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Design and Construction of Watertight Plugs in Permeable Karst Collapse Columns in Restoration of Flooded Mines: A Case Study at Dongpang Mine, China

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Abstract: *An emergency watertight plug was constructed within a karst collapse column (KCC) in Dongpang Mine, China to restore the flooded mine. Unlike traditional plugs that are typically constructed in horizontal tunnels, this plug was constructed in a vertical discontinuity. The KCC, which was more than 460 m high and buried more than 300 m below ground surface, functioned as a pathway through which pressurized groundwater in the underlying limestone suddenly flowed into the mine. Design of the plug was based on years' experience with water inrush mitigation in China and site-specific knowledge with general reference to some best design practices in South Africa, United States, and United Kingdom. Thirty exploratory boreholes were used to determine characteristics and three-dimensional geometry of the KCC. The grouting was conducted on the surface by drilling boreholes and pumping grout into the collapse column. The completed watertight plug is 73 m high. Construction of such a large in-situ plug in an underground collapse feature overcame several engineering challenges. A key component to the success of this project was that all the grout holes including primary, secondary and tertiary ones were directed precisely to the designed locations by directional drilling. In addition, the technique, quality control, grouting stages, and completion criteria of each stage were tailored to the site-specific conditions. The effectiveness of completed plug was evaluated by core samples collected at boreholes and underground water-injection and water drainage tests.*

1. WATERTIGHT PLUGS IN UNDERGROUND MINING

Construction of watertight plugs is a water control technique by cutting off hydraulic connections between water sources and working areas in underground mines. They can be important structures in preventing water hazards such as water inrushes or restoring flooded mines.

1.1. Types of Watertight Plugs in Underground Mining

Table1. *Types of watertight plug in underground mining*

Watertight Plug	Characteristics
Precautionary plug	▪ Constructed in underground roadways to limit the area of flooding if water inrushes occur. ▪ Watertight doors are often built into them which can be shut when any danger of flooding arises. ▪ Installed as a safety measure prior to development in areas known to be potential water-bearing zones. ▪ Such plugs are designed to withstand full hydrostatic pressure from surface level.
Control plug	▪ Constructed to seal off or control the inflow of water from abandoned mining areas. ▪ When constructed in boundary pillars between adjacent mines, such plugs are referred to as boundary plugs and serve to prevent water flowing from abandoned areas of one mine into the workings of an adjacent mine. ▪ No means of access to the sealed off areas is provided through control plugs. Drain pipes, with valves, are often cast into the plugs. ▪ Such plugs are designed to resist full hydrostatic pressure from surface level or the pressure imposed by the head of water to the highest overflow point.
Emergency plug	▪ Constructed to seal off unexpected inrushes of water either temporarily or permanently. ▪ No means of access to the sealed-off areas is provided in such plugs. ▪ They are usually designed to withstand full hydrostatic pressure from surface level.

Temporary or consolidation plug	▪ Constructed to allow inflow water to be controlled or stopped while simultaneously providing the resistance for high pressure grouting and consolidation operations. ▪ Such plugs are removed after the water pressure zones are sealed. ▪ Full hydrostatic pressure from surface level may be the dominant design parameter for these plugs.

Table 1 lists four types of plug that are often constructed in underground mining. As underground mining moves to areas with greater hydrogeological complexities, the ability to safely control and impound water underground will become increasingly important. These plugs are to be designed and constructed sufficiently thick and properly anchored in surrounding strata to (1) withstand the hydrostatic pressure; (2) prevent water leakage, and (3) resist deterioration by mine water. Most of the reported watertight plugs were horizontal and constructed in man-made tunnels or adits. Construction of vertical plugs within geological formations is rare.

