THE FLOODING AT THE WEST DRIEFONTEIN MINE*

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SYNOPSIS

The paper gives a background to the water hazards of the so-called West Wits Line and refers to some geological aspects of the Wonderfontein Valley in which the West Driefontein mine property lies. It describes the precautions taken against flooding during shaft sinking, development and stoping; these include cover hole drilling with cementation, provision for water storage in worked-out areas, installation of water tight bulkhead doors and pumping capacity well in excess of the mine's normal requirements. The plan of campaign adopted after the inrush took place is explained and the reorganization and expansion of pumping arrangements are discussed. When it became clear that the only means of saving the mine was to construct plugs to isolate the inrush while still maintaining all emergency pumping measures, the work was put in hand with the utmost dispatch. In considering the design of the plugs a number of major factors had to be taken into account. The most vital was the need for speed which was complicated by difficulty of access to the plug sites because of the huge volumes of water flowing in the drives. How this and other factors affecting plug design and construction, leakage problems, sequence of operations and final closure of the valves, were overcome is detailed in the paper.

INTRODUCTION

The inrush of water into the West Driefontein mine, the biggest gold producer in the world, which began on 26th October, 1968 and continued unabated for 23 days before the flood was stemmed, was unprecedented in the history of mining. Also unprecedented were the tremendous efforts and organization which made it possible to bring the situation under control and so save the mine from being lost for a matter of years.

BACKGROUND

Before going into details of how the situation was brought under control it is appropriate to consider the background to the event. The water problem on the so-called West Wits Line has been recognized for many years. As far back as 1912 the Pullinger Shaft, situated in the centre of what is now the Venterspost mine property, had to be abandoned because there were no means then available of coping with the large quantity of water encountered.

To appreciate what vast quantities of water were involved, it helps to refer to some geological aspects of the Wonderfontein valley in which the West Driefontein property lies. The narrow tabular Witwatersrand gold-bearing beds, which dip at about 25 degrees towards the south, are overlain by a surface layer of dolomite which reaches a maximum thickness of some 4,000 ft at West Driefontein. A number of vertical syenitic dykes intersect the valley more or less at right angles to its course (Fig. 1). These dykes are about 150 ft thick on average and are spaced at about eight-mile intervals in the West Driefontein area. They penetrate both the dolomite and the Witwatersrand beds and as they are impervious to water they divide the valley into a series of large underground water compartments.

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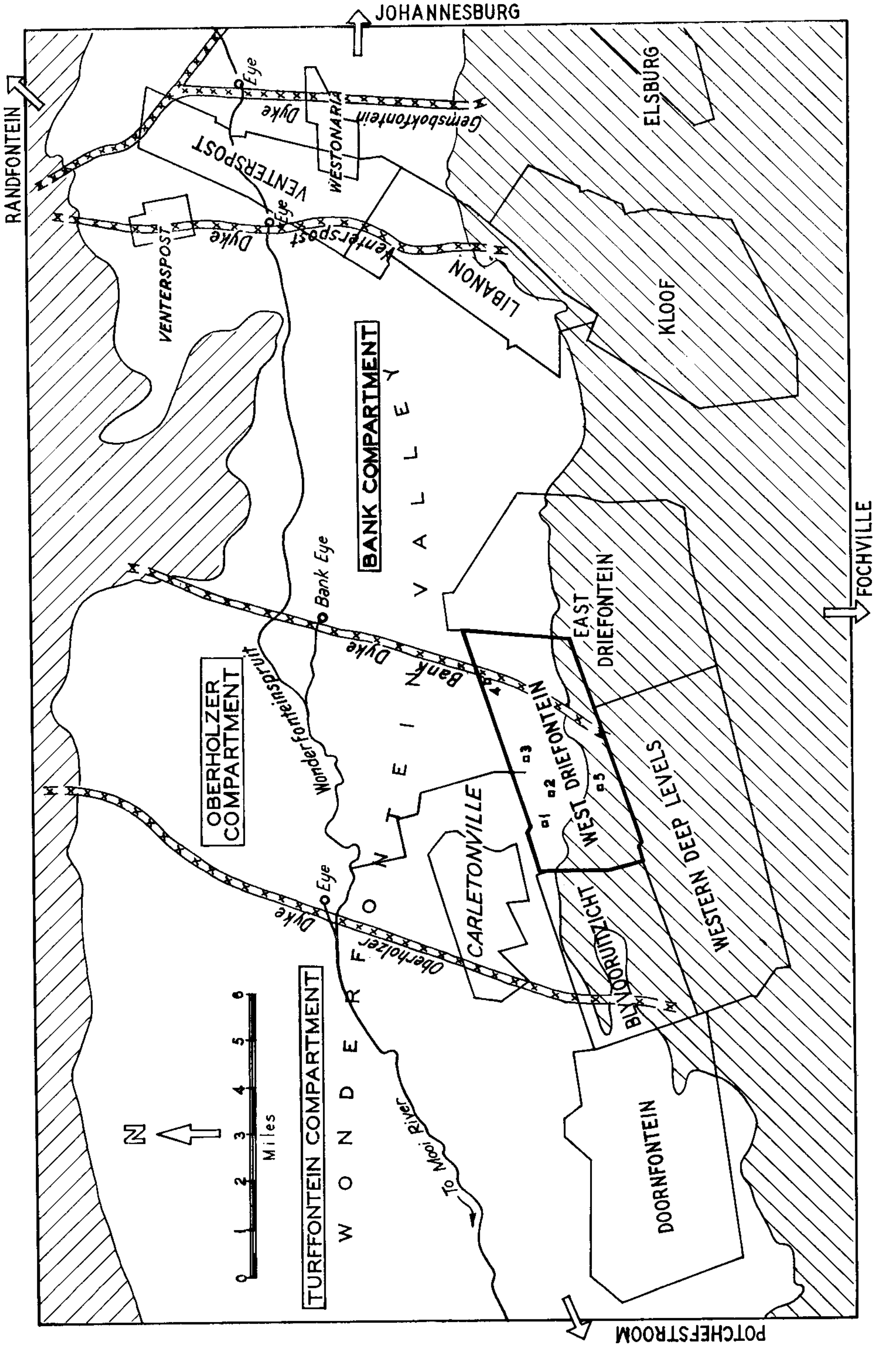

Fig. 1—Dolomitic compartments of the West Wits Line

The major portion of the West Driefontein mine occurs in one of these compartments, the Oberholzer compartment, but the eastern portion extends into the Bank compartment. Water flows from one compartment to another over the dykes at their lowest points of outcrop. These overflow points are the plentiful 'eyes' or springs that discharge into the Wonderfontein Spruit at successively lower levels in a westerly direction towards the Mooi River. Salient data about the eyes, before large scale pumping began, are given in Table I:

TABLE I

ELEVATION AND YIELDS OF THE EYES IN THE DOLOMITIC COMPARTMENTS

Eye	Elevation above sea level, ft	Dolomitic compartment	Average yield million gals/day
Gemsbokfontein	5,114	Gemboksfontein	1·9
Venterspost	5,051	Venterspost	4·6
Bank	4,927	Bank	10·8
Oberholzer	4,816	Oberholzer	11·9
Turffontein	4,650	Turffontein	4·05

Annual rainfall in the area averages about 26 in. and it has been estimated that some 10 per cent of this, or approximately 37½ million gallons per square mile per annum, drains into the dolomite through solution fissures that were originally faults, bedding planes and joints. The rainwater contains small quantities of humic and carbonic acids which, over the aeons, have dissolved away portions of the dolomite rock with the result that large interconnected subterranean caverns and open fissures now exist within it, providing storage for an enormous volume of water. Some of these caverns intersected during the sinking of No. 1 Shaft are shown in Fig. 2. The Bank compartment alone, which covers a dolomite area of about 60 sq. miles, is thought to contain more than 100,000 million gallons of water. This water would not be an embarrassment to mining operations, other than in the sinking of shafts from surface, were it not for the presence of tension faults which cut through the dolomite and down into the Witwatersrand beds and so to the mine workings. These act as channels through which water from the dolomite reservoirs can flow, as shown diagrammatically in Fig. 3. Tension faults are more numerous in the West Driefontein and Venterspost areas than in the Libanon and Doornfontein areas, which explains why there are big differences in the quantities of water that, in normal circumstances, have to be pumped from the four mines. This is illustrated in Table II.

PRECAUTIONS

By the time West Driefontein came into production a great deal of information about the water hazard on the West Wits Line and methods of overcoming or at least minimizing it had been accumulated. Most of this knowledge was gathered as a result of operations at Venterspost and on the basis of this experience a standard precautionary code of practice was evolved. A brief description of some of the measures laid down follows.

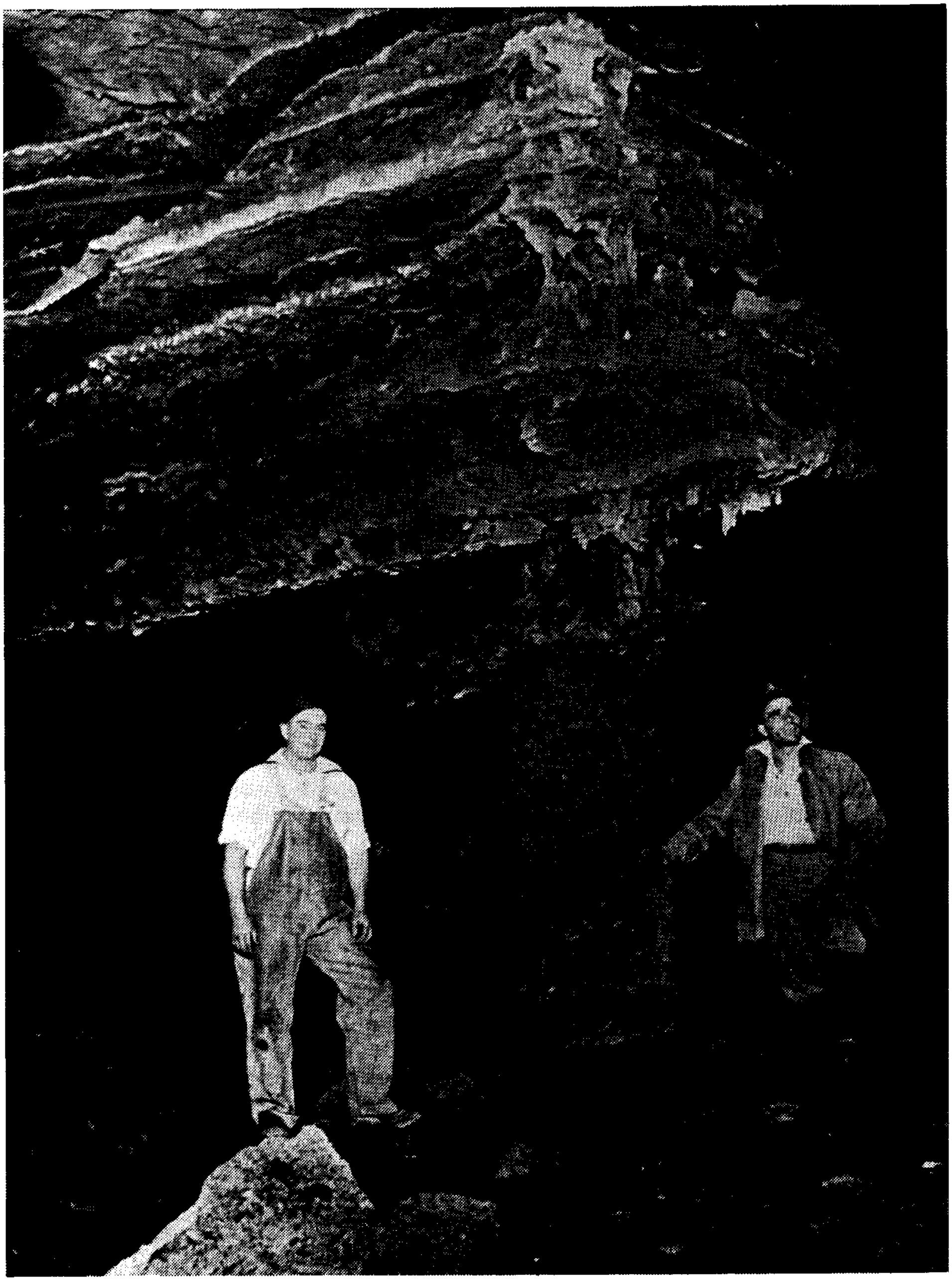

Fig. 2—Solution cavity broken into at 400 ft below surface, while sinking No. 1 Shaft

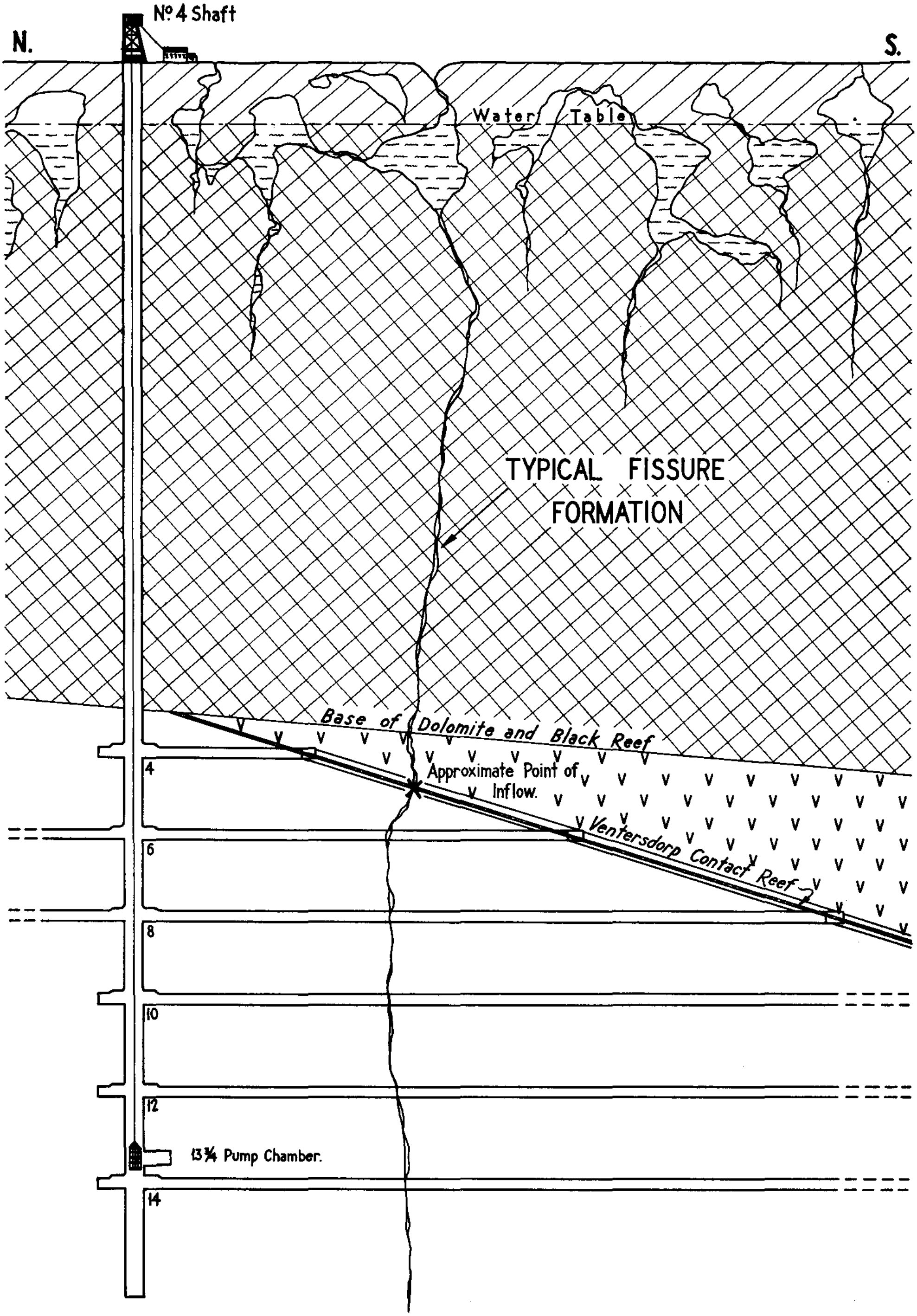

Fig. 3—Diagrammatic section showing solution cavities and fissure formation

TABLE II

WATER PUMPED FROM THEIR WORKINGS BY CERTAIN MINES ON THE WEST WITS LINE

	Pumping position October, 1968		Maximum quantities pumped	
	Available pumping capacity m.g.p.d.	Water pumped to surface m.g.p.d.	Rate m.g.p.d.	Period
Doornfontein	7	3·0	3·5	1960
West Driefontein	63	17·0	32·0	1963
Libanon	9	2·8	2·8	1968
Venterspost	20	9·5	12·5	1961

