DRILLED SHAFT CONSTRUCTION AT CROWNPOINT, NEW MEXICO

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ABSTRACT

The Wyoming Mineral-Conoco Crownpoint Project represents the first time that big hole drilling has been exclusively used to develop totally a privately financed mine below a depth of 1000 feet.

Three shafts, one ten feet in diameter and two six feet in diameter, were successfully drilled to depths of 2243', 2188' and 2188' respectively and cased with hydrostatic designed steel casing. The largest shaft is to be used to handle muck and water from station excavation and enlargement of the other two shafts.

A reverse circulation system with potassium chloride base mud was used for drilling. The mud was cleaned by settling in steel lined tanks with assistance by cyclone type separators. Deviation was monitored by gyroscopic surveying at 30 foot intervals and did not exceed 16 inches off plumb. Casing was designed to withstand hydrostatic pressures with a 1.5 safety factor to the large shaft and 1.25 safety factor for the smaller shafts. All casing welds were x-ray tested prior to lowering the casing in the hole. Cementing the annular space completed the operation.

The total time to mobilize, drill and move off the three shafts was 363 days. The drilling operation was completed ahead of schedule and under budget.

INTRODUCTION

The Crownpoint Project is a joint venture between Wyoming Mineral Corporation and Conoco Inc. The operator of the Crownpoint Project is Conoco Inc. Reference hereafter to the Crownpoint Project is intended to mean the joint venture in this paper.

The Crownpoint Project is located in the Grants Mineral Belt approximately 60 miles northwest of Grants, New Mexico, in Section 24, T17N, R13W, McKinley County, about 1/2 mile west of townsite of Crownpoint, New Mexico.

PROJECT OBJECTIVES

The objective of the Crownpoint Project is to develop an underground uranium mine in the Westwater formation at an approximate depth of 2180 feet below the surface.

The plan requires that one hole be used as a work shaft to enlarge the other two holes which will become the main production shaft and the ventilation shaft.

Shaft No. 1 has been completed in a 120-inch diameter hole drilled to a depth of 2243 feet and cased with 85-inch I.D. steel casing. This hole will become the work or development shaft.

Shaft No. 2 has been completed in a 72-inch diameter hole drilled to a depth of 2188 feet and cased with 36-inch steel casing. This hole will be enlarged to a finished diameter of 18-feet to become the main production shaft.

Shaft No. 3 has been completed in a 72-inch diameter hole drilled to a depth of 2188 feet and cased with 36-inch steel casing. This hole will be enlarged to a finished diameter of 18-feet to become the ventilation shaft.

The three large diameter shafts drilled 100-feet apart will be joined together at the station level at about 2180-feet. All muck and water encountered in enlarging Shafts No. 2 and 3 will fall downward through the 36-inch steel casing to the station level and thereafter be removed through Shaft No. 1 back to the surface for disposal.

Figure No. 1 illustrates the Crownpoint drilled shaft development concept.

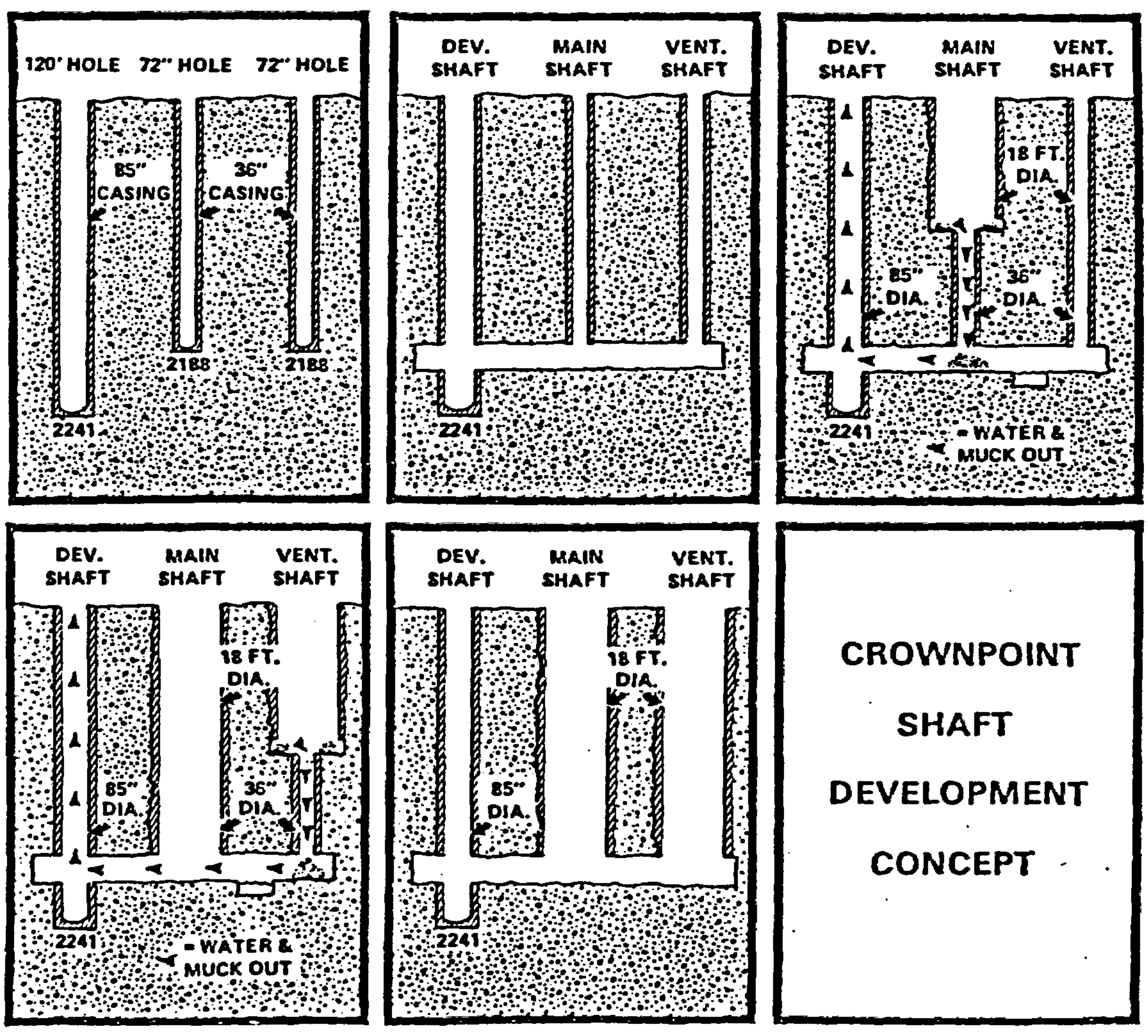


Figure No. 1

DRILLING MANAGEMENT

The drilling supervision team was provided through Conoco's Production Engineering Services, (P.E.S.), Houston, Texas. See Figure No. 2 for Organization chart. P.E.S. furnished all the necessary technical and supervision drilling personnel to engineer and direct all the large diameter drilling activities. The P.E.S. effort was coordinated with the Crownpoint management staff.

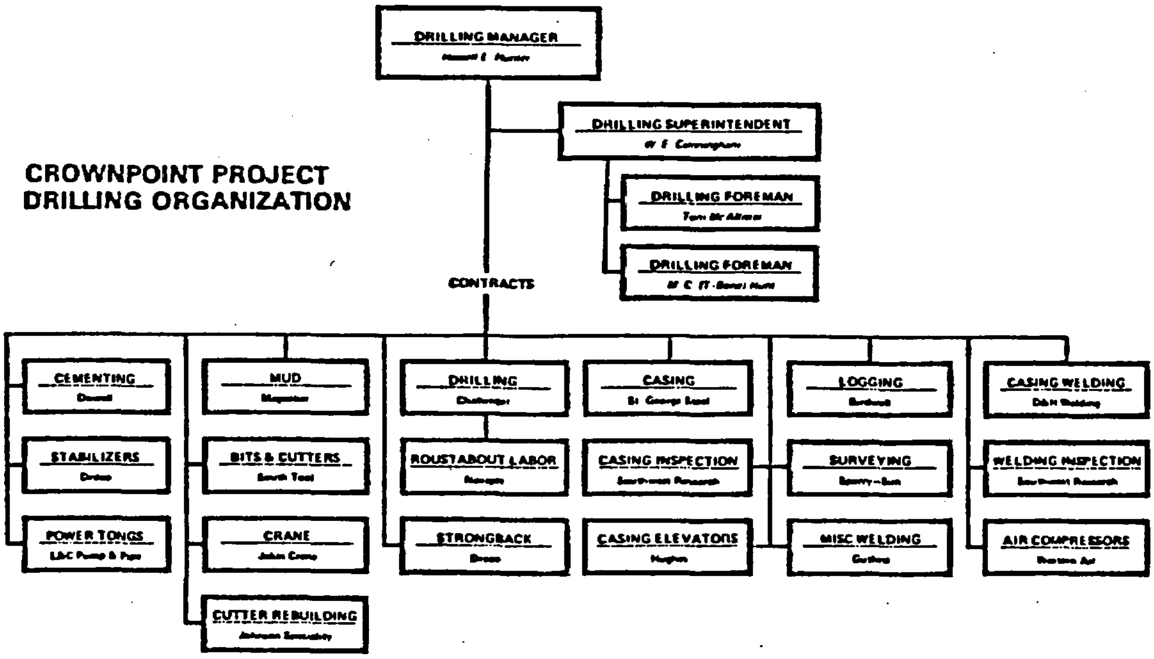


Figure No. 2

The drilling management and supervision team consisted of four persons. Those persons by functional title and name are as follows:

1. Drilling Manager - Hassell E. Hunter
2. Drilling Superintendent - W. E. Cunningham
3. Drilling Foreman - Thomas A. McAllister
4. Drilling Foreman - Marion C. Hunt

The Drilling Manager had overall responsibility for engineering, planning and direction of the drilling project. The Drilling Superintendent was specifically responsible for the physical drilling operations at the drill site. The Drilling Foreman directed the daily drilling operation.

One or more of the above four persons were on location at all times. The drilling operation was supervised by P.E.S. around the clock, (24 hours a day/seven days a week) for the duration of the Crownpoint drilling.

The Crownpoint Project was organized to be executed in a manner quite similar to other Conoco oil field drilling operations. The drilling operation was planned and directed by Conoco drilling personnel and the drilling plan was executed through several coordinated contractors working in harmony with each other. Figure No. 2 identifies the function performed by each contractor. All contracts were prime contracts to Conoco. There were no subcontractors. All risks of successful completion were borne by Conoco through total direction of the drilling operations.

DRILLING SHAFT NO. 1

Shaft No. 1 commenced mobilization on April 1, 1980. Drilling of a 120 inch diameter hole, began on April 13, 1980 and reached a total depth of 2243' RKB in 129 days ending August 20, 1980.

Upon reaching total depth, a caliper log was run and thereafter 85-inch ring stiffened casing was run to a setting depth of 2194 ground level measurement and cemented back of surface. The casing was pumped and bailed dry to complete the drilling and casing operation. Shaft No. 1 was completed in 183 days.

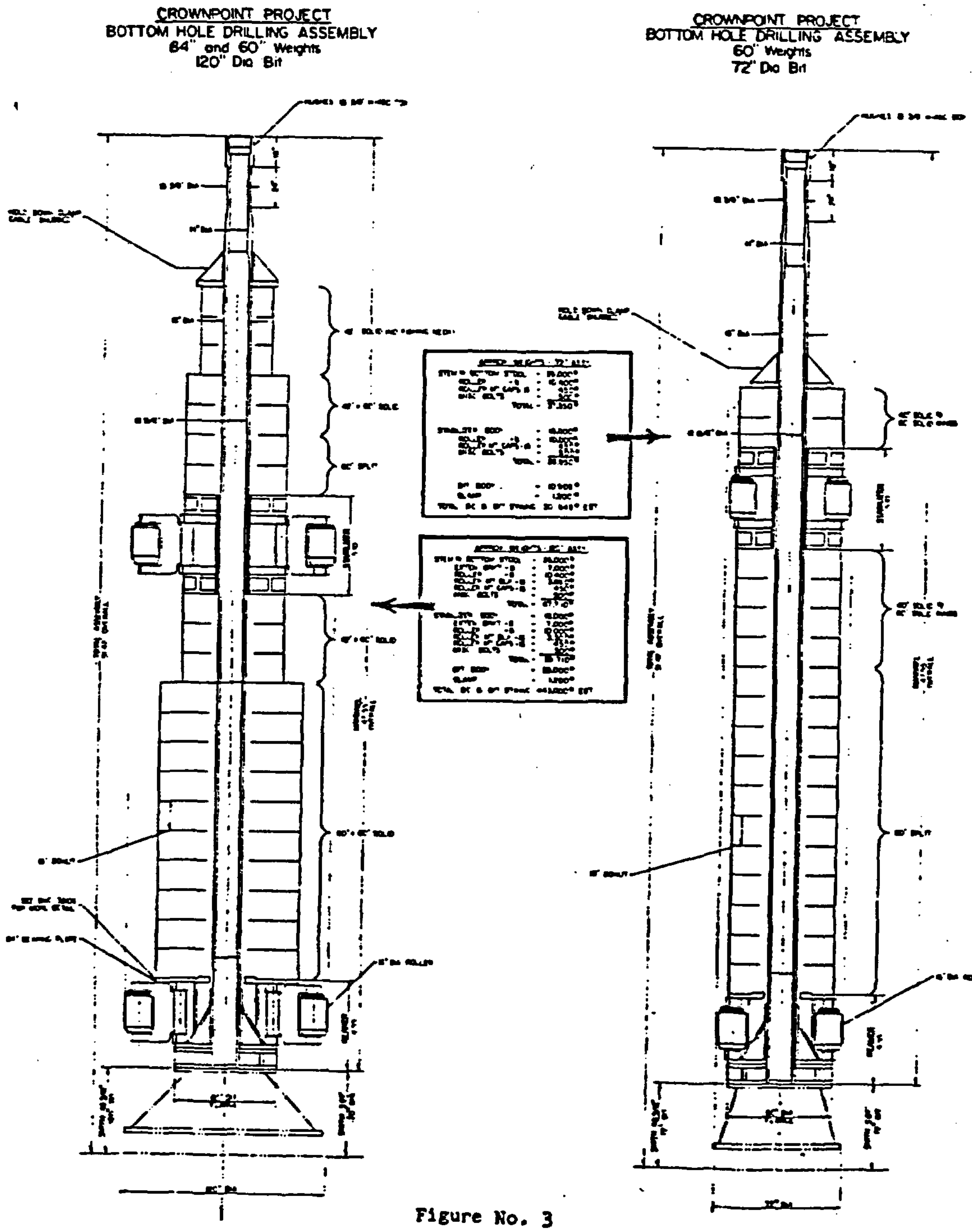


Figure No. 3

Drilling Equipment

The basic rig consisted of a 112 ft. mast with a hook load capacity of 1,000,000 lbs. The substructure was 6-1/2 ft. high with sliding rotary beams that enable the bit and bottom hole drilling assembly to be pulled through the substructure into the mast.

The 120 inch bottom hole drilling assembly was designed to accomplish two basic purposes. To apply weight to the bit and to keep the hole straight. Both purposes are accomplished by use of an extremely heavy drill collar assembly having a weight in mud of about 360,000 lbs. Figure No. 3 illustrates two different bottom hole assembly arrangements used in the drilling Shaft No. 1 and Shafts 2 & 3.

The prime mover for the drill rig consists of two Caterpillar D-379 turbo-charged diesel engines rated at 615 hoisting horsepower each. These engines drive the drawworks and rotary table through a torque converter and a compound.

The circulation system is activated with three Atlas-Copco rotary compressors rated at 1200 cubic feet per minute at a maximum effective pressure of 290 psi.The drill pipe sections were 13-3/8 inch O.D. by 30 foot long.

Bit Weight & Rotary Speed

Weight on the bit and rotary speed are controlled drilling parameters by the Conoco drilling supervisor. The shale formations are the hardest to drill so the highest weight on the bit will be used with a RPM to run smooth without excessive vibration or bit chatter.

The sand formations drill quite rapidly and usually penetration rates had to be held to 4 to 5 feet per hour to prevent overloading the circulation system with excessive drilled cuttings. The RPM sometimes was also slowed down as a vibration limiting factor.

Hole straightness and penetration rate are controlled by weight on the bit and the speed of rotation. The objective at Crownpoint was to achieve a reasonable penetration rate while keeping the hole as perpendicular as possible. This is accomplished by using a heavy bottom hole assembly and using approximately 30% of the weight of the drill collar as weight on the bit and the remaining 70% acting like a pendulum to keep the hole straight.

Maximum penetration rates and maximum hole straightness are seldom achieved at the same time. In order to achieve maximum penetration rates, most of the weight of the drill collar would be carried on the bit. This would minimize the pendulum effect of the bottom hole assembly and cause the hole departure to increase. The horizontal departure of the hole is usually in an updip direction. The two roller stabilizers help keep the bit headed in the same direction thus minimizing hole departure, but it is still largely due to the pendulum effect of the heavy drill collar that keeps the hole straight.

The weight on the bit and the rotary speed were determined by the P.E.S. drilling supervisor and executed through the drilling contractor. There are other considerations when determining a weight on bit and rpm. The rotary table is the weakest link in the rotary drive. Increases in weight on the bit cause increases in the torque that the rotary table must transmit to the drill string. It is possible to increase this torque to a point where the rotary table will fail. Another objective in selecting the correct weight on bit and rotary speed is to keep the string together and not leave any part of the drilling assembly in the hole. Combinations of weight and rpm are often changed to achieve smooth rotating conditions.

Penetration Rates

There were twelve bit runs made on Shaft No. 1 to a total depth of 2243' RKB measurements. The rotating hours during the drilling of the hole was 2294.75 hours or 96 days out of a total of 129 drilling days. This amounts for about 74% of the time on bottom rotating. Average penetration rate was 0.98 feet per hour or 17.39 feet per day overall.

There were nine bit runs made on Shaft No. 2 to a total depth of 2188' RKB measurements. The rotating hours during the drilling of the hole was 1212 hours or 51 days out of a total of 62 drilling days. This amounts for about 80% of the time on bottom rotating. Average penetration rate was 1.8 feet per hour or 36.46 feet per day overall.

There were eleven bit runs made on Shaft No. 3 to a total depth of 2188' RKB measurements. The rotating hours during the drilling of the hole was 1258.5 hours or 52 days out of a total of 66 drilling days. This amounts for about 79% of the time on bottom rotating. Average penetration rate was 1.7 feet per hour or 34.19 feet per day overall.

Mud

The top portion of Shaft No. 1 was drilled using a freshwater gel mud. This was done in the interest of economy. The mud system was converted to a potassium chloride system at 638 feet before entering into the Mancos Shale. The Mancos is a bentonitic shale that swells and heaves upon contact with freshwater. A KCL system is the best drilling fluid to inhibit this swelling situation.

The entire mud plan was predicated on inhibiting shale swelling. Hole stabilization was the objective of the mud program. The hole could have been drilled with water had it not have been necessary to stabilize the walls of the hole.

Drilling progess is shown in Table No. 1 showing the time to drill Shaft No. 1.

TABLE 1. Drilling Progress Shaft No. 1

	Time to Complete
Mobilization	12 days
Drill 120" hole from 66' to 2243'	129 days
Prepare Rig To Run 85 inch Casing	5 days
Run 85 inch Casing to 2194 G.L.	21 days
Cement Casing	10 days
Total Shaft No. 1	183 days

CIRCULATING SYSTEM

All large diameter drilling operations require some type of air assist method. The reverse fluid air assist circulation system used on the Crownpoint Project is the simplest of all air assist circulation methods. No fluid pumps are used.

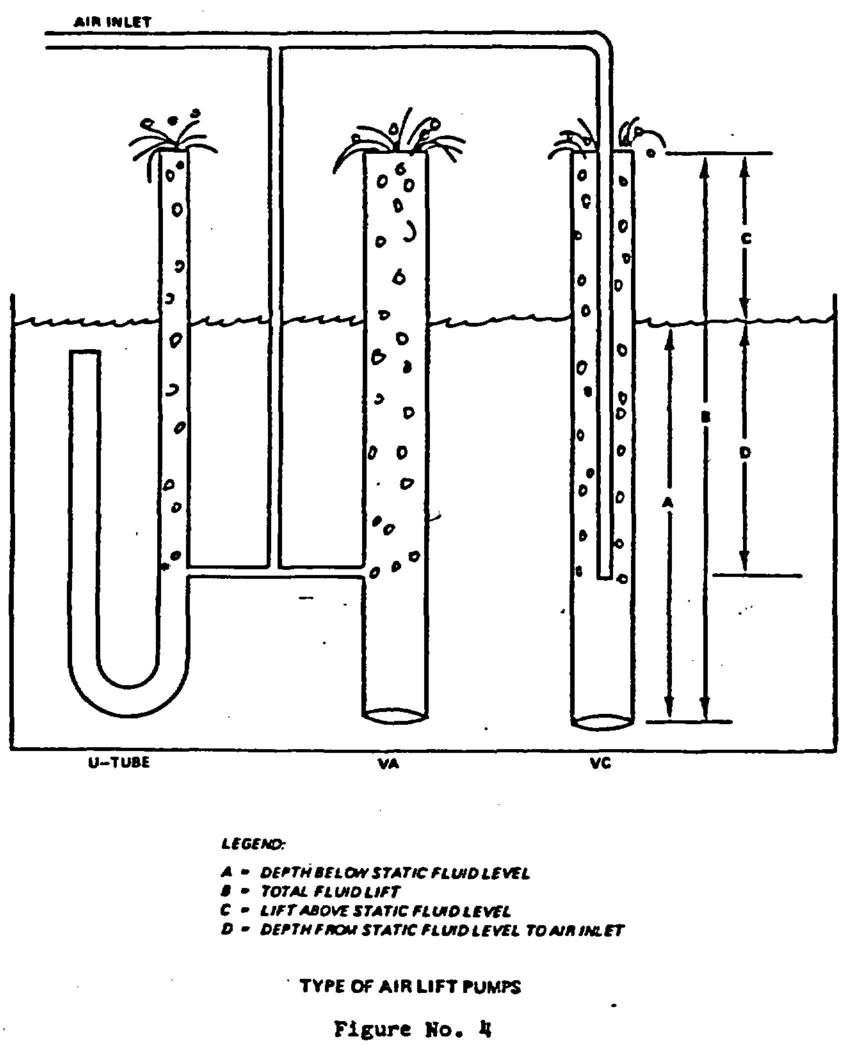
The hole is kept full through a gravity return line from the pits. Circulation is created by injecting air into a 3 1/2" line suspended inside the drill pipe below the hole fluid level in the annulus at about 320 feet. The column of fluid inside the drill pipe is aerated and the column of fluid in the annulus is not aerated. The unequal hydrostatic heads of fluid inside and outside the drill pipe at the bit result in a reverse flow across the face of the bit. Cuttings are picked up and returned to the surface at a velocity of about 700 feet per minute. At the Crownpoint Project, air injected at the rate of 2000 cfm produced mud returns of over 4000 gpm.

The cuttings are discharged out the blooie line into the working pits where the cuttings are removed and hauled away to a waste disposal area. Cuttings removal from the mud is largely due to settling in the surface working puts and removed with a crane and clamshell. Further cuttings removal is made with cyclone type separators.

How The Reverse Fluid Air-Assist System Works

The air lift pump may be described in its most basic terms as a "U" tube having unequal legs as shown in Figure No. 4. If the "U" tube is submerged in a tank of water as in Figure No. 4 where the shorter leg of the tube is flooded, and air is injected at some point into an outlet on the longer leg, flow will occur up the longer leg to an elevated point. Flow will occur only if the hydrostatic pressure of the longer leg is less than that of the shorter leg. This is the fundamental principle of air lift pumping; a hydrostatic pressure imbalance caused by the injection of air into one leg of the "U" tube.

The amount of flow depends on several predicable parameters. The height of the discharge above the static reservoir level, the amount of air injected, the depth of submergence of the air inlet below the static fluid level, and the pressure all have an effect on the amount of flow. Fortunately, most of these parameters can be calculated within reasonable accuracy to achieve maximum efficiency. The descriptions and equations that follow are the result of experience, Ingersoll-Rand's extensive testing data, field test experiment results made by the Atomic Energy Commission and field performance data from Conoco's Florence, Arizona, project. The reader should be cautioned that air pumping is not an exact science and that field adjustments may be required to achieve maximum efficiency in actual practice. The equations fairly well match the field data so adjustments should be minor.



TYPE OF AIR LIFT PUMPS

Figure No. 4

Types of Air Pumps

There are two types of air lift pump systems but the pumping principle is the same as the U-tube, as illustrated in Figure No. 4. One system requires an outside air line which is run in the annular space between the hole and the tubing. The outside air line system, or VA system, has the air flow line and the production tubing running side by side in the well. Because of the nature of this parallel tubing configuration, this system is not conducive to rotation of the tubing that is required in a drilling system. While this, the VA type, is slightly more efficient than the other concentric string type system (VC), it will not be stressed in this paper. The second type is the concentric string type, or VC system, where the air line is suspended inside the production tubing becomes drill pipe in a rotating mode. The suspended air line string should have left hand thread to lessen the possibility of backing off the coupling as the drill string rotates to the right. The VC system is slightly less efficient than the VA system because of the increased friction caused by the concentric strings having less than the full open diameter in the VA type. However, the VC system lends itself to rotation and is used by most contractors who are engaged in drilling large diameter holes. It was with this type circulating system that the Atomic Energy Commission drilled a 90" diameter hole to a depth of 6200' in a single pass. This is the deepest large diameter hole that has been drilled to date. The same system was used to drill 10 foot diameter hole to 5500 feet.