2. Design Principles Of Watertight Plugs

Factors that need to be considered when selecting watertight plugs as the preferred alternative to control water include (1) plug location in relation to the prevailing rock and working conditions; (2) water head to be withstood by the plug; (3) condition of and stress in the rock surrounding the plug; (4) strength of and stress in the material of the plug; and method of plug construction. The design and construction of watertight plugs must therefore follow best mining and civil engineering practice (Littlejohn and Awart, 2006), tailored to the purpose of constructing the plug and site-specific hydro geological conditions. Because there are no universal rules for watertight plug design, any published design data should be used for reference purpose only and should not be used directly. The most critical design factor of a watertight plug is the plug length required to ensure proper sealing rather than the strength of the concrete or structural grout used to build it. Table 2 presents selected formulae to estimate the plug length based on experience in South Africa (Garrett and Pitt, 1961; Auld, 1983), United States (Chekan, 1985), China (China State Administration of Coal Mine Safety, 2011), and United Kingdom (United Kingdom Health and Safety Executive, 2018).

Table2. *Equations for designing plug length*

Model	Assumption	Equation	Annotation*	Reference
Thin-plate	▪ Plug is to act as a simply support ed thin plate, spanning the width of the entry. ▪ Structural behavior under static load is characterized by bending at midspan. ▪ Bending failure is governed by tensile strength.	$T = 0.865\, a \sqrt{p/f_t}$	*T* - plug thickness (feet) *a* - maximum entry dimension (feet) *p* - hydrostatic pressure (psi) f_t - allowable tensile strength of construction materials (psi)	Chekan (1985)
Thick-plate	▪ Plug is to recess the ends into the surrounding strata, allowing the structure to act as a flat arch. ▪ Failure occurs at both sides and along the midspan by fracturing. ▪ Failure is governed by compressive strength	$T = 0.670\, a \sqrt{p/f_c}$	*T* - plug thickness (feet) *a* - maximum entry dimension (feet) *p* - hydrostatic pressure, psi *fc* - allowable compressive strength of construction materials (psi)	Chekan (1985)
South African parallel-sided model	▪ Load, induced by hydrostatic pressure, is transmitted from concrete plug to the rock as punching shear around the perimeter of the plug and along its full length. ▪ Failure is governed by shear strength.	$T = \frac{pab}{2(a+b)f_s}$ To prevent leakage, pressure gradient should be: 1) ≤11 psi/foot after grouting plug/rock interf ace before grouting the rock; 2) ≤400 psi/foot after grouting rock.	*T* - plug thickness (feet) *a* - width of entry (feet) b - height of entry *p* - hydrostatic pressure, psi *fs* - allowable shear strength of construction materials or rock, whichever is the lesser (psi)	Garrett and Pitt (1961)

South African parallel-sided (tapered shape)	▪ Load, induced by hydrostatic pressure, is transmitted from concrete plug to the rock as punching shear around the perimeter of the plug and along its full length. ▪ Failure is governed by compressive strength.	$T = \frac{pab}{(a+b)f_c}$	T - plug thickness (feet) a - width of entry (feet) b - height of entry (feet) p - hydrostatic pressure (psi) fc - allowable compressive strength of construction materials or rock, which ever is the lesser (psi)	Garrett and Pitt (1961)
Circular plug model	▪ Load, induced by hydrostatic pressure, is transmitted from concrete plug to the rock as punching shear around the perimeter of the plug and along its full length. ▪ Failure is governed by shear strength.	$T = \frac{pr}{f_s}$ or $\frac{T}{r} = \frac{p}{f_s}$	T - plug thickness (feet) r - radius (feet) p - hydrostatic pressure (psi) fs - allowable shear strength of construction materials or rock, which ever is the lesser (psi)	Auld (1983, 1996)
Water inrush coefficient analog model	▪ Based on water inrush data in China's coal mines. ▪ Empirical equation applicable to rocks. ▪ May be considered as conservative estimates for concrete.	$T = \frac{p}{\alpha}$ where: 1) α = 60 Pa/m in areas with fractures; 2) α = 100 Pa/m in areas without fractures.	T - plug thickness (m) α - water inrush coefficient (Pa/m) p - hydrostatic pressure exerted to bottom of plug (Pa)	China State Administration of Coal Mine Safety (2011)
United Kingdom plug model	▪ Plug/rock interface shearing is the governing failure mechanism. ▪ Assume a 4:1 factor of safety against interface shear failure. ▪ Permissible interface shear stress is the lesser of the permissible shear stress in the plug material and the permissible shear stress in the host rock.	$T \geq 2a$ with the following conditions: 1) $(pab)/[2(a+b)T] \leq$ 350 kN/m^2; 2) $p/T \leq$ 500 kN/m^2/ m.	T - plug thickness (m) a - maximum entry dimension (m) b – minimum entry dimension (m) p - hydrostatic pressure (kN/m^2) fs - allowable shear strength of construction materials or rock, whichever is the lesser (kN/m^2)	United Kingdom Health and Safety Executive (2018) http://www.hse.gov.uk/mining/circulars/waterplu.htm