Shaft sinking

The bottom of a sinking shaft is advanced within a spiral pattern of cover holes up to 150 ft in length and so drilled that coverage of up to 10 ft outside the periphery of the shaft is ensured (Fig. 4). When a new series of holes is drilled, an overlap of 40 ft is allowed. If water is intersected in a fissure ahead of the face, additional holes may be drilled and all are injected with cement under high pressure.

If the fissure is a large one the shaft bottom is advanced near to it after the initial cementation process has been completed. A concrete mat is then cast on the bottom of the shaft and the shaft walling extended to match it. After the mat has set, further holes are drilled through it to intersect the fissure which is again cementated until it is certain that the water has been sealed off—a condition that is determined by the drilling of test holes. Sinking is then continued, using the same precautionary measures. Each fissure encountered is carefully surveyed in order to determine its probable further course.

Development

The records of fissures met with during sinking operations help to determine the most suitable pattern of cover holes to be drilled in development. But in any event every development end is advanced between at least two 450 ft holes the ends of which are never less than 195 ft ahead of the face and are always cementated (Fig. 5). When a waterbearing fissure is intersected by the cover holes the face is advanced to within not less than 20 ft of it. Further holes are then drilled into the fissure which is again cementated. As in shaft sinking, development then continues after test holes have established that the fissure has been effectively sealed off.

In addition to the foregoing precautions, pilot holes 4 ft longer than the longest drill steels are put in before the round is drilled, two of them at an angle of 20 degrees up and out and two at an angle of 20 degrees down and out. If water appears on the face, development is stopped and further holes are drilled and cementated. As a final precaution, there is available to each developer a set of emergency equipment to deal with any water that may be encountered in the face holes.

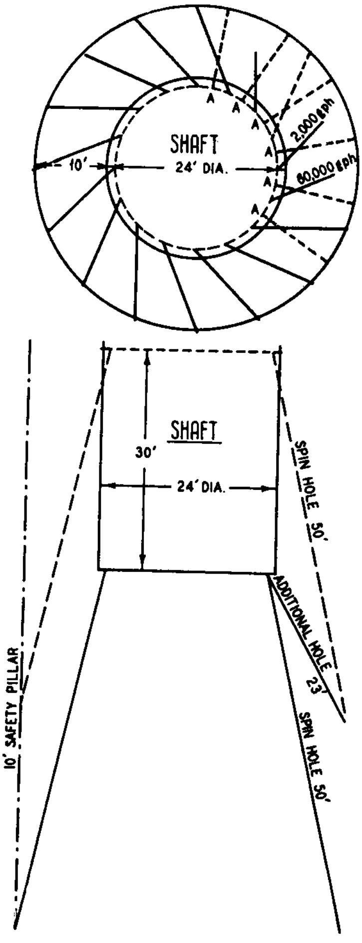

Fig. 4—Cover drilling in shaft sinking

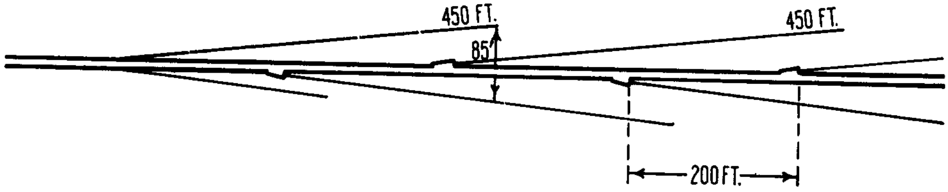

Fig. 5—Pattern of cover holes in lateral development

Stoping

Control of the inflow of water during stoping operations presents a more difficult problem. Experience at the Venterspost mine had shown that any method that relied on cementated cover holes in the hanging wall was only effective during the early stages of stoping. As stoping progressed and the distance excavated on strike increased, general sag of the hangingwall followed, causing the formation of fracture zones which extended above the stoped-out areas and beyond the ground previously cementated. Water then flowed through the fractures into the workings. Not unexpectedly the same pattern of events took place at West Driefontein and it soon became clear that there were no suitable methods available of providing an effective seal against the inflow of water into stopes. Such flow had to be accepted as inevitable and it therefore became necessary to devise means of coping with it and also of furnishing protection against any greater rate of flow than that which might reasonably be expected.

The solution arrived at was to increase the mine's pumping capacity to a point well in excess of its normal foreseeable requirements and to dewater the Oberholzer compartment in which most of the mine lies. Tests carried out early in the life of the mine, using dyes and radioactive isotopes, showed that water pumped from underground and discharged on surface seeped back into the mine workings within a few days. In order to prevent this a surface canal was built which conveyed the water to a point of no return (Fig. 6). This was towards the end of 1957 by which time the mine's pumping rate had climbed steadily to 18½ m.g.p.d. and its installed pumping capacity to 38½ m.g.p.d. The installation of the canal had the effect of levelling off the quantity of water made underground at the former figure and this state of affairs continued until the end of 1959 by which time installed capacity had been increased to 54 m.g.p.d. Between 1960 and mid 1964 an expansion of mining along the strike gave rise to an increased inflow of water into the workings and during the latter half of this period an average of 32 m.g.p.d. was pumped to surface. By then the installed capacity had been increased to 63 m.g.p.d. From this time on there was a steady decrease in the rate of inflow until 1967 when it steadied at around 17 m.g.p.d. of which 3 m.g.p.d. was coming from the Bank compartment. This was the position at the time of the inrush (Fig. 7).

Emergency storage

In addition to providing pumping capacity well in excess of the mine's normal requirements, it was thought wise to create emergency underground reservoirs in mined-out areas bordered by dykes and interconnected so that one could overflow into another. This storage capacity was designed to fill two needs, namely the ability to handle sudden inrushes, and the provision of surge capacity in the event of a general power failure. One such emergency reservoir, isolated from the other workings by concrete plugs, alone had a capacity of 11 million gallons. Water from any of the upper levels could be diverted into this reservoir and could be handled either by the pumps at No. 2 Shaft or the pumps at No. 5 Shaft; alternatively it could be passed into the Ventersdorp Contact Reef (V.C.R.) workings. Shafts Nos. 2, 5, 3 and 5A Sub-vertical were to comprise the main pumping system. Watertight bulkhead doors were constructed in the levels leading off Nos. 2, 5 and 5A Sub-vertical Shafts and mining was concentrated in the areas round these shafts thereby producing large potential reservoirs. Thus, in an emergency the bulkhead doors could be closed and the flood waters allowed to rise around the isolated shafts and their pump stations which would remain operative.

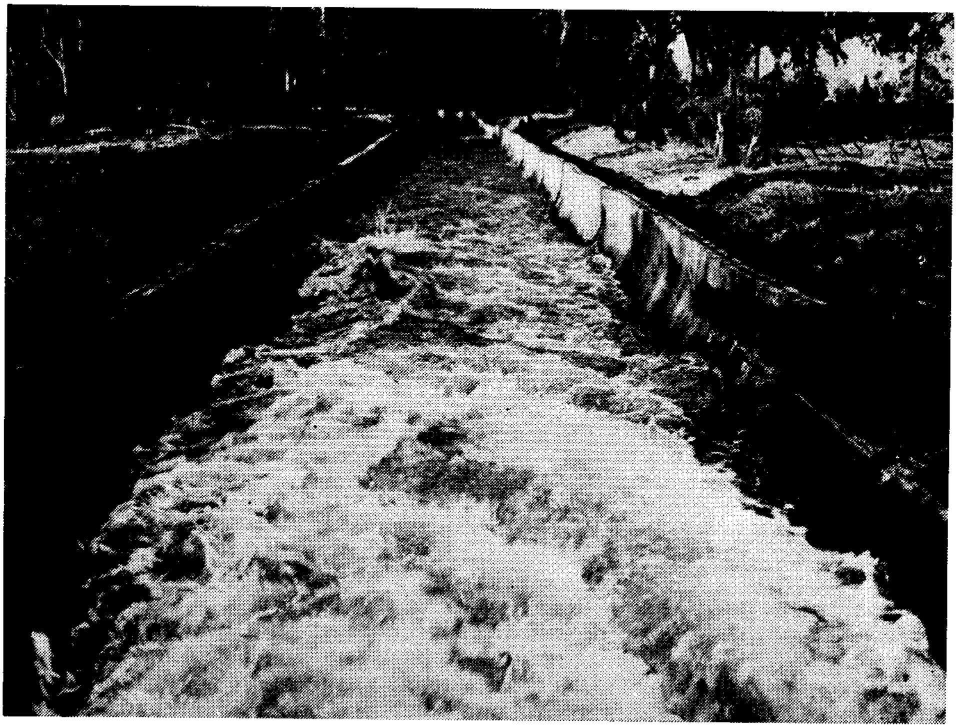

Fig. 6—Photograph of surface canal

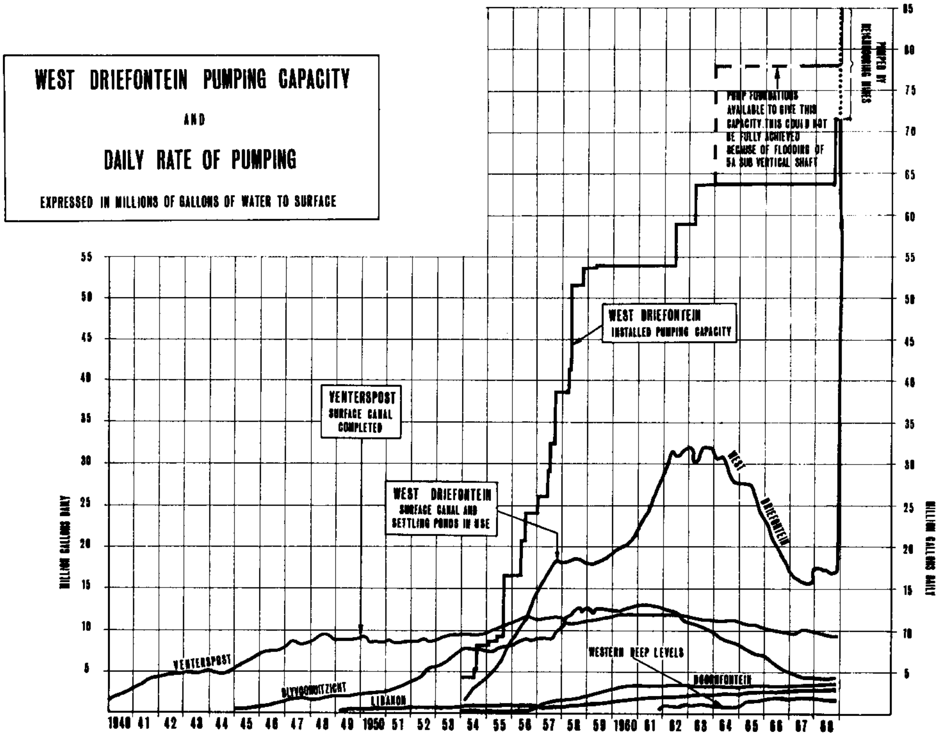

Fig. 7—Graph showing pumping rates of mines on the West Wits Line, and the installed pumping capacity at West Driefontein

The various pump stations were so situated that, by means of a series of interconnecting pipe columns and drains, water made in any part of the mine could be diverted to whichever of the different shaft pump stations had available capacity at any given time; complete flexibility of pumping was therefore provided. However, the success of the system was dependent on the bulkhead doors holding and the pumps having sufficient capacity to cope with the inflow of water.

Unfortunately, though not disastrously as it turned out, when this storage area was being filled after the October inrush, valves and flanges in the concrete bulkheads on 32 level, 5A Sub-vertical Shaft, sprang uncontrollable leaks and the shaft and its pump station had to be evacuated. (Pumping arrangements are discussed in more detail in a later section).