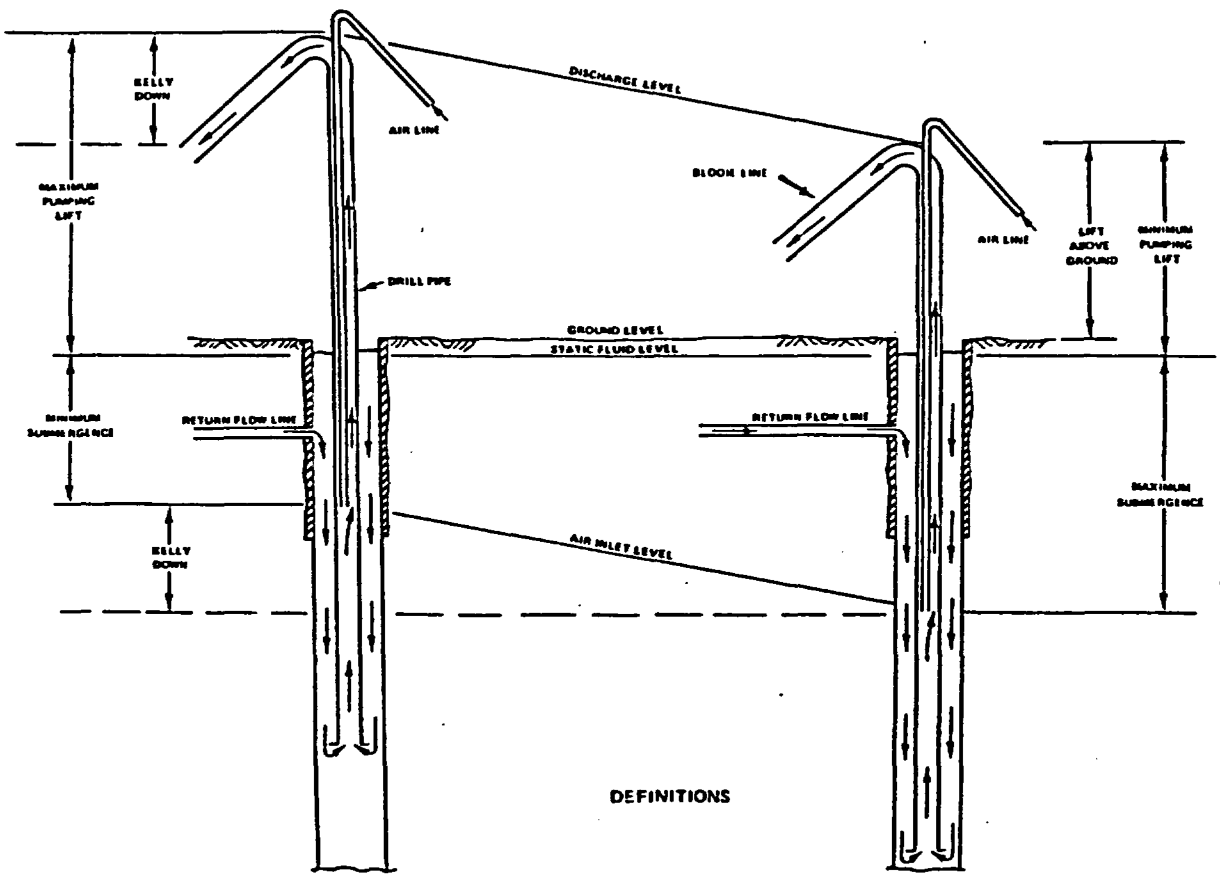


Figure No. 5

Terminology

The following glossary of terms and equations originally developed for a water well application have been modified to relate to a drilling situation. The operating modes are almost identical. Terminology of the reverse fluid air assisted circulating system illustrated in Figure No. 5 is as follows:

Discharge Level is the level of the discharge pipe above the ground. This is the height of the gooseneck attached to the swivel in the suspension system, and varies in height above ground level as the kelly is raised and lowered.

Static Fluid Level is the distance from ground level to the fluid surface in a static condition. The static fluid level should be near ground level.

Pumping Fluid Level is the same as the Static Fluid Level except in unusual conditions such as lost circulation. Hole is kept full by either pumps or gravity flow return lines.

Air Inlet Level is the distance from ground level to the discharge point at the air line tubing suspended inside the drill pipe.

Lift Above Ground is the distance from the ground to the top of the gooseneck.

Kelly-Down is the distance the kelly travels while drilling down a joint of drill pipe while circulating at rated capacity from the static or pumping fluid level.

Minimum Pumping Lift is the distance from the static water level to the top of the gooseneck discharge outlet when the kelly is in the down position.

Maximum Pumping Lift is the distance from the pumping fluid level to the top of the gooseneck discharge outlet when the kelly is in the up position.

Minimum Submergence is the distance from the pumping fluid level to the air inlet level when the kelly is in the up position.

Maximum Submergence is the distance from the pumping fluid level to the air inlet level when the kelly is in the down position.

Notations:

The symbols that apply in the formulas that follow are:

A_1 = Net inside area of discharge pipe at top, sq. in.

A_4 = Net inside area of discharge pipe at bottom, sq. in.

B = Atmospheric pressure, psi A

C = Efficiency Constant

E = Expansion of air volume per foot of discharge pipe, cfm

f = Air line friction loss, psi

L = Maximum pumping lift, ft.

M_w = Mud or fluid weight, ppg

P_s = Starting minimum submergence air pressure, psi G

P_m = Maximum submergence air pressure, psi G

Q_{a1} = Volume of dense air at air inlet level, cfm

Q_{a2} = Volume of dense air at air inlet level, cfm

Q_w = Volume of fluid flow, cfm

r = Air compression ratio at air inlet level, dimensionless

S = Pumping submergence, ft.

%S = Percent pumping submergence, %

S_m = Maximum submergence, ft.

T = Total free air required, cfm

V_a = Quantity of free air required per gal., cfm/gpm

V_d = Discharge velocity, ft./min.

V_e = Entrance velocity, ft./min.

W = Fluid flow, gpm

Basic Equations:

Percent pumping submergence, %

$$\%S = \frac{100S}{L+S} \qquad \text{(Eq. 1)}$$

Total Free air required, cfm

$$T = Q_{a2} = V_a W \qquad \text{(Eq. 2)}$$

Air compression ratio at the footpiece

$$r = \frac{(.052)(M_w)(S)}{B} + 1 \qquad \text{(Eq. 3)}$$

Volume of fluid flow, gpm

$$Q_w = \frac{W}{7.481} \qquad \text{(Eq. 4)}$$

Volume of air at top of discharge pipe, cfm

$$Q_{a2} = T \qquad \text{(Eq. 5)}$$

Volume of dense air at inlet level, cfm

$$Q_{a1} = \frac{T}{r} \qquad \text{(Eq. 6)}$$

Expansion of air volume per foot of discharge pipe, cfm

$$E = \frac{Q_{a2} - Q_{a1}}{L + S} \qquad \text{(Eq. 7)}$$

Maximum submergence air pressure, psi G

$$P_m = (.052)(M_w)(S_m) + f \qquad \text{(Eq. 8)}$$

Starting minimum submergence air pressure, psi G

$$P_s = (.052)(M_w)(S) \qquad \text{(Eq. 9)}$$

Discharge velocity, fpm

$$V_d = \frac{144}{A_1}(Q_w + Q_{a2}) \qquad \text{(Eq. 10)}$$

Entrance velocity, fpm

$$V_e = \frac{144}{A_4}(Q_w + Q_{a2}) \qquad \text{(Eq. 11)}$$

Quantity of free air required, cfm/gpm

$$V_a = \frac{L}{C \log_{10}\left(\frac{S + 34}{34}\right)} = \frac{\text{cfm}}{\text{gpm}} \qquad \text{(Eq. 12)}$$

Basic Theory

In order to estimate the amount of air required, depth of submergence, air pressure and pipe sizes to achieve optimum values, the first number that must be determined is how much lift above the pumping fluid level will be necessary. This will usually be from near ground level to the top of the gooseneck with the kelly in the up position. Generally, this will be 50 to 70 feet above the pumping/static fluid level.

The depth of submergence is the next item that must be determined. Submergence is usually expressed as a percentage of the total length of the air line tubing. Thus, an air line string 200 feet long with 140 feet submerged below the static or pumping fluid level with 60 above the fluid level would have a submergence percentage of 70% as calculated by Equation 1.

$$\%S = \frac{100S}{L + S} = \frac{(100)(140)}{(60) + (140)} = 70\%$$

The best percentage submergence for maximum efficiency for most drilling rigs will be 65-70%. With this optimum known, Equation 1 can be rearranged to determine the footage submergence required on any given rig when the total pumping lift (L) is known as follows:

$$S = \frac{(\%S)(L)}{100\ (1-\%S)} = \frac{(70)(60)}{100\ \left(1-\frac{70}{100}\right)} = 140'$$

The amount of free air required in cubic feet of air per gallon of fluid pumped can be estimated empirically from Equation 12.

$$V_a = \frac{L}{C \log_{10}\left(\frac{S + 34}{34}\right)}$$

The air volume requirement Equation 12 above can be modified [20] to account for higher fluid densities associated with the lifting of the drilling mud and cuttings for large diameter rotary drilled holes. The modified equation is as follows:

$$V_a = \frac{L \times SP.\ Gr}{C \log_{10}\left(\frac{(S \times Sp.Gr) + 34}{34}\right)} \qquad \text{(Eq. 13)}$$

Equation 13 was used to calculate the cubic feet of air required to produce a gallon of drilling fluid on the Crownpoint Project.

The Constant C in Equation 12 & 13 may be taken from the Table No. 2, below:

TABLE 2. Constant for Air Volume Formula

From Loomis[2]

%Submergence	Inside Air Line	Outside Air Line
30	135	188
35	160	216
40	186	245
45	214	272
50	240	295
55	264	316
60	288	336
65	305	345
70	320	357
75	330	367
80	335	375

Allen[20] noted that the Constant "C" changes to a lower value with increases in air volume and further, that the system efficiency factor is not applicable for drilling operations at great depths. Allen[20] calculated the value of factor "C" with air volumes of 1000 cfm, 1500 cfm and 2000 cfm in the following Table No. 3:

TABLE 3. System Efficiency Constants

From Allen[20]
Mud Weight = 9.0 ppg
Drill Pipe = 13 3/8" O.D.
Inside Air Line

Air Volume = 1000 cfm

Submergence%:	50	55	60	65	70	75	80	85	90
"C" Value :	250	285	316	337	335	310	265	219	205

Air Volume = 1500 cfm

Submergence%:	50	55	60	65	70	75	80	85	90
"C" Value :	200	217	230	238	237	222	193	172	185

Air Volume = 2000 cfm

Submergence%:	50	55	60	65	70	75	80	85	90
"C" Value :	158	173	182	186	181	170	155	139	125

This unreliability of the factor "C" was noted further in the Crownpoint drilling as it was calculated daily. Time does not permit a thorough analysis of Equation 12 & 13 with the publication of this report, but it is suggested that further study is necessary to determine the effect of friction in the circulating system.

Substituting values in Equation 12, an estimate of cfm/gpm at maximum pumping lift with the kelly up and minimum submergence is obtained as follows:

$$Va = \frac{60}{320 \log_{10}\left(\frac{140 + 34}{34}\right)}$$

$$= 0.26 \text{ cfm/gpm}$$

This estimate does not consider friction and the air required (cfm/gpm) will be increased due to the high volume and high velocity created in a drilling application.

The start-up pumping pressure of the above example is determined from Equation 8 assuming the use of a 9 ppg mud:

$$P_s = (.052)(M_w)(S)$$

$$= (.052)(9)(140) \qquad \text{(Eq. 8)}$$

$$= 66 \text{ psi}$$

The Crownpoint Circulating System

The complete circulating system concept for the Crownpoint Project is illustrated in Figure No. 6.

The collar shaft was completed to a depth of approximately 60' in all three shafts when the concrete foundation for the rig was constructed by others. The working pits were lined with steel plate into 8 separate compartments containing a total volume of 10,150 barrels. The hole is kept full of fluid through a 30" pipe with connects between the last working pit compartment and the hole.

When the kelly is drilled down, the maximum submergence pumping air pressure is estimated with Equation 9.

$$Pm = (.052)(Mw)(Sw) + f \quad \text{(Eq. 9)}$$

$$= (.052)(9)(180) + 10$$

$$= 90 \text{ psi}$$

The loss of pressure due to the friction of gas flowing through the pipe can be calculated from the basic Fanning equation. In the preceding example, the air friction pressure losses were estimated at 10 psi. Conversely, the drill pipe and air line sizes should be selected to keep the friction losses between 5 and 10 psi. Sizing of the drill pipe is an important function in the design of the efficient operation of a reverse fluid air assist circulating system. There are two types of losses in the pipe; (1) the slippage of air through the drilling fluid and, (2) normal friction losses of the drilling fluid with the pipe. As the fluid velocity increases, the air slippage through the fluid will decrease and friction losses will increase. The reverse is true as velocity decreases. Friction losses will decrease and air slippage will increase until at some point heading or intermittent unloading will occur. This is an undesirable situation and adjustments are necessary to increase velocity to obtain a steady flow. A constant pipe velocity cannot be achieved because velocities will increase from the footpiece to the surface because of the expansion of the injected air bubbles. The footpiece at the end of the air line is essentially a perforated nipple that disperses the injected air into as many small bubbles as is possible. Friction losses can be reduced by the use of larger diameter drill pipe opposite the length of maximum air line submergence. The hole is always the same level as the fluid level in the last pit.

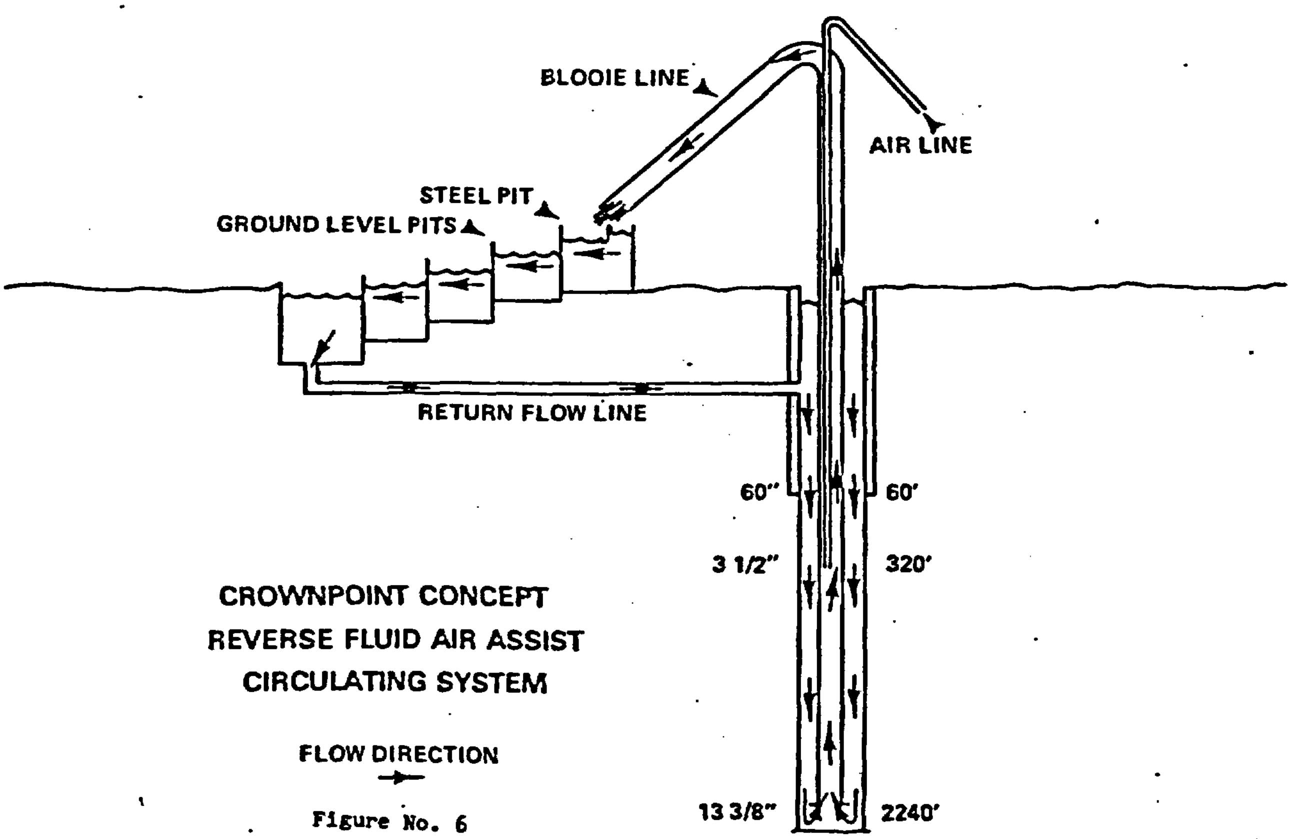


Figure No. 6

The air was supplied by 3-Atlas Copco rotary screw compressors rated at 1200 cfm at 310 psig. Each rotary compressor is powered by a G.M. 16V-71 internal combustion engine rated at 532 brake horsepower. At the Crownpoint altitude each compressor had an output of about 1000 cfm at the 170 psi operating pressure required to activate the circulating system.

Two compressors were used to activate the circulating system while the third compressor was standby capability when either of the other two compressors were down for any reason.

Because of the predetermined air volumes and pressures available in the compressors selected for the project, a submergence of 90% was chosen for use on the Crownpoint Project. Basic theory from the literature as discussed above states that a submergence of 70% presents the optimum use of air and horsepower. The Crownpoint Project used a submergence of 90% which was determined through trial and error in obtaining maximum flow with the available equipment selected to furnish the air requirements. While this is a higher percentage of submergence above the 70% submergence set out in the literature, it did provide maximum practical fluid returns for the Crownpoint Project.

Measurement of Fluid Returns

The Crownpoint Project was carefully planned where fluid returns in the circulating system could be accurately measured. In fact, this is the first time that mud returns, air input and pressure have been accurately measured on any large diameter hole.

The fluid or mud returns from the hole carrying drilled cuttings first passed through a mud-air separator to remove as much air as possible before the cutting laden drilling fluid entered the first compartment in the working pits. The flow from one compartment to another was through a 1' x 5' measuring weir. By measuring the height of the fluid flowing through the weir, the volume of fluid returning from the hole could be accurately measured.

Solids Removal

Approximately 90% of the drilled cuttings dropped out of suspension into the first pit compartment. These cuttings were removed from the pit by use of a crane and clam shell and hauled away with a dump truck to a waste disposal area. The remaining 10% of the cuttings either settled out in the remaining pits or remained in the mud in suspension.

Further solids removal was achieved through the use of hydrocyclone type separators. At first only one 18" Linatex separator was used on Shaft #1. This was increased to 3 units on Shafts #2 & #3. Each hydrocyclone separator had an input slurry capacity of 730-910 gpm. This solids removal equipment effectively removed fine particles from the system helping the mud weight low.

The Crownpoint Field Test

On February 27, 1981, a test was conducted on Shaft #3 to determine the change in the flow rate by varying the air input. This was accomplished by using one compressor, following with two compressors and finally three compressors while measuring the stabilized fluid returns at the weir in the working pits.

Table 4 and Figure 7 shows the results of this flow test during routine drilling operations at a depth of 1958 feet. The depth of submergence (S) of the air line was 337 feet and lift (L) was 32 feet above hole annulus fluid level. Percent submergence was 91%.

TABLE 4. Flow Rate Test - Shaft No. 3
February 27, 1981

CFM	GPM	PSI	CFM/GPM	Constant "C"
1000	3260	158	0.31	105
2000	3956	168	0.50	64
3000	4412	205	0.68	48

It should be noted tht the increase in air volume and horsepower by a factor of 3 produced only a 35% increase in drilling fluid returns. Note, also, that Constant "C" in Equation 1 and 2 does not correlate with data from Tables 2 and 3.

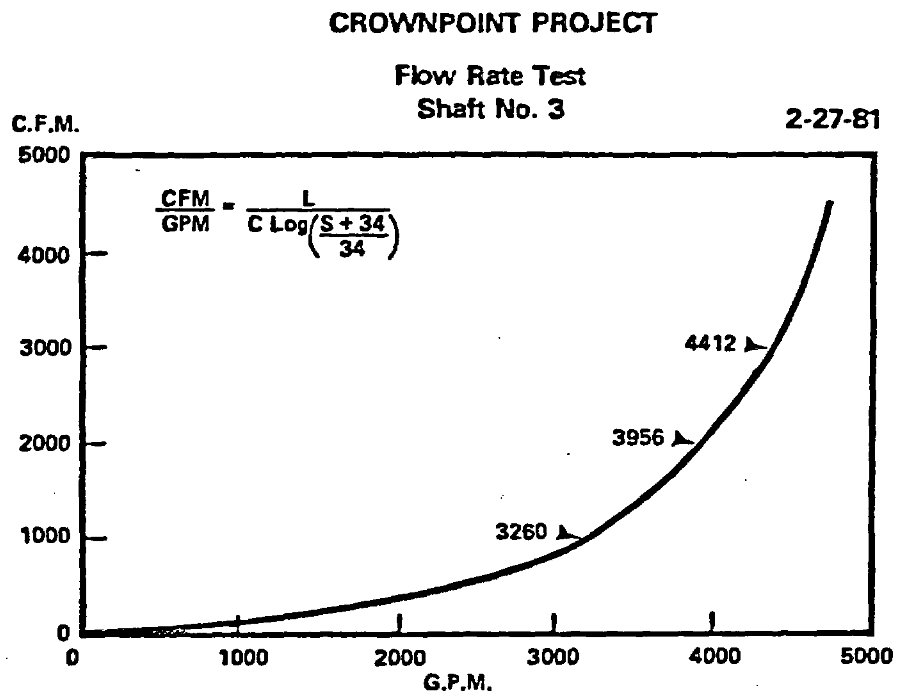


Figure No. 7

CASING DESIGN SHAFT NO. 1

Large diameter ring stiffened steel casing was designed to withstand a predetermined collapse environment. The collapse parameter chosen for the 85 inch I.D. casing for the Shaft No. 1 was 1 1/2 times a full hydrostatic head of water.

Design considerations for collapse are determined by the thickness of the steel plate, the casing diameter - plate thickness ratio, the strength of the material, out of the roundness tolerance, the height and length of the stiffener rings and the spacing of the stiffener rings.

The initial design for the Crownpoint casing is tabulated in brief in Table No. 5.

The 85-inch casing weight in air was over 3,000,000 pounds. The casing design was later modified in part in order to expedite timely delivery.

TABLE 5. 85-inch Casing Design

Design From	Design To	Design Pressure	Wall Thickness	Stiffner Rings	Ring Spacing	Material
2201'	1976'	1428	1-1/8"	4" x 5"	18"	A-588
1976'	1436'	1166	15/16"	4" x 5"	18"	A-588
1436'	1076'	932	15/16"	4" x 5"	24"	A-588
1076'	725'	697	13/16"	4" x 5"	36"	A-588
725'	613'	470	11/16"	3 1/2" x 5"	42"	A-441
613'	496'	398	1/2"	3 1/2" x 5"	39"	A-441
496'	320'	321	1/2"	3 1/2" x 5"	48"	A-441
320'	0'	207	1/2"	3" x 5"	60"	A-441

RUNNING & WELDING CASING

Running and welding of the 85 inch diameter casing is a combination of effort between the welding contractor and the drilling contractor. The casing joints are moved to the rig by the welding contractor, upended into the mast and lowered into the hole by the rig after welding operations are complete. The casing is designed as a closed pressure vessel having a hemispherical steel head as the shoe joint. The casing is floated into the hole by controlling the buoyed weight of the casing. The rig only carries a portion of the total casing weight.

The entire casing string is run into the drilled hole before any cementing operations are commenced.

After mobilization is complete, the welding contractor's first responsibility is to attach four 3-1/2" O.D. grout line guides on 90° centers on all casing joints to be run.

The welding contractor arranges the casing into its running sequence during grout line guide installation. He furnishes a crane and low-boy truck to transport the casing from the storage yard to the rig.

The welding contractor's crane places one set of casing elevators onto the casing joint in front of the rig and lifts the joint into the V-door whereby the rig crew can attach the elevator slings to the elevator.

The rig hoists the casing into a vertical position while the welders crane holds the opposite end of the casing. Alignment of the casing joint is the next task after up-ending the casing. The closure pass or root bead is the first weld. The root bead is inspected using radiographic procedures before further welding is permitted. The circumferential weld is completed using seven welders working simultaneously.

Quality control for welding was provided through a separate contractor reporting to Conoco drilling. The QC personnel consisted of one welding inspector and two X-ray technicians.

After the weld is complete and the casing elevator is removed from the joint that has just been welded, four splice joints are placed in the area occupied by the elevators to make each grout line guide continuous.

A fifth grout line guide is welded in place near one of the other grout line guides.

The purpose of the fifth line is to provide free and independent access to the annulus for the Nuclear Annulus Investigation Log which is run on a wireline during cementing operations.

The hole is kept full of drilling fluid during the entire casing operation. As the forty foot joint is lowered in the hole, the casing string will gain buoyancy which is equal to the weight of the fluid which has been displaced from the hole into the pits.

When the casing is lowered 40 ft., about 120,000 lbs. of buoyancy is gained per joint, thereby reducing the rig load. Thereafter, water is added inside the casing to cancel out the buoyancy effect and bring the rig back to a pick up load of approximately 330,000 lbs.

This procedure is repeated each for each joint to maintain a constant rig load.

There are two pieces of casing handling equipment which are mandatory in running casing. These are the casing elevators and the casing strongback. The casing elevator system consists of two sets of elevators for the casing size being run, one set of cable slings and a lifting yoke.

The casing strongback is a structural box with a cover plate upon which the casing elevator must bear.

LOGGING

There are two types of geophysical electric logs used in large diameter drilling. A caliper log is run upon reaching total depth of the hole. The purpose of the caliper log is to determine hole configuration and hole volume.

The Nuclear Annulus Investigation Log, (NAIL) is used during cementing operations to determine the top of the cement slurry. It is a wireline tool which measures the difference in density between the cement slurry and the drilling mud in the annulus to determine the interface between the different density fluids. All three large diameter holes drilled at the Crownpoint site had identical logging programs.

SURVEYING

Hole straightness was measured on the Crownpoint drilled shafts through the use of gyroscopic directional surveys run on a wire line inside the stabilized drill collar.

Horizontal displacement from perpendicular on the completed three large diameter shafts is as follows:

	Horizontal Displacement		
	Feet	Direction	Depth
Shaft No. 1	1.03'	S30 Deg. 17'W	2243'
Shaft No. 2	1.33'	S21 Deg. 17'W	2188'
Shaft No. 3	0.84'	S75 Deg. 38'W	2188'

All three shafts were drilled within the hole straightness criteria established by the WMC/Conoco Minerals Department.

Directional borehole surveying was provided by Sperry-Sun. The contract required Sperry-Sun to provide all down hole gyroscopic surveying tools and a resident survey engineer to remain on call at all times. The engineer did in fact reside in a Sperry-Sun trailer house in the Conoco trailer park.

CEMENTING

Three types of cement slurries were used. Chem Comp is a cement which expands when setting up. The expanding cement was used on all three shafts to cover The Westwater Formation and other sand sections in the hole. The objective of the expanding cement was to achieve a good bond in the shale sections between major sand formations. Prehydated 2% gel cement was used on Shaft No. 1 on stages requiring only a filler cement. Neat cement was used as a filler cement on Shafts No. 2 and 3.