* Original units are preserved in the formulae to facilitate reference to original works.

The first two models of Table 2 address the plug design from a mechanical point of view, whereas the last five models emphasize both the mechanical failure and hydraulic seepage. Figure 1 shows the relationship between plug length and water head based on different models when designing a circular 25-m radius plug. It is apparent and understandable that the various design formulae provided a wide range of options. In general, the leakage model based on the South Africa experience gives the most liberal estimate on the plug length, while the water inrush model based on experience from China's coal mines provided the most conservative estimate. To withstand a water head of 600 m, a plug of approximately 34 m long would be sufficient with a safety factor of 10 based on the South Africa model and a plug of approximately 60 m long would be needed from the China's model. Based on the United Kingdom code of practice, the plug length would be 89 m.

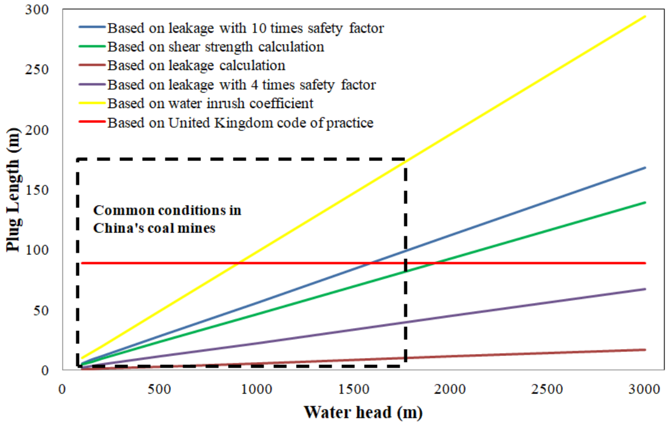

Fig1. *Designed watertight plug length versus water head in underground mining*

The discussions above illustrate that design of watertight plugs is a complicated process. All the design equations oversimplify the failure mechanisms of a watertight plug against hydrostatic pressures, dynamic water head, permeation, deterioration, and hydro-fracturing. Because of these uncertainties, any plans on plug design, location, and construction should be reviewed by subject matter experts and approved by regulatory agencies. The following example demonstrates the step-by-step approach that led to successful construction of a vertical watertight plug within a karst collapse column (KCC) in a coal mine, China.

3. Water Inrush Though Karst Collapse Column In Dongpang Mine, China

Dongpang Mine is located in Xingtai, China. As a large-scale coal mine, it has an annual production of 3 billion kilograms. The mine extracts primarily the #2 coal seam in the Shanxi Formation of the Permian Period. Mining takes places on two levels, -300 m and -480 m below sea level (bsl), respectively. A water inrush at the maximum flow of 70,000 m^3/h occurred in 2003 at working panel #2903 on the second level (-480 m bsl) when an undetected KCC was intercepted (Fig. 2). The incident flooded the entire mine. Water level and geochemical data indicated that the Ordovician limestone was the water source. The Ordovician limestone underlies the coal measures and is a highly karstified aquifer. Not only is the limestone several hundred meters thick, but the water in it is confined and pressurized. The KCC acted as a passageway for the pressurized groundwater in the Ordovician limestone to flow upward into the underground working area, resulting in the water inrush (Fig. 3).

