosion failure.
May 1975	All accessible regular anchors secondary-grouted.

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