The cementing plan called for one cementing stage per day. The amount of cement that could be pumped in 2 1/2 hours was about 6500 sacks or 1550 barrels of cement slurry per stage. Setting time for the cement was five hours. Only 1/2 of the time was used for pumping and the other 1/2 of the time was used in pulling the four tubing lines out and above the cement level in the annulus.

All cement slurries were displaced through four 1.9 inch O.D. tubing lines simultaneously. The tubing was run inside the 3 1/2 inch O.D. grout line guides that were attached to the casing when the casing was welded and run into the hole. The 5th grout line guide was used to run a wireline Nuclear Annulus Investigation Log (NAIL) to monitor the location of the interface between the cement slurry and the drilling fluid in the hole during cementing operations. Figure 8 illustrates the second stage of cement in process. The 1.9 inch tubing is pulled up and above the cement level when each stage is complete.

The time between stages was used to refill the cement bulk storage bins on location. Two stages of bulk cement were maintained on location for flexibility in case of interruptions in logistics or weather. The cementing contractor furnished 15,500 cubic feet of bulk storage space for the cement. Often, half of the storage was filled with Chem Comp and Type I Class A in the other half.

CROWNPOINT PROJECT
CEMENTING

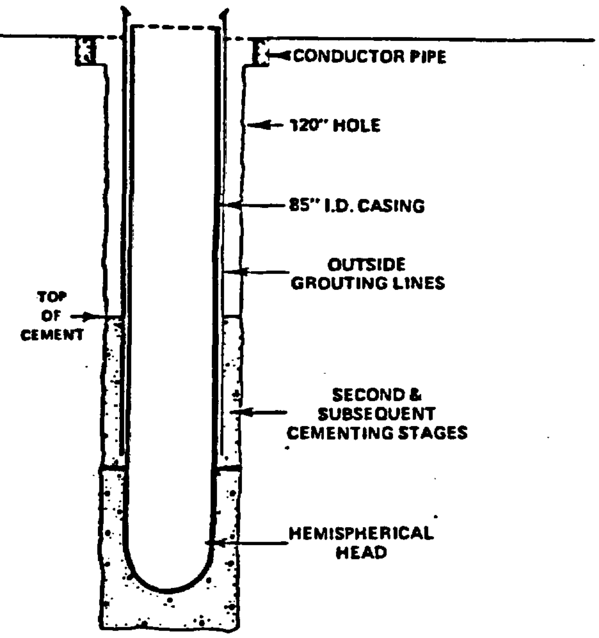


Figure No. 8

Cementing equipment consisted of three pumping units, cement silos, an instrument bus, instrumented manifold, water surge tank, compressor and cement flow lines. The cementing contractor furnished all cementing material delivered to the site.

The slurry volume required to cement the annulus of Shaft No. 1 was 55,160 cubic feet; Shaft No. 2 was 42,547 cubic feet; Shaft No. 3 was 44,047 cubic feet.

PUMPING AND BAILING CASING

Upon completion of the cementing operation, the inside of the large diameter casing is full of fresh water. The final operation to be performed is the removal of water from the casing.

A centerlift centrifugal pump is run into the 85" casing to bottom on 7" tubing. The pump used was 10-3/4" O.D. body, 520 horsepower tandum motor, with 17 stages rated at 1675 gpm.

Water was pumped out of the casing in a few hours and then the centerlift pump was removed from the casing. The water remaining in the casing was removed by running a 30" x 20' fabricated dart bailer on a wireline.

DRILLING SHAFT NO. 2

The concrete rig foundation containing a 96" finished inside diameter collar shaft (surface hole) cased with CMP pipe to a depth of 60 ft. (G.L.,) was constructed by the general contractor. The collar shaft hole was excavated with an auger rig.

The drill pad was released to the drilling contractor on September 30, 1980. Rig up was complete and Shaft No. 2 was spudded on October 6, 1980. Drilling commenced at a depth of 59 feet rotary kelly bushing measurement (RKB).

The 72 inch bottom hole drilling assembly shown in Figure No. 3. Using a drill collar assembly having a weight in mud of about 260,000 lbs. only about 1/3 of this weight is used for weight on the bit to achieve a satisfactory penetration rate.

A reverse fluid air assist circulating system was used to circulate cuttings from the hole. The method was exactly the same as used on Shaft No. 1 as discussed above. The cfm air input and gpm return were almost identical in behavior as in Shaft No. 1.

Shaft No. 2 was drilled to a total depth of 2188' RKB. Thereafter, 36 inch ring stiffened casing was run to a setting depth of 2146' RKB (2132 ground level) measurement and the annulus was cemented back into the collar shaft. Casing design, running and welding casing and cementing casing were similar to that discussed above for Shaft No. 1.

The casing design for Shafts No. 2 and 3 had a unique feature that was different from Shaft No. 1. Off the shelf 36" line pipe complying to API 5L, Grade B, standards was ring stiffened with A-36 steel. The original casing specifications are tabulated in Table 6.

TABLE 6. 36-Inch Casing Design

Design From	Design To	Design Pressure	Wall Thickness	Stiffner Rings	Ring Spacing	Material API-5L
2200'	1960'	1191	3/4"	2" x 1-3/4"	12"	B
1960'	1720'	1060	3/4"	2" x 1-3/4"	14"	B
1720'	1360'	931	5/8"	2" x 1-3/4"	14"	B
1360'	1040'	736	5/8"	2" x 1-3/4"	18"	B
1040'	720'	563	5/8"	2" x 1-1/2"	21"	B
720'	520'	390	5/8"	2" x 1-1/2"	24"	B
520'	320'	281	5/8"	2" x 1-1/2"	30"	B
320'	0'	173	1/2"	2" x 1-1/2"	40"	B

After cementing operation had been completed, the casing was pumped and bailed dry of fluid.

Drilling progress including the location move is tabulated in Table 7 showing the time to drill Shaft No. 2.

TABLE 7. Drilling Progress Shaft No. 2

Shaft No. 2	Time to Complete
Location to location move	7 days
Drill 72" hole from 59' to 2188'	64 days
Prepare rig to run 36 inch casing	2 days
Run 36 inch casing to 2132' G.L.	8 days
Cement casing	5 days
Pump and bail casing dry	4 days
Total Shaft No. 2	90 days

DRILLING SHAFT NO. 3

The concrete rig foundation containing a 96" finished inside diameter collar shaft (surface hole) cased with CMP pipe to a depth of 60 ft. (G.L.) was constructed by the general contractor. The collar shaft hole was excavated with an auger rig.

The drill pad was released to the drilling contractor on December 27, 1980. Rig up was complete and Shaft No. 3 was spudded on January 1, 1981 at 0700 hours.

Drilling commenced at a depth of 56 feet rotary kelly bushing measurement (RKB).

The 72 inch bottom hole drilling assembly was identical to that used on Shaft No. 2 illustrated in Figure 3. The mud program also was identical to Shaft No. 2.

Shaft No. 3 was drilled to a total depth of 2188' RKB. Thereafter, 36 inch ring stiffened casing was run to a setting depth of 2145' RKB, (2131 ground level) measurement and the annulus was cemented back into the collar shaft. Casing design, running and welding casing and cementing casing was almost identical to that of Shaft No. 2.

After the cementing operation had been completed, the casing was pumped and bailed dry of fluid.

Drilling progress including the location move is tabulated in Table 8 showing the time to drill Shaft No. 3.

TABLE 8. Drilling Progress Shaft No. 3

Shaft No. 3	Time to Complete
Location to location move	4 days
Drill 72" hole from 56' to 2188'	66 days
Prepare rig to run 36 inch casing	1 day
Run 36 inch casing to 2131' G.L.	7 days
Cement casing	7 days
Pump and Bail Casing Dry	2 days
Demobilize	3 days
Total Shaft No. 3	90 days

FISHING

There were two quickly solved fishing jobs. The 3 1/2" air line separated and dropped on Shaft No. 1 and a cutter was lost off the bit on Shaft No. 2.

The air injection line on Shaft No. 1 separated at the gooseneck and dropped inside the drill pipe to the bit. The 3 1/2" tubing was recovered with an overshot run on 2-7/8" tubing. The fishing job was complete in nine hours including delivery time of the rented overshot tool.

A cutter was lost at 1406' in Shaft No. 2 due to a bit saddle failure. A 66-inch rotating side door basket was fabricated on the Crownpoint site. The cutter and broken saddle were recovered on the first fishing run into the hole. The fishing job required 75 hours to complete including the time to acquire the steel plate and rolling the plate into a 66 inch tube. Welding consumed about 28 hours of the time to fabricate the fishing basket. The run into the hole to catch the fish and come out of the hole consumed eight hours.

TIME AND COST

Mobilization of the drill rig on Shaft No. 1 commenced on April 1, 1980. Demobilization of the rig from Shaft No. 3 was final on March 29, 1981. A lapsed time of 363 days was used from the beginning to the end of drilling activity. The drilling construction was finished 205 days ahead of the project plan schedule and 19.6% under budget.

A comparative cost-percent of total analysis is presented in Table 9 and Table 10.

TABLE 9. Comparative Cost - Percent of Total

Site Preparation	6.9 %
Rig	20.2
Air	3.6
Bits & Stabilizers	10.9
Mud	5.3
Logging & Surveys	1.4
Cuttings Disposal	1.9
Casing	21.0
Casing Welding	12.0
Quality Control	1.2
Casing Elevators & Strongback	1.9
Cementing	6.9
Fuel	2.6
Supervision	2.9
Taxes and other	1.3
Total	100.0 %

TABLE 10. Shaft Cost

	Percent of Total
Shaft #1	53
Shaft #2	24
Shaft #3	23
Total	100 %

Acknowledgement

The author is grateful to Conoco Inc. and Wyoming Mineral Corporation for permission to publish this paper.

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Chapter 23

EXPLORATORY SHAFTS AND UNDERGROUND TEST FACILITY FOR THE WASTE ISOLATION PILOT PLANT (WIPP)

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ABSTRACT

During the past few years, the first stage of the Waste Isolation Pilot Plant, (WIPP) which consists of an exploratory shaft, a ventilation shaft, shaft stations, access drifts and test rooms, has been under construction. Construction of the first stage is scheduled to be completed in the summer of 1983. The U.S. Department of Energy (DOE) is constructing the WIPP facility to demonstrate the feasibility of disposing of defense-related nuclear waste in the bedded salt deposits of the Delaware Basin near Carlsbad, N.M. The project promises to be a proving ground for underground nuclear waste storage facility design and construction.

The completed underground plant will extend over an area of 647,500 m^2 (160 acres), and 1,360,500 metric tons (1,500,000 tons) of salt will be excavated. The first stage of the project, the Site and Preliminary Design Validation (SPDV) Program, provides for full-scale construction of the major components of the underground facility. Shaft construction and underground development are described. Rock bolt installation is performed as required for roof control. Geotechnical instrumentation is installed to monitor the behavior of underground openings. Direct observation of the constructed shafts, shaft stations, and underground openings is expected to confirm the adequacy of the underground design and allow full-scale construction of the first U.S. underground nuclear waste storage facility to proceed.

INTRODUCTION

In the United States, the accumulation of radioactive waste and its safe disposal have become key elements in determining the safe use of nuclear energy. In parallel with the program dealing with disposal of waste from nuclear power projects, safe and reliable methods of permanently disposing of radioactive waste accumulated from the defense program must be demonstrated. The U.S. Department of Energy is responsible for a major effort to resolve the issue of safe underground disposal of nuclear waste. Construction of the Waste Isolation Pilot Plant (WIPP) by the DOE will demonstrate the

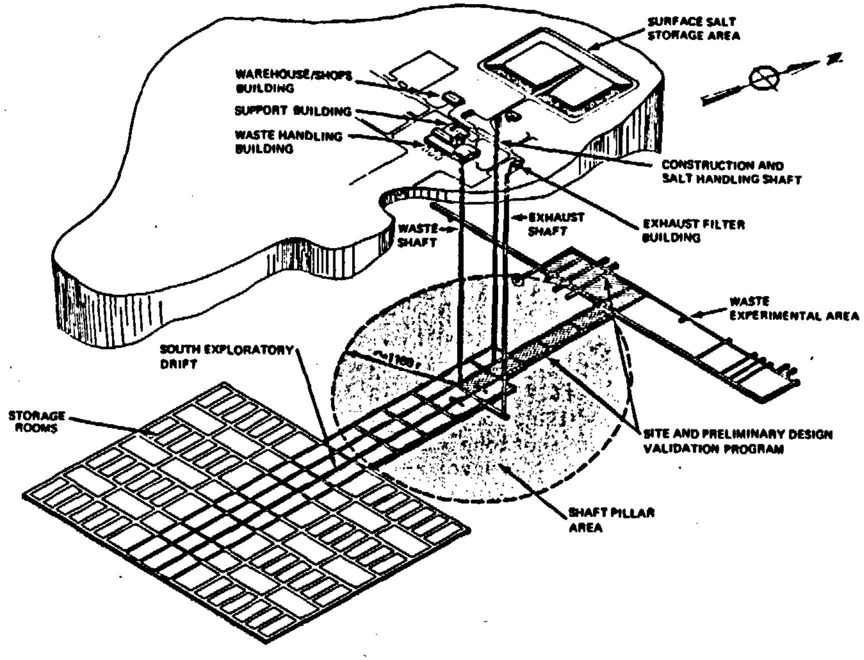

FIGURE 1 LAYOUT OF THE WASTE ISOLATION PILOT PLANT. THE CROSS-HATCHED AREA SHOWS THE DRIFTS AND ROOMS EXCAVATED UNDER THE SPDV PROGRAM.

feasibility of disposing of defense-related nuclear waste in the bedded salt deposits of the Delaware Basin near Carlsbad, N.M.

The layout of the completed underground facility is shown in Figure 1. The project master plan calls for phased development of the facility. The completed plant will extend over an area of 647,500m^2 (160 acres). Before the full design capability can be realized, the WIPP Site and Preliminary Design Validation (SPDV) Program has to be completed to confirm the adequacy of the site and the underground design. The program includes construction of the major components of the underground facility including, access shaft and ventilation shaft, shaft stations, several thousand feet of entries and test rooms. The layout of the SPDV facility is included as part of Figure 1. The facility provides for mapping of local geology and direct observation of the behavior of the representative underground openings and shafts. If necessary, adjustments will be made to the continuing design development based on observed performance. Construction of the SPDV facility is almost completed and tests and data evaluation are now under way.

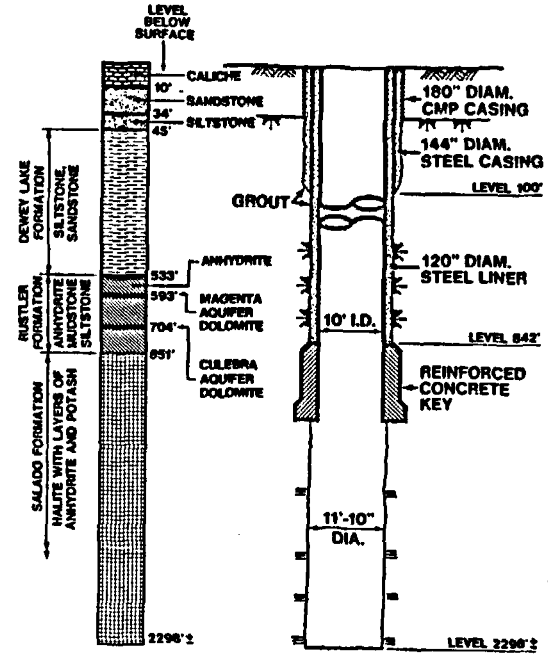

FIGURE 2 GENERALIZED GEOLOGIC COLUMN AND SHAFT CONSTRUCTION

DESCRIPTION OF SITE AND GEOLOGY

The WIPP facility is located in southeastern New Mexico, about 40.2 km (25 miles) east of Carlsbad, N.M. in a semiarid region of high desert. The site receives about 30.5 cm (12 inches) of rain a year. In the upper 244 m (800 ft) the geologic formations are relatively soft and consist of sandstone, siltstone, mudstone, and dolomite. Two water-bearing zones were penetrated at a depth of 181 m (593 ft) and 215 m (704 ft). These zones yield very little water, on the order of 0.063 liter per second (1 gallon per minute) in the unlined ventilation shaft. Below that, the bedded salt of the Salado formation is about 610 m (2,000 ft) thick (Figure 2). Underground development is carried out at a depth of about 655 m (2150 ft) in thick deposits of the Salado formation. The Salado formation at that depth consists of slightly dipping layers of halite (salt), polyhalite, and anhydrite. The thickness of the layers varies from thin layers of only several inches to thicker layers of 3 to 6 meters (10 to 20 ft). At some of the contacts of the layered deposits, clay seams vary from almost imperceptible thickness to several inches in thickness.

SHAFT DRILLING

Both the 3.66 m (12 ft) drilled diameter exploratory shaft and the 1.83 m (6 ft) drilled diameter ventilation shaft were drilled by the down-hole drilling method. Shaft drilling was performed with drilling equipment including a derrick and hoist works capable of supporting a suspended load of 454 metric tons (500 tons) in the shaft. The drilling assembly consists of a drilling mandrel, drill bit, stabilizers, and weights (Figures 3 and 4). During drilling it was suspended in the brine filled shaft from the traveling block mounted in the drill derrick. The drill bit is securely attached to

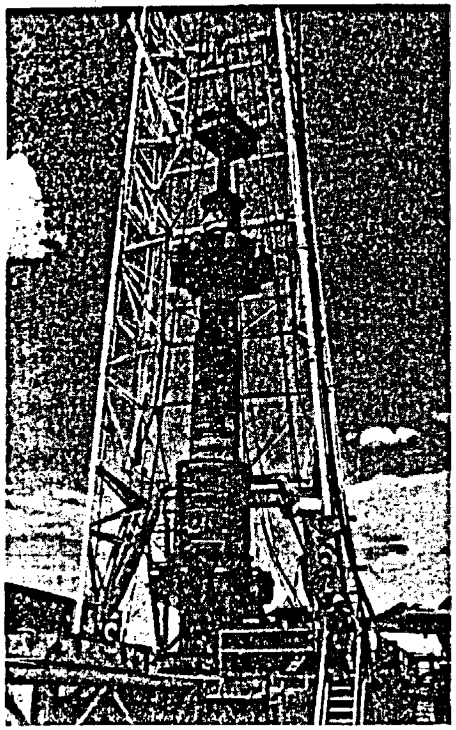

FIGURE 3 DRILL DERRICK WITH DRILLING ASSEMBLY

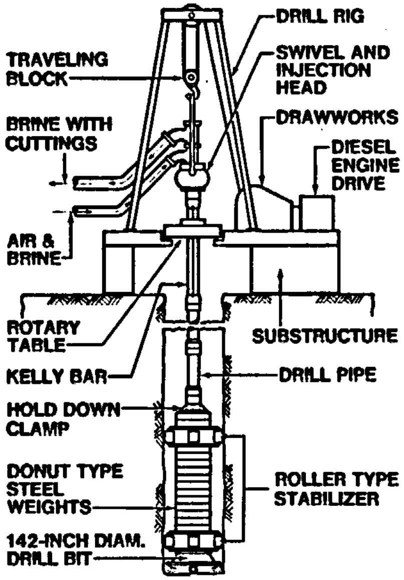

FIGURE 4 GENERAL ARRANGEMENT OF DRILLING EQUIPMENT

the bottom flange of the drilling mandrel. It utilizes a full face, rolling cutter head equipped with high strength, steel milled, teeth cutters (Figure 5). Rotary drill speeds were maintained between 12 and 16 RPM.

The drill pipe was delivered in 13.72 m (45-ft) sections. When drilling had proceeded for the length of one drill pipe, drilling was stopped. A new section of drill pipe was connected to the drill string and drilling was resumed.

Cuttings were removed by reverse circulation and air lift (Figure 6). A dual string drill pipe was used. The outer casing of the drill pipe was 34 cm (13-3/8 in.) diameter. The inner casing was 17.78 cm (7 in.) in diameter. A mixture of drilling fluid and air was circulated down the annular space between the outer and inner casing. When the mixture reached the bottom of the drill bit, it was released through jet openings and air separators. The separated air was injected directly into the inner casing to rise as air bubbles to the surface, thereby reducing the combined specific

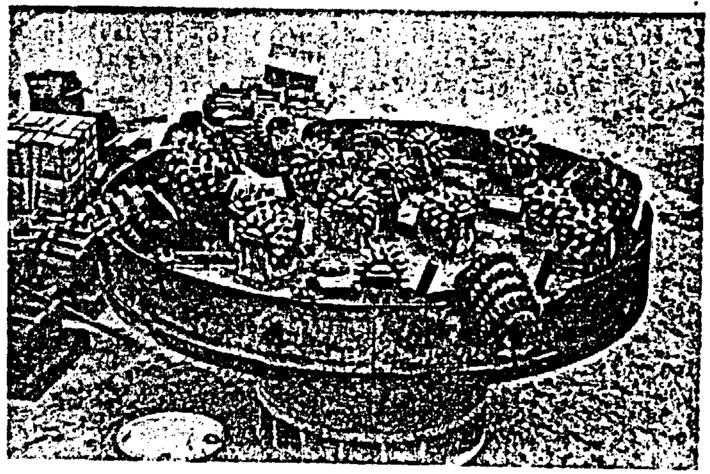

FIGURE 5 DRILL BIT

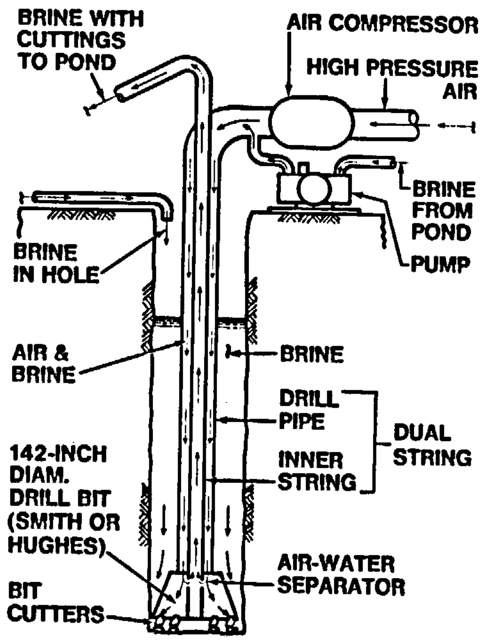

FIGURE 6 REVERSE CIRCULATION DRILLING SCHEMATIC

gravity of the air-fluid mixture. The fluid component was circulated across the bit face. Then, together with the drill cuttings, the fluid was drawn into the inner string casing to be forced to the surface by the difference in specific gravity of the drilling fluid in the drilled shaft versus the lighter fluid in the inner casing. The level of drilling fluid in the shaft was always kept at least 61 m (200 ft.) above the drill assembly. A drilling fluid pond was provided at the surface with a storage capacity of 12,490 m^3 (3,300,000 gal). Drilling fluid removed from the drill hole was discharged into the drilling fluid pond. The cuttings were allowed to settle to the bottom of the pond, and the drilling fluid was cleaned and recirculated in the shaft.

The maximum penetration rate for one 24-hour three-shift day is 7.62 m/day (25 ft/day) in rock and 19.81 m/day (65 ft/day) in salt for the 3.66 m (12 ft) diameter shaft, and 18.29 m/day (60 ft/day) in rock and 40.84 m/day (134 ft/day) in salt for the 1.83 m (6 ft) diameter shaft.

To control deviation in the shaft as drilling progressed, surveying was conducted at intervals of 13.72 m (45 ft), each time a new drill pipe was installed. A Sperry Sun multishot gyroscope was suspended down the inner casing of the suspended drill pipe and stopped in a location between the upper and lower stabilizers of the drilling assembly. The maximum deviation from centerline at the bottom of the shaft was 0.48 m (1.59 ft) for the 3.66 m (12 ft) exploratory shaft and 0.69 m (2.25 ft) for the 1.83 m (6 ft) diameter ventilation shaft.

SHAFT LINING

In the 3.66 m (12 ft) drilled diameter exploratory shaft, a 3.05 m (10 ft) inside diameter steel lining was placed to protect the hoist

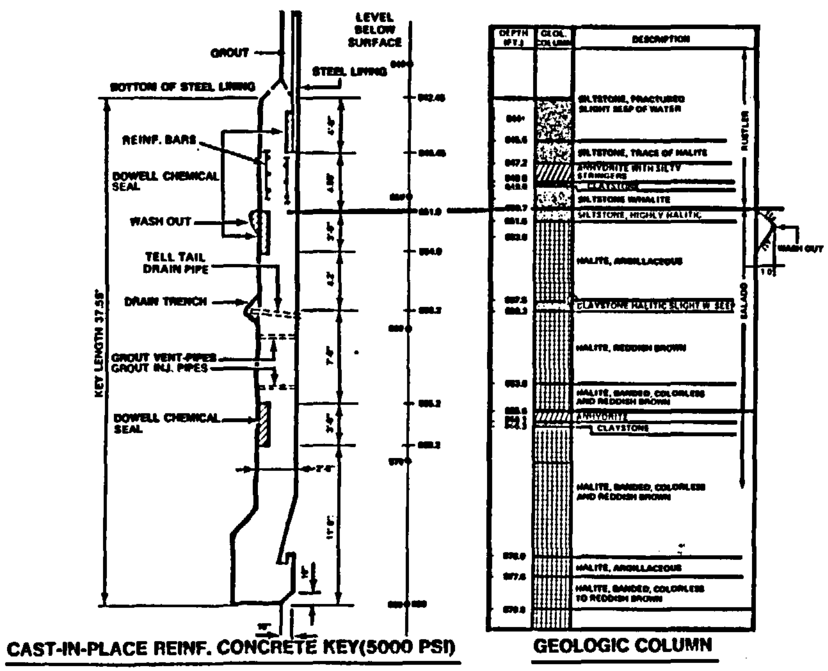

FIGURE 7 SHAFT KEY

furnishings in the overburden rock. No lining was required in the salt portion of the shaft (Figure 2). A concrete shaft key was constructed at the rock-salt interface to key the lining into the salt interface and to prevent water leakage at the bottom of the steel lining (Figure 7).