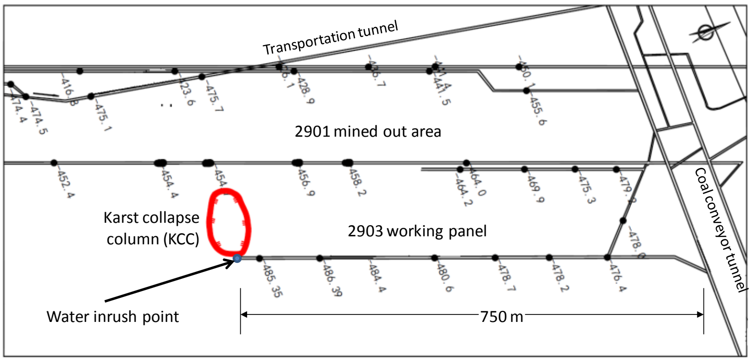

Fig2. *Position of working panel #2903*

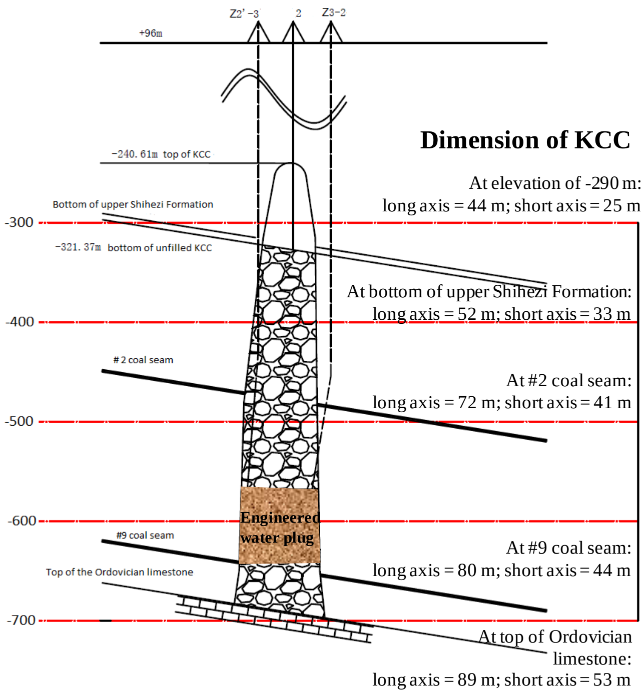

Fig3. *Cross-section of KCC intercepted at working panel #2903*

Characteristics and three-dimensional geometry of the KCC (Fig. 3) were investigated with 30 exploratory boreholes. Figure 4 shows projected surface locations of these boreholes in relation to the identified KCC. Table 3 summarizes the characteristics of the KCC as observed in the borehole logs. As presented in Table 3, branch-out and directional drilling was used for target exploration. Seven boreholes, Z1, Z2, Z2’, Z3, Z4, Z5, and Z5’ were master boreholes. Multiple directional boreholes were advanced from these master boreholes at designed kickoff points. For example, Z1 is a master borehole, which consists of three branched-out directional boreholes, Z1-1, Z1-2, and Z1-3. The shape of the KCC was delineated by advancing approximately 8,000 m of exploratory drilling. The KCC is shaped like a bowling ball when projected on surface, a larger area in southeast than northwest. The KCC was more than 460 m high with the top at -240.61 m bsl and the bottom in the Ordovician limestone below -700 m bsl. Because the surface elevation is approximately 96 m asl, the KCC is buried more than 300 m below ground surface (bgs). A fracture zone was also identified around perimeter of the KCC.