SITUATION PRIOR TO THE INRUSH

At the time of the inrush mining by West Driefontein, along a strike distance of about 3,000 ft, had already taken place from No. 4 Shaft in the neighbouring Bank compartment in the north eastern corner of the property (Fig. 8). This compartment, in contrast to the Oberholzer compartment, had not been dewatered. The

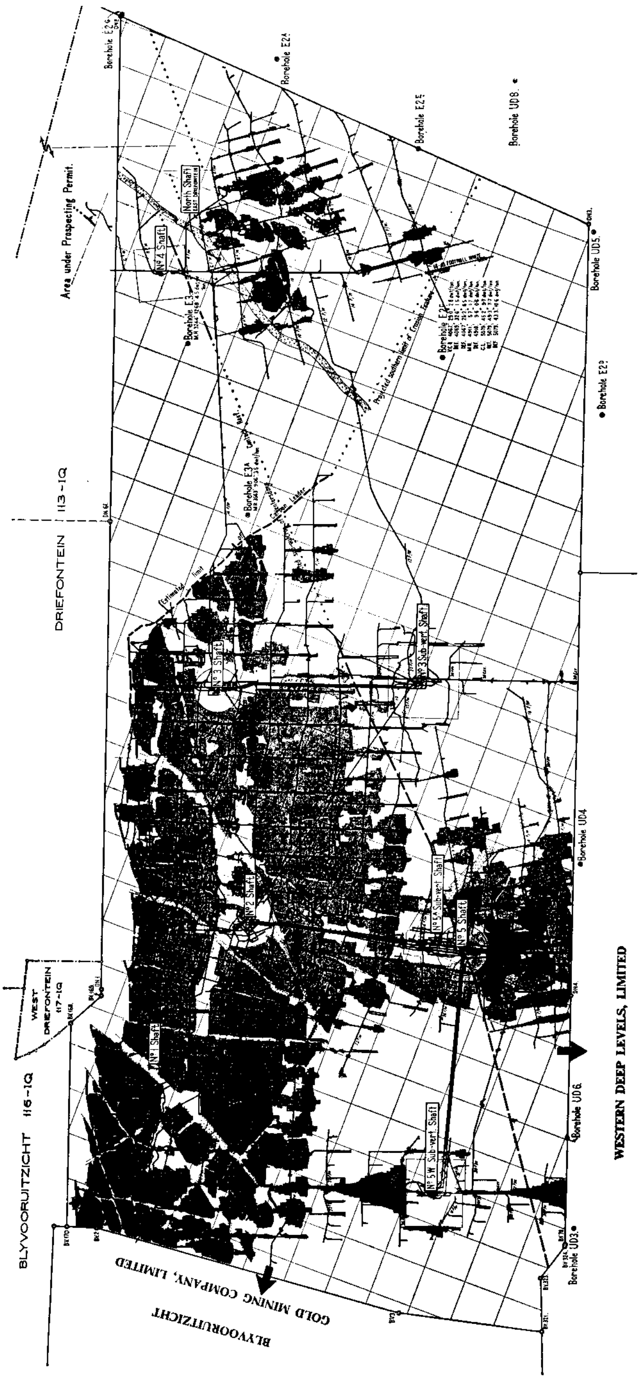

Fig. 8—Plan of mine workings. (Grid squares are 1,000 ft × 1,000 ft)

Libanon and Kloof mines had also mined in this compartment—particularly Libanon where stoping had taken place in the Bank compartment for virtually the whole strike length of the property—and at no stage had there been any evidence of the existence of fissures through which large quantities of water could flow into the mine workings. Had these been present it could have been expected that they would have been detected by the normal precautionary measures adopted.

The defences of the West Driefontein mine at this time against danger from flooding appeared virtually impregnable. Pumping capacity was 63 m.g.p.d. and foundations were available for the installation of further pumps which could increase this figure to 78 m.g.p.d. to give a factor of safety at the then existing rate of inflow of over four and a half. In the worked-out areas of the mine, in extent some 43 million sq ft equivalent to 1·54 square miles at an average width of about 45 in., mainly on the Carbon Leader Reef horizon, there was an estimated storage capacity for 1,000 million gallons of water, which seemed large enough to accommodate any surplus that the pumps might not be able to cope with should an unprecedented inrush of water occur. The biggest inflow ever experienced in these fields had been 6 m.g.p.d. at Merriespruit and 4 m.g.p.d. on the West Wits Line. The rate of inflow on the West Wits Line, however, was not maintained for more than few days, after which they dwindled.

NOTIFICATION OF THE INRUSH

On Saturday morning, 26th October, 1968, when the miner in charge of a stope on 6 Level, No. 4 Shaft, came on shift he was worried—the hangingwall of the stope between 4 and 6 Levels, at a depth below surface of 2,830 ft, had moved and some water was flowing in. He reported this to his shift boss who in turn informed the mine overseer. At 9.30 a.m. a fissure opened in the stope and large quantities of water rushed into the mine.

In view of the situation already described it is not surprising that no immediate alarm was felt. First reaction was that it was a minor emergency that could be dealt with in a routine manner—as so many others had been—by plugging the fissure or simply diverting the water. To those underground, however, it soon became clear that this was no ordinary inflow and that there was serious danger to a great many people working in many parts of the mine, not least in the No. 4 Shaft pump stations. A very wise decision was taken to start withdrawing personnel immediately and some highly courageous actions were performed.

At the time there were approximately 13,500 men in the mine, of whom about 1,200 were in the No. 4 Shaft area. Of these some 400 were hoisted up No. 4 Shaft while the remainder scrambled along 10 and 12 Levels, the only connections to No. 3 Shaft and other operating shafts. Even at this stage it was not realized what tremendous quantities of water were pouring into the mine; nor was it appreciated at the time that what were thought to be over-generous precautions to meet any such emergency would prove inadequate and would have to be supplemented by drastic measures that were only just able to avert complete flooding of the mine. Among these measures was the relief provided by the Blyvooruitzicht and Western Deep Levels mines, both of which made diamond drill hole access with West Driefontein through which they were able to draw off, between them, more than 11 million gallons of water a day.

PLAN OF CAMPAIGN

By Sunday morning there was no doubt at all that the flow of water entering the mine was quite exceptional even by West Wits standards. The photograph, Fig. 9, shows large quantities spilling into No. 4 Shaft. A command post had been set up at

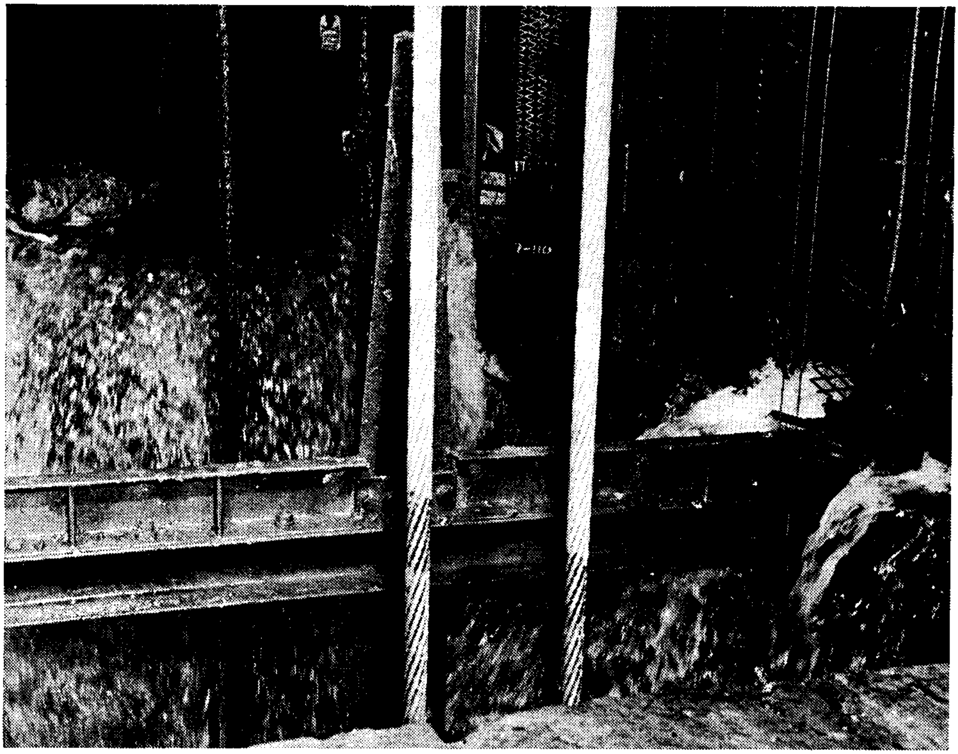

Fig. 9—Water from the inrush overflowing into No. 4 Shaft

No. 4 Shaft and the whole attention of management was directed at getting accurate information about what was happening underground and controlling the directions in which the water was flowing to ensure that it did not flood any of the pump stations. As in a military operation planning fell into three main categories: tactical planning; the establishment and maintenance of supply lines; and long term strategic planning.

Tactical planning

Immediate plans were made and put into effect for the routing of the water to the pumps that could best handle it; for the storage of the excess water, which the pumps could not handle, in stoped-out areas; for the closing of the watertight bulkhead doors to prevent flooding of the shafts; and for the protection of essential equipment from water damage. This was no easy task. Both 10 and 12 Levels were running about 4 ft deep in water, the force of which was enough to overturn empty hoppers and to carry in its path all manner of debris which caused blockages and added greatly to the difficulties of operation.

These activities were all controlled from the command post at No. 4 Shaft which was continuously staffed by at least one, usually two, and often three of the most senior officials on the mine.

Supply lines

Urgent attention was given to the organization and supply of European and Bantu personnel so that teams could be formed to do the required work. Additional teams had to be kept on immediate standby to deal with unforeseen emergencies that might arise and a roster of relief teams had to be drawn up.

Arrangements were made for the supply of materials and tools, of food and drink for the men working long hours underground, for the replacement of cap lamps where necessary, for the provision of meals on surface and even dormitory facilities for key men at the shaft heads.

Of vital importance were the arrangements for the ordering, receipt, handling and installation of additional pumps, cables, transformers and similar equipment (Fig. 10.)

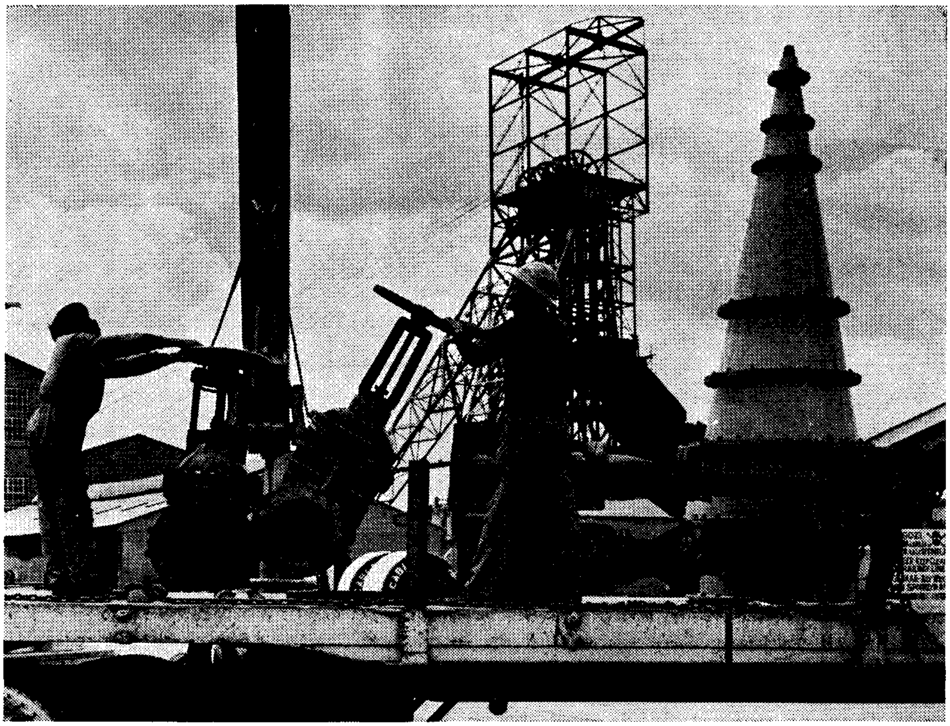

Fig. 10—High pressure valves arriving on the mine

Ambulance stations were kept fully manned and a doctor was on immediate call. It is of great credit to all who played a part in the rescue operation that from start to finish there were no fatalities or even serious accidents.

Strategic planning

As more information became available, it was possible by Tuesday, 29th October, to make the first reasonably accurate estimate of the volume of the inrush of water, which was measured at 85 million gallons a day (40·4 m.g.p.d. on 10 Level and 44·6 on 12 Level), so that, together with the normal quantity of 14 m.g.p.d. made from the Oberholzer compartment, the total ingress amounted to nearly 100 m.g.p.d.

The two neighbouring mines which have common boundaries with West Driefontein—Blyvooruitzicht and Western Deep Levels—offered to put boreholes through their boundary pillars below the level of the flood waters and so supplement the pumping capacity of West Driefontein iteslf. These offers were gratefully accepted.

Western Deep Levels had to reopen and re-equip 7,000 ft of drive, develop a tunnel for 150 ft through solid rock and then put through diamond drill holes into the lower portion of the V.C.R. workings in West Driefontein. Blyvooruitzicht had to do similar work and West Driefontein had to drive a tunnel towards the boundary pillar. The holing took place on West Driefontein 20 level horizon (Fig. 8).

Despite these offers of valuable assistance, which in any case would, it was thought, take two or three weeks to materialize, consideration had to be given to the fact that should the water rise above the critical 14 Level horizon, the main pump stations at Nos. 2 and 5 Shafts would be drowned and all hopes of saving the mine would be lost for a matter of years.

It was realized that even with increased pumping capacity there would be a limit to the volume of water that could be removed from the mine, and that it might not be possible to maintain this limit because of potential power failures or breakdowns, or both. It was also known that the margin of safety in terms of time, even assuming that there were no calamities, was virtually nil. Accordingly plans had to be drawn up and preparations made for the installation of plugs in the shafts immediately below the relay pump stations, so that future dewatering of the mine could be tackled from these points and not from surface.

In the meantime every possibility of increasing pumping capacity by any means had to be investigated and, if possible, planned for and put into effect. These investigations included the installation of further pumps and their ancillary pump columns, cables and transformers. Consideration was also given to the use of skips for baling from the shaft bottoms. The feasibility of choking the fissure and so reducing the flow of water into the mine was examined. It had been calculated that with the pressure exerted by a head of 2,800 ft of water the equivalent size of the opening where the inrush occurred could be as small as 1 sq ft in area; the nature of the opening or openings was not known but it was thought probable that they consisted of a large number of narrow cracks extending over a considerable distance. However, the location of the fissure by means of boreholes from surface would alone have taken months and the sealing process, even if possible, would have been a very lengthy one.

The possibility of supplementing existing pumping capacity from 4 Level, No. 4 Shaft, was considered. The plan was to install two sets of pumps, each set having a capacity of 4 m.g.p.d., pumping through a 14 in. diameter column. A start was made on the placing of the power cables, the installation of the first column and the casting of the pump foundations on 4 Level. When it was subsequently decided to isolate No. 4 Shaft by putting in plugs on Nos. 10 and 12 Levels, which, if successful, would result in complete flooding of the shaft, this work was discontinued.

It was evident from observations of the existing surface boreholes, which were immediately reopened, and also from the drying up of the Bank 'eye' that the inrush was connected with the whole vast reservoir in the Bank compartment and therefore there was little hope of the inflow diminishing within a reasonable time.

While considering all these possibilities it became clear that the only positive way of controlling the inflow would be to isolate No. 4 Shaft from the rest of the mine by means of massive plugs on 10 and 12 Levels which were the only connections between that shaft and the rest of the mine. How this was done is dealt with later. In the meantime reorganization and expansion of the pumping arrangemenst had to continue with all possible speed.