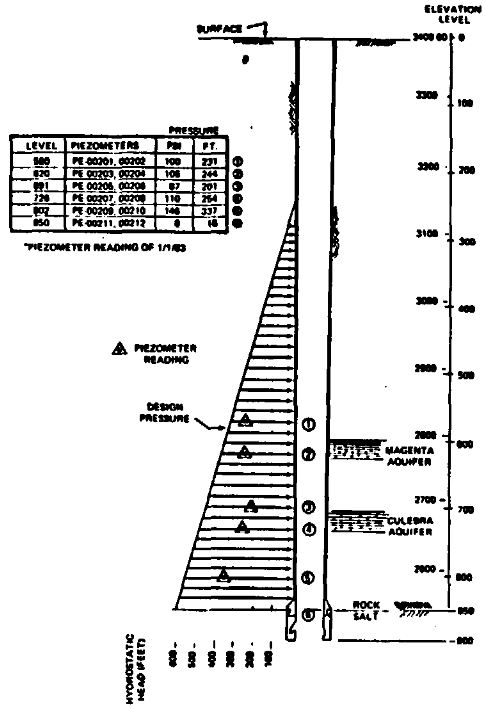

FIGURE 8 COMPARISON OF DESIGN HYDROSTATIC PRESSURE ON SHAFT LINING AND AS-BUILT PIEZOMETRIC PRESSURE ON LINING

To study the aquifers and to allow geologic mapping of the entire geologic column overlying the facility, the 1.83 m (6 ft) drilled diameter ventilation shaft was left unlined and local support was provided only where it was required for the safety of the personnel inspecting the shaft and for the geologists performing the geologic mapping. It was found that the sandstone in the Dewey Lake formation was stable. The siltstone and mudstone beds in the Ruster Formation showed some washouts from the drilling operation and subsequent erosion from the aquifer water running down the shaft wall. In the washout zones, liner plate was installed to protect the wall.

The lining and lining stiffeners in the 3.66 m (12 ft) drilled diameter exploratory shaft were designed for hydrostatic pressure from the aquifers and made of A-441 steel (Figure 8). They were fabricated into 6.1 m (20 ft) sections by full penetration bevel welds between the lining plates. All sections were double joint welded to 12.2 m (40 ft) sections by full penetration bevel welds.

The thickness of the steel lining and its stiffener rings increased with the depth of the hydrostatic pressure. The thickness varied from 1.59 cm (5/8-in.) to 3.81 cm (1-1/2 in.) for the upper 58 m (190 ft) to 3.81 cm (1-1/2 in.) for the lower 30.5 m (100 ft). The lining was floated into the brine-filled shaft relying on the buoyancy of the lining which was closed at the bottom by a temporary concrete float shoe. One 12.2 m (40-ft) lining section after another was lifted by the traveling block onto the derrick, and then lowered into the shaft (Figure 9).

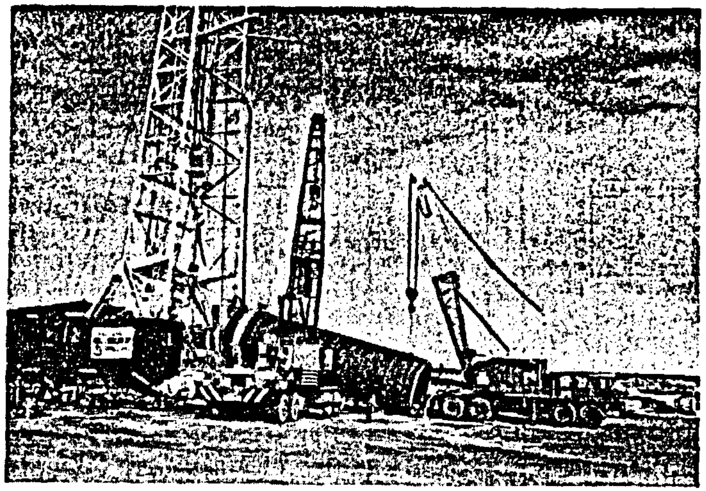

FIGURE 9 LINING SECTION BEING LIFTED INTO THE DERRICK

Weld connections of the 12.2 m (40-ft) lining sections were made while the lining sections were suspended from the derrick. After all lining sections were installed, the annulus between the lining and the shaft wall was grouted with cement grout through grout pipes which were attached to the outside of the lining sections. After the grout of the lining was set, the concrete shoe at the bottom of the liner was drilled through and the brine in the shaft was pumped out into the drilling fluid pond.

UNDERGROUND DESIGN

Layout and development of the WIPP underground facility was aided by the fact that there are a number of potash mines in the Carlsbad area in comparable lithology and stratigraphy. Also, there are salt and potash mines in similar geologic formations in other parts of North America. Room and entry width as well as pillar spacing for WIPP were developed based on the experience with those mines. In addition, the selected room and pillar sizes were checked by methods described in the SME Mining Engineering Handbook, Chapter 13.7 "Roof Control Through Beam Action and Arching" (Ref. 1).

A more rigorous finite element analysis of the behavior of the WIPP underground openings is in progress and it is expected that the observed behavior and measurements of the Preliminary Design Validation Program will provide the in-situ, site-specific data for completion of this analysis.

In the design and construction of underground openings at the depth of the WIPP underground facility, several factors can significantly influence the design of the opening.

Stress Condition. The expected stresses around the opening are relatively high. In-situ stresses in the undisturbed ground are on the order of magnitude of 13,790 KPa (2,000 psi). Stress redistribution around the excavated opening creates stress concentrations in the range of 20,685 to 27,580 KPa (3,000 to 4,000 psi). These stresses are higher than the unconfined compressive strength of salt and, therefore, local spalling with associated stress redistribution have to be expected and provided for (Figure 10).

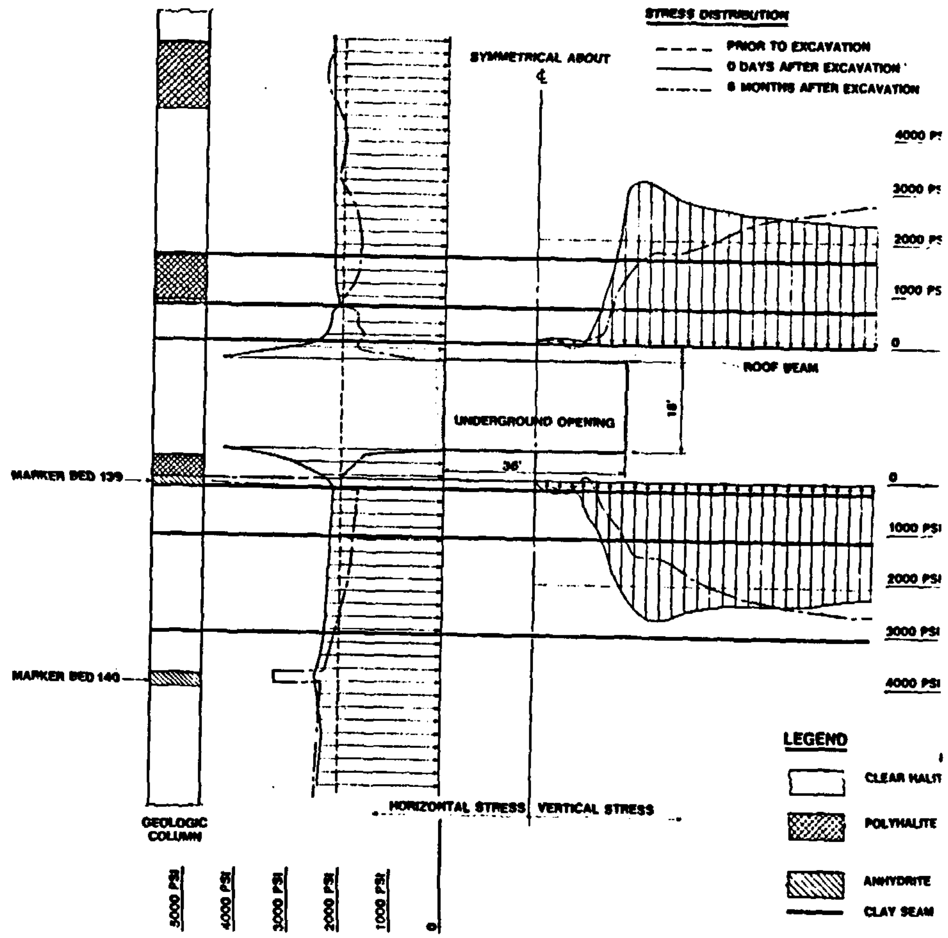

FIGURE 10 STRESS DISTRIBUTION AROUND A HORIZONTAL OPENING

Plastic Deformation. In conjunction with the relatively high stresses, high plastic creep deformations and resulting large convergence of underground openings typically occur in rock salt.

Discontinuities between rock strata. Discontinuites between the rock strata caused by the clay seams in proximity to roof or floor of an underground opening can be expected to affect the immediate roof or floor of the opening. To diminish the effect of the discontinuity induced into the layered deposits by clay seams, the room elevation was carefully chosen to ensure that the roof beam and floor beam of the opening were adequate (Figure 11).

Non-uniformity of the Rock Mass. Most methods of analysis, including finite element methods, are based on the assumption that the rock mass is uniform within a defined rock stratum and that, for example, the immediate roof or floor is free from cracks. Because of the non-uniformity of the rock mass, the occurance of zones of weakness or cracks should be taken into consideration. The cracked roof beam behaves like an arch. This behavior is different from that of solid beams and insights gained from the analysis of solid beams may not be valid.

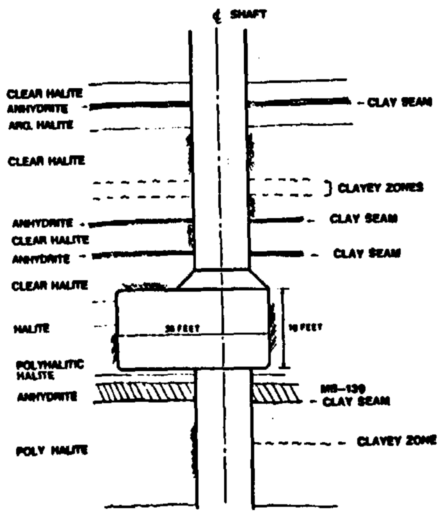

FIGURE 11 DISCONTINUTIES CAUSED BY CLAY SEAMS AROUND AN UNDERGROUND OPENING

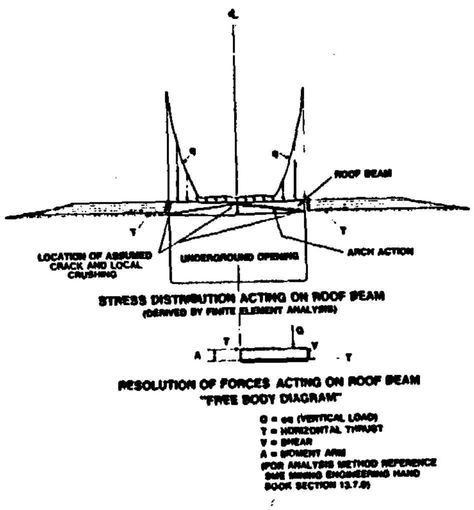

FIGURE 12 ARCHING ACTION OF ROOF BEAMS IN AN UNDERGROUND OPENING

Stability analyses of the roof and floor beams for each opening of the WIPP facility were prepared taking the above described factors into consideration. In addition to the analysis of the openings using beam action and the assumption of an uncracked section, arching action in roof beams was also investigated assuming that cracks may develop in the roof or floor (Figure 12).

Once a crack has developed, such a beam can fail in three ways. If the horizontal thrust (T) is not great enough, the blocks of rock could simply slide down and the roof would collapse. A second possible mode of failure is by the rock crushing at points of high compressive stress, permitting the rotation of blocks and consequent collapse. A third possible mode of failure is by elastic buckling where the rock at the abutments and center could deform to such a degree that blocks can rotate without exceeding the crushing strength of the rock at point of rotation.

In addition to these analyses, convergence measurements are made in the underground openings. Most of the potash mines use instrumentation, such as extensometers and convergence measurement points, to monitor the stability of underground openings. Records of their instrumentation measurements (for example Ref. 2) are being compared with the measurements obtained from WIPP instrumentation to evaluate the performance of the WIPP underground openings.

UNDERGROUND DEVELOPMENT METHOD

After completion of the construction of the exploratory and ventilation shafts, the underground development contractor pursued the following objectives:

- Connect the exploratory and ventilation shafts as soon as possible to facilitate ventilation
- Provide a secondary escape way to the surface in order to comply with mine regulations without restricting the number of people utilized underground
- Produce space for the assembly of continuous excavating mining equipment in the exploratory shaft station
- Postpone furnishing of the shaft until the continuous mining machine and one load haul dump unit (LHD) were lowered underground in the unobstructed shaft
- Minimize the utilization of drill and blast method and implement continuous miner excavation
- Develop, as soon as possible, haulage roads to the exploratory shaft station and the skip loading hopper
- Provide space for material storage and temporary mechanical shop

To achieve these objectives the contractor proceeded as follows:

First, the hoisting machine, the steel headframe, the working platforms and its hoisting machine were installed including hoist and guide ropes, bucket and collar doors.

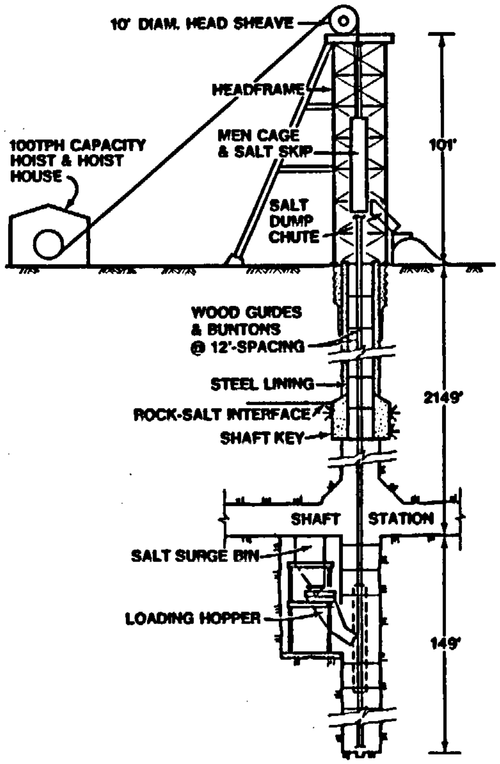

FIGURE 13 GENERAL ARRANGEMENT OF THE OUTFITTED EXPLORATORY SHAFT

After the working platform and the bucket were operational, the area of the shaft wall below the lining at the rock-salt interface was excavated to allow construction of a concrete key 0.76 m (2-1/2 ft) thick and 11.28 m (37 ft) high which was constructed as an integral part of the lining (Figure 7). Two chemical seal rings were placed between the concrete key and the salt wall to prevent any potential migration of water from behind the lining down to the salt formation. The monitoring of water drained through the telltale holes below the seal during the operating life of the shaft would indicate whether there was significant water passage behind the key to the salt formation, and whether injection of grout would be required to prevent leakage and to maintain the integrity of the shaft. Then, the working platform and hoisting bucket were lowered to the exploratory shaft station and the excavation of the shaft station was started, hoisting men and materials in the bucket. The excavated salt was stored in the shaft sump and loaded into thebucket with a Cryderman mucking machine. After space was gained in the shaft station, an Eimco overhead mucking machine and a LHD unit, 2.3 cubic meter (3 cubic yard) capacity, was lowered into the station. A 6.1 m (20 ft) wide by 2.75 m (9 ft) high drift toward the ventilation shaft was excavated by drill-and-blast method, connecting the two shafts. Subsequently, the Dosco LH 1300 continuous miner was lowered and assembled in the shaft station. While the Dosco mining machine was assembled, the shaft sump was enlarged and deepened. After the completion of the above-mentioned

mining operations, the furnishing of the shaft station and the shaft was completed with the installation of the 72.6 metric ton (80 ton) capacity salt surge bin, steel buntons, wooden guides and skip, thus getting ready for a continuous miner production (Figure 13). All work prior to this stage was done using the shaft bucket as conveyance and for hoisting of salt.

EXPLORATORY SHAFT STATION

The shaft station was excavated to provide the following:

- A salt transfer installation from the underground material handling system to the salt hoist skip
- Adequate space for unloading material and supplies from the shaft, for initial mining excavating equipment and initial electrical equipment providing power and light to workings and equipment
- An air plenum space to decrease air velocity of fresh air sent down the exploratory shaft to ventilate workings

The exploratory shaft station was excavated from the exploratory shaft, using a drill-and-blast method (Figure 14).

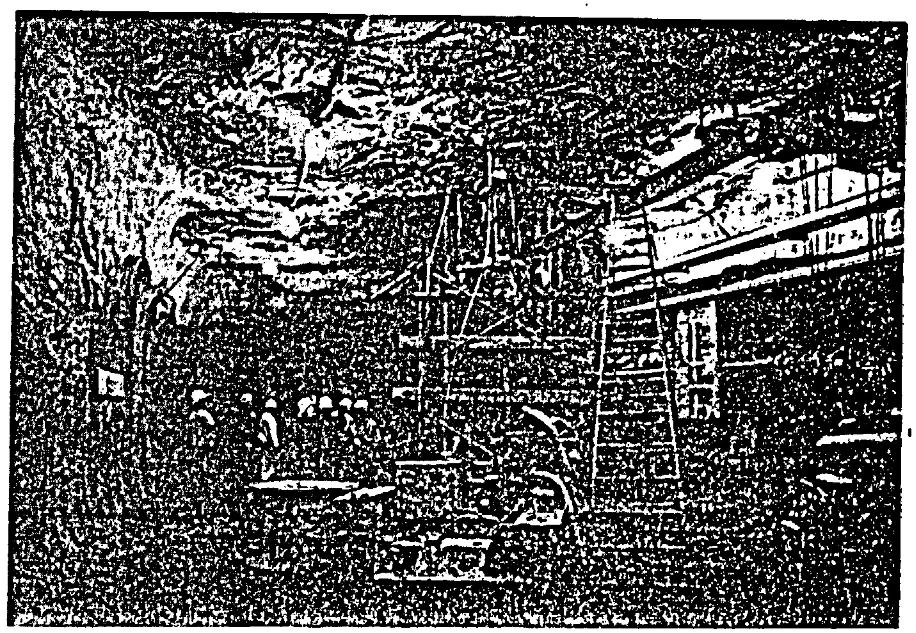

FIGURE 14 EXPLORATORY SHAFT STATION. MAN ON SCAFFOLD IS INSTALLING AN EXTENSOMETER IN THE ROOF.

The excavation to the south was done in several stages. The upper portion was started with a pilot drift 3.66 m (12 ft) wide by 2.75 m (9 ft) high. Fifteen blasting rounds were fired to advance the drift 16.5 m (54 ft) to the south. Once this was completed a parallel drift, 3.66 m (12 ft) wide by 2.74 m (9 ft) high, was blasted on the west side of the station slabbing the rock into the pilot drift. The remaining portion of the station was excavated by further slabbing of the west side into the top bench (Figure 15).

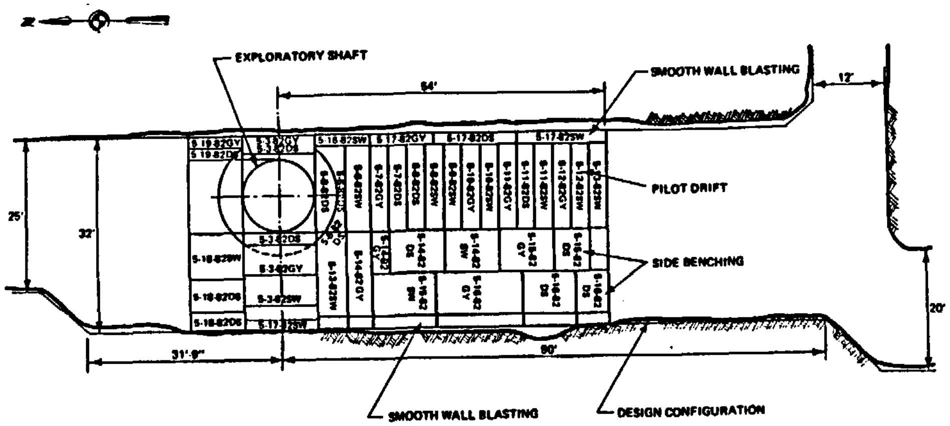

FIGURE 15 EXPLORATORY SHAFT STATION TOP BENCH BLASTING SCHEDULE

After completion of the excavation of the exploratory shaft station top bench, the station was deepened in the immediate area of the shaft to full station height. At the same time as the station was excavated, a slot 3.66 m (12 ft) wide by 2.74 m (9 ft) high was driven along the west side of the station, 16.5 m (54 ft) long.

At this point the excavation of the drift connecting to the vent shaft in the south started. The drift was 3.05 m (10 ft) wide by 2.44 m (8 ft) high.

This drift provided the first connection between the exploratory shaft station and the vent shaft. The drill and blast drift was excavated, leaving one foot of salt on either wall and also 0.6 m (2 ft) of roof salt and 0.3 m (1 ft) of floor salt.

The Dosco LH 1300 continuous miner was used for final trimming of the walls, roof and floor to obtain final excavation dimensions of the drift. The final dimensions of the west and east walls of the station were obtained by line drilling along the walls and smooth wall blasting, to reduce overbreak. The lower southern portion of the station was drilled vertically and bench blasted toward the north, throwing the rock toward the exploratory shaft, bench height is 2.44 m (8 ft).

Excavation of the station toward the north was done by a similar method: a pilot drift 3.66 m (12 ft) wide by 2.75 m (9 ft) high in the upper portion, slabbing of the west side into the pilot drift and then line drilling and smooth wall blasting. The lower portion of the station was vertically drilled and bench blasted toward the exploratory shaft. The bench height was approximately 2.75 m (9 ft); a total of 13 blast rounds was fired.

Excavation of the station required several different blast hole patterns. In the pilot drift the blast rounds were based on standard blasting rounds for drifts. For the bench-type blasting the rounds were based on standard blasting rounds for slabbing.

The blast holes were drilled 4.13 cm (1-5/8 in) in diameter and 2.44 m (8 ft) in depth. Drifting rounds consisted of a modified burn, or V-cut. The 3.66 m (12-ft) wide by 2.75 m (9-ft) high pilot drift had approximately 52 blast holes and four 7.62 cm (3 in) diameter holes for the burn cut. The slabbing rounds had 16 production blast holes and 8 line holes to diminish overbreak. The explosives used were cartridge water gels, 3.81 cm (1 1/2 in) in diameter, initiated by electric detonators placed in the back of each hole loaded.

Excavation of the upper portion of the exploratory shaft station was started on May 2, 1982 and was completed after 18 days. Excavation of the lower portion was started on May 20, 1982 and was completed after 24 days. Excavation of the station was completed on June 16, 1982.

ROCK BOLTING

The roof of the exploratory shaft station is secured with rockbolts. The rockbolts were installed while excavating the upper portion of the station. After each 2.44 m (8 ft) deep blasting round, the roof was scaled and 4.76 cm (1 7/8 in) diameter holes were drilled into the roof on a 1.83 m by 1.83 m (6 ft by 6 ft) pattern. Rock bolts made of No. 8, grade 60 rebar, 2.44 to 3.05 meter (8 to 10 ft) long resin grouted, were installed in the roof. The resin-grouted roof bolts were installed with hand-held air-powered impact wrenches. Five cartridges of resin were installed with each rock bolt. A number of these bolts had to be cut off, redrilled and reinstalled due to the fast setting time of the resin. Hand-held impact wrenches used to install the bolts did not provide enough torque to rotate the bolts into the hole filled with resin.

During the excavation of the lower portion of the shaft station it was noted that the roof required frequent scaling due to the occurence of slabs. In addition, the threaded portion of several installed rockbolts broke off. The damaged rockbolts were replaced with new bolts. Of the total of 420 rock bolts installed in the station, about 30 bolts have failed by breaking at the threads.
Bolt failure may have been caused by the following:

- Grade 60 rebars were used as rockbolts. The threaded part of bolts may be work-hardened and brittle due to the threading technique employed.

- Rock/creep deformation may have stressed the bolts to yield strength and may have exceeded the capability of the bolt in some cases (Figure 16).

- Installation details, including the lack of beveled washers and improper nut torqueing, may have resulted in the bending of the bolts at the roots of the threads.

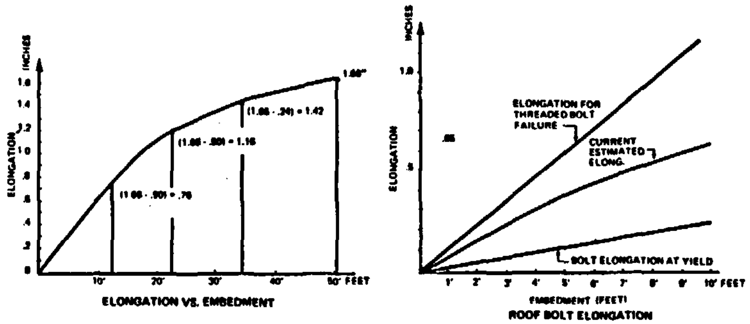

FIGURE 16 MEASURED ROOF BOLT DEFORMATION

While drilling into the middle third of the roof it was noted that the roof apparently separated along the first clay seam located approximately 0.92 m (3 ft) above the immediate roof of the station. The separation of the 0.92 m (3 ft) thick roof beam showed a measurable gap. At that time, a decision was made to rebolt the entire roof of the station. Rock bolts 3.05 to 3.66 m (10 to 12 ft) long, on a 0.61 m by 0.61 m (2 ft by 2 ft) pattern, with wooden block 15.24 cm by 15.24 cm (6 in by 6 in), 6.35 cm (2-1/2 in) thick, were installed, to secure the roof.

INSTRUMENTATION

To monitor the ground movement in the station and specifically the behavior of the 0.92 m (3 ft) thick roof slab, geomechanical instrumentation was installed (Figure 17).

Additional instrumentation is installed in the north and south drifts and test rooms. The instrumentation consists of multi-point extensometers, inclinometers, rockbolt load cells, and convergence pins. The purpose of the instrumentation is to monitor the ground movement and closure rates. Readings are taken on a regular basis.

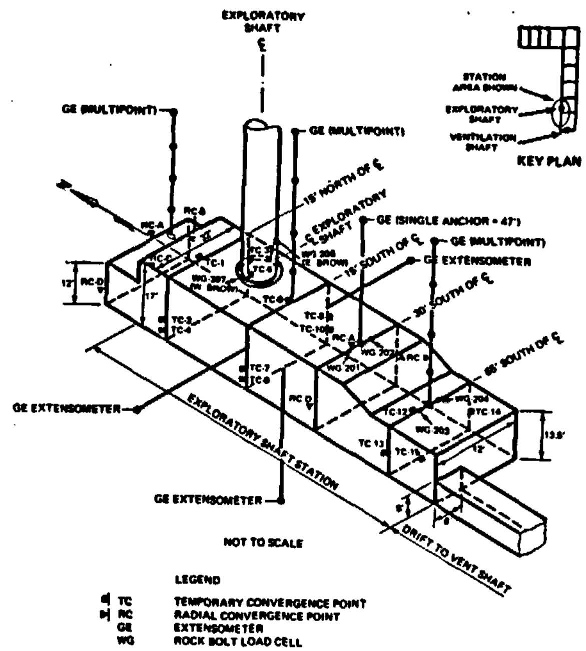

FIGURE 17 INSTRUMENTATION LOCATION EXPLORATORY SHAFT STATION

Holes cored for the instrument installation also provided geologic information on the salt strata surrounding the opening.