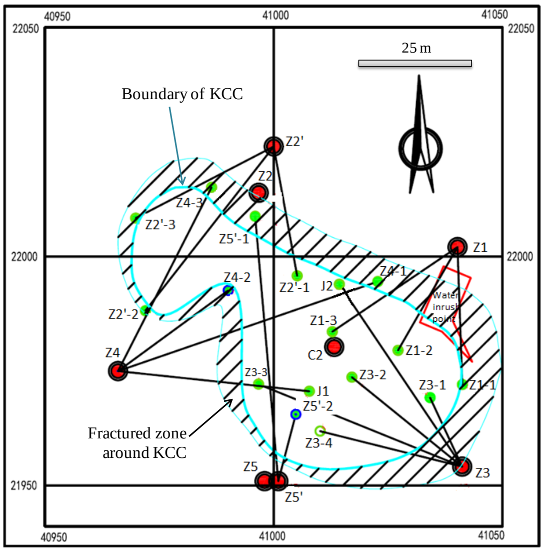

Fig4. *Locations of exploratory boreholes*

In order to restore the flooded mine and prevent future water inrushes, construction of a grout-plug within the KCC was chosen as the preferred alternative to permanently cut off the hydraulic connection. Figure 1 was used to provide a general reference for plug length determination. The plan consisted of both surface and underground operations. Concrete mixture was pumped into the KCC and the surrounding fractures through the grout holes, which were drilled on the surface. Aquifer tests at directionally drilled boreholes were conducted underground to evaluate quantitatively the effectiveness of the plug. The top of the plug was at -568 m bsl, while the bottom was at -641 m bsl. The constructed grout-plug was 73 m high (Fig. 3).

Table3. *Summary of exploratory boreholes*

Bore hole ID	Borehole parameters		Depth of circulation loss (m bgs)	Parameters of KCC			Height of detected cavity (m)	Offset at bore hole termination（m）	Note
	Depth (m bgs)	Advancement (m)		Depth of encountering KCC (m bgs)	Offset at total circulation loss (m)	Offset when intercepting KCC (m)			
C2	336.61	336.61	325	336.61		0.51	80.76	0.51	grouting hole for aggregates
Z1	550	550		368					Z1-1, Z1-2, Z1-3 are branched-out bore holes from master borehole Z1.
Z1-2	737.2	187.2	608.21	630	8	16.5		26.98	fractures noted at 608.21 m
Z1-3	740.1	215.1						32.34	More circulation loss noted at 708.5 m
Z1-1	737.93	237.93						29.35	
Z2	366.2	366.2	365				1.2		Z2-1, Z2-2 are branched-out bore holes from master borehole Z2.
Z2-1	342	142						20.58	

Z2-2	391	91	371.5	371.5	20.94		15		grouting hole for aggregates
Z2'	550	550	365						Z2'-1, Z2'-2, Z2'-3 are branched-out boreholes from master borehole Z2'.
Z2'-2	742.6	192.6	613	640	10.6	17.7		46.19	fractures noted at 613 m
Z2'-3	738.55	213.55							
Z2'-1	739.3	239.3	640		15.97			28.77	
Z3	550	550						0.86	Z3-1, Z3-2, Z3-3, Z3-4 are branched-out boreholes from master hole Z3.
Z3-2	740.18	190.18	572.6	580	0	4.6		32.03	borehole at boundary of the fracture zone
Z3-3	742.68	217.68						48.04	
Z3-1	738	238						20.06	
Z3-4	740.6	230.6						32.63	
Z4	550	550							Z4-1, Z4-2, Z4-3 are branched-out boreholes from master borehole Z4.
Z4-2	757	207						33.41	
Z4-1	745.5	220.5						59.95	started circulation loss at 667 m
Z4-3	740	230						40.02	
Z5	716.16	716.16						0.608	Z5-1, Z5-2 are branched-out boreholes from master borehole Z5.
Z5-1	582	147		528.13		8.91		15.15	increase in circulation loss at 516.45 m
Z5-2	708.81	208.81	615	631	6.1	8		16.82	
Z5'	550	550						0.73	Z5'-1, Z5'-2 are branched-out boreholes from master borehole Z5'.
Z5'-2	738.06	188.06	699.88					15.04	
Z5'-1	748.2	188.2						58.49	J2 is a branched-out hole from master borehole Z3.
Z3-J2	742	247						48.27	J1 is a branched-out hole from master borehole Z4.
Z4-J1	741	246						41.75	

4. CONSTRUCTION OF THE WATERTIGHT PLUG

4.1. Grouting Method and Materials

The watertight plug was constructed in stages. The grouting was carried out at three levels: the upper level at -544 m bsl, the middle level at -568 m bsl, and the lower level at -641 m bsl. The boreholes were drilled and grouted sequentially from the upper to the lower levels. Secondary and tertiary grouting holes were drilled after the primary grouting holes until the KCC and its surrounding fractures were completely sealed. Because the grouting was in a collapse feature, there were uncertainties. Therefore, the actual operations were kept flexible. Under certain circumstances when continuous drilling was impossible, for example, large voids were encountered; the drilling tools were deflected; or the drilling fluid was lost; the grouting target was adjusted accordingly. All the grout holes were thoroughly cleaned within 4 h after injection of single-cement slurry, immediately after injection of double-cement slurry, or immediately when pressure at borehole inlet was reset to zero.