REORGANIZATION AND EXPANSION OF PUMPING ARRANGEMENTS

At the time of the inrush the mine had a total of 72 pumps, pumping in stages to surface, each with a capacity of 2 million gallons per day. Within 12 hours every pump that had power to it was in operation. This was a remarkable achievement ranking high in the annals of maintenance. It should be borne in mind that this also entailed distribution of the abnormal flow of water throughout the mine to the appropriate pump sumps.

Distribution of water

Figs. 11 and 12 show the route along which the inflow from the Bank compartment of 85 m.g.p.d. travelled. In addition there was the normal inflow of 14 m.g.p.d. from the Oberholzer compartment. Of this total 72 m.g.p.d. were pumped out of the mine. Of the initial inflow of water 35 m.g.p.d. flowed in the main crosscut on 12 Level and most of it entered No. 3 Sub-vertical Shaft and ran along 30 Level to Nos. 5A and 5W Sub-vertical Shafts. The remaining 50 m.g.p.d. flowing in 10 Level crosscut (Fig. 13) was diverted to the shafts which could best handle the water.

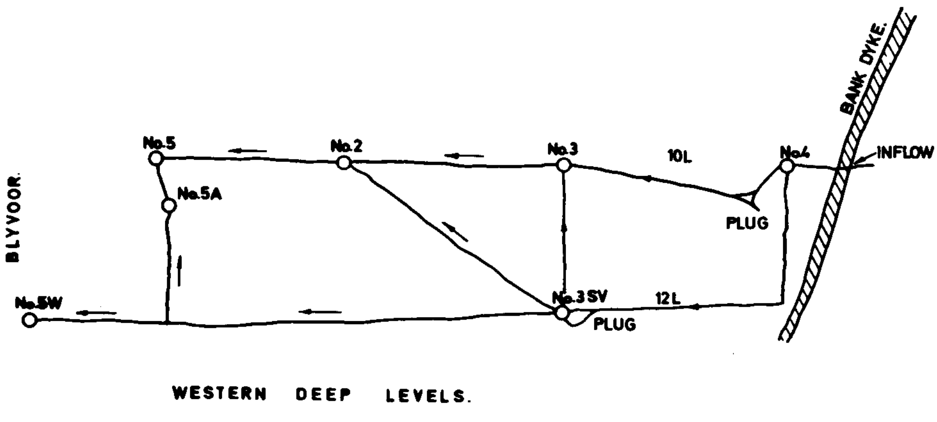

Fig. 11—Plan of routes along which inflow from the Bank Compartment travelled

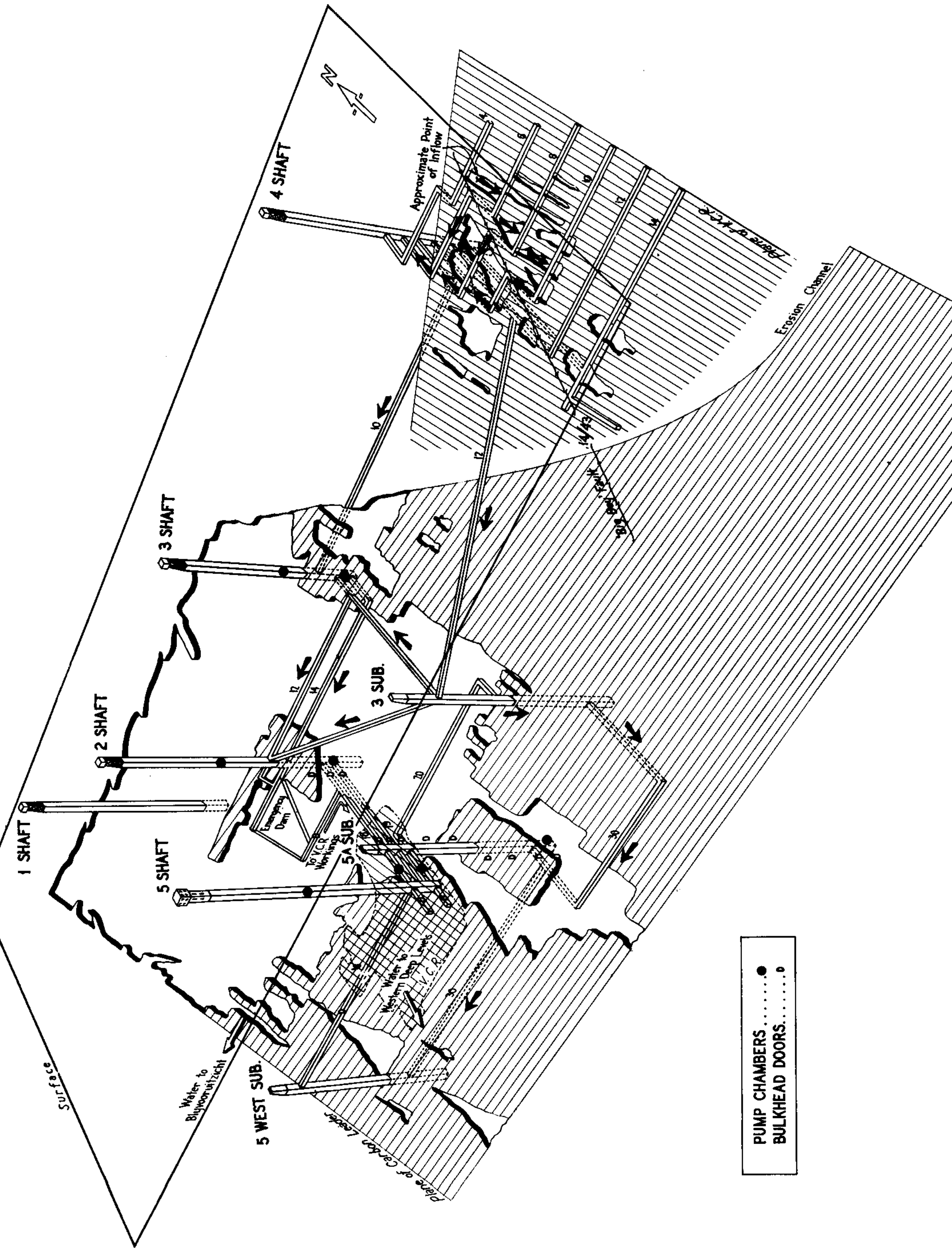

Fig. 12—Diagrammatic view showing routes along which inflow from the Bank Compartment travelled

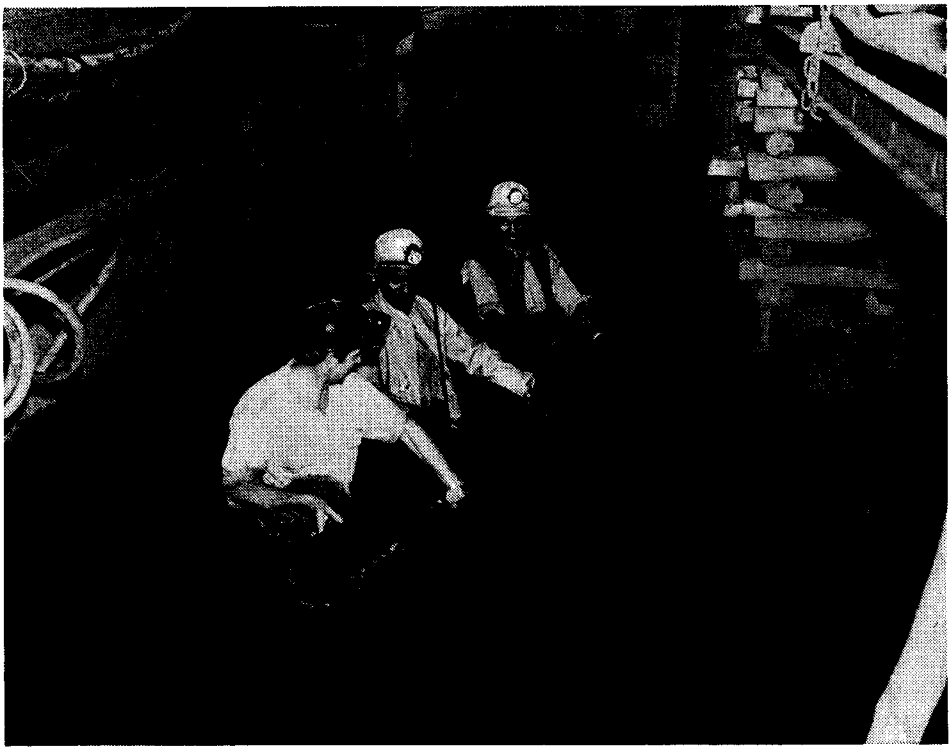

Fig. 13—Flow of 50 million gallons per day on 10 Level

Permanent pumping arrangements

The original pumping arrangements whereby 32 pumps pumped 63 m.g.p.d. to surface was increased to 36 pumps pumping 72 m.g.p.d. by the use of equipment borrowed from mines of the Gold Fields Group and other mining companies (Fig. 14).

The installation of 10-stage pumps, each complete unit weighing approximately 10 tons, was a major undertaking; high pressure valves had to be found, delivery piping specials of $\frac{3}{4}$ in. wall material manufactured, suction piping specials made, switchgear installed and in some cases modified, and foundations cast.

The pumping arrangement was so designed that, in the event of the development of sinkholes in the No. 2 Shaft area—where the crushing plant had been lost in the 1962 disaster—and consequent possible loss of that shaft, water could be transferred to Nos. 3 and 5 Shafts.

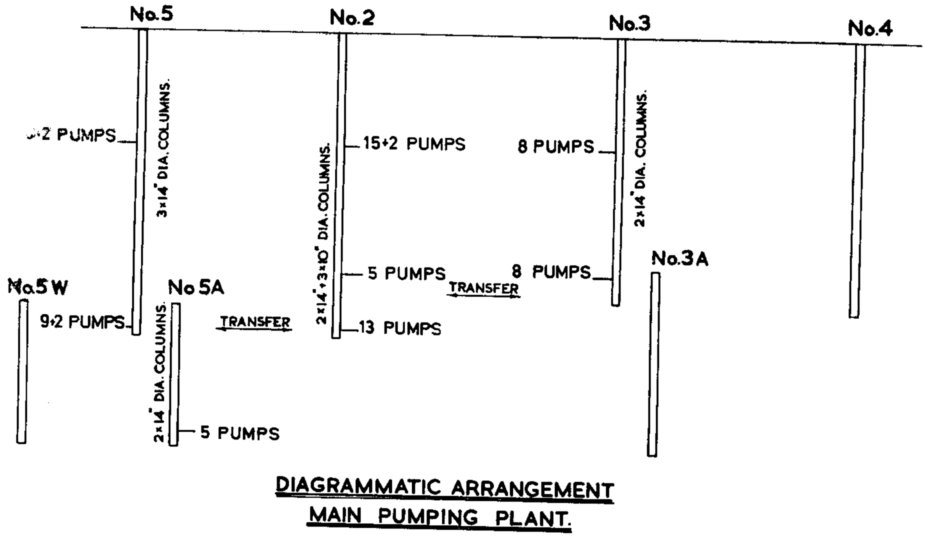

Fig. 14—Diagrammatic arrangement of main pumping plant

Electric power supply

The normal mine load is 78 MW; this jumped to 125 MW within the first day after the inrush and reached a peak of 137 MW. When the concrete plugs on 10 and 12 Levels were closed it dropped to 48 MW. Because of the overload at No. 5 Shaft the Electricity Supply Commission (ESC) were asked to install two additional 3·2 MVA transformers; this they did in 32 hours without stopping other supplies. This operation entailed moving the 15-ton transformers over bare ground a distance of 60 ft under live 40 kV conductors.

As it was proposed to install pumping plant at No. 4 Shaft, if time permitted, the ESC were requested to replace four existing 2·5 MVA transformers with four new 3·2 MVA units and, in addition, provide three 5 MVA transformers under the same conditions as existed at No. 5 Shaft.

To distribute extra power from No. 2 to No. 5 Shaft in order to cope with the extra pumping load, a total of 26,000 ft of 6,600 volt, 0·25 sq in. cable was laid on surface and a further 21,000 ft of existing similar cable was picked up and relaid underground. One of the mines of the Group provided a 9-panel, 6,600 volt switchboard, and with its own electricians, installed and commissioned it in 19 hours.

Under these conditions it can be appreciated what apprehension prevailed on the mine whenever there were thunderstorms in the area. A power trip-out, and these happen fairly frequently, with consequent loss of pumping capacity, could have been disastrous. However, the electrical department, in conjunction with the ESC, were so well organized that when a trip-out did occur power was restored in 2 minutes 40 seconds.

Pumping arrangement—No. 5 Shaft and No. 5A Sub-vertical Shaft (Fig. 15)

It had been foreseen that a sudden uncontrollable inflow of water might occur and No. 5A Sub-vertical had been made a pumping shaft in case the rest of the mine should have to be abandoned. As previously mentioned each level was protected by bulkhead doors on either side of each shaft station and a pumping installation with settlers was constructed on the lowest level of the mine—32 Level. The water was pumped 2,360 ft up the shaft through two 14 in. diameter columns to the pump station on 18 Level, No. 5 Shaft, which was also protected by bulkhead doors, and then up the shaft, a further vertical distance of 5,275 ft. The bulkheads were constructed with pipes through them for drainage, compressed air, water service and ventilation, and a special pipe to carry cables; each pipe was provided with a high pressure valve on the dry side.