CONTINUOUS MINER EXCAVATION

The drifts were excavated utilizing a Dosco LH 1300 boom-type continuous mining machine. The cutting width of the machine is 5.7 m (18.67 ft) and the cutting height is 3.6 m (11.75 ft). The excavation of the 7.63 m (25 ft) wide and 6.1 m (20 ft) wide by 2.44 m (8 ft) high drifts was done in two passes, cutting 3.81 m (12.5 ft) wide and approximately 13.73 m (45 ft) deep. The excavation of the 12 feet wide by 9 feet high drifts and crosscuts was done in one pass (Figure 18). The general specification for the Dosco LH 1300 continuous miner is given below:

Miner energized by a 1,000 volt trailing cable.

Overall weight	36.3 metric tons	(40 tons)
" width	3.0 meter	(9' 10")
" height	2.06 meter	(6' 9")
" length	11.9 meter	(39')
Tramming speed	19.2 m/min	(63 ft/min)

Cutting speed	2.44 m/min	(8 ft/min)
Loading capacity	11.47 m^3/min	(15 cyds/min)
Conveyor width	43 cm	(17")
Total installed h.p.	269 kw	(360)

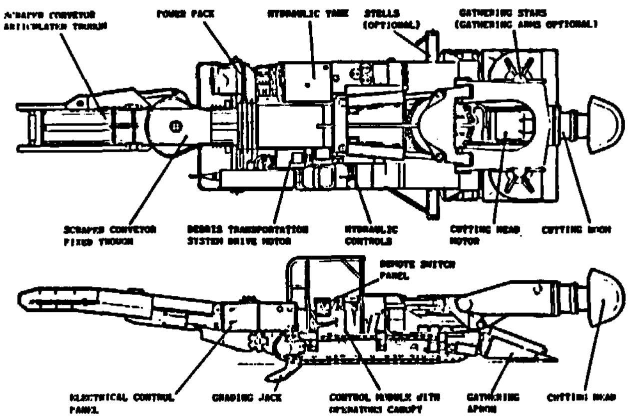

FIGURE 18 DOSCO MINER

The excavated salt was loaded by the conveyor of the continuous mining machine directly into diesel-powered LHD, later on into trucks (Figures 19 and 20). The haulage equipment consisting of a LHD and two trucks was manufactured by Eimco.

Trucks were model ST-980, payload 10 tons

Width	2.0 meter	(6' 8")
Length	6.2 meter	(20' 2")
Height	1.78 meter	(5' 10")
Weight	7,892 kg	(17,400 lbs)

Diesel engine, Deutz F6L-714-, 101 kw h.p. 135
USBM-approved Schedule 24 for non-gassy mines
Fresh air requirements 425 m^3/min 15,000 cu ft/min

To clean the production face from debris a 3 cubic yard LHD unit was used. The LHD was a Model 913 with a 110 h.p. Deutz diesel engine

Length	4.9 meter	(15' 4")
Height	1.52 meter	(5')
Width	1.83 meter	(6')
Weight	12,750 kg	(28,000 lbs)

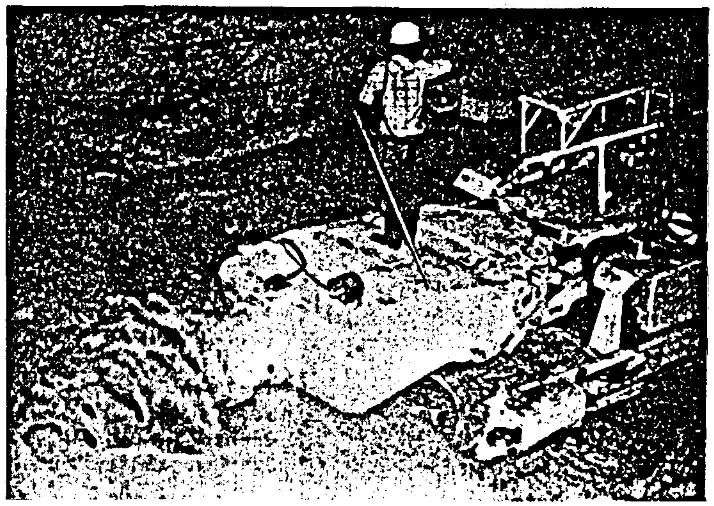

FIGURE 19 DOSCO FRONT END SHOWING BOOM EXCAVATION

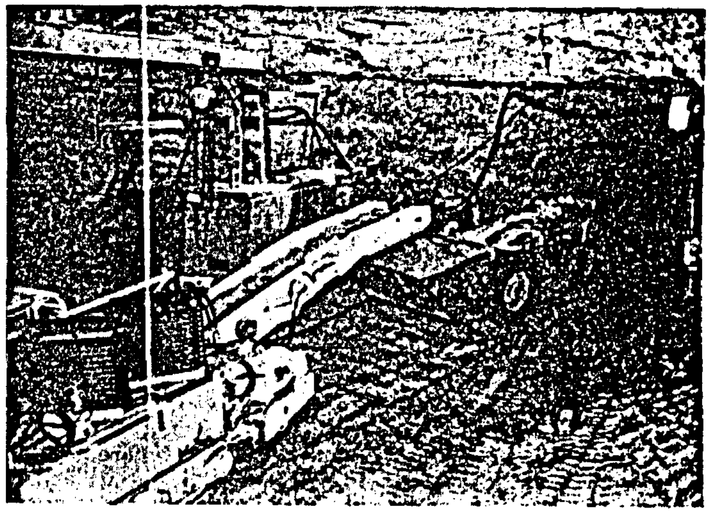

FIGURE 20 TAIL END OF DOSCO MINER LOADING INTO LHD

NORTH DRIFTS

For the Site and Preliminary Design Validation Program (SPDV) two drifts and four test rooms are to be excavated toward the north (Figure 21).

FIGURE 21 TYPICAL DRIFT EXCAVATION

The west drift starts at the vent shaft crosscut. The drift is 6.1 m (20 ft) wide by 2.44 m (8 ft) high and 90.9 M (298 ft) long. At this point the drift connects to the exploratory shaft station. The drift extends north beyond the exploratory shaft at a width of 7.72 m (25 ft) by 2.44 m (8 ft) high and has a total length of 440 m (1,440 ft).

The east drift starts also at the vent shaft crosscut. The drift is 3.66 m (12 ft) wide by 2.44 m (8 ft) high and has a total length of 561 m (1840 ft). The west and east drifts are interconnected by crosscuts at intervals of 94 m (308 ft).

Excavation of the two drifts projected to the north, and the non-waste experimental test rooms is not completed at the time of this writing (Feb. 1983). Excavation advanced north about 61 m (200 ft).

SOUTH EXPLORATORY DRIFT

Excavation of the south exploratory drift starts at the vent shaft crosscut and proceeds toward the south. The total length of this drift is 995 m (3,264 ft), it extends through the length of the shaft pillar area 366 m (1,200 ft) and into the future waste storage area 629 m (2,064 ft). The drift is 7.62 m (25 ft) wide and 2.44 m (8 ft) high. It was excavated utilizing a Dosco LH 1300 boom-type continuous mining machine.

Excavation of the exploratory drift south started on December 2, 1982. Excavation was completed on February 1, 1983 in a total of 51 construction days.

PRODUCTION RATES

During the initial excavation of the north drift, during machine start up, the daily rate of production was erratic and low. Because of frequent machine breakdowns and necessary repairs and adjustments to the hydraulic installation, the continuous mining machine produced in average only 272 metric tons (300 tons) per day during the start up period.

After completion of the start up period, during excavation of the south drift on the average, 290 metric tons (320 tons) of salt were excavated per shift and 870 metric tons (960 tons) per day. The maximum production rate per day was 1,180 metric tons (1,300 tons).

VENTILATTON

After the connection was made to the ventilation shaft, fresh air was downcasted through the exploratory shaft and exhausted through the 1.83 m (6 ft) diameter vent shaft. An exhaust fan was installed at the surface, exhausting air through the vent shaft. The fan capacity was 1,416 cubic meter per minute (50,000 ft^3/min).

The production faces in the west, east and south drifts and crosscuts were ventilated using an exhaust-type ventilation method. At all times fresh air traveled in the drift, the return air was exhausted through a vent compartment erected in the drifts. The vent compartment was built out of brattice cloth fastened to the roof and floor of the drift, thus creating a partition. The brattice cloth partition was erected approximately 0.76 m (2.5 ft) away from the rib.

Once sufficient space was excavated, permanent fans were installed underground. The fans were located near the vent shaft station, exhausting into the vent shaft. Temporary bulkheads were used to isolate the fans and direct the air flow.

PRESENT STATUS OF COMPLETION

Excavation of the shafts, drifts and test rooms included in the Site and Preliminary Design Validation Program (SPDV) (Figure X) is scheduled to be completed in the summer of 1983. The full WIPP facility is scheduled for completion in 1987.

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Chapter 31

SINKING THE SILVER SHAFT

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ABSTRACT

For many major mining projects the time frame from initial planning to production can be measured in terms of decades. Hecla's Silver Shaft project, which is scheduled to go into production by the end of this year and includes a complete surface facility, shaft with an ultimate depth of 2,348 meters (7,700 feet), complete ore pass and loading pocket system, and 610 meters (2,000 feet) of development drifting and stope preparation, was initially concieved in the Fall of 1979. This duration of a mere four years from conceptual engineering to production is a testimonial to what can be accomplished when a progressive mining company and an innovative contractor work together as a team rather than developing into two warring camps as often happens.

INTRODUCTION AND ENGINEERING

In the Fall of 1979, Hecla Mining Company awarded a contract to J. S. Redpath Corporation to perform feasibility investigations and design engineering related to a proposed new shaft facility at the Lucky Friday Mine.

The Lucky Friday Mine is located at Mullan, Idaho. It is a major producer of silver, as well as significant quantities of lead and some zinc.

In late 1979, stoping was in progress down to the 4,600 level. The existing main shaft had been developed to the 5,000 level. At that depth the shaft had reached the ultimate capacity of the existing hoist. Further deepening of the shaft would require a major new installation. This alternative was also unattractive due to poor ground conditions which resulted in substantial repair and associated down time. The Lucky Friday ore body was projected to extend to depths substantially below those accessible through the existing shaft. Hecla also anticipated an increase in mining rate over that permitted by the existing facility. These factors dictated consideration of a new facility capable of producing 2,000 tons per day from an ultimate mining depth of 2,285 meters (7,500 feet).

The scope of the engineerig program included the evaluation, selection and detailed design of the shaft facility, stations, loading pockets, hoist plant, headframe and various surface ancillary systems.

An important element was the selection of the shaft configuration and design of the shaft lining. The rocks of the Couer d'Alene district are geologically complex and are subject to high tectonic stresses. Severe ground conditions are encountered and rockbursting is experienced in the brittle quartzitic rocks of the district. Redpath was directed to evaluate the technical feasibility and economics of various circular concrete and rectangular timbered approaches to this project.

All previous shafts in the district were rectangular in section, equipped either with timber or steel sets. There was a widely held belief, on the part of operators experienced in the Couer d'Alenes, that a circular concrete shaft facility would fail under the combined affects of difficult ground and high horizontal (tectonic) stresses. This opinion was reinforced by some limited and unsuccessful experience with the use of concrete in short sections of several existing shaft facilities. Shaft repair resulting from ground squeeze and movement was also an accepted fact of life in the district. This contributed to the opinion that the repair was inevitable and, if so, it was a cheaper and simpler procedure for a timbered shaft than would be the case with a concrete-lined opening.

In addressing the technical and economic aspects of the shaft type selection, three (3) principal areas were investigated:

1. Rock Mechanics

   The geology and available rock data were reviewed. This led to definition of the loadings that would be imposed on the shaft opening and lining system.

2. Lining Design

   The behavior of various proposed lining designs was evaluated in the context of the rock mechanics and geological information.

3. Economics

   The capital and operating costs of various circular concrete and rectangular timbered facilities were compared. Various untangible factors were also evaluated.

It was concluded that a circular concrete configuration was technically feasible and that it represented the most economic combination of capital and discounted long-term operating costs. Various intangible factors also favored this selection, including operational reliability and ease of maintenance, resistance to fire, superior ventilation characteristics and contruction schedule. Rock mechanics and geotechnical evaluation indicated that the proposed concrete lining would withstand the loadings. The facility would require less maintenance than the existing Lucky Friday shafts and other facilities in the Couer d'Alenes. The details of the engineering that led to this conclusion are beyond the scope of this paper. The analysis can, however, be summarized by reference to Figure 1. Horizontal tectonic stresses ranging up to 760 KG/CM² (10,800 PSI) have been measured at the Lucky Friday. Maximum stresses of around 630 KG/CM² (9,000 PSI) were predicted at a depth of 2,285 meters (7,500 feet). There are significant differences between the maximum and minimum principal stresses with the ratio ranging between 1.25 and 2.75. Although the maximum principal stress at the shaft site is generally oriented in a northwest-southwest direction, its direction varies widely with changes in structure at differing elevations. Very high and potentially troublesome stress concentrations develop at the corners at any rectangular opening, even if it is oriented parallel to the direction of principal stress. Predicted loadings indicated probability of some failure and associated long term repair expense even where the ideal shaft centerline/principal stress orientation was maintained. This condition would be magnified in those sections of the shaft where this relationship could not be maintained. In contrast, the circular concrete lined alternative would adequately handle these conditions irregardless of the orientation of principal stress.

The selected configuration is 5.5-meter (18-foot) diameter shaft with a nominal 0.30-meter (1-foot) concrete lining. The shaft has four (4) compartments; consisting of two (2) 1.88-meter by 1.88-meter (6-foot 2-inch by 6-foot 2-inch) hoisting compartments, a 1.57-meter by 2.54-meter (5-foot 2-inch by 8-foot 4-inch)

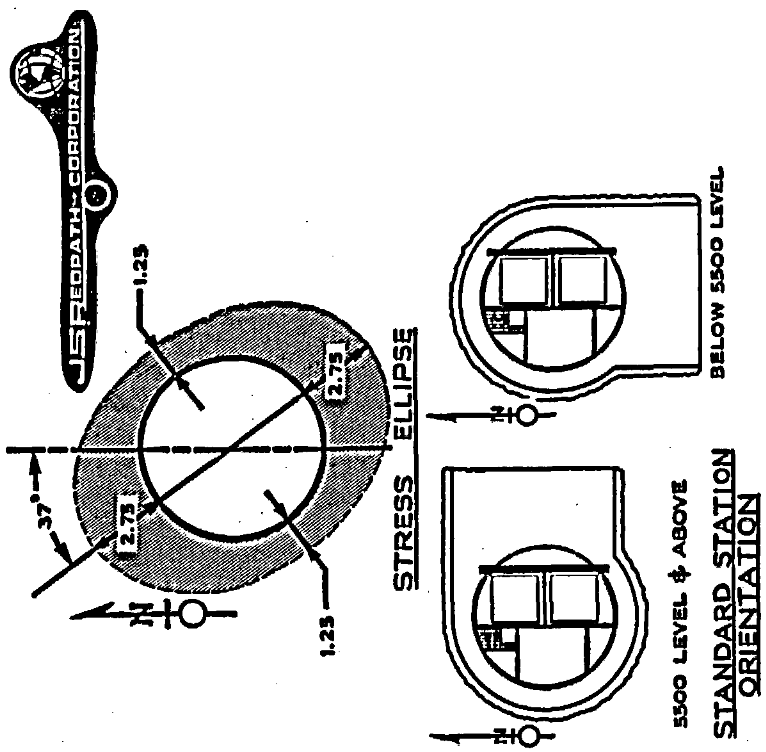


FIGURE 1

compartment for a future service cage and a manway. Skip compartments are equipped with 127-millimeter by 152-millimeter (5-inch by 6-inch) tubular steel guides. These provide an extremely smooth and low maintenance guide system. Shaft configuration in the sinking phase is shown in Figure No. 2.

The production hoist is a double-drum (3.66-meter) (12-foot) diameter, 2,236,000 Watt (3,000 HP), direct current drive unit equipped with thyristor convertors. Hoisting speed is 686 meters (2,250 feet) per minute. Hoist rope is 44.4-millimeters diameter (1-3/4 inch). Rated capacity, from the 7,500 level, is 125 tons per hour.

The general arrangement concept and shaft configuration were finalized in December 1979. Detailed engineering commenced in January 1980. During this period a contract for construction of the plant and sinking of the shaft was awarded to Redpath. To comply with the scheduled start of on-site construction operations in April 1980, it was necessary to perform the detailed design and procurement on a fasttrack basis.

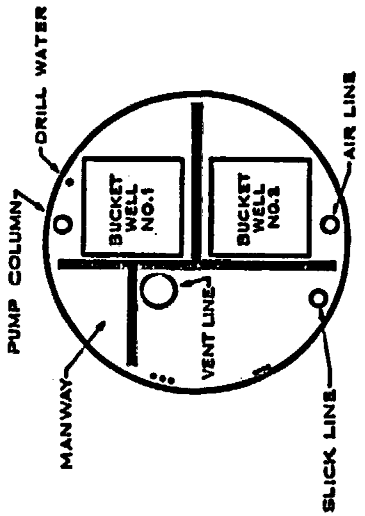
Ultimate depth of the shaft will be 2,350 meters (7,700 feet), with mining levels on 61-meter (200-foot) intervals beginning at the 4,900 level. Loading pockets will be installed below every third mining level. Bored ore passes will connect each set of three (3) levels. This is illustrated Figure 3.

EQUIPMENT SELECTION

While detailed engineering was in progress, planning, specification and procurement in preparation for shaft sinking were initiated. Long lead time items such as hoist and headframe were ordered on a fast-track basis as engineering data became available.

Hoisting Plant

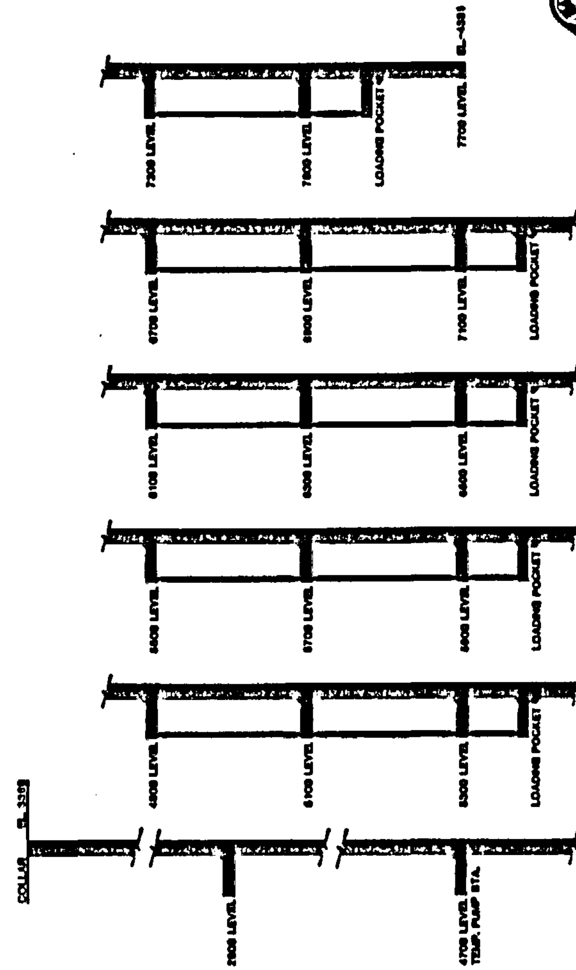
The hoisting plant is the life line of a shaft sinking operation. A schematic of the Silver Shaft hoisting system is shown in Figure 4. The three (3) stage winches are located in front of the sinking hoist, with their ropes going over sheaves on the second deck of the headframe. The sinking hoist is located behind the stage winches. Sheave wheels for the sinking ropes are on the top deck of the headframe. These buckets form the main access to the shaft, as the work deck is held captive below the steel sets as they are installed.



PLAN OF 18' DIAMETER SHAFT

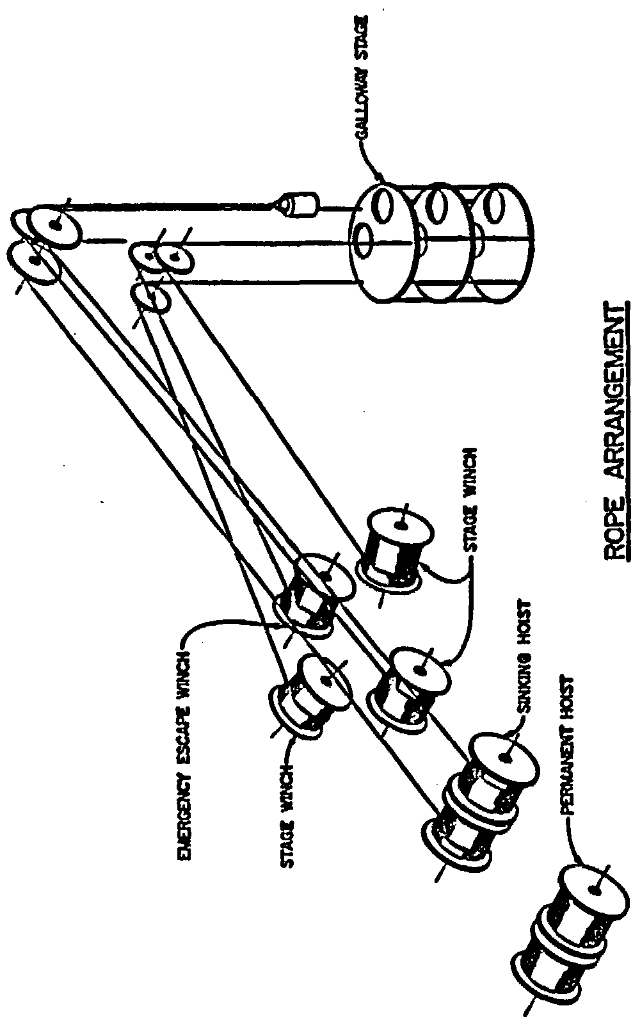
JS REDPATH CORPORATION

FIGURE 2



JS REDPATH CORPORATION

FIGURE 3



ROPE ARRANGEMENT

FIGURE 4

The permanent production hoist is located directly behind the sinking hoist and will utilize the same sheave wheels in the headframe.

Headframe

Supporting the sheave wheels is the headframe. The Silver Shaft headframe is 41 meters (135 feet) high and contains nearly 270 tons of steel. This will serve as the permanent headframe. It also incorporates dump doors and dump chutes required during the shaft sinking phase.

Stage Winches

The sinking stage and mucker are supported by three (3) ropes which are attached to three (3) stage winches. Each has a 2.0-meter (6-foot 8-inch) diameter drum with a 2.0-meter (6-foot 8-inch) face. They will accommodate 2,500 meters (8,200 feet) of 39.7-millimeter (1-9/16 inch) rope. Line pull is 310,000 newtons (70,000 pounds). Hoisting speed is 7.6 meters per minute (25 feet per minute). They are equipped with 56,000 watt (75 HP) motors and disc brakes. A fourth winch was installed as an emergency escape device for use in the event that the main hoist was disabled. This fourth winch is powered by its own generator in case of a power outage at the site. It is also used for shaft cable installation.

Sinking Hoist

Due to the long lead time of the permanent hoist, which was designed and built by Rexnord and Westinghouse, a temporary sinking hoist was used during sinking of the first 1,525 meters (5,000 feet) of shaft.

This is a Rexnord double-clutched hoist powered by a 930,000 watt (1,250 HP) DC motor. Drums are 2.7-meter (8-foot 10-inch) diameter with 1.93-meter (76-inch faces) capable of spooling 1,585 meters (5,200 feet) of 38.1-millimeter (1-1/2-inch) rope. The sinking hoist had a line pull of 204,600 newtons (46,000 pounds) at 550 meters per minute (1,800 feet per minute) and was equipped with Westinghouse solid state thyristor controls. This hoist proved to be very reliable throughout its service and minimal down time was experienced.

Permanent Hoist

The permanent hoist is currently being used for sinking. It is powered by two (2) 1,118,000 watt (1,500 HP) DC motors and has two (2) 3.66-meter (12-foot) diameter drums, with 2.44-meter (8-foot) faces. It also is double-clutched and is equipped with parallel post type brakes. This hoist is capable of spooling 2,350 meters (7,700 feet) of 44.0-millimeter (1-3/4-inch) rope on each drum and has a line pull of 344,700 newtons (77,500 pounds) at 685 meters per minute (2,250 feet per minute). Again, this hoist was Westinghouse solid state thyristor control. The permanent hoist is installed directly behind the temporary hoist and utilizes the same sheave wheels in the headframe.

Galloway

The work deck, shown in Figure 5, was also designed and ordered early in the course of the project. The sinking stage has three (3) decks. The tope deck is utilized during steel installation. The other two (2) decks are employed for the stripping, lowering and setting of the 4.57 meter (15-foot) concrete forms. Spacing of the three decks is a 4.57 meters (15 feet), to match the concrete forms and the steel spacing. The bottom deck provides a work area and is the suspension point for the mucking unit.

Mucking Unit

The cactus grab is supported from the center of the stage on a rotating bearing. The operator's console hangs directly under the bottom deck of the stage and is mounted on one end of a tracked beam. On the other end of the tracked beam rides a large air-powered tugger. The tugger is moved back and forth in a horizontal path by a large air cylinder. The beam is rotated beneath the stage by means of two (2) swing motors. Suspended from the tugger is the 0.60 cubic meter (21 cubic foot) cactus grab unit. This mucking system was selected both for its rapid muck handling capability and its versatility in mucking either full-face or bench type shaft rounds. It also has the advantage of being suspended above the shaft bottom and is not hampered by wet or sloppy conditions on the shaft bottom.

Total weight of the galloway with the mucking unit is slightly under 35,000 kilograms (78,000 pounds).