The grouting materials used in this project included standard Portland cement 42.5, water from a local well, and soluble silicate as accelerator with module 3.0 and concentration 45 degree Beaume. Early strength agent of 0.03-0.05% triehanolamine (purity ≥ 95%) and 0.3-0.5% cooking salt were added in single-cement slurry. Formulary test was carried out on site to determine the compositions of the cement mixture, specific gravity, gelatification times (initial and final gelatification time), and other performance indices.

4.2. Control of Grouting Boreholes

Injection of grout was performed through directionally drilled boreholes. On the surface, the injection boreholes were arranged to cover the approximate extent of the KCC as uniformly as possible. Because the plug was constructed at three sections and the boreholes were drilled in angles (Fig. 5), each borehole had three projected locations on the plane maps (Fig. 5). The projected area of the KCC on surface was 3,546 m^2. A total of 17 grout holes were drilled, which resulted in coverage of 222 m^2 per borehole. Of the 16 grout holes, six were primary ones with borehole spacing from 9.5 to 30 m. The primary holes had a control radius of approximately 14 m. There were five secondary and five tertiary grout holes, respectively, and they were drilled with smaller borehole spacing because of their relatively smaller control radii. Table 4 summarizes the parameters of the grout holes.

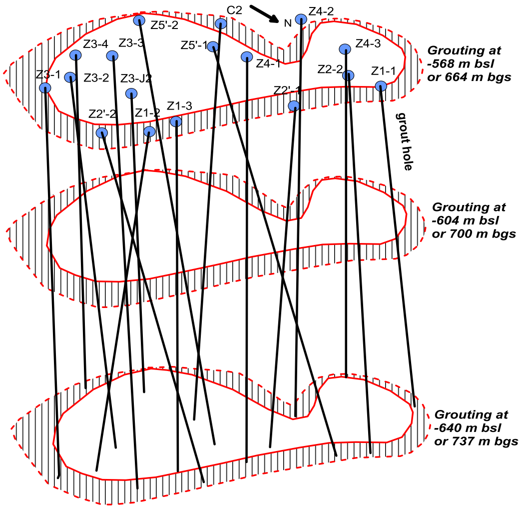

Fig5. *Three-dimensional grout holes on three elevation levels*

Table4. *Summary of grout boreholes*

Parameter	Primary holes	Secondary holes	Tertiary holes
Number of boreholes (borehole IDs)	7 (Zl-2, Z2'-2, Z3-2, Z4-2, Z5-2, Z5-2', Z5'-2)	5 (Zl-3, Z2'-3, Z3-3, Z4-1, Z5'-l)	5 (Zl-1 Z2'-1, Z3-1, Z4-3, Z3-4)
Control radius (m)	14.3	10.6	8.8
Spacing between boreholes (plane projection, m)	9.5 - 30	8 - 24	5.2 - 17

4.3. Grouting Stages and Procedures

The grout engineering was divided into the following four stages:

Stage I - Filling Grout

This was the initial stage of each primary grout hole. The main purpose at this stage was to inject a great amount of cement slurry to fill voids or caves. Cement slurry was mainly single slurry of high concentration. Injection was carried out quantitatively and repeatedly under static water pressure without pressure accumulation at the borehole inlet. The main frame of the grout-plug was created by sealing major water passage ways.

⇓

Stage II - Increasing Pressure

The purpose of this stage was to seal relatively large fractures and solution-enlarged fissures and to consolidate the grouting results of the early stage. Single cement slurry was primarily used. Injection was carried out under static water pressure. During the injection process, the pressure at the borehole inlet was gradually increased. However, the pressure at the borehole inlet was controlled to be no more than 2 MPa.