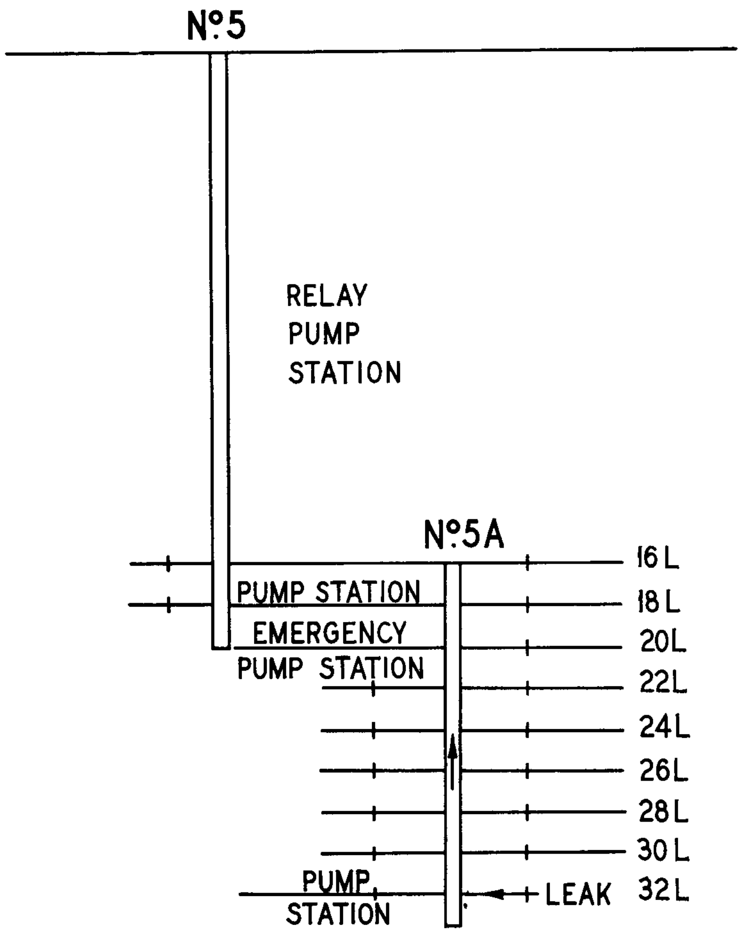

Fig. 15—Pumping arrangement—No. 5 Shaft and No. 5A Sub-vertical Shaft

After the inrush, as the water crept up the mine on the outside of the closed bulkhead doors, with the pressure steadily increasing, leaks through the valves and flanges began to occur on 32 Level, making it necessary to install pumps stripped from the reduction plant to lift this water into the pump sumps. (32 Level was the sump chamber elevation.) In the meanwhile sufficient water was being drained through a valve on 30 Level to supply the five pumps on 32 Level (10 m.g.p.d.).

When the water was 1,500 ft above 32 Level a plug used to block a hole drilled for a pressure gauge in the bulkhead door blew out. Notwithstanding repeated attempts to plug the hole, the inflow increased. A major leak also occurred through a 4 in. pipe when the gasket in a blank flange blew out. At the high pressure then prevailing any small leak rapidly eroded bolts and flanges away. Eventually the inflow increased beyond the means to pump water into the main sump and the shaft had to be abandoned.

A new problem now existed—the water was now flowing into and up 5A Sub-vertical Shaft inside the bulkhead doors and as the level increased the water threatened eventually to drown the No. 5 Shaft pumps on 18 Level. The solution was to install pumps on 20 Level station to collect the water flowing up the shaft at this level and pump it to the 18 Level pump chamber. Eventually, after two weeks, ten 2 m.g.p.d. pumps with 250/300 hp motors had been installed. Of these pumps eight (two were available on the mine) had to be manufactured, pipes and specials fabricated, electric power provided with switchgear, and all installed on a normal shaft station without any lifting facilities whatsoever, and with limited space (Fig. 16). Because of the small area available, sump capacity was minimal and the pump station operated on a 5 minute surge capacity.

The difficulty in operating this pump station was that as the difference in head of the water on the outside and inside of the shaft increased so did the flow, which in turn caused greater erosion through the leaks, still further increasing the flow. There was therefore an unrelenting battle to install sufficient pumps to meet the ever-increasing flow.

As a last ditch stand, the 18 Level pump chamber was walled off from the shafts and arrangements were made to let the water flow directly into the pump suctions.

From the foregoing it is clear that the only means of saving the mine was to construct plugs on 10 and 12 Levels while still maintaining all emergency pumping measures.

MAJOR FACTORS IN THE BUILDING OF THE PLUGS

The need for speed was the vital factor in the construction of the plugs. This was complicated by difficulty of access to any possible plug site because of the huge volumes of water—40 to 50 m.g.p.d.—flowing on each of the two Levels. Also there was a potential head on 12 Level of 3,830 ft, so it was clear that the plugs would have to be very big structures and that any relief pipes built into them to carry the flow would have to be capable of withstanding high pressures. It was obvious that such relief pipes should be of the largest diameter possible compatible with the ability to manhandle them through rivers of water 3 to 4 ft deep and to fit valves capable of standing up to the high pressures that would be encountered. However, it also had to be borne in mind, when considering the size of the pipes, that the space occupied by them in the plugs must not be such as to endanger the structure, which would have to be strong enough to withstand the head required to force the water through the limited number and size of the pipes it would be possible to use. The volume of water flowing in the drives was so great that no cofferdam could be built against it strong enough to force the water through relief pipes in a plug. The only solution was to follow normal dam building practice and divert the flow.

As will be seen from the plan in Fig. 17 there were two possible plug sites on 10 Level and one on 12 Level. The site on 12 Level was near the station on No. 3 Sub-vertical Shaft and would involve a relatively short distance of travel against the flow of water. Neither of the sites on 10 Level was easily accessible. However, it was not yet known whether the water could be diverted nor whether the rock structure was suitable for the building of plugs. Reconnaissances made on 29th October revealed that both of these conditions could be met, the former with difficulty. Of the two sites on 10 Level only one was found to be practicable and even this would involve a travelling distance of about 1,000 ft through running water.

Owing to the difficulty of access it was realized that transport of materials would be a major problem. While such items as pipes and valves would have to be manhendled to the sites, the transport of concrete or even stone pack for actually building the plugs was out of the question in view of the very large volumes involved. The plugs would therefore have to be built of sand/cement mortar which could be mixed on surface and pumped directly into place, thereby solving handling and transport problems. The grout ranges through which the mixture would be pumped were

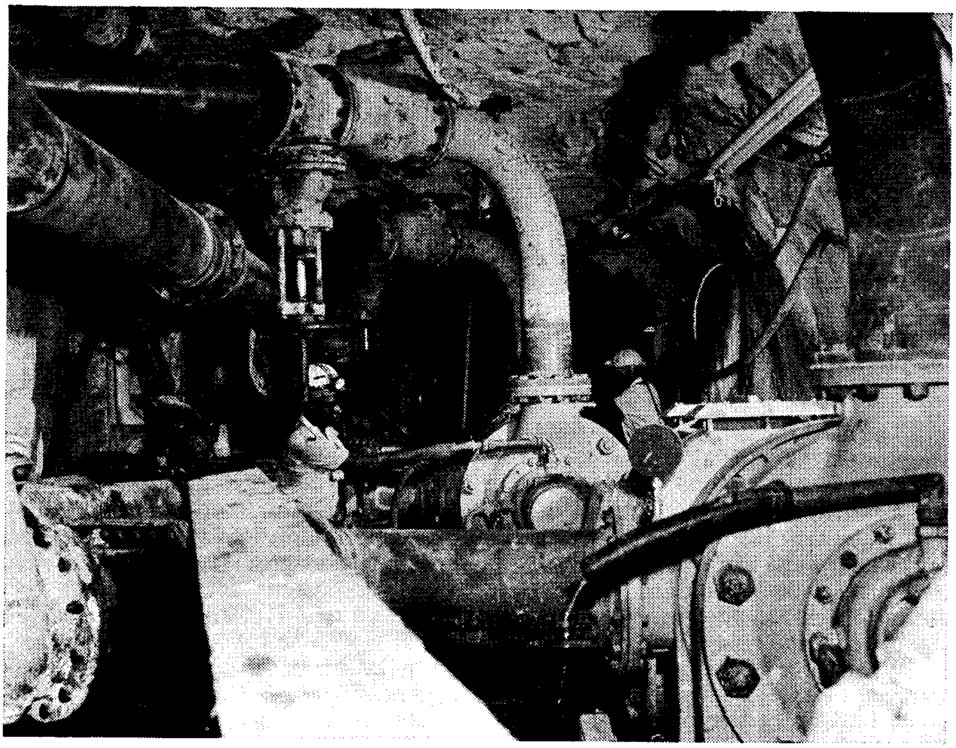

Fig. 16—Emergency pumps installed on 20 Level station at No. 5A Sub-vertical Shaft

already in place in Nos. 3 and 4 Shafts and consisted of standard 1½ in. high pressure pipes with special joints, while the ranges partly in place underground were of standard 1 in. piping.

The distances from the pumping stations on surface to the selected plug sites on 10 and 12 Levels were 6,000 ft and 11,000 ft respectively. Previous experience indicated that although a wide variety of grout mixes could be pumped over long distances, there was always the serious danger of blockages. This danger led to the decision to use a 1:1 sand/cement mortar, with rapid-hardening cement, having a water: cement ratio of 0·8:1, as likely to be strong enough, without reinforcing, and to be least liable to form blockages (the grading of the sand used is given in Appendix 1). The use of aluminous cement or additives for high early strength was considered but not adopted as they would have introduced risk of blockages, added complication or simply delay in getting started. The guiding factor was to avoid blockages at all costs, and to improve the chances of success further, only experienced men with the necessary knowledge were allowed to mix the grout.

DESIGN CONSIDERATIONS

Design assumptions

The accepted code of practice for plug construction by the mortar intrusion or 'colcrete' process calls for a plug to be built of packed clean rock into which a grout mixture of water/cement ratio of 0·6:1 is introduced. Because neither of these important specifications could be adhered to in the circumstances prevailing at West Driefontein and because several site and design factors were bound to remain

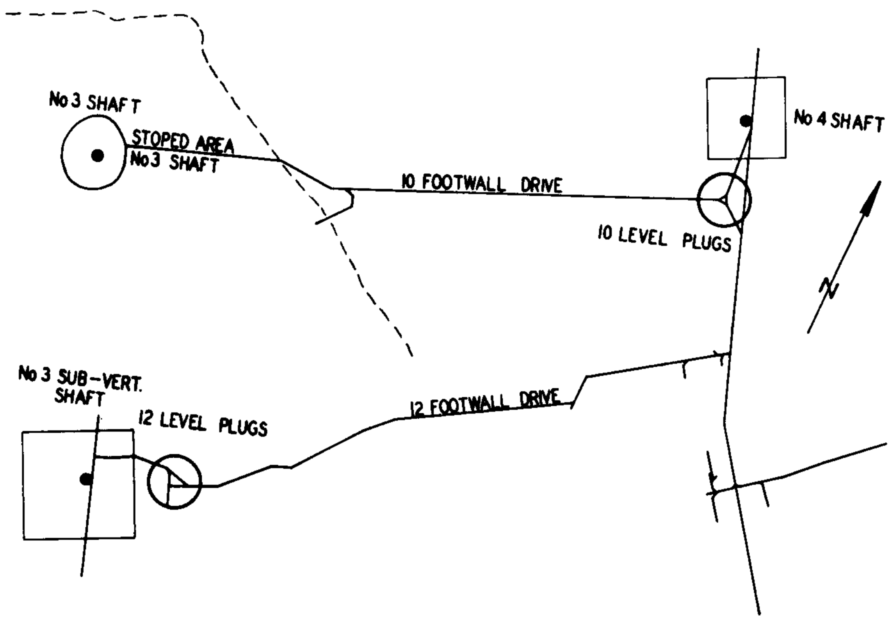

Fig. 17—Plug sites on 10 and 12 Levels

uncertain until the actual work was in progress, preliminary designs had to be prepared on broad assumptions.

It was acknowledged from the start that any plugs built would not be reinforced and that no enlargement of excavations would be undertaken to give added grip. Experimental work carried out in 1957 and 1958 proved beyond doubt that these precautions were entirely unnecessary if plugs were constructed in rough-sided parallel excavations: the saving in time by their omission would be critical in cases of emergency construction.

The volumes of water flowing in each of the drives was taken as 40 m.g.p.d. the size of the drives as 10 ft × 10 ft and the full static head as 3,830 ft on 12 Level and 3,472 ft on 10 Level.

Design formulae

The accepted formula for design of plugs envisages the plug being resisted by a punching shear around its perimeter. In two papers by Campbell Pitt and Garrett[1, 2], the behaviour of a number of plugs subjected to pressure was studied and the conclusion reached that the stress conditions in a plug were extremely complicated and that 'Whatever the correct theory, therefore, as to the stress conditions in the concrete, the rock becomes the governing factor'. Further they state: 'From a number of practical results that have been obtained in recent years, it is the authors' belief that leakage past a plug and through the rock surrounding it is likely to be the most important factor in design for high pressure under our conditions on the South African gold mines, and that being the case, it is necessary to design plugs of greater length than would be required for purely structural reasons.'

There was no doubt from the test work done that a parallel plug in a rough rock excavation will resist enormously great loads, but that the leakage past it can be a far more critical factor.

The most important point arising in the design was to determine the earliest time at which the plugs could be expected to withstand the full pressure safely.

Plug strength

Using accepted and well known cement technology, supported by tests made on grouts, curves of strengths of proposed mix against age were prepared (Fig. 18) and used to assess the probable strength of the plugs at various ages. From the curve for rapid-hardening Portland cement grout, the unconfined compressive strength of the material could be expected to reach 2,300 lb/in.^{2} after a 16-day setting period, and 1,500 lb/in.^{2} after a 7-day setting period. Shear strength was taken at 1/20th of this latter figure and used in the accepted formula for the length of a plug.

The greatest unknown factor of all was when the plugs would be required to resist pressure and for safety this period was taken as 7 days. Sixteen days was selected as being about the time the strength of the grout proposed should take to reach the equivalent of concrete made of ordinary Portland cement of the same water/cement ratio at 28 days.

The orthodox calculations for unreinforced plug length in rough-sided parallel excavations are given in Appendix 2 using the foregoing assumptions, together with confirming calculations for the plugs as actually built.

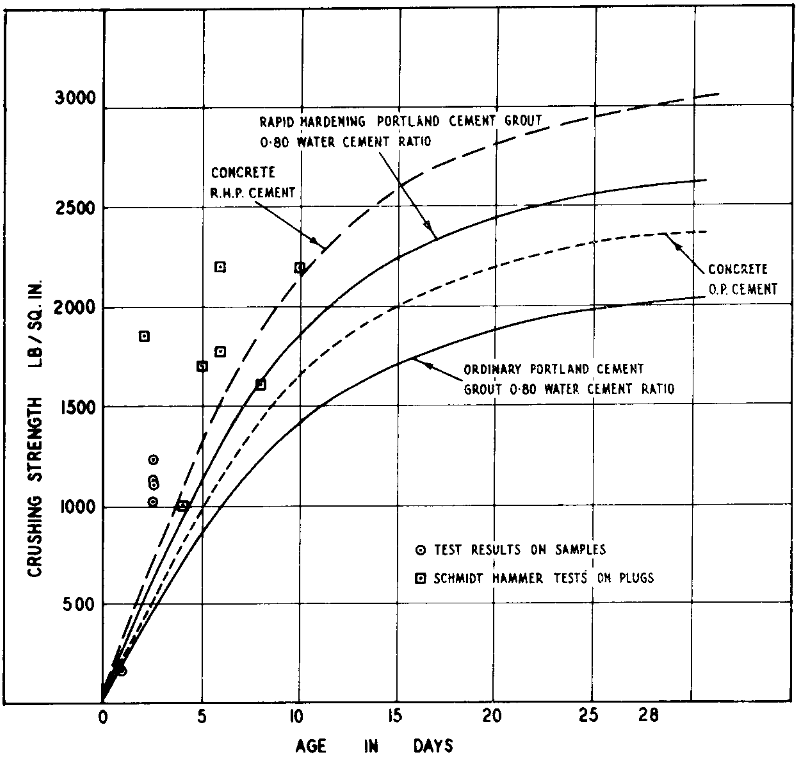

Fig. 18—Age-strength relationship of grouts consisting of 1:1 sand cement mix.
Sources of information:
Concrete technology—Fulton
Design of concrete mixes—Road Research Laboratory
Work tests—strengthening of Churchill Dam

After the design of the plugs had been decided upon and construction of them begun, samples of the grout as pumped were recovered, as the work proceeded, and tested by the Portland Cement Institute. The results are recorded in Appendix 3 and also on Fig. 18. From these it can be seen that actual strength exceeded design expectations. There were two factors making for greater strength on site than in design, both of which had been ignored throughout. Firstly, much of the surplus water in the grout mixture was forced out as it was placed, thereby improving strength. A check made by the Portland Cement Institute on the water/cement ratio of a grout sample as placed gave figures of from 0·63:1 to 0·65:1. Secondly, the heat of hydration in the plugs probably produced temperatures approaching boiling point and this would have greatly accelerated setting and increased early strength. Finally, Schmidt hammer tests (Appendix 3) on the finished walls confirmed the tests on the material. It was assumed, therefore, that the strength of the material of which the plugs were made was at least equal to that shown on the curves of Fig. 18.