Concrete Forms

Concrete is poured in 4.57-meter (15-foot) lifts. Forms consist of a 0.91-meter (3-foot) curb ring, two (2) 1.83-meter (6-foot) wall sections and a match ring for pouring closures. Form length was dictated by the 4.57-meter (15-foot) set interval. Inserts for the sets are incorporated into the curb ring. Forms are supported by five (5) 25.4-millimeter (1-inch) diameter threaded hanging rods.

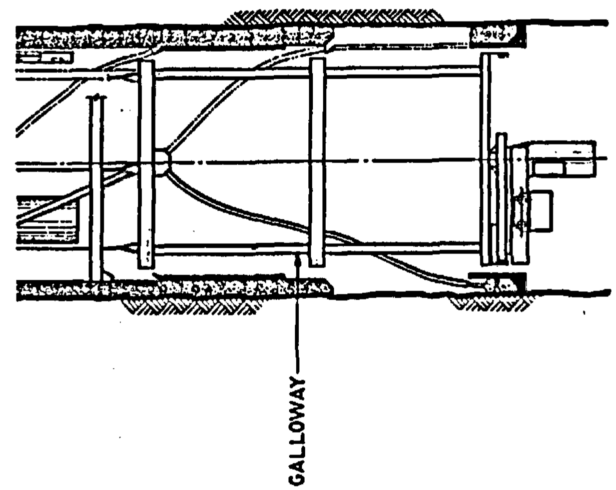


FIGURE 5

Drill

A benching method was selected for excavation of the shaft, reasons for which will be discussed under the sinking part of this paper. Selection of this method dictated use of sinking hammers, or pluggers, as opposed to a jumbo drilling. Pluggers have the advantage of being versatile with good availability and have a very quick set-up time. The major disadvantage of pluggers is the difficulty of pulling steel in bad ground. This has not presented too much of a problem as the rock, for the most part, has been suitable for drilling with hand held machines.

MOBILIZATION AND SURFACE CONSTRUCTION

On-site operations commenced April 1, 1980. At that time, the site had been cleared and leveled by Hecla. In a little over six (6) months an operable sinking plant including headframe, hoist houses, offices, shops, etc., were erected on the site and sinking operations had commenced. Work accomplished during this six (6) month period is described in this section.

Collar

The shaft collar was excavated using a crane and an Eimco 630 overshot mucker to a depth of 32.6 meters (107 feet), where competent rock was encountered. This excavation was temporarily supported by 6-meter (20-foot) diameter liner plate and ring beams. Grout was placed behind this liner plate affording a secure anchor into the surrounding overburden. After completion of the collar excavation and the installation of the temporary support, the steel shaft forms were lowered into the collar and the final 5.5 meter (18 foot) concrete lining was placed. Rebar and anchoring eyes for the headframe were installed and the collar and the headframe foundation were poured simultaneously.

Galloway Erection

With the completion of the concreting of the shaft collar and headframe foundation, assembly of the three (3) deck shaft work stage began. The carrier and the support beam for the mucking unit were assembled in the bottom deck of the stage. The stage was lowered into the shaft and then the other two (2) decks were attached to the main vertical supports.

Headframe Erection

Following installation of the stage, the collar was bulkheaded for safety reasons and headframe erection commenced. Headframe

procurement and erection represented the critical path from mobilization to shaft sinking. Erection of the headframe required 105 days.

In parallel with collar construction and headframe erection, the remainder of the surface plant was installed and commissioned. This included underground utilities, concrete pads for the project buildings and erection of prefabricated insulated buildings providing shop facilities and dry and a temporary hoist house.

Hoist and Winch Installation

The other major activity occurring at this time was the excavation, forming and pouring of the concrete foundations for the temporary hoist and for the three (3) large stage winches. The temporary sinking hoist was delivered in June 1980. Mechanical installation of the hoist began on 3 July 1980, and was completed 5-1/2 weeks later. The hoist was fully commissioned on 6 September 1980.

After the galloway winches and the emergency winch were set on their foundations, the sinking hoist and the winches were roped up durng the first week of October 1980. As final preparations for shaft sinking were completed, air and water headers were incorporated into baskets for lowering the jackhammers to the shaft bench, and the shaft bentilation system was installed. Sinking buckets and crossheads were suspended from the hoist ropes and headframe. On 15 October 1980, exactly 6-1/2 months after the start of on-site operations, the first bench was taken in the Silver Shaft. On 20 October 1980, sinking commenced on a three shift per day basis ending the mobilization phase of the job and starting the shaft sinking phase.

SHAFT CONSTRUCTION

The next part of the presentation will illustrate how the previous portions fit together into an efficient shaft sinking operation. The work which goes on in the shaft is cyclical in nature, and the success of the project, as measured by the rate of progress in the shaft, is completely dependent on the details of each cycle and dovetailing the different cycles together. For our bird's-eye view of shaft sinking, we will assume that the shaft crew is just completing the installation of a steel set in the shaft. Please refer to Figure 6 which represents the bottom 25 meters (80 feet) of the shaft where all shaft construction activity occurs.

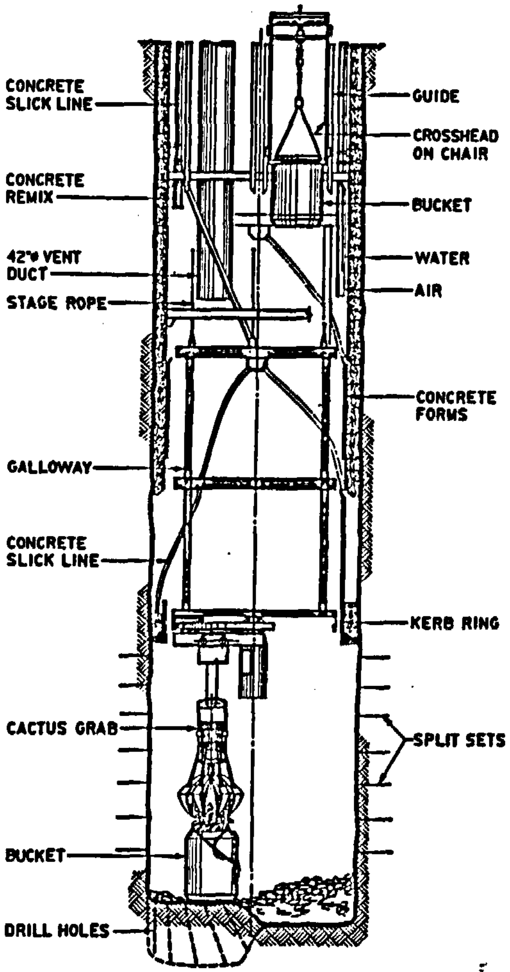


FIGURE 6

Steel installation is done from the top deck of the stage. Wall brackets are bolted to inserts which have been cast into the concrete, and the steel dividers are then bolted onto these brackets. The tubular steel guides, which enable high speed travel in the shaft, are then bolted to the steel dividers and lined up to within fractions of a meter. Utilities, consisting of pipes and electrical cables, are also bolted to brackets for which inserts are poured into the concrete. Alignment of the steel, guides and utilities is done by plumb lines which hang from the surface of the shaft; these, of course, are checked on a regular basis.

To date 394 steel sets have been installed for a total of 791,940 kilograms (1,773,000 pounds) of shaft steel.

As crews complete the steel installation, the last step of which is careful alignment of the guides, part of the crew prepares the concrete forms for pouring. This is done by stripping the bottom 0.91 meter (3 feet) of the concrete form or curb ring, lowering it into position for the next 4.57 meter (15 feet) pour with chain blocks mounted underneath the top deck of the galloway and installing five (5) 25.4-millimeter (1-inch) diameter hanging rods. These hanging rods suspend the curb ring from the previous pour in its proper position and are threaded and provide accurate adjustment for elevation. The curb ring is positioned by means of the plumb lines and is blocked into place with jacks. At this time, the inserts which form the anchors for the wall brackets for steel installation are also attached to the curb ring.

With this completed, scribing pins are placed through the bottom part of the curb ring out to the shaft wall; a floor of expanded metal is placed on top of these pins; and the curb ring is filled with concrete. Concrete is delivered through a 15.2-millimeter (6-inch) slick line which is shown on the left wall of the shaft. Concrete transfers from the slick line into a distribution hopper on the top deck of the galloway. This hopper has distribution hoses running from th bottom of the hopper connecting to the curb ring elevation. Upon completion of the pouring of the curb ring, the wall forms are stripped from the previous pour, lowered by the same set of chain blocks and blocked into place. The top set of hoses can quickly be released from the connecting sleeves and used to fill the remaining forms, or matcher ring, as shown on the right side of the schematic. Pouring operation is repeated and the concrete cycle is completed.

To date 23,729 cubic meters of concrete (31,222 cubic yards) have been poured in the Silver Shaft lining.

The steel and concrete cycle is usually done with a fresh muck pile on the bottom. This serves two purposes: first, the loose muck serves to soak up any spilled concrete and wash water and secondly, the time used in mucking out the broken rock at the bottom of the shaft allows the concrete to take its initial set before any blasting is done, the shock of which might injure the fresh concrete.

Muck removal is accomplished by the use of the cactus grab hanging from the bottom of the stage. This mucking unit is mounted on a bearing in the center of the galloway which allows it to rotate through 350°. The cactus grab itself is slung from a tugger mounted on the end of the muckin unit beam. This tugger travels in and out from the center of the mucking unit to the rib, allowing full coverage of the shaft bottom. The cactus grab loads into the sinking bucket which is then hoisted to surface and dumped by means of the dump doors in the headframe.

To date 64,241 cubic meters (84,528 cubic yards) of broken rock have been removed from the Silver Shaft.

As the shaft walls are exposed while mucking, split-set rockbolts and wire mesh are installed from the top of the muck pile. These provide temporary support and protection for the miners working on the bottom. This support becomes more critical as the shaft depth advances, as the rock in the shaft at depth is subject to high stress buildup which must be relieved prior to the placing of the concrete lining. Rockbolts allow the shaft walls to relieve themselves in a controlled manner.

To date there has been 75,777 linear meters (248,547 linear feet) of rockbolts installed in the shaft wall and 82,309 square meters (885,645 square feet) of wire mesh installed as temporary shaft wall support.

When the muck pile has been removed from the highest part of the shaft or the bench, the drill header is lowered to the bottom of the shaft and holes are drilled into the bench as shown in the lower left portion of the schematic, loaded with explosives and detonated. Length of drill holes varies from 2.43 meters (8 feet) to 3.05 meters (10 feet), therefore, each bechn round advances half of the shaft bottom from 2.43 meters (8 feet) to 3.05 meters (10 feet) giving an overall advance of 1.07 meters (3-feet 6-inches) to 1.37 meters (4-feet 6-inches) per blast. This cycle is repeated until the shaft bottom is again approximately 9 meters (30 feet) from the bottom of the curb ring, at which time the concrete and steel cycles are repeated. Benching has four (4) major advantages over full-bottom excavation:

1. Water handling is easier due to the availability of a sump.

2. Benching is safer, as the bench can be blown clean to check for misfires.

3. The force of the blast is directed into the shaft wall rather than up into the galloway minimizing damage from fly rock.

4. Rockbursts are also minimized as the shaft bottom has a chance to relieve itself in sections.

MINE DEVELOPMENT

During the course of shaft sinking, associated mine development work has also been done. Referral to Figure 3 at the beginning of the paper will give the reader an idea or perspective of the work that has been accomplished.

Shaft Stations

The shaft station which connects to the existing Lucky Friday shaft has been installed at a depth of 854 meters (2,800 feet). A temporary pumping station has been installed at a depth of 1,393 meters (4,570 feet). The first mining level was installed at a depth of 1,494 meters (4,900 feet). This station will be the first development level from the Silver Shaft. Incorporated into this station were chambers which allowed the set-up of a Robbins 82R raise drill for the raiseboring of the ore pass system between the 4,900 and 5,300 levels. Mining levels have also been constructed at a depth of 1,555 meters (5,100 feet) and 1,616 meters (5,300 feet). On the 5,300 level, access was also provided for the attachment of the raise bore reaming head again for the development of the first set of ore passes.

Three other mining levels have also been constructed at depths of 1,677 meters (5,500 feet), 1,738 meters (5,700 feet) and 1,799 meters (5,900 feet). Of these levels, the 5,900 level has been developed to accomodate a deepening station in the event that the shaft is not taken to its ultimate depth in this first drive. This deepening station has been designed to accomodate materials handling, installation of stage winches, methods for transporting shaft muck into the Silver Shaft skipping system, and a dumping system above the 5,900 level.

Loading Pockets

The 5,300 level loading pocket has been excavated and concreted. The loading pocket mechanical items have also been procured and installed and used on an interim basis for muck removal from the ore passes. Other work involved in the loading pocket was the conventional driving of two finger raises from the loading pocket level which sits at a depth of 1,646 meters (5,400 feet) up to the 5,300 level. Final mechanical adjustments are being made to the loading pocket which will enable it to be used for the transfer of development drift muck into the shaft mucking system.

Crews are currently in the progress of excavating the 5,900 level loading pocket which is located at a depth of 1,829 meters (6,000 feet) and will be used at a later date for loading ore from the lower levels into the skipping system.

To date a total of 10,835 cubic meters (14,171 cubic yards) of broken rock have been excavated from the various stations and loading pockets. 3,250 cubic meters (4,251 cubic yards) of concrete have been placed in the stations and loading pockets and 137,170 kilograms (304,823 pounds) of structural steel have been placed in the stations and loading pockets. 32,079 kilograms (71,286 pounds) of reinforcing steel have been placed in the concrete and 33,788 linear meters (110,853 linear feet) of resin rockbolts have been installed for station ground support.

Upon completion of the 5,300 level a Robbins 82R drilling machine was transported to the site by Redpath and lowered into the Silver Shaft at the 4,900 level to construct the first set of ore passes. The machine was set up on the 4,900 level. A 12-1/4-inch pilot hole was drilled to the 5,300 level in each of two locations and an 8-foot reamer head was attached and the pilot hole was back reamed to the 4,900 level. This gave a total of 244 linear meters (800 feet) of 8-foot diameter raise bored ore pass which was constructed in conjunction with other construction activities. Upon completion of the boring of these levels another small headframe, instead of stage winches, was installed. A small galloway was constructed and crews reinforced the walls of the ore passes with 1.5 meter (5-foot) resin bolts (1.2 meter by 1.2 meter) (4-foot by 4-foot) pattern.

Diamond Drilling

Also during the course of construction, a diamond drilling program was carried out on the 4,900 level by Redpath crews to verify the existence of the silver vein that helped deliniate the course of the development drifts that were to be driven at a later date. Three (3) holes were drilled amounting to a total of 662 linear meters (2,172 linear feet) of BX core. Redpath crews accomplished this on a two shift per day basis using a Dymec 250 core drill.

Underground Construction Work

In addition to the mining crews, Redpath has had three construction crews on-site for the last year, which have been involved in mechanical installations and other concrete instructual projects. To date these crews have installed two (2) concrete and steel grizzlies on the 5,300 level and two (2) concrete and steel grizzlies on the 4,900 level. The next project is to be the equipping of the 4,900 level for a track drifting operation. This will enable mining crews to start driving development drift toward the silver vein concurrently with the shaft sinking effort. During this work, muck will be stored in the ore passes; when the ore passes are filled, shaft crews will be moved to other work for a period long enough to pull the ore passes when shaft sinking and drifting will resume on a concurrent basis again.

SUMMARY

As of the end of February 1983, 39 months has elapsed since the start of engineering work on the Silver Shaft and 35 months has elapsed since the start of on-site construction operations. In that period of time the following items of work have been accomplished:

Quantities as of 2/28/83

	Linear Feet	Linear Meters	Cubic Meters	Cubic Yards	Pound	Kilogram
Shaft Excavation (Depth)	1,830	6,004				
Shaft Excavated			64,630	84,528		
Shaft Concrete			23,872	31,222		
Shaft Steel	1,801	5,910			1,773,000	791,940
Shaft Split-set -Rockbolts	75,758	248,547				
Station Excavation			10,835	14,171		
Station Concrete			3,250	4,251		

	Linear Feet	Linear Meters	Cubic Meters	Cubic Yards	Pound	Kilogram
Station Steel					304,823	136,154
Station Resin Bolts	33,788	110,853				
Split-Set Rock-bolts	11,800	38,714				
Mechanical Bolts	451	1,480				
Rebar					71,286	31,841
Raisebore Holes	241	790				
Diamond Drilling	662	2,172				

As of February 1, 1983 the project was 638 days ahead of schedule. The safety record is superior to the National Mining Average. Two North American records for constructing completed shaft have been set. These were 36.9 meters (121 feet) of completed shaft including excavation, concrete and equipping in one week, and an advance of 146.3 meters (480 feet) of completed shaft in a 30 day period. This latter record was accomplished twice.

Completed to its ultimate depth of 2,348 meters (7,700 feet) this shaft will be the deepest single lift shaft in North America.

Chapter 32

SINKING A FREEZE SHAFT WITH INSTALLATION OF A WATER-TIGHT, FLEXIBLE LINING

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ABSTRACT

The sinking and lining of a large mine shaft with 8.0 m i.d. and 1,133 m total depth is described in detail. The ground freezing method was applied to safely sink the shaft through unstable, waterbearing strata. Following a brief introduction to the geological and hydrological conditions and the owner's shaft requirements, is the general technical concept to sink and line the shaft. Particular emphasis is put on the lining system which has to provide stability and absolute watertightness for the lifetime of the shaft, even though mining takes place in its immediate proximity. Machinery and equipment used to execute the work are mentioned and commented. Furthermore, the excavation and lining of the large filling stations using rock anchors and shotcrete are described. Details on performance rates are given.

INTRODUCTION

Coal mining in the Ruhr district, Germany will be further extended to the North. The thickness of the unstable, waterbearing strata is increasing. The Ruhrkohle AG is also planning to mine the coal inside the shaft safety pillars. Therefore, new shaft sinking and lining concepts had to be developed. The shafts "Haltern 1 and 2", presently under construction by Deilmann-Haniel, are used as an example to document todays sinking and lining methods employed in freeze shaft construction in Germany.

The shafts "Haltern 1 and 2" are located in the center of the future "take area" of the Haltern mine. The shafts are 150 m apart and will have finished diameters of 8.0 m each. The downcast shaft (20,000 m³/min) "Haltern 1" will be 1,133 m deep serving for manriding and material handling. The upcast shaft "Haltern 2" will be 1,120 m deep. Three and two two-entry filling stations will be constructed.

Long wall mining using the caving method will begin inside the safety pillars. Resultant strata movements will affect the shafts. Therefore, the owner requires a shaft lining system which meets the following requirements:

In the unstable, waterbearing overburden
- Long term stability against earth- and water pressures;
- Absolute watertightness;
- Insensibility to vertical and lateral strata movements;
- Sealing system to prevent penetration of water from the waterbearing strata alongside the shaft lining into the competent rock and into the shaft below;
- Resistance to corrosion.

In the competent, dry rock
- Flexibility in the vertical direction to keep lining damage due to strata movements to a minimum.

Both shafts have received identical linings. Sinking of the shaft "Haltern 1" started in 1979, construction of the shaft "Haltern 2" began appr. 1.5 years later.

GEOLOGY AND HYDROLOGY

Knowledge of the classification and characteristics of the soil and rock strata are fundamental in determining the shaft sinking and lining methods. Therefore, an exploratory test hole was drilled to a depth of 900 m at the proposed shafts location. At depths from 20 to 300 m and 600 to 900 m continuous cores were taken. In situ drill stem tests as well as geophysical logging in the exploratory hole, geotechnical investigations, and

laboratory analyses of the acquired cores gave the results which are presented in a condensed form in TABLE 1.

TABLE 1 Results from exploratory hole

Geologic Epoch	Strata Description and Classification	Classification for Shaft Sinking	Depth in m
Santon and Coniac	Halterner Sands fine to medium sands	unconsolidated, unstable, waterbearing (below 45 m) quick sands	
	Wulfener Facies calcareous fine to medium sands		130
	Recklinghäuser Sandy Marl fine marly sands to fine sandy marl	little consolidated, consolidation increasing with depth, unstable, waterbearing	210
		competent, possible fissure water	250
	Emscher Marl clay marl, initially fine sandy, then silty fine grained clay marl	competent, dry	650
Turon	Calcareous Marl and Limestone partly clayey	competent, possible fissure water	
Cenoman	Essener Greensands sandy limestone and fine grained sandstone	little consolidated to competent, dry	845
Carbon	Lower Coal Measures	competent, dry	

The main conclusions drawn from those results are:

- At least to a depth of 210 m a temporary excavation support and groundwater control system has to be used to sink the shaft safely through the unstable, waterbearing strata;

- A watertight final shaft lining system is required in the unstable, waterbearing strata to avoid any inflows of water and/or quick soils;

- Due to the possibility of fissure water in the Turon strata special precautions have to be taken during sinking to avoid uncontrolled water inflows into the shaft;

- Rock characteristics of the Emscher Marl, Turon, Cenoman, and Carbon strata did not indicate any significant peculiarities or anomalies that might cause problems during shaft sinking. Those rock characteristics are well documented from previous shaft sinking experience in the Ruhr district.

SINKING AND LINING OF FORESHAFT (0 - 45 m)

Frequently, the moisture content of the soils above the groundwater table is too low to provide sufficient stabilization by ground freezing. Depending on the depth of the water table various special construction techniques are applied. In this case, a secant concrete pile wall provided the temporary excavation support to a depth of 32 m, consisting of a series of bored, cast-in-place, lean concrete piles (Ø 0.7 m; overlap 0.1 m). Every second pile contained a steel beam as the main bearing element. During sinking, internal ring beam bracing was installed.

From 32 to 45 m depth liner plates, grouted in-place, provided the temporary excavation support.

The foreshaft was sunk as an open excavation using a hydraulic loader at the shaft bottom, and a crane and bucket for muck removal.

Above the water table a watertight lining is not required. To a depth of 38 m the final reinforced concrete lining was poured in 3 m lifts without mechanical joints. From 38 to 45 m depth a 0.6 m thick outer lining of reinforced concrete was installed in sections with waterstops.

SINKING AND LINING OF FREEZE SHAFT (45 - 227 m)

The ground freezing method was used to stabilize and seal the unstable, waterbearing soils to a depth of appr. 210 m during shaft sinking and installation of the watertight final lining.

The basic concept of ground freezing is the extraction of heat from the ground causing the in situ pore water to freeze and serve as a bonding agent between the soil grains to increase their combined strength and make them impervious. A freeze wall is formed around the periphery of the planned excavation. This provides a stable excavation support and groundwater control system. Ground freezing has been successfully utilized in shaft sinking for several decades. The safety and reliability of this method are permitting precise, detailed planning of time and economic factors.

Drilling and Installation of Refrigeration Pipes

39 refrigeration pipes were equally spaced on a 15.0 m Ø circle to create the freeze wall for each of the shafts, resulting in a theoretical pipe spacing of 1.21 m. The refrigeration pipes extended to a depth of 217 m to socket the freeze wall safely into the dry, competent rock.

Collar casings (245 mm o.d.; 15 m long) were set and grouted in place at each bore hole location during construction of the shaft collar and the freeze cellar, which will later hold all manifolding for the refrigeration pipes. Two rotary drill rigs, suitable to drill to 1,000 m depth, were used. Each rig was mounted on a frame running on tracks to facilitate pivoting around the centerline of the shaft. The bit had 216 mm Ø, and bentonite mud was used to stabilize the hole and transport the cuttings.

The distance between adjacent refrigeration pipes controls the time necessary for complete closure of the freeze wall. Therefore, alignment of each hole is critical to ensure freeze wall closure within the scheduled time. Corollary, all refrigeration bore holes have to remain within a specified zone, defined by two concentric circles of 14.0 and 15.0 m Ø around the center of the shaft. Furthermore, the maximum spacing of two adjacent holes was limited to 1.60 to 1.95 m depending on the depth. Heavy drill pipes and stabilizers were used for precise drilling. Continuous alignment controls during drilling and - if required - the use of directional drilling tools to avoid excessive deviations permitted that all holes remained well within the specified limits.

During drilling operations the alignment of each hole was checked using a single-shot magnetic compass survey instrument which was lowered into the drill pipe, into the non-magnetic drill collar above the bit. Generally, 22 surveys per hole were taken to measure drift angle and direction of the bore hole. Altogether, the directional drilling tool was utilized 30 times; 19 of the 43 holes were drilled without using this tool. The 43 bore holes (9,400 m) - 39 refrigeration holes, 3 temperature monitoring holes, and 1 pressure relief hole - were completed in 83 working days. The average overall performance rate was 4.7 m/h, the average drill advance rate was 6.3 m/h. The record was set by completing one hole in 31 hours.

Seamless pipes (127 mm o.d.; wall thickness 6.3 mm) made of special steel with high strength at low temperatures were installed as refrigeration pipes by a crane. The refrigeration pipe was capped at the bottom, and the pipe sections were threaded and coupled using Omega-connections. The use of hydraulic power tongs guaranteed precise tightening with a maximum torque of 5.5 kN m. After installing the refrigeration pipe to the final depth of 217 m, the pipe was pressure tested for watertightness to 5.5 MPa for a period of 15 minutes. Then, the location of the pipe was checked again with a surface recording gyroscopic survey instrument.

Installing and Maintaining the Freeze Wall

Inside the refrigeration pipes open end PE-pipes (75 mm o.d.; wall thickness 4.3 mm) were placed. A distribution header was attached to the top of the refrigeration pipe with a quick-connect coupling. This header was connected to main manifold lines. A central pumping station was circulating the chilled coolant - a calcium chloride brine at an approximate temperature of -25° C - through the main manifold line to the shaft, into the ring manifold which equally distributed the brine into each refrigeration pipe (Fig. 1). The brine, warmed by the heat extracted from the soil, was pumped back to the refrigeration station and was re-cooled. As heat was extracted a cylinder of frozen earth was formed around each refrigeration pipe. As time passed the frozen cylinders around adjacent pipes grew together, forming a continuous, stable, and watertight wall of frozen earth. The freeze wall had then to thicken with time at least to the required thickness according to the structural design, varying with depth from 2.0 to 4.3 m.

During the pre-freezing period, when the freeze wall was formed, 100 % of the installed refrigeration capacity was required. When the freeze wall section just below the groundwater table had reached the minimum thickness to continue shaft sinking, the refrigeration load was reduced and the maintenance freezing process was regulated to always provide the minimum freeze wall thickness according to the structural design as excavation proceeded and to minimize encroachment of the freeze wall into the excavation area.

The freezing process had to be controlled. The brine circuit was constantly monitored by installation of 45 temperature sensors. Furthermore, brine volume and pressure were measured.