⇓

Stage III - Hydrodynamic Grouting

Each injection borehole was under static water pressure. After completion of stage II, the pressure at the borehole inlet was relatively high as a result of the reinforcement grouting. The directionally drilled aquifer test holes in the -480 m level drift were examined for drainage. The residual inflow rates at these boreholes provided the data to evaluate the water-sealing effect of the early stages. On the other hand, if there was a significant water flows at the holes, the artesian flows produced an artificial flow field around the holes. Under such a hydrodynamic condition, carefully designed grouting procedures were used to seal the water-conducting passages.

⇓
Stage IV - Reinforcement Grouting This was the last stage of all injection boreholes. During the grouting, pressure began to increase at the borehole inlets. High pressure injection sealed small fractures between injection boreholes and increased the strength of grout-plug. Single cement slurry was primarily used at this stage. Grouting under static and hydrodynamic pressures was carried out alternatively.

The Stage I grouting was carried out at boreholes Zl-2, Z2-2, and Z5-2. They were the first boreholes drilled, in which cement slurry was mainly vertically diffused. No cement slurry crossing was observed between boreholes. A total of 8,908 m^3 of cement slurry was injected. Appearance of pressure at the borehole inlets from the water-pressing tests was the indicator of the end of Stage I and the beginning of the Stage II grouting. At Stage II grouting, there was no initial pressure at the borehole inlets. Lateral diffusion was also noted at this stage as the pressure at the later d1illed Z3-2 was measured at 1.0 MPa.

Many fractures were filled at the late period of the pressure grouting. As a result, the number of open fractures was reduced, thus the cement intake decreased. The height of the slurry column in the boreholes rose continuously. When the boreholes were completely filled with the cement slurry, the pressure began to rise at the mouth of boreholes. This symbolized the starting of the reinforcement grouting stage. At this stage, the pressure before injection at the borehole inlets was generally more than 2 MPa. The water-pressing pressure at the mouth of boreholes continued to increase with the increase of the injection pressure. The grouting stopped when the water pressure at the mouth of boreholes was more than 3MPa while the injection pressure was at 2 MPa. Then, the water-pressing tests were carried out to check whether the grout-plug was effective in stopping water flows.

Boreholes Zl-2, Z2-2, and Z5-2 were drilled into the KCC. They were characterized by soft strata, fast footage, and unstable walls. Collapses or cave-ins occurred frequently during drilling. The washing fluid (drilling mud) consumption was all greater than 50 m^3/h. The rocks in the KCC were highly permeable. In order to reduce the washing fluid consumption, the drilling was carried out after grouting in the upper section of the grout-plug. Grouting was performed immediately regardless of the designed target if the borehole was in the grouting section but the loss of circulation liquid was very serious and the drilling was too difficult to continue. Grouting of the entire section was often accomplished through multiple intermittent grouting.

For the late drilled primary boreholes Z3-2, Z4-2 ad Z5-2, their drilling conditions were obviously improved, except at borehole Z3-2. A relatively large fluid loss occurred at Z3-2. There was basically no or very little consumption of washing liquid at Z4-2 and Z5-2. Although the drilling was fast and the efficiency was high, the borehole walls were relatively stable. For the last drilled secondary and tertiary holes, the washing fluid consumption decreased significantly. None of them had a continuous drilling liquid loss.

Boreholes Z3-J2 and C2 were used for filling skeletal materials in the upper part of the KCC and also used for water level observation. At the time of grouting, the skeletal material injection was carried out in these two boreholes. With gradual formation of the grout-plug, the water inrush at boreholes Z3-J2 and C2 became more frequent as the water body in the upper KCC was progressively confined and isolated. Because the water level in the Ordovician limestone near the KCC was at elevation of 36 m asl, the water in the upper KCC did not represent the water level of the Ordovician limestone.