Accepting the 'punch shear' analysis of strength, it becomes necessary to assume a safe shear stress for examining the plugs at various ages. Information is recorded in Table III for several plugs which were accurately tested at known ages, as reported by Campbell Pitt and Garrett[2]. The average shear stress with grouted plugs using packed stone and normal Portland cement in the case of the first four plugs was 108 lb/in.^{2} at 28 days. (The fifth plug leaked.) This may be taken as a safe shear stress.

TABLE III

SHEAR STRESS AND PRESSURE GRADIENT ACROSS PLUGS ACCURATELY TESTED AT 28 DAYS

Mine and location	Dimensions	Pressure lb/in.^{2}	Shear stress lb/in.^{2}	Pressure gradient wet to dry face lb/in.^{2} ft
West Driefontein 8 Level	13 ft × 10 ft × 41 ft 8 in.	1,827	124	43·5
West Driefontein 12 Level	13 ft × 10 ft × 41 ft 8 in.	1,827	124	43·5
Virginia 31 Haulage South	13 ft 6 in. × 10 ft × 63 ft	1,650	75	26·2
Virginia/Merriespruit boundary plug ..	12 ft 3 in. × 11 ft × 36 ft	1,340	108	39·2
Virginia/Merriespruit temporary plug ..	12 ft 3 in. × 11 ft × 12 ft	1,340	324	111·7

Using this value and relating it back to the material used (Fig. 18) it is possible to draw a curve for the West Driefontein plugs giving safe stresses at varying ages (Fig. 19) and to plot on this graph the actual ages of the plugs assuming closure of the valves five days after completion of the last plug on 10 Level. The actual shear stress may be calculated as full load is developed on the plugs and it will be noted that the stresses are below the expected safe shear resistance of the material of which the plugs are made (Appendix 2).

Leakage factor

The earlier work of Campbell Pitt and Garrett[1, 2] indicated that leakage would be a vital factor and this aspect therefore received very close attention. Fortunately the same experimental results given in Table III provided a means of assessing safe conditions insofar as leakage past a plug is concerned. These results indicate that if the pressure gradient from the wet to the dry side of the plug is less than 38 lb/in.^{2} ft of plug, leakage will not take place. It is a reasonable assumption that erosion will bear a relationship to compressive strength and therefore a fairly reliable forecast can be made of safe resistance to leakage at any given age (Fig. 20). Furthermore, there is a record of a temporary plug built at Virginia Mine[3], 12 ft thick, being tested at the same time as a 36 ft thick plug (Table III). The latter plug was proof against leakage whereas the former leaked badly. This information can be plotted to give an indication of a limiting condition of resistance to leakage at any given age (Fig. 20).

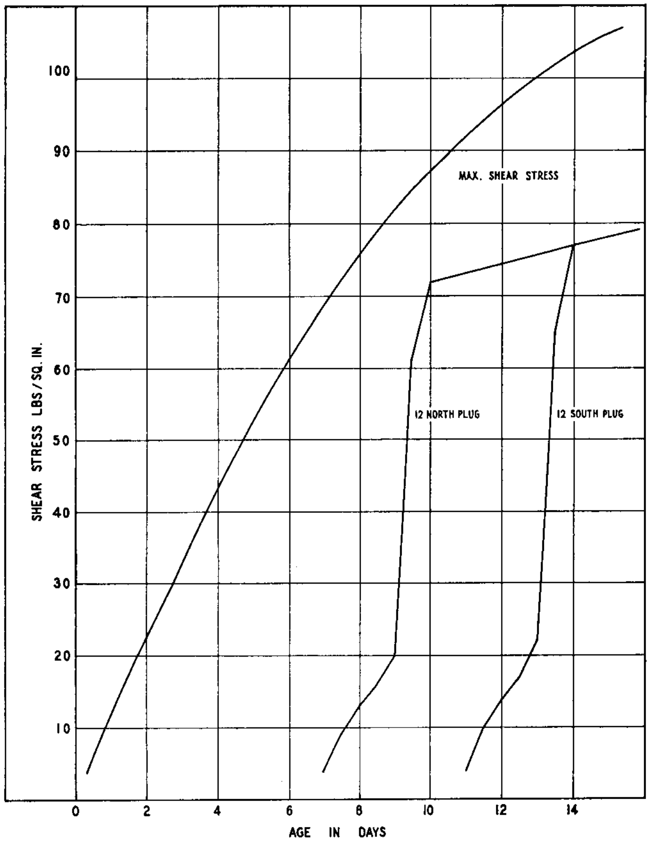

Fig. 19—Structural strength of concrete plugs—safe shear stresses at age in days after placing

Preliminary design sizes

As a result of the foregoing major factors and design considerations, the important dimensions decided upon were that the plugs on 12 Level should be 55 ft in length and those on 10 Level 50 ft.

As already mentioned, the number and size of relief pipes had to be settled by compromise. Table IV shows the volume of water that may be expected to flow through one 10 in. pipe 60 ft long under various heads after allowing for entry and exit friction losses.

The accommodation of more than five 10 in. pipes with valves would have been very difficult, apart from involving lengthy delay in installation. From the table it can be seen that five pipes could cope with the flow on 12 Level alone (about 40 m.g.p.d.) under a head of 50 ft, and with the entire flow (85 m.g.p.d.) under a head of approximately 200 ft.

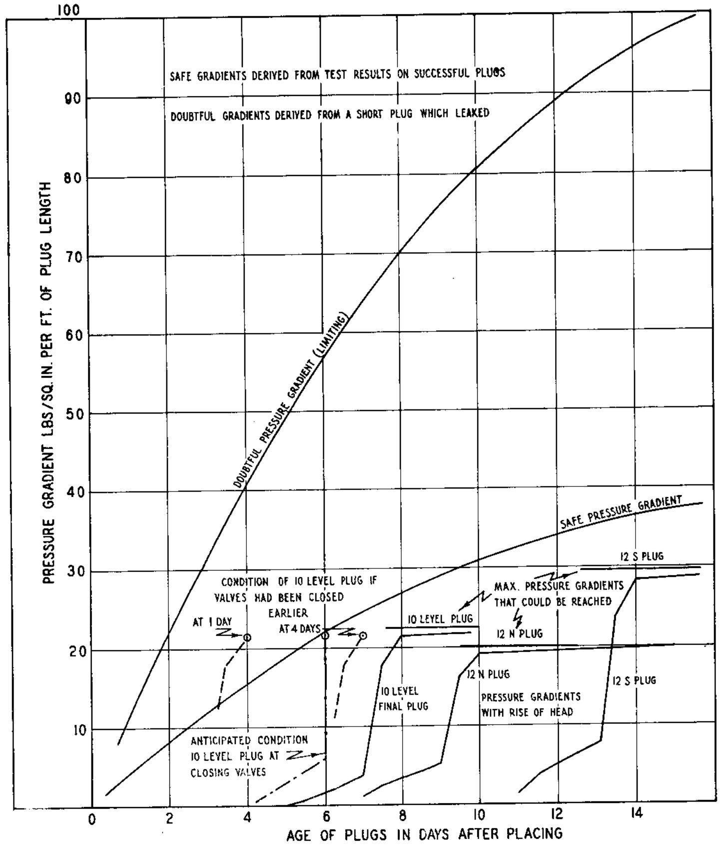

Fig. 20—Leakage resistance—safe pressure gradients in lb/in.^{2} ft of plug at age after placing

TABLE IV
FLOW THROUGH 60 FT LENGTH OF 10 IN. PIPE

Head, ft	Flow in m.g.p.d.
10	3·7
20	5·0
30	6·1
40	7·1
50	7·9
60	8·7
100	11·1
200	15·8
300	19·2

While a head of 50 ft would be too much for a sandbag cofferdam to withstand, it would be well within the capacity of a temporary plug only 8 ft thick, and the difference in elevation of 10 and 12 Levels being 358 ft made 12 Level the ideal place for the setting of the relief pipes and control valves to carry the whole flow. Apart from this consideration, easy access to the plug site from No. 3 Sub-vertical Shaft station was a decisive factor.

The temporary plug had to be built with openings adequate to carry the entire flow under a head of 1 ft above the openings if cofferdam building was not to be made very difficult.

The volumes of water estimated to flow through an 8 ft length of 30 in. ventilation pipe under varying heads, after allowing for entry and exit friction losses, are shown in Table V.

TABLE V

FLOW THROUGH 8 FT LENGTH OF 30 IN. PIPE

Head above top of pipe in in.	Flow m.g.p.d.
4	19·9
6	22·5
8	23·6

PLANNED SEQUENCE OF OPERATIONS

Operational plan for 12 Level

(1) Construct a deflecting cofferdam in the north crosscut 4 to 5 ft high so as to divert the whole flow of water into the south crosscut. Construct a temporary plug in the north crosscut (Fig. 21) 8 ft long with waterways consisting of three 30 in. ventilation pipes and a fourth one as an access way high up. Fit all pipes with flap valves on the wet side capable of withstanding a head of 50 ft.

(2) Break down the deflecting cofferdam in the north crosscut and re-erect it in the south crosscut, but this time build it in such a way that it does not break up when water passes over it as a weir. This precaution is necessary to avoid at a later stage debris from the cofferdam being swept down the south crosscut and blocking the 10 in. relief pipes. Construct a main plug in the south crosscut 55-60 ft long, installing five 10 in. relief pipes and high pressure control valves (Fig. 22). Test each pipe assembly to 1,500 lb/in.^{2} pressure to ensure that gaskets do not fail under pressure and thereby endanger the plug.

(3) Close the flap valves in the temporary plug and divert the entire flow on 12 Level through the 10 in. relief pipes in the plug in the south crosscut. Block the 30 in. ducts through the temporary plug to extend the plug in the north crosscut to a length of 60 ft excluding the temporary plug (Fig. 23).

Operational plan for 10 Level (concurrent with operations on 12 Level) (Fig. 25)

(1) Divert flow as on 12 Level and construct a temporary plug to withstand 12 ft head (Fig. 24) this being the pressure required to divert water over the highest point on 10 Level crosscut south.

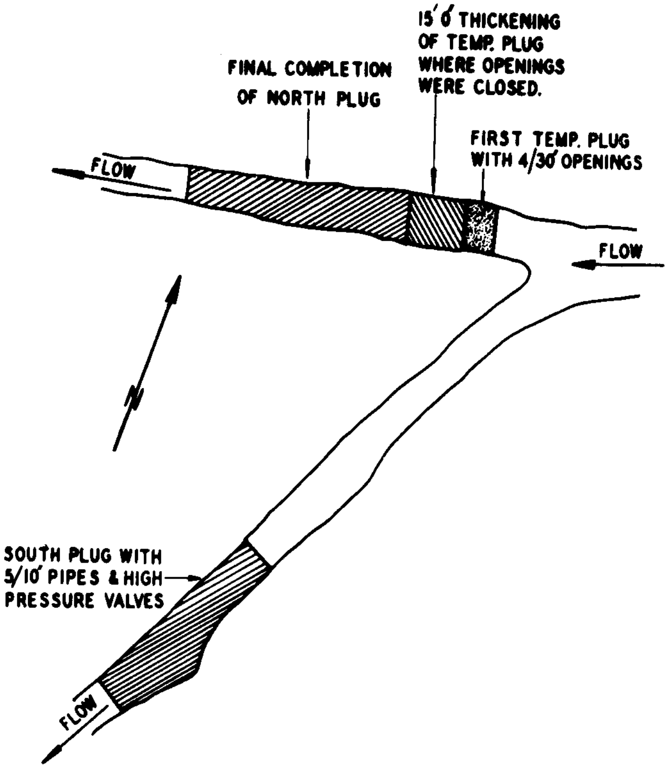

Fig. 21—Plugs on 12 Level

(2) Construct a main plug in the south side of the triangle leading into 10 Level footwall drive. Pierce this plug with a 30 in. ventilation pipe as a duct to release water if necessary during the construction of the final plug in the other side of the triangle. Complete these two operations by the time Operation 3 on 12 Level is finished.

(3) Change the course of all water from 10 Level down to 12 Level by closing the flap valves in the temporary plug and diverting the water flowing from the shaft. Build the final plug in the north side of the triangle thus finally blocking off 10 Level footwall drive (Fig. 25).

Final operation

After completion of Operation 3 on 10 Level, that is the construction of the last plug, close the valves on 12 Level after making due allowance of time for the plugs to set.