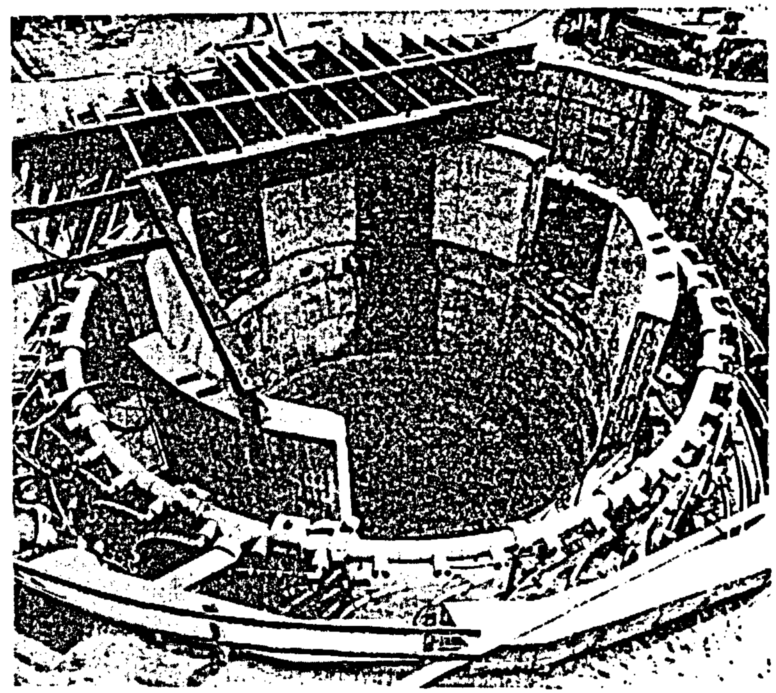

Fig. 1 Freeze cellar with main manifold system

To monitor the freezing process in the soil 3 temperature monitoring pipes were installed in the ground, 1 inside and 2 outside of the refrigeration pipe circle. After a temperature profile in these holes was established over the entire freeze shaft depth, fixed temperature sensors were installed at critical depths.

Furthermore, a pressure relief hole - cased below the groundwater table with a filter screen - was placed approximately in the center of the shaft. Rising water inside this relief pipe usually indicates closure of the freeze wall.

In addition, ultrasonic measuring techniques were used to check the progress, extent, and continuity of the freeze wall. This method is based on the concept that ultrasonic waves have considerably different velocities in frozen and unfrozen soil.

The data gained from all the measurements and their changes with time provided sufficient information on shape and condition of the freeze wall to avoid any "surprises" during shaft sinking.

Because sinking of both shafts was staggered by about 1.5 years, simultaneous freezing was not required minimizing the necessary refrigeration capacity. The central refrigeration station consisted of 3 independent refrigeration plants equipped with screw compressors. At a brine temperature of -25° C the total installed refrigeration capacity was 4,700 MJ/h.

The Lining System

A system consisting of a flexible outer lining and a watertight, flexible inner lining separated by an asphalt layer best fulfilled the owner's shaft requirements in the unstable, waterbearing strata (Fig. 2).

The construction sequence is as follows:

The shaft is first sunk through the unstable, waterbearing formations with installation of the outer lining only. In the competent, dry rock a main reinforced concrete foundation ring is poured, which will later transfer the loads of the inner lining and the asphalt layer to the competent rock. Then, the inner lining is installed starting at the top of the foundation ring. Finally, the annulus between the outer and inner linings is backfilled with asphalt.

The inner lining is a steel-concrete composite, which fundamentally consists of a reinforced concrete cylinder surrounded on the outside by a fully welded steel liner. For freeze shaft depths to 250 m the concrete cylinder is designed to carry all loads, and the steel liner only provides watertightness. For greater depths the steel liner is increased in thickness and then part of the structural design. For freeze shaft depths exceeding 400 m a steel liner is usually also installed on the inside.

The required insensibility to strata movements and the seal to avoid penetration of water into the shaft below the watertight lining system are accomplished by a set of structural means.

First of all, the annulus between the inner and outer linings is backfilled with asphalt, a viscous fluid, to

permit relative movements between the watertight inner lining and the strata. The asphalt is made of bitumen with limestone powder as a filler. Usually, the asphalt will have a density of 1,300 kg/m³ in accordance to the approximate lateral pressures of water and earth.

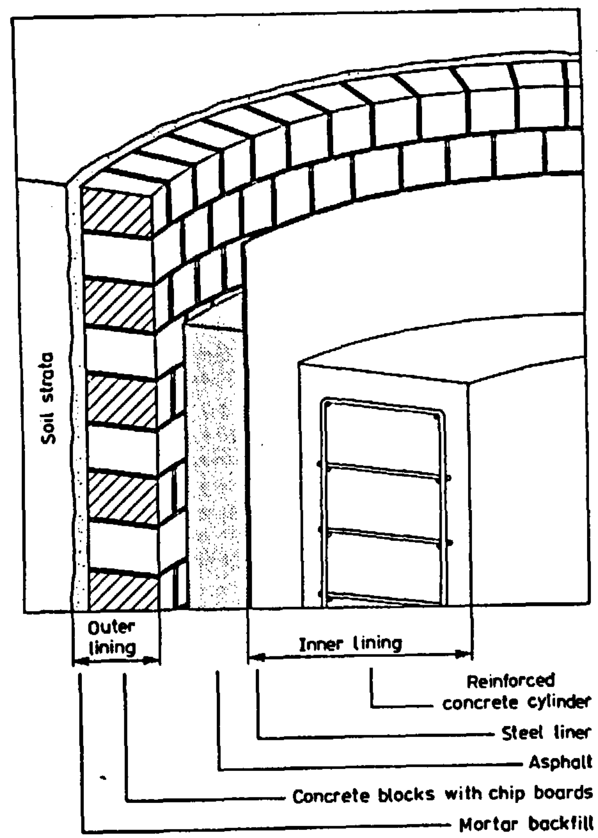

Fig. 2 Structure of lining system

The asphalt layer is providing a protective zone for the inner lining, because lateral strata movements will only affect it, if the movements exceed the thickness of the asphalt layer. Therefore, this thickness will be determined according to the expected deformations from the mining activities.

If the movements are exceeding the asphalt layer thickness or if the main concrete foundation ring is tilted, the inner lining will be bent. Nevertheless, it has to remain stable and watertight. Therefore, the reinforced concrete cylinder is separated into single rings without mechanical joints. When being bent the joints between the concrete rings can open, diminishing tension and avoiding concrete damage. At the same time the steel liner will be elongated. To avoid its overstressing by concentrating the elongation to the joint area the inside of the steel liner is lubricated with a 2 mm bitumen coating which reduces friction; thus, relative movements between steel liner and concrete cylinder are possible.

Special structural means have to be taken at the top and bottom of the inner lining. In the zone immediately above the main foundation ring the inner lining is in direct contact with the rock to provide a firm fixation, avoiding damage of the foundation and the inner lining even when the foundation ring is tilted. At the top of the inner lining a guide ring is installed to only permit relative vertical movements between inner lining and shaft collar but no lateral movements which could be detrimental to the hoisting facilities (Fig. 3).

The outer lining has to secure the shaft walls to protect the miners from any spalling of frozen material and has to provide additional long term support for the freeze wall until the watertight inner lining is installed. In view of the future strata movements the outer lining must be constructed in a way which prevents damage to the watertight inner lining. A lining made of prefabricated concrete blocks with a limited flexibility best fulfills those requirements. Wooden chip boards placed in all vertical and horizontal joints between the concrete blocks provide the outer lining with the necessary flexibility in vertical and circumferential directions. The chip boards can be compressed to 40 - 50 % of their original thickness with almost no lateral strain. Perimeter reductions up to 1 m can thus be achieved without destroying the outer lining. However, should locally concentrated strata movements later destroy part of the outer lining, only small chunks of concrete could spall and penetrate into the asphalt filled annulus, hardly representing a threat to the watertight inner lining.

The flexibility of the outer lining is also beneficial when frost pressures develop due to the formation of ice lenses during freezing.

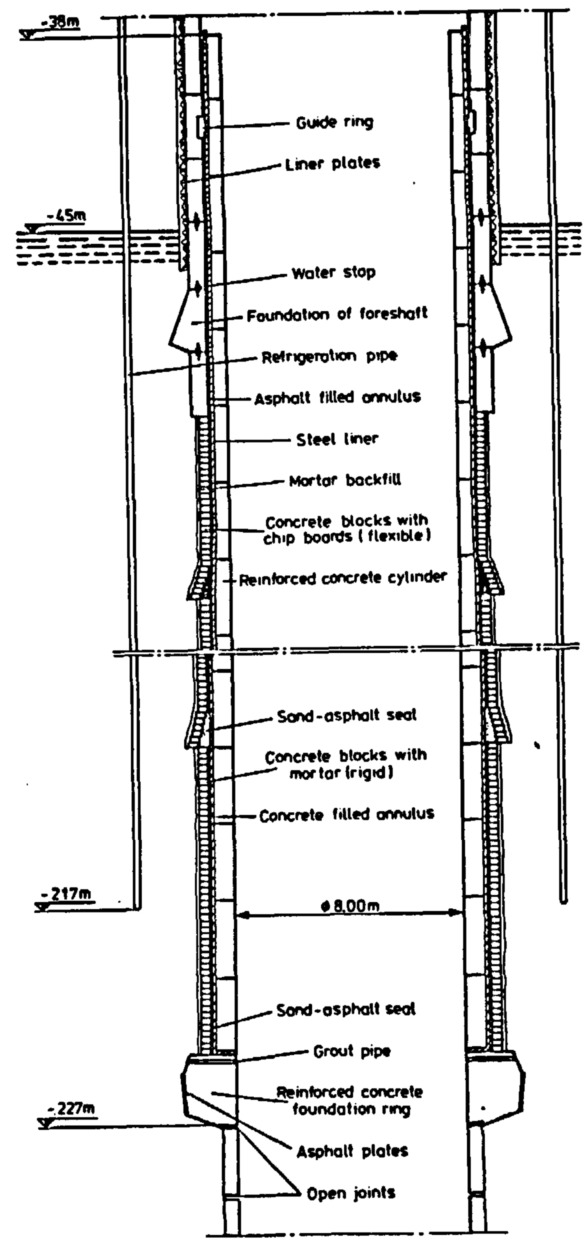

Fig. 3 Lining system in freeze shaft section

In special cases, the outer lining can be designed to later carry the entire soil pressure. Then, the density of the asphalt can be reduced resulting in a decreased pressure on the inner lining.

Sinking and Installation of Outer Lining

After completion of the foreshaft and during the pre-freezing period the shaft sinking plant was installed, i.e. sinking headframe, bobbin hoist, winches, and sinking stage (Fig. 4).

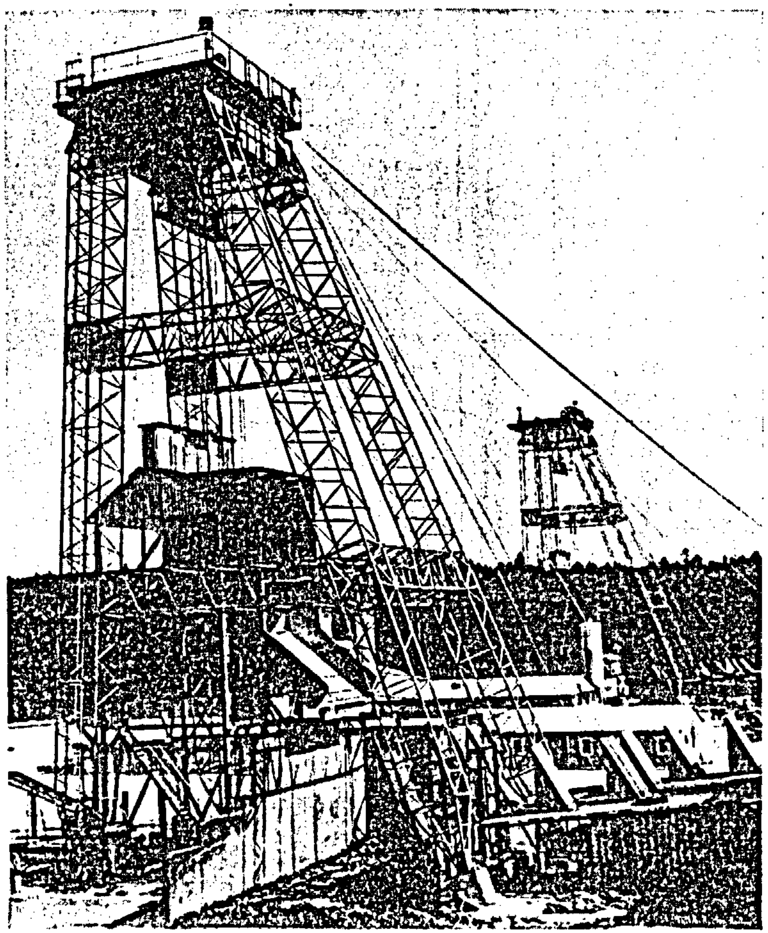

Fig. 4 Sinking plant for shafts "Haltern 1 and 2"

5 m^3 buckets and a 5-blade cactus grab with 0.8 m^3 capacity attached on a rotational frame to the two-deck sinking stage were used. After a period of 2.5 months of pre-freezing and upon completion of a final check of the freeze wall integrity with ultrasonic measuring techniques sinking was resumed.

In the soft, unfrozen sand layers mucking was easily handled by the cactus grab. Drill and blast techniques were used in the sandy marl or where the freezing had encroached too much into the excavation area. The recent improvements in steel quality and pipe connections permit employing this method also in freeze shafts. However, careful blasting techniques were applied to avoid refrigeration pipe damage. The number of detonators and the amount of explosives for each period delay were restricted. Depending on the soil conditions the depth of a blasting round was limited to 1.5 - 2.0 m. Sinking rates up to 3 m/day working four 6 h-shifts were achieved with this method without damaging a refrigeration pipe.

Conical shaped, prefabricated concrete blocks (f_c = 55 MPa) with the dimensions of 0.2 x 0.2 x 0.3 m were used as flexible outer lining. The chip boards placed in the horizontal and vertical joints were 8 mm thick, permitting a possible maximum perimeter reduction of 0.5 m.

The outer lining was placed in 12 m sections. Each section had a small conical bearing set - also made of concrete blocks - resting on frozen soil (Fig. 3). The concrete blocks were set from the sinking stage, leaving never more than 24 m of shaft unlined (Fig. 5). A mortar backfill provided direct contact between concrete blocks and frozen soil.

Fig. 5 Installation of outer lining

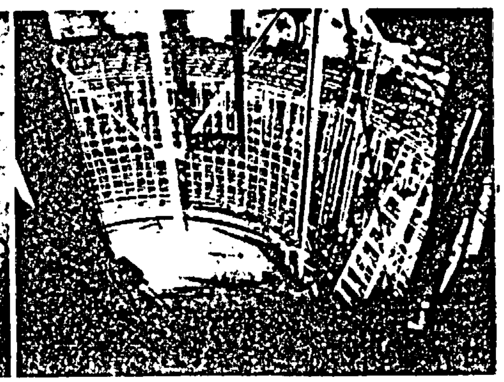

Fig. 6 Reinforcement of main foundation rin

Main Concrete Foundation Ring

The reinforced concrete foundation ring (f_c = 45 MPa) (3 m height; average width 2.5 m) was placed 7 m below the tip of the refrigeration pipes in the stable, competent rock (Fig. 6) to transfer the loads of the watertight inner lining and the asphalt layer (appr. 15,000 tons) to the competent rock. Immediately below the foundation ring two reinforced concrete support rings (thickness 0.6 m; height 3 m) were poured. Open joints were left between the foundation ring and support rings to avoid direct load transfer from the foundation ring onto the lining.

9 days were required to construct the support rings and the main foundation ring.

Installation of Watertight Lining

The functioning of the flexible, watertight lining was already described earlier.
The thickness of the reinforced concrete cylinder increased with depth from 0.53 m to 0.75 m in 5 increments. The thickness of the steel liner (f_y = 370 MPa) was 8 mm from top to bottom. The asphalt layer was 0.15 m thick.

Design calculations had indicated that - due to strata movements from mining activities - a bending radius of 10,000 to 12,000 m of the shaft center line might occur. Therefore, the reinforced concrete cylinder (f_c = 35 to 45 MPa) was poured in 3.6 m sections permitting a bending radius of 5,000 m without exceeding the allowable stresses. Corollary, the steel liner was also installed in 3.6 m high sections.

To install the watertight lining two decks were added to the sinking stage. The two top decks were used for installation of the steel liner, the lower decks for concreting.

The steel liner segments - 5 segments formed an entire ring - and the reinforcing cages - prefabricated at the shaft surface - were lowered by an auxiliary hoisting system into the shaft to the sinking stage, where a rotating positioner and chain falls were placing the segments into final position. Steel liner and concrete cylinder were installed alternately, beginning with the steel liner. A steady working rhythm was established for the crews that 3.60 m of watertight lining were installed in 24 hours. The steel liner segments were first tack welded. Horizontal welds were manually applied by certified personnel while fully automatic welds were made on the vertical seams. All field welds were ground smooth and ultrasonically inspected, all flaws were marked and repaired.

The concrete cylinder was poured using a collapsible jump form. The form was attached to the chain falls, collapsed, and then raised to the next concrete elevation. The concrete was transported in 2.5 m³ bottom dump buckets from the surface batch plant to the sinking stage, where a distribution system and a hydraulically operated chute on the bottom deck was feeding the concrete to the forms.

Immediately on top of the concrete foundation ring and above the fixation zone of the watertight lining a sand-asphalt seal was installed to prevent penetration of asphalt into the foundation area. After the limestone powder had been mixed with the hot bitumen (130° C) in a batching plant at the shaft site the asphalt was placed in the annulus with the tremie method using a tremie pipe (88.9 mm o.d.). The annulus was backfilled to a level just below the top of the inner lining, which is ending at a depth of 38 m sufficiently above the present water table at 45 m depth as margin for possible variations in the water table or future subsidences due to mining activities (Fig. 7).

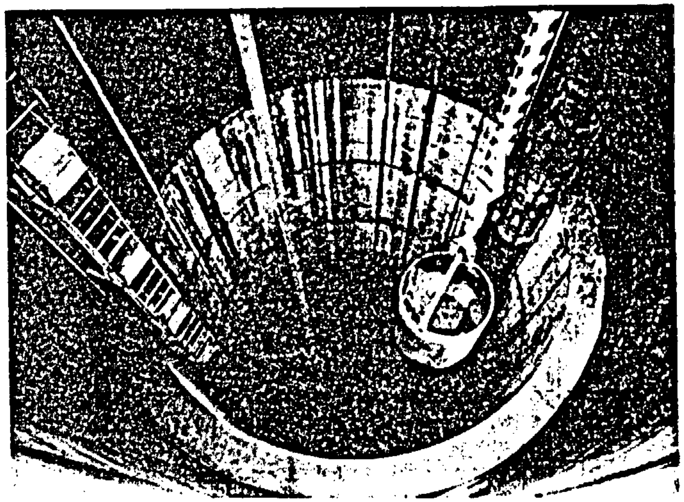

Fig. 7 Top of watertight inner lining system in freeze shaft

Once installing the watertight lining and backfilling the annulus with asphalt were completed freezing was no longer required, and the refrigeration plants were turned off. The freeze wall can now thaw naturally. After the thawing process will have progressed sufficiently the PE-down-pipes will be pulled, and the refrigeration pipes will be backfilled with cement grout.

22 months were required from starting construction of the shaft collar until completion of installation of the watertight lining.

SINKING AND LINING IN COMPETENT ROCK (227 - 1,133 m)

The application of special shaft sinking methods were not required in the competent rock below a depth of 220 m. Therefore, conventional drill and blast techniques were used to sink the shaft. Fig. 8 provides information on the major equipment and machinery used during sinking and lining of the shaft, which was done alternately and sequentially. 8 m of shaft could be left unsupported before the final lining was installed. In rock of bad quality wire mesh and rock bolts were applied.

A 4-boom drill jumbo - specially designed and built by Deilmann-Haniel - equipped with Turmag P 2/4 rotary percussion drills was used to drill the blast holes. The depth of a blasting round was 4.5 m. The 4-deck sinking stage, 5 m³ muck buckets and the 0.8 m³ cactus grab were the same as used during sinking and lining of the freeze shaft.

Systematic advance probe drilling in the Turon and Cenoman formations was carried out to detect any fissure water. The probe drilling was done in 35 m sections with an 8 - 12 m overlap. When water was encountered, cement grouting was performed to an extent that shaft sinking could proceed safely through these layers. However, very little grouting was required in both shafts.

The lining in the competent rock was of concrete (f_c = 25 MPa) with a wall thickness of at least 0.4 m. A reinforcing mat was placed at the inside of the concrete lining to avoid spalling of large concrete chunks when the expected high stress and strains from future mining activities will be imposed on the lining.

The concrete was poured in 4.2 m lifts. The lifts were separated from each other by a continuous 0.30 m high joint which was only closed on the side exposed

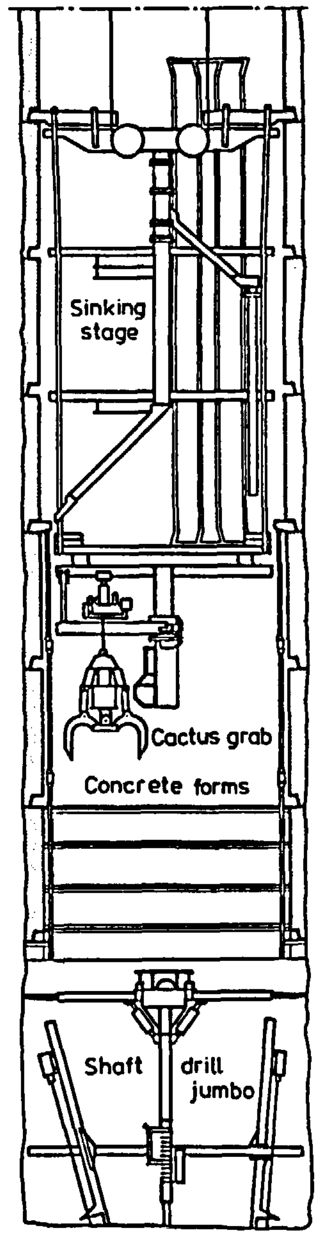

Fig. 8 Shaft sinking in competent rock

to the rock by a 70 mm thick concrete curtain. The joints which were used during shaft sinking to seat the sinking stage will remain open.

The separation of the lining into single concrete rings is providing some flexibility in the vertical direction, minimizing damage due to high vertical strains in the surrounding rock from future mining activities.

A 4.2 m high steel form was used following sequentially the sinking operation. The form was placed on a curb ring. The curb ring was suspended from hanging rods (length 13.5 m) which were fixed to anchor plates placed on the already cured concrete.

Pouring concrete was done the same way as during installation of the watertight lining.

An average daily production rate of 4.5 m completed shaft (sinking and lining) was achieved in the Emscher Marl and Cenoman strata; in the Turon formation the advance rate was slightly less. The crew at the shaft bottom consisted of 7 miners including the walker and supervision, working four 6-h shifts. Including all hourly and salaried personnel an average of 55 men-shifts were used during a working day.

SHAFT STATIONS AND DRIFTS

Because sinking of the shaft "Haltern 1" was far ahead of schedule and in order to permit earlier connection of the shaft to the existing mine, the contract was extended to drive ventilation drifts at the first two-entry station at the 870 m level. It was decided to line the large filling stations and the drifts with rock anchors and shotcrete, a lining system Deilmann-Haniel had already successfully used for the first time when constructing one of the large filling stations of the shaft "An der Haard 1". The work consisted of 2 major parts:

- construction of a two-entry station with a maximum cross-sectional excavation area of 160 m²;
- driving appr. 140 m of drifts at each entry with cross-sectional areas varying from 160 to 28.5 m².

The lining system was designed by the Bergbau-Forschung (Mining Research Institute), Essen, Germany.

The top heading and benching method was used to excavate the large station (Fig. 9). Immediately following the excavation a 0.1 m thick shotcrete layer (f_c = 25 MPa) was applied to consolidate the rock. Then, rock anchors (Ø 33 mm; length 5.15 and 6.15 m) were set using a quick-set mortar providing an anchor bearing capacity of appr. 480 kN within a short period of time. The pattern of the anchors varied but as an average 1.2 anchors per m² were installed. The anchors were not prestressed.

Together with the anchor plates a reinforcing mat (Ø 6 mm; grit pattern 0.15 x 0.15 m; weight 3.01 kg/m²) was installed. Then another 0.05 m thick shotcrete layer was applied to reduce corrosion of the anchor plates and nuts. In the drifts the same lining system was installed, except that a resin anchor system (Ø 24 mm; length 3.65 m) was used providing an anchor bearing capacity of appr. 250 kN.

The following equipment was employed:

- (1) crawler-mounted twin boom drill jumbo equipped with Tamrock drills for blast hole drilling;
- (1) LF-7 Gutehoffnungshütte (GHH) diesel driven front-end loader (LHD; 3.6 m³) to haul the muck;

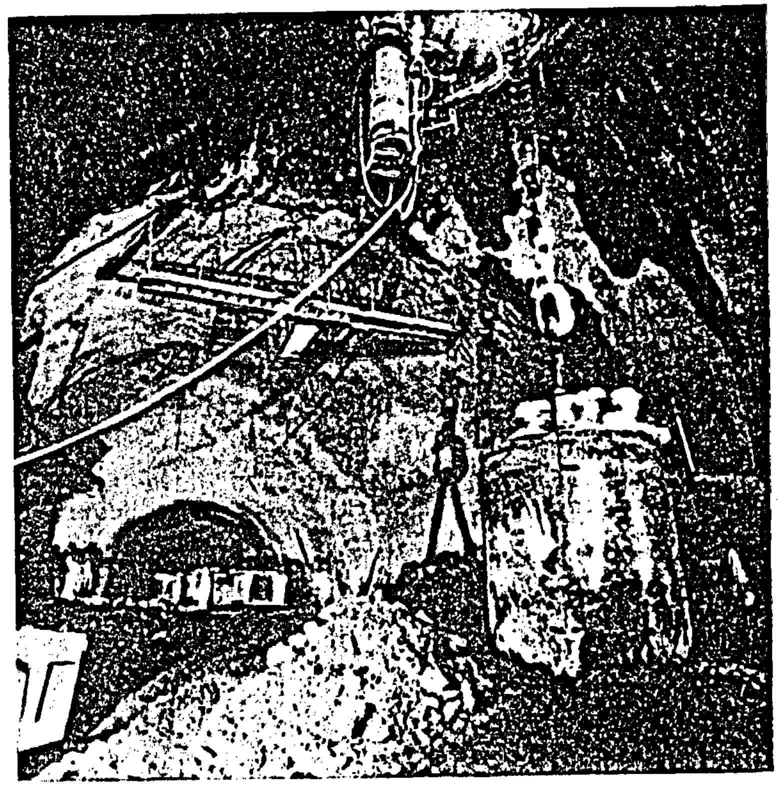

Fig. 9 Top heading of shaft station

- (1) crawler-mounted G 210 Deilmann-Haniel electric driven hydraulic side tip loader (2.0 m³) to load the spoil into the buckets;
- (1) crawler-mounted, electric driven drill carriage to drill anchor holes and set anchors, specially assembled by Deilmann-Haniel;
- (2) crawler-mounted, air operated telescopic working platforms;
- (2) GM 90 Meynadier shotcrete machines (6 - 7 m³/h);
- (2) 70 kW-fans for ventilation.