ACTUAL ACHIEVEMENT

While the planned design and sequence of operations were carried out in close conformity with the original plan, there were some variations. The estimated and

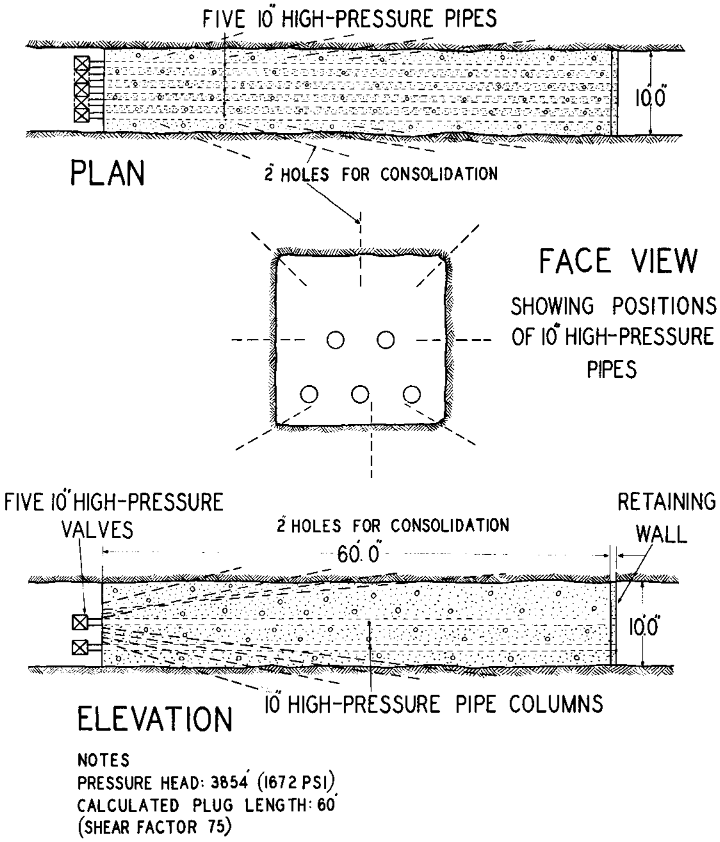

Fig. 22—12 Level South plug

actual timing is best illustrated in Table VI which shows that the original programme was bettered despite the many difficulties encountered.

VARIATIONS FROM PLAN AND CONSTRUCTION DIFFICULTIES

Cofferdams

The original intention was to construct preliminary deflecting barricades of horizontal 9 in. × 3 in. planks against timbers wedged between footwall and hanging, and to build sandbag cofferdams behind these barricades. In actual execution the timbers were wedged horizontally across the headings and short 9 in. × 3 in. planks placed vertically. The cofferdams were then built up against the timber work. Sandbags and cement pockets were used for this purpose in 12 and 10 Level cofferdams and cement pockets only in 12 Level south. It was naturally difficult to make these

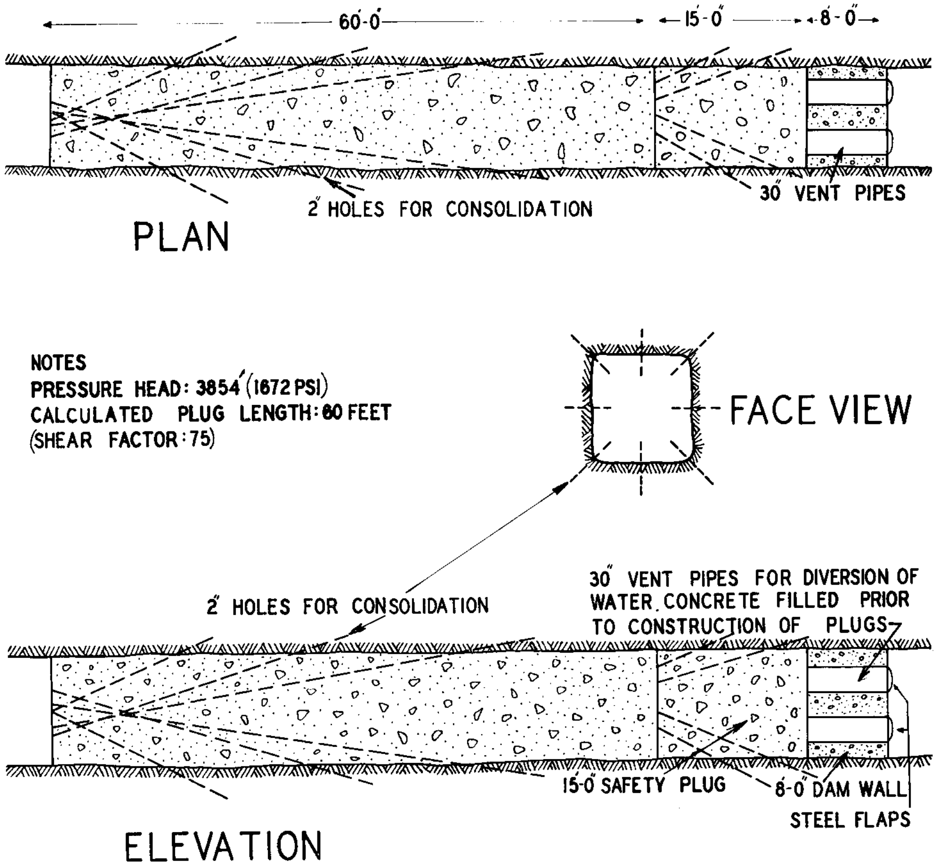

Fig. 23—12 Level North plug

cofferdams watertight as rail tracks and loose material on the footwall were always troublesome.

The actual walls for casting the plugs were made of timber shuttering strutted to resist the pressure of grout rising in the plugs, scribed to the side walls, and sealed with cement mortar doped with calcium chloride for quick setting. Much of this shuttering was built concurrently with the grouting.

Between the cofferdams and the plug shuttering there were thus sumps formed which were kept dry with Quimby pumps in the early stages of plug pouring. When the grout level in the plugs had risen high enough, pumping of these sumps was not necessary and was stopped.

12 Level temporary plugs

The flap valves closing the 30 in. ducts did not work well, mainly because of obstructions to closing on the wet side. On 10 Level this trouble was eliminated.

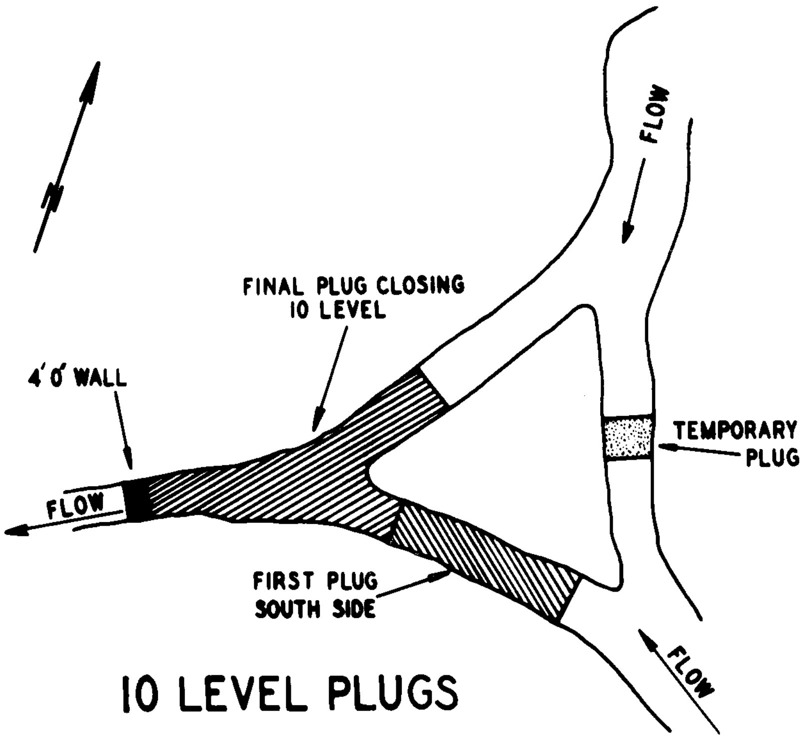

Fig. 24—Plugs on 10 Level

12 Level south plug (Fig. 22)

The actual site adopted was rather far from the cofferdams but was chosen because the ground was better than that nearer the intersection. The setting and testing of the 10 in. pipes proved very difficult and because the lengths of pipe worked out conveniently to 60 ft the final plug was only 56 ft long. Cleaning the footwall was more extensive than had been expected and the actual size of excavation was nearer 10 ft × 12 ft than 10 ft × 10 ft.

12 Level north plug (Fig. 23)

When the flap valves on the 12 Level temporary plug were closed, the difficulty of sealing was greatly alleviated by diverting much more of the flow on 12 Level to 10 Level than had earlier been thought possible. This would have greatly delayed work on 10 Level unless the whole flow from No. 4 Shaft could be diverted back to 12 Level quickly. In the original plan the intention had been to complete the full 60 ft of plug in 12 Level north, but 24 hours or more were saved by thickening the temporary plug by 15 ft as a matter of urgency, followed by the remainder of the work in a more leisurely way. Once again footwall cleaning was a major labour.

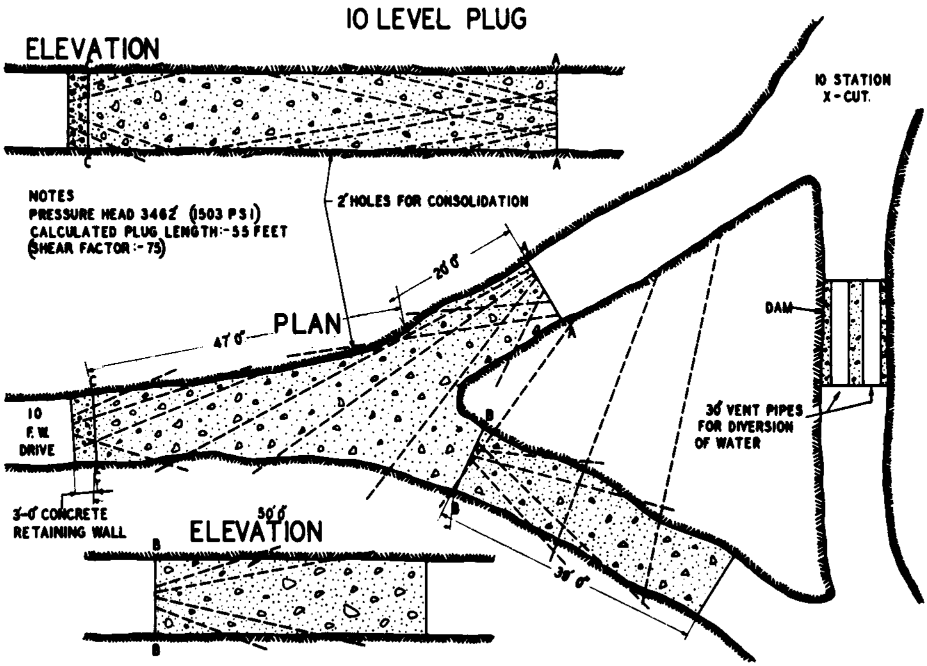

Fig. 25—10 Level plug

10 Level south plug (Fig. 24)

The intention to provide a possible relief duct 30 in. in diameter through the plug was a failure as the duct could not be held in place under the extremely difficult conditions prevailing. It was consequently filled with grout and the plug built solid.

10 Level north—final plug (Fig. 25)

When it was found possible to deflect the entire flow from 10 Level down to the lower 12 Level, and thus through the relief pipes and control valves, the building of the final plug on 10 Level became an easier proposition. It was accordingly re-sited to back up the south plug and cover the potentially weak ground in the apex of the triangle. In so doing, advantage was taken of the wedge shape of the excavations formed by the junction of the two headings with 10 Footwall Drive. In both plugs on this level footwall cleaning involved removal of very large volumes of filling leaving the final sizes about 10 ft $\times$ 14 ft.

Final grouting

It was unavoidable in the method of construction that surplus water would accumulate on top of the rising grout in the plugs and while it was bled off regularly, it was inevitable that water would accumulate on rock/concrete interfaces and particularly at the hanging.

TABLE VI

PLANNED AND ACTUAL CONSTRUCTION SCHEDULE FOR THE PLUGS ON 10 AND 12 LEVELS

Operation	Planned start	Planned finish	Actual finish
General preparation—cleaning debris and getting to plug sites, 12 Level	29.10.68	2.11.68	
12 Level:			
(1) Cofferdam and temporary plug	29.10.68	3.11.68	7 a.m. 2.11.68
(2) Move cofferdam and build main south side plug	4.11.68	7.11.68	2 a.m. 7.11.68
(3) Close temporary plug and complete north side plug	9.11.68	11.11.68	3 a.m. 11.11.68
General preparation, clearing debris and getting to plug sites, 10 Level	31.10.68	3.11.68	
10 Level			
(1) Cofferdams and temporary plug	4.11.68	7.11.68	8 a.m 3.11.68
(2) Place south side plug	7.11.68	9.11.68	5 a..m. 6.11.68
(3) Divert all water to 12 Level and build final plug in North side	10.11.68	14.11.68	3 a.m. 13.11.68

This is normal and it is standard practice to grout up the contacts after the material of a plug has set, for which purpose 2 in. pipes are laid out and cast into the body of a plug so that they intersect the rock concrete contact planes at predetermined points. It is also standard practice to drill through these pipes to get a clean intersection of the contact planes; the holes are then extended into the surrounding rock to pick up any cracks adjacent to a plug. In this case there was not time to follow normal procedures and grouting contact planes with neat cement had to be done within 48 hours after plug placement. On this work a maximum pressure of 500 lb/in.^{2} was used after 48 hours setting, followed by a second injection later, as and how it became possible, at 1,000 lb/in.^{2}. Some drilling for further intersection of potentially weak spots was done through the plugs, but this was kept to a bare minimum, and the satisfactory result of the grouting was in fact a useful guide to soundness of the plug construction. Statistics of the plugs as built are given in Table VII, and layout of grout pipes is shown in Figs. 22, 23 and 25.

TABLE VII

STATISTICAL INFORMATION—PLUGS AS BUILT

Plug	Size	Materials pumped cu ft	Est. % stone pack	Pumping time, hours	Grout used to tighten cu ft
12 Temporary	10 ft × 12 ft × 8½ ft	960	70	5	—
12 N Thickening	10 ft × 12 ft × 15 ft	2,800	0	7	40
12 N Main	10 ft × 12 ft × 60 ft	8,600	30	23	415
12 S Main	10 ft × 12 ft × 56 ft	7,140	10	23	170
10 Temporary	10 ft × 14 ft × 8½ ft	2,186	0	3	—
10 S	10 ft × 14 ft × 40 ft	5,810	0	17	24
10 N Main	10 ft × 14 ft × 70 ft	13,000	80	27	224

Finally it was realized that the greatest risk of trouble with the plugs was that leakage along the contacts might take place and set up erosion of the partially set material of which they were constructed. To counter this one grout pipe was left through each plug with an open end on the wet side. As soon as the valves were closed on 12 Level and water started to build up, neat cement grout was pumped from surface through these pipes to the wet side of all plugs. The intention was that cement pulp circulating up against the plugs would tend to be drawn into any cracks or leakage planes and automatically form a seal.

The volumes of cement pumped in this way are given in Table VIII:

TABLE VIII

VOLUME OF CEMENT PUMPED TO PREVENT LEAKAGE

12 North plug	10,000 cu ft
12 South plug	6,466 cu ft
10 South plug	14,386 cu ft
10 Final plug	16,262 cu ft

In fact the leakages on rock/plug interfaces were negligible when the full hydrostatic pressure was applied.