To have more efficient use of men and machines dual headings were driven. The total underground crew consisted of 8 men exclusive supervision working four 6-h shifts.

Work started in August '81 and was completed in January '82 including installation of 2 fan lines (Ø 1.2 m), 3 utility lines, and 1 high-voltage cable in the shaft. A total of 15,700 m^3 were excavated, 7.600 tons of shotcrete installed, and 7,963 rock anchors set.

WORK STATUS

In February '83 the shaft "Haltern 1" was 1,103 m deep, and the third station and 450 m of drifts and cross-cuts were being built. After construction of the shaft sump the shaft will be equipped.

The shaft "Haltern 2" was at the 870 m level, and the first station (no drift work) was being constructed.

All work will be completed ahead of schedule before the end of the year.

Chapter 33

BIG HOLE DRILLING -- THE STATE OF THE ART

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ABSTRACT

The "Art" of Big Hole Drilling has been in a continual state of evolution at the Nevada Test Site since the start of underground testing in 1961. Emplacement holes for nuclear devices are still being drilled by the rotary drilling process, but almost all the hardware and systems have undergone many changes during the intervening years.

The current design of bits, cutters, and other big hole drilling hardware results from contributions of manufacturers and Test Site personnel. The dual string, air lift, reverse circulation system was developed at the Test Site. Necessity was really the Mother of this invention, but this circulation system is worthy of consideration under almost any condition.

Drill rigs for Big Hole drilling are usually adaptations of large Oil Well drill rigs with minor modifications required to handle the big bits and drilling assemblies. Steel remains the favorite shaft lining material, but a lot of thought is being given to concrete linings, especially precast concrete.

INTRODUCTION

The "Art" of Big Hole Drilling has been developed at the Nevada Test Site as a part of the nation's nuclear testing program. Agreements with the Soviet Union specify that the radioactive products of a detonation shall not pass the national boundaries. Therefore, nuclear experiments are conducted underground mostly in drilled shafts. There have been more than 450 big holes drilled at the Test Site for this purpose. These big holes were at least 48 inches in diameter and 500 feet deep.

Twenty "Big Holes" were drilled in 1981, 64 inches to 96 inches in diameter to an average depth of 1,590 feet. These holes were drilled at a rate in excess of 100 feet per day and with a bottom offset of only slightly over one foot per 1,000 feet of depth. This drilling rate of 100 feet per day includes the actual drilling time, the time required for trips and connections, as well as, time spent surveying the hole. The bottom offset was determined by optical surveys of the completed hole, whereby the bottom of the hole is located by surveying from the surface with a theodolite.

This directional control was achieved by using the "plumb bob" affect with the drilling assembly whereby only 40 percent of the weight of the drilling assembly is used to drill the hole, the other 60 percent of the weight is used to keep the hole vertical.

The geology of the Nevada Test Site is almost entirely of volcanic origin. There may be some sedementary material included in the alluvium, but the underlying formations are volcanic tuff. Compressive strength of the hardest densely welded tuffs may run as high as 25,000 psi but the average compressive strength of the material drilled is less than 4,000 psi.

Drilling costs are meaningful only when these costs are accompanied with an explanation of the operational procedure and an indication of incremental labor costs. Big Hole drilling costs at the Nevada Test Site are contained in two reports. The first covers the surface hole, and the second the remainder of the hole. The surface hole is normally drilled 120 feet deep with an auger. The hole is lined with steel casing which is cemented in place. Costs of this surface hole vary from $80,000.00 to $200,000.00, depending on the material being drilled and the size of the casing. Zero Station drilling costs reflect the cost of drilling the hole from the bottom of the surface hole to total depth. NTS Zero Station costs for uncased holes are from $250.00 to $400.00 per foot. These costs reflect a drilling crew labor cost of $240.00 per hour and certain operating conditions. (1) Most rig moves are by skidding, therefore mobe and demobe costs are minimal. (2) Cutter costs are controlled by retipping and rebuilding cutters to extend cutter life. (3) Equipment cost approximately equals normal rental rates.

Figure 1 is a one page history of the evolution of Big Hole drilling tools and techniques at the Nevada Test Site. This presentation does not begin to indicate the difficulties that were encountered using various techniques and undersized equipment until the current equipment and techniques were developed.

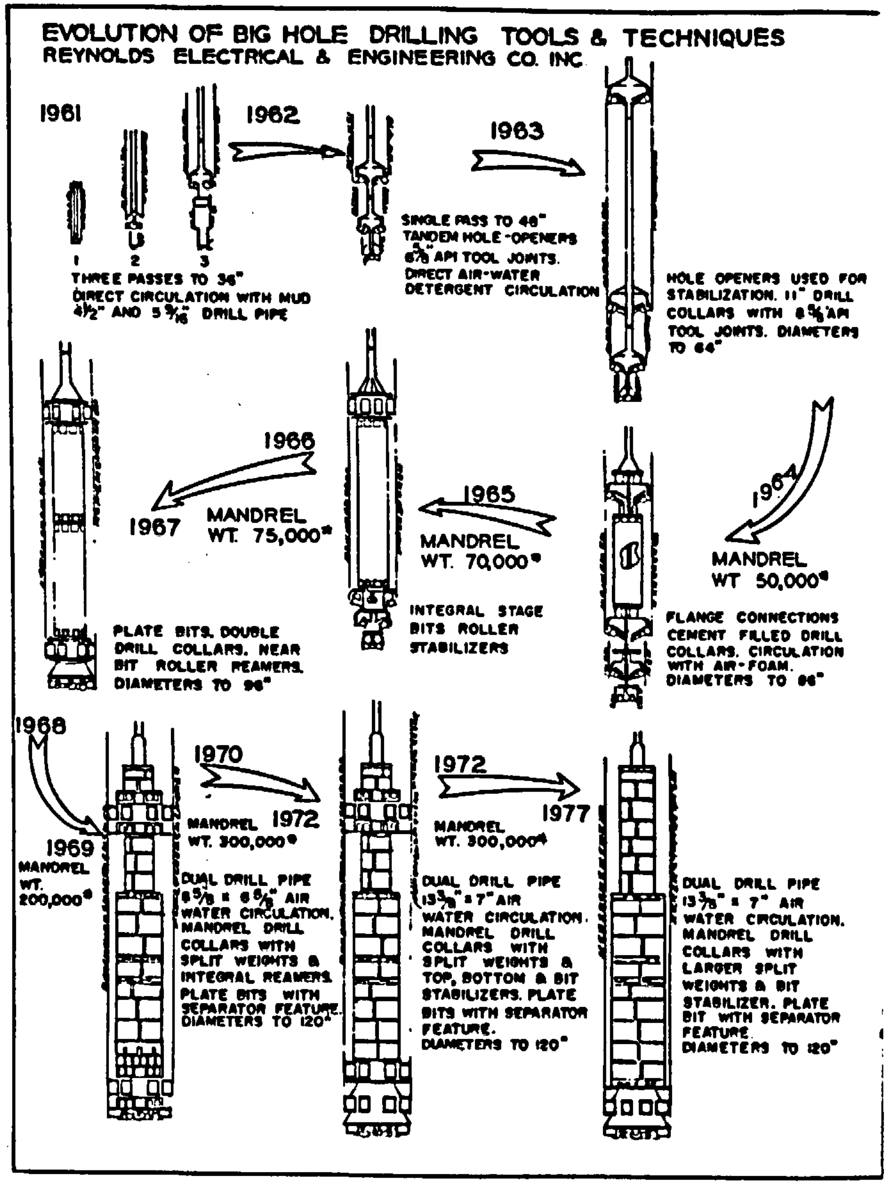
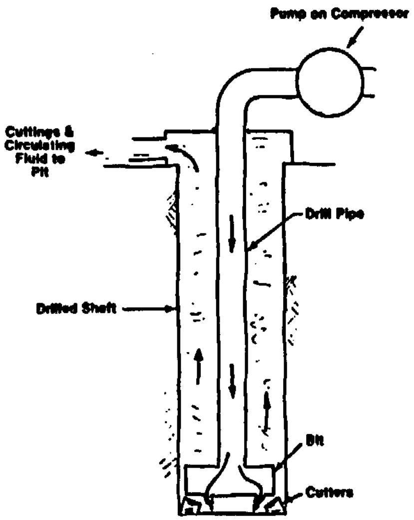


Figure 1

CIRCULATION SYSTEM

The rate of penetration relates to the efficiency of cleaning the bottom of the hole which, in turn, is a function of the circulation system. Direct circulation, Figure 2, is normally used in Oil Well drilling. This system has been used for Big Hole drilling but its use is limited because of the huge volumes of air or liquid required to produce an acceptable annular velocity. Reverse circulation, in which the flow is the reverse of that shown in Figure 2, has been used to a limited extent with air pressure or air vacuum. Air pressure reverse circulation results in a pressure in the annulus and in the formation. This pressure must be bled off each time a joint of pipe is added to the string. Air vacuum limits the circulation rate because of the small amount of pressure drop available to the circulation system. Air vacuum is susceptible to formation fluids and requires extensive filtration equipment to protect the blowers that provide the vacuum, but vacuum drilling does not disturb the formations being drilled.

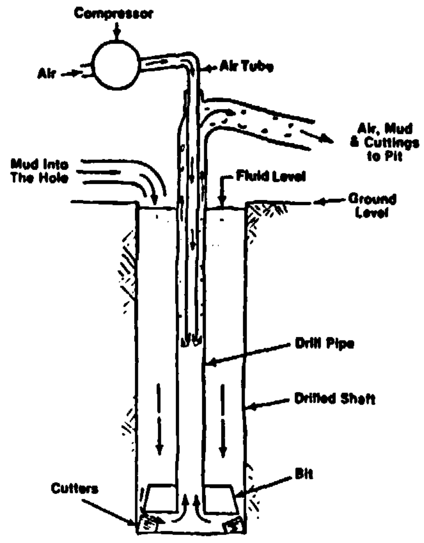


DIRECT CIRCULATION

Figure 2

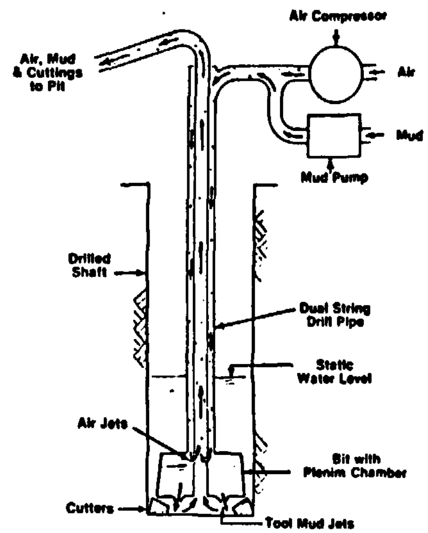
Most Big Hole drilling is done with a reverse circulation air lift system as shown in Figure 3 or with a dual string circulation system shown in Figure 4. Both systems use an air lift system to lift the drilling fluid and formation cuttings up the drill pipe. In the air lift system, shown in Figure 3, the hole is kept full of fluid and air is injected through tubing suspended inside the drill pipe. Fluid circulation rates should equal or exceed 4,000 cfm. J. H. Allen (1) recommends 7,000 cfm for 84" to 120" holes, however, shafts have been drilled sucessfully with circulation rates of 2,000 cfm or less.

The dual string circulation system was developed at the Nevada Test Site because formations would not support a hole full of fluid. This system requires a special drill pipe which consists of a large diameter drill pipe with an added inner string. Air and drilling fluids are introduced into the drill pipe annulus and travel together to the bit where they are essentially separated by gravity and centrifugal force. The drilling fluid goes out the bit jets to clean out the bottom of the hole. Air goes through the air jets and provides the air lift. Circulation rates are much lower, in the order of 300



AIR LIFT CIRCULATION

Figure 3



DUAL STRING CIRCULATION

Figure 4

to 600 gpm.

Air lift pumping operates when the pressure due to the submergence exceeds the pressure created by the aerated column of fluid inside the casing. The mechanics of air lift pumping has been described by A. W. Gibbs (2) and others (1)(3)(4). This system was defined for lifting water, it must be modified for drilling fluids. It has been next to impossible to get good correlation between actual operating conditions and the conditions projected by Gibbs. More theoretical work and experimentation needs to be done to define this operating system.

The dual string system requires more power than the conventional air lift system. Operating pressures of the compressors and pumps are much more than required on the air lift system. We feel that the dual string system does a better job of cleaning the bottom of the hole due to the effect of the high velocity water jets and the resulting turbulance they create.

SURFACE EQUIPMENT

Most Big Holes are drilled with large Oil Field type drill rigs, usually with only minor modifications. These rigs must be equipped with large heavy duty type rotary tables to supply the necessary drill pipe torque. Rotary table failures were a part of the early history of Big Hole drilling, however, the present 37-1/2" tables have been adequate for holes to 120" diameter. Rotary table supporting structure modifications are indicated since the table and supporting structure must be removed whenever the drilling assembly is pulled to change the bit. Special equipment is required to make and break the drill pipe tools joints. 13-3/8" drill pipe requires 100,000 ft. lbs. of make up torque. The proposed 20" drill pipe will require at least 500,000 ft. lbs. of torque.

The Micon division of Hughes Tool Company has taken a different approach to design a rig specifically for Big Hole drilling. This rig has a top drive, instead of a rotary table, to rotate the drill pipe and bit. The top drive also makes and breaks the tool joints replacing the normal spinning and torqueing device(s) normally required on rotary type rigs. Unfortunately this machine pulls 30 foot joints which must be layed down whereas an Oil Well type rig usually pulls 90 foot stands which are racked in the derrick.

Making and breaking the drill pipe tool joints presents a problem in rotary table type rigs because of the high torque involved. Oil Field practice is to spin the joint up with a chain and torque it with drill pipe tongs. 13-3/8" drill pipe cannot be spun with a chain. It can be spun with a rope but rope costs are high because of frequent breakage. Spinning is best accomplished with casing tongs. Only recently have drill pipe tongs been offered which were designed for the torque required for 13-3/8" drill pipe. Self contained, hydraulically operated torque wrenches are available which make and break tool joints more safely than with drill pipe tongs. Drill pipe tongs are not suitable for 20" drill pipe, a self contained torque wrench must be used.

DRILLING TOOLS

The drilling tools for a dual string circulation system are shown in Figure 5. The swivel must be of adequate capacity and with a large bore. Swivels designed to be used with 13-3/8" drill pipe have a 12-1/2" diameter bore and are available to a working capacity up to 700 tons. This swivel must be adapted to the dual string circulation. This requires the addition of the inner string wash pipe, packing, and gooseneck. The return flow of fluid, air, and cuttings is quite abrasive, particularly where the fluid is forced to change direction. A 1-inch replaceable rubber pad under the gooseneck cover absorbs the impact of the returning fluid and cuttings with less wear than steel.

The inner string wash pipe is screwed into the swivel saver sub. This connection has been designed so that the inner string can float and seek the center of the packing. Torque is transmitted from the rotary table to the drilling assembly through the square kelly and the drill pipe, all units of which are equipped with an inner string. Details of this inner string installation are shown in Figure 5. Also shown are the details of the circulation system.

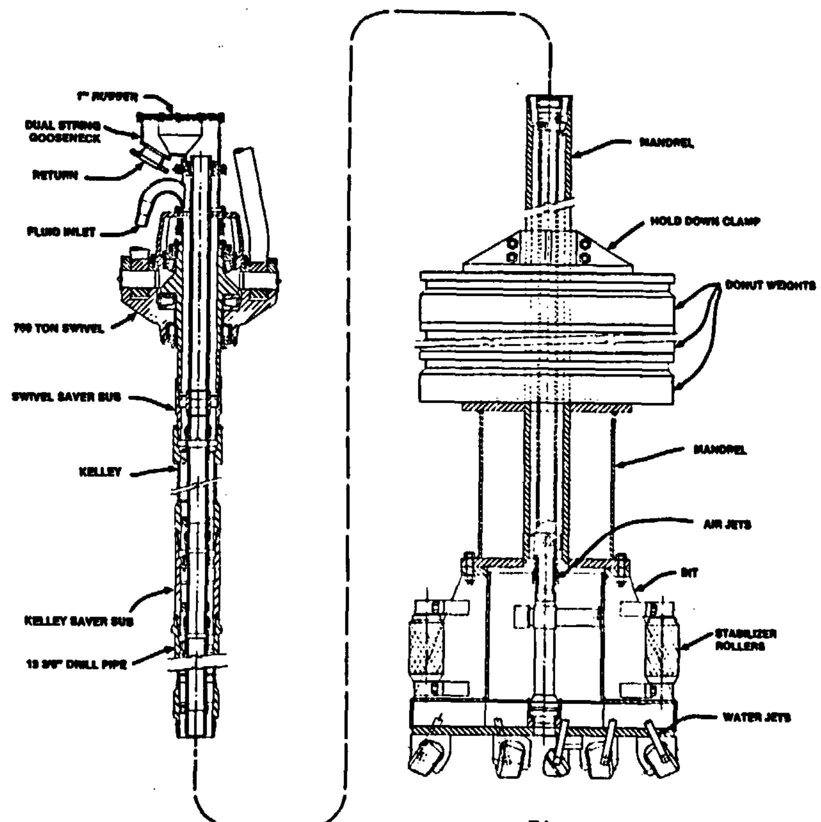


Figure 5

BITS AND CUTTERS

Three examples of bit cutters are shown in Figures 6, 7, & 8. Each is equipped with ball and roller bearings to take the radial and thrust loads imposed during normal drilling operations. The seals are critical components of any cutter. Bearing life is dictated by these seals in that the bearing failure usually results from seal failure which allows contamination of the lubricant with drilling fluid and cuttings. The three illustrations show three of the basic types of cutter structures. The Reed cutter (Figure 6) has a so-called Mill Tooth cutting structure. The Hughes-Micon cutter (Figure 7) is equipped with Tungsten Carbide Inserts distributed across the cutter periphery. The Drilco-Smith cutter (Figure 8) is called a kerf cutter. It has Tungsten Carbide Inserts located in rings with space between the rings. One type of cutter, not shown, is the disc cutter offered by the Robbins Company which is widely used in raise and tunnel boring.

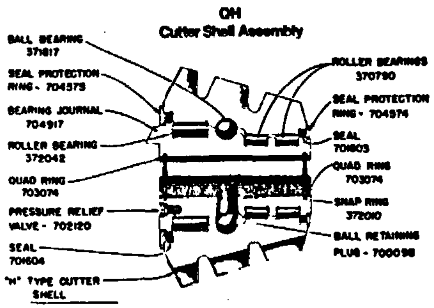


Figure 6

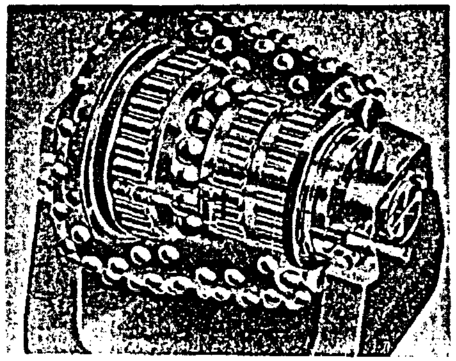


Figure 7

There is no mystery to the arrangement of cutters across the face of the bit. (Figure 9) There must be at least one cutter for each radial protion of the hole, that is, there must be full coverage. The cutter locations should be balanced; obviously you don't want to put all the cutters on one side. There should be extra guage cutters to insure that the hole is kept at full diameter. Cutters should not be placed at equal angles, this will contribute to bit bounce.

The location and sizing of the fluid jets is almost a case of by guess and by golley. Current practice is to have one jet per cutter and to have a total fluid jet area of approximately 2.75 sq. in. with the ratio of fluid jet area to air jet area equal approximately 4:1. Experience has shown that as the hole gets deeper, the jet ratio should increase.

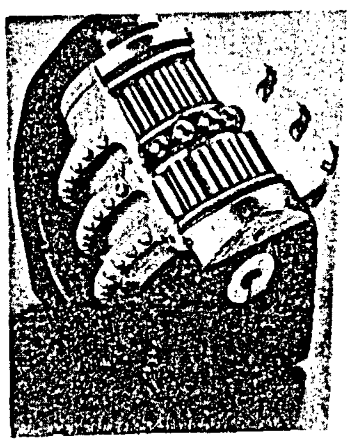


Figure 8

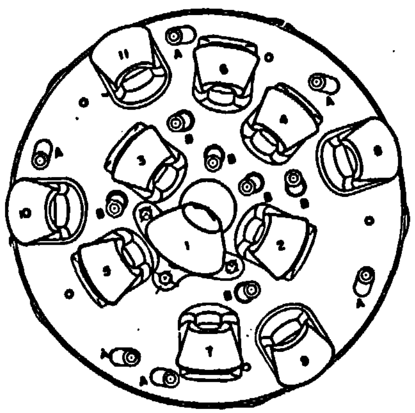


Figure 9

CASING AND CEMENTING

The "State of the Art" liner material for big holes is steel. Although concrete has been used to a limited extent, steel has predominated. The cost of these steel liners has made an adverse contribution to the economics of drilled shafts since the cost of a steel liner, installed and cemented in place, can be as much as two thirds the total cost of the shaft. Precast concrete would seem to represe an economical alternative, particularly using high strength concrete (5) Shot creting might be considered if the shaft can be pumped dry (6) An interesting hybred was used in a shaft in Southern Canada. This shaft was lined with prefabricated sections consisting of preca concrete in a steel shell. The first section was closed with a hemispherical head. As each section was added the steel shell was welde to the previous sections. Since the shaft was full of fluid, the liner floated. Lowering was accomplished by pumping fluid into the liner section. (7)

Cementing has, historically, been done by Oil Well Cementing Companies. This has been an unnecessary expense since these companies pump a grout of cement and water whereas a sand cement grout is much cheaper; it can be mixed in a rented portable batch plant and pumped into the annulus outside the casing.

OPERATING PROCEDURES AND PARAMETERS

Surface holes are drilled with an auger rig, as indicated earlier Normal procedure is to auger a 104" diameter hole and ream the hole the desired finished diameter. Surface casing is set by a crane and cemented in place with batch plant cement.

The 100 feet per day penetration rate at the Nevada Test Site is achieved with drilling parameters somewhat different from cutter manufacturer's recommendations. A 96" bit with 18 cutters will seld be loaded to more than 120,000 pounds and rotated as much as 22 rpm. This equates to less than 7,000 pounds per cutter, whereas cutter manufacturers recommend cutter loading at 20,000 to 30,000 pounds. This lower cutter loading is acceptable because of the relatively soft formations. The 22 rpm rotating speed is much higher than norm recommendations. This speed has not adversely affected cutter life, it does directly affect the penetration rate.

Bits are run no more than 100 hours and then pulled and changed. This is done to prolong cutter life and to guard against the possibility of leaving a cutter in the hole. Mill tooth cutters are then r moved from the bit and retipped. Normally the cutters are retipped twice and then disassembled for inspection. Bearings and seals are replaced as necessary until the outer race in the cutter shell becom damaged. Tungsten Carbide Insert cutters are relubricated after the first run and reused. After the second run, they are disassembled and repaired. Normal life for a mill tooth cutter is 4.4 runs or 260 hours, for TCI cutters 3.2 runs or 310 hours.

Rotating torque is less than predicted by cutter manufacturers. Average recorded torque can be approximated by the following formula:

$$T = .05\ WD$$

T = Torque in foot pounds

W = Weight on bit in pounds

D = Bit diameter in feet

Penetration rates have been as high as 12 feet per hour on a 96" hole for prolonged periods of time. This equates to an excavation rate of 600 cubic feet or 22.2 cubic yards per hour. This was accomplished through a 7" inner string with 2,000 to 4,000 scfm of air, depending on depth, and 400 gpm of water.

PROBLEMS AND SOLUTIONS

Two principal problems affect drilling operations at the Nevada Test Site. The first problem, lost circulation formations, led to the development of the dual string circulation system. Even so, it is sometimes difficult to maintain the desired submergance to get dual string circulation. This can usually be cured by use of lost circulation materials but sometimes it is necessary to drill without proper circulation.

The second, and major problem, involves hole sluffing. A considerable portion of the drilled hole is unsupported by hydrostatic pressure, since drilling is accomplished with 200/300 feet of water in the hole. Portions of this unsupported hole have been known to collapse over the bit. The most successful procedure to recover the drilling assembly, has been to pump fluid into the hole to the top of the fill and fluidize the fill over the bit. Fortunately we have been able to recover the drilling assembly during recent years using this procedure. If sluffing is not too severe, drilling operations can procede without further remedial action. In cases of severe sluffing, it has been found necessary to plug the hole back with concrete to fill the cavity and then redrill the hole.

CONCLUSIONS

Big Hole Drilling represents a "State of the Art" method of Blind Shaft Construction. Holes to 180" diameter can be drilled, in most places, with the current 13-3/8" drill pipe and associated drilling assemblies. Larger holes will require larger drilling tools. Twenty (20") inch drill pipe and compatible drilling assemblies should be sufficient for twenty (20) to twenty-four (24) foot shafts.

Shaft sinking is a hazzardous occupation. The accident rate is high. Shaft drilling is much safer since the work is done from the surface and no one goes into the shaft until it is completed.

Shaft drilling is much faster than conventional shaft sinking. Penetration rates of 100 feet per day may not be achieved, but drilling rates should always be ahead of the rates which result from drilling, blasting, and mucking.

Shaft drilling is most adaptable to some areas that offer severe problems to conventional shaft sinking methods. An area with severe water problems would cause major problems to conventional shaft sinking but no problems to shaft drilling. Water flows would be controlled by keeping the hole full of fluid.

These comments were not designed to serve as a text for Big Hole Drilling. Rather, they were intended to define the state of the art for anyone who has a shaft to sink and provide him with some basic information about the parameters of Big Hole Drilling.

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