General

Measurements of water inflow were at all times difficult to make as it was impossible to establish gauging weirs and the total of water flowing into the mine was therefore assessed as the volume pumped plus volume stored in the lower levels. The calculation of volume stored was inevitably open to some doubt as the size of the old stopes could not be exactly known. The first reasonably accurate calculation of the volume of water entering No. 4 Shaft was therefore made when the entire flow was concentrated in the five 10 in. relief pipes through 12 Level South plug.

The head on this plug over the period before closing the valves was measured (Table IX), and the total volume of water flowing from the No. 4 Shaft area was accordingly estimated at 85 million gallons per day.

TABLE IX

HEAD OF WATER ON 12 LEVEL PLUGS

Date	Time	Head
11.11.68	10 a.m.	213 ft
12.11.68	2.30 p.m.	213 ft
14.11.68	6 a.m.	210 ft
15.11.68	10 a.m.	210 ft

When the original plan was drawn up the universally accepted opinion was that it would be impossible to divert all the flow from either 10 or 12 Level to the other. The whole plan was built around this assumption, but as time went on, the manipulation and control of the water improved and at the end it was in fact possible to reduce the flow on either 10 or 12 Level footwall drives to very small dimensions. Four major plugs and two temporary plugs were built, but if this control had existed at the beginning, it would have been necessary to build only two plugs.

The precautions taken to avoid range blockages were extremely successful and no breakdowns or hold-ups occurred in the pumping of nearly 4,000 tons of grout and cement pulp.

VALVES, PIPES AND CLOSING CONDITIONS

General

The 10 in. pipes through 12 South plug were capable of safely resisting a pressure of 3,000 lb/in.^{2} and the valves were of the gate type and designed for a safe working pressure of 1,500 lb/in.^{2}. With the background of the Merriespruit disaster where failure occurrred as a result of a series of gaskets blowing on flanged pipe joints, the joints on the West Driefontein 10 in. pipes were of necessity regarded with suspicion although all joints were tested to 1,500 lb/in.^{2}. The gaskets used were Klinkerite and it was fortunate that in fact the joints were tested as despite the use of high tensile bolts and torque wrenches to do the tightening, leaks occurred during the testing process.

The valves too, although they were of high quality, were under suspicion as the velocity of flow through the 10 in. pipes was of the order of 60-70 ft/sec., and some sand or fine silt must surely have been present in the water. It was feared that these conditions might produce erosion of the valve seats, or alternatively, it might not be possible to close some of the valves fully. In either event the restricted passage of water past a valve not completely seated would cut the seat away very rapidly.

The head behind the 12 North plug when all the water was passing through the 10 in. pipes was a little over 210 ft and remained remarkably constant, confirming the estimate of volume flowing and the fact that it was not diminishing. It was expected that the pressure would rise very rapidly to about 200 lb/in.^{2}, then slowly for about two days while the worked-out stopes in the No. 4 Shaft area filled, and finally very rapidly as the water rose in No. 4 Shaft itself.

There would thus be little danger of any sort at the time of closing and there would be a relatively long period when it would be safe to work on the valves and pipes and render them beyond any risk of failure as the full pressure developed later.

It was decided, therefore, that all valves should be closed as rapidly as possible to stop the entire flow, so that, if there was going to be any serious difficulty, there would then be no trouble in opening the valves again to allow a scheme to be developed to cope with it. Thereafter, because all flanged joints were suspect, the valves would be closed off with blank flanges and the pipes and valves through the plug grouted up solid. To do this the flanges on the lower three valves were fitted with connections and high pressure cocks top and bottom, while the top two were fitted with 2 in. high pressure cocks at the centres of the flange. Immediately after closing, the bottom pipes were grouted up by injecting through the lower connections and bleeding through the top, while the fittings on the top two pipes were used to pump bulk cement filling through to the wet side of the plug.

Arrangements for closing valves

As the last operation of closing the valves was vital, much thought was given to it. It was realized that the discharge of a volume of water of the order of 85 million gallons per day into the drive would produce a standing wave completely filling the heading. Each valve was therefore fitted with a short length of light gauge piping bolted to it, to carry the flow clear. Fitting over each of these open ends, there was a 14 in. pipe strongly anchored and arranged to discharge some distance further away from the plug and past the junction with another crosscut (Fig. 26).

Fig. 26—Water being discharged at the rate of 85 million gallons per day through five extention pipes to a position away from the plugs

This crosscut was blocked off by a rubble embankment and a high level timber platform erected above the 14 in. discharge points thus providing a completely protected access way to the valves.

Immediately clear of the valves a low concrete wall was built to form a small chamber for the valves protected against back flow from the discharges.

The actual valve position was extremely cramped (Fig. 27) and difficult of access and the organization for closing had therefore to be precise. At a given command all valves were to be closed simultaneously and if they closed completely, as in fact was the case, each of the short discharge pipes was to be cut off and blank flanges bolted on. To ensure that there was no hold-up a most impressive array of spare flanges, bolts, etc., and tools was provided.

The operation went off very smoothly and the flow into the mine was closed off at 1 p.m. on 18th November, 1968.

As a special precaution to observe the behaviour of the plug closed circuit television systems were installed on the dry side of 12 South and 10 Level plugs with a viewing position at No. 3 Shaft.

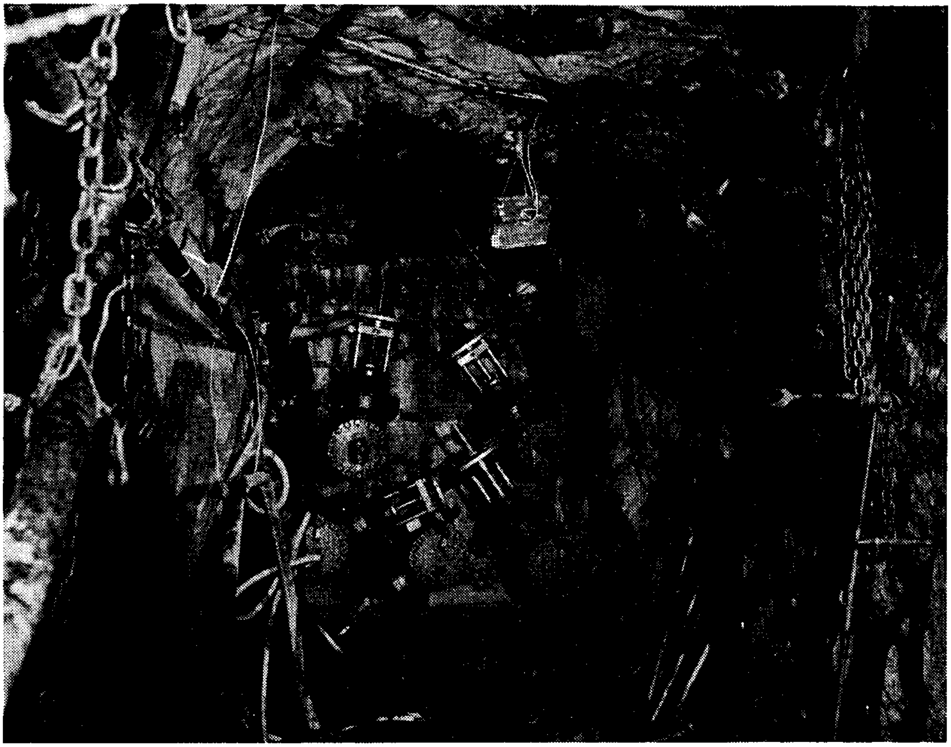

Fig. 27—High pressure valves on the end of the 10 in. relief pipes through 12 South plug, after closure and fitting of blank flanges, under surveillance of a television camera suspended from the hanging

The pressure build-up behaved much as expected and reached almost full static pressure in the early hours of Thursday morning, 21st November, as shown in Table X.

TABLE X

BUILD-UP OF HEAD OF WATER AGAINST 12 LEVEL PLUG (IN FT)

Date	4 a.m.	8 a.m.	12 m.d.	4 p.m.	8 p.m.	12 m.n.
18.11.68	210	210	210	396	442	489
19.11.68	533	592	665	706	747	804
20.11.68	871	964	1,029	1,127	1,254	3,085
21.11.68	3,661		gradual rise after this			

After closing the valves and during the pressure build-up, a continuous watch was kept on the plugs for leakage and some very minor injections were made into the rock surrounding the plugs. The actual leakage was, however, negligible, being only about 700 gallons per hour from all plugs.

SAFETY PLUGS

Because the mine would have to continue working with the No. 4 Shaft area flooded, it was considered advisable to make assurance doubly sure with safety plugs constructed behind each of the original plugs. These plugs were designed to be entirely orthodox. Their only unique feature was that they were constructed with a gap between them and the previous plug, the purpose of which was to act as a pressure chamber for testing both plugs to a load 50 per cent higher than full static head. After these tests it was possible to state emphatically, for the benefit of men working in the mine, that the plugs on which their lives depended had been tested to more than full load and when the intervening space was finally filled by grouting the total combined size of the plugs was such that there was a very large factor of safety.

ACKNOWLEDGEMENT

The authors would like to acknowledge a debt of gratitude to all employees of the West Driefontein mine and of The Cementation Company for their patient fortitude, and to those in the head office of Gold Fields of South Africa who contributed so much to the successful outcome of a titanic struggle. They would also like to express their thanks and appreciation for the help given by neighbouring mines and by the many allied industries throughout the land.

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APPENDIX 1

GRADING OF SAND—B.S.S. STANDARD SERIES

Series No.	*Percentages*
+ 14	0·65
− 14 + 25	3·89
− 25 + 52	51·57
− 52 + 100	27·04
− 100 + 170	10·46
− 170	6·39

Over a number of years it has been common practice to pump sand/cement grouts long distances for underground cementation by the Colcrete Process. For the process to be an economic proposition it is necessary that there should be small danger of range blockage and the sand should be readily available without having to be regraded for use. This particular sand meets these conditions and had been proved reliable in the past.

APPENDIX 2

PLUG DESIGN DETAILS

PRELIMINARY DESIGN

12 Level main plugs

Head 3,830 ft
Drive size 10 ft × 10 ft
Assumed safe punching shear stress 75 lb/in.^{2}

$$\text{Design length} = \frac{3{,}830 \times 10 \times 10 \times 62{\cdot}5}{10 \times 4 \times 75 \times 12 \times 12} = 55 \text{ ft.}$$

12 Level temporary plug

Head 200 ft maximum
50 ft probable working
Drive size 10 ft × 10 ft

$$\text{Design length} = \frac{200 \times 10 \times 10 \times 62{\cdot}5}{10 \times 4 \times 75 \times 12 \times 12} = 3 \text{ ft}$$

$$\text{Minimum length} = \pm 8 \text{ ft.}$$

10 Level plugs

Head 3,472 ft
Drive size 10 ft × 10 ft
Assumed safe punching shear stress 75 lb/in.^{2}

$$\text{Design length} = \frac{3{,}472 \times 10 \times 10 \times 62{\cdot}5}{10 \times 4 \times 75 \times 12 \times 12} = 50 \text{ ft.}$$

SHEAR STRESS IN PLUGS AS BUILT

12 Level south plug

Actual drive size 10 ft × 12 ft
Actual length 56 ft

$$\text{Shear strength maximum } \frac{3{,}830 \times 10 \times 12 \times 62{\cdot}5}{(10 + 12) \times 2 \times 12 \times 12 \times 56} = 81 \text{ lb/in.}^2$$

12 Level main plug north

Actual drive size 10 ft × 12 ft
Actual length, ignoring temporary plug = 60 ft

$$\text{Shear stress maximum} = \frac{3{,}830 \times 10 \times 12 \times 62{\cdot}5}{(10 + 12) \times 2 \times 12 \times 12 \times 60} = 70 \text{ lb/in.}^2$$

10 Level plugs

By siting the final plug in the position shown, it is retained in position not only by the rough rock walls, but also by the wedging action achieved by the shape.

Consequently, although the water pressure may at worst be applied over a very much greater area than the net face of the plug, the simple shear analysis used on the other plugs is not applicable. The structural strength must, however, be far greater than for a parallel plug and it is therefore necessary only to consider this plug from the point of view of leakage past it.

PRESSURE GRADIENT UNDER FULL PRESSURE ON PLUGS AS BUILT

12 Level south plug

Pressure 3,830 ft
Length 56 ft

Pressure gradient $= 3{,}830 \times \frac{62\cdot 5}{144} \times \frac{1}{56}$

$= 29\cdot 7$ lb/in.^{2} ft.

12 Level north plug

Pressure 3,830 ft
Length 8 ft + 15 ft + 60 ft = 83 ft

Pressure gradient $= 3{,}830 \times \frac{62\cdot 5}{144} \times \frac{1}{83}$

$= 20$ lb/in.^{2} ft.

10 Level final plug

Pressure 3,472 ft
Length 67 ft

Pressure gradient $= 3{,}472 \times \frac{62\cdot 5}{144} \times \frac{1}{67}$

$= 22\cdot 5$ lb/in.^{2} ft.

APPENDIX 3

TEST RESULTS ON GROUT SAMPLES

Grout proportions as mixed

Rapid hardening cement 1 packet
Sand 1 cu. ft
Water 7 gallons
Nominal water/cement ratio 0·75
Water/cement ratio as determined from samples 0·63 to 0·65.

Random samples

Excess water was continually bled off the top of the rising grout, and at the end of each bleeding operation some good grout was allowed to run to waste.

Two such samples were recovered from the construction of 12 Level S plug and after 30 hours subjected to unconfined compressive tests. Test pieces were cut from the rough hand samples.

Sample 1

Yielded two test pieces which gave an average compressive strength of 215 lb/in.^{2}

Sample 2

Yielded three test pieces which gave an average compressive strength of 310 lb/in.^{2}

Actual results could not fail to be better than these figures which represent reject material.

Grout samples as placed

In order to simulate placing conditions, grout was allowed to flow into and fill up a 3 ft length of 4 in. piping. The depth of grout thus corresponded in some measure to actual placing conditions with excess water collecting on the surface.

Test samples were cut from these tubes and subjected to tests at 2½ days after placing, the compressive strength being:

Open end	*Mid point*	*Bottom end*
1,010 to 1,040 lb/in.^{2}	1,050 to 1,210 lb/in.^{2}	1,120 to 1,350 lb/in.^{2}

Schmidt hammer tests

As soon as it was conveniently possible to do so, Schmidt hammer tests were carried out on the surface of the plugs.

Readings in excess of 1,000 lb/in.^{2} for each surface were obtained.

Plug	*Age*	*Compressive strength*
12N	2 days	1,850 lb/in.^{2}
12S	6 days	1,775 lb/in.^{2}
12N	6 days	2,200 lb/in.^{2}
12S	10 days	2,200 lb/in.^{2}
10N	3 days	Well below 1,000 lb/in.^{2}*
10N	4 days	1,000 lb/in.^{2}

*The hammer was found to be defective and these results are therefore unreliable.