At Stage I, there was basically no cement slurry crossing between injection boreholes. As the grouting pressure was increased and the grouting entered into the subsequent stages, the cement slurry crossing and water inrush became frequent. For example, cement mud crossing occurred between Z5-2' and Z2-2, Zl-2 and Z5-2, Z3-2 and Z5-2. The slurry crossing indicated that the slurry movement changed from the initial vertical diffusion to lateral diffusion subsequently. As the fracture passages between boreholes were gradually filled, the cement slurry injected at each borehole overlapped with each other to form one continuous solid grout-plug.

During reinforcement grouting, no more cement mud crossing was observed between boreholes. However, the water inrushes at monitoring boreholes became more frequent, which suggested that the large fractures had been sealed and the grout mixture was pushed into the small and micro fractures.

Under relatively high injection pressure, the residual water in small fractures was squeezed out and drained through the non-grouted boreholes. This artificially created flow field also produced the conditions for hydrodynamic grouting in small fractures.

4.4. Completion Criteria of Grouting

Injection pressure and pump delivery rate are commonly used standards in determining the completion of a grouting project (Liu et al., 2008). During the initial stage of grouting, the cement slurry had to overcome the maximum static water pressure to push out the water in voids and fractures. Based on the control radius of each borehole and past engineering experiences, the injection pressure used in the project was approximately 1.5 times the maximum hydrostatic pressure. It should be pointed out that the grouting process itself was a dynamic and iterative process. The grouting parameters were adjusted constantly to the dynamic geological conditions of the boreholes according to any additional geological and hydro geological data that were obtained during the grouting. The completion criteria should also consider factors such as the length of injected section and cement mud performance parameters to be more objective.

In order to precisely describe the permeability of the injected section and to reflect actual grouting effect, we proposed in this project the use of water absorbance rate as the completion criteria. The water absorbance rate was defined as the amount of water intake along 1 m of grout section under 1 m hydraulic head pressure within 1 minute. It was determined by conducting specifically designed water pressure tests at the end of grouting. Such water pressure tests are in principle similar to the Lugeon test that is often used at dam sites (Milanovic, 2000). The Lugeon test takes place in section of 5 m intervals. The water is injected into the section isolated by packers, which are mechanically, hydraulically or pneumatically expanded and pressed against the borehole walls. A Lugeon unit is defined as the amount of water received by rock mass within 1 m borehole length at a pressure of 10 bars in 1 minute.

The water absorbance rate is closely related to the permeability of a rock mass. A very small water absorbance rate usually suggests that the rock mass is less permeable to water. Table 5 shows seven levels of rock permeability and their relationship to water absorbance.

Table5. *Relationship between permeability and water absorbance in rock masses*

Rock Description	Permeability K (Cm/S)	Water Absorbance Q (L/Min.M.M)	Characteristics Of Rock Mass
Impermeable	$K < 10^{-6}$	$Q < 0.001$	Competent rock with fractures of equivalent opening < 0.025 mm
	$10^{-6} \leq K < 10^{-5}$	$0.001 \leq q < 0.01$	Rock mass with fractures of equivalent opening of 0.025 - 0.05 mm
	$10^{-5} \leq K < 10^{-4}$	$0.01 \leq q < 0.1$	Rock mass with fractures of equivalent opening of 0.05 - 0.1 mm
	$10^{-4} \leq K < 10^{-2}$	$0.1 \leq q < 1$	Rock mass with fracture of equivalent opening of 0.1 - 0.5 mm
	$10^{-2} \leq K < 100$	$q \geq 1$	Rock mass with fractures of equivalent opening of 0.5 - 2.5 mm
Permeable	$K \geq 100$		Rock mass with fractures of equivalent opening ≥ 2.5mm

The water absorption rate was a comprehensive index that took into account the following factors such as hydraulic pressure in grout section, pressure at the mouth of water-pressing boreholes, and the length of grout section. The result was not influenced by the cement mud performance. Such a completion criteria reflected more precisely the function of the grout-plug as a hydrologic barrier. Because the purpose of each grouting stage was different, the threshold values for completion at each stage were different.

- Water absorbance threshold for the early grouted boreholes Zl-2, Z5-2, and Z2'-2 was 0.05 L/min.m.m. The relaxed threshold was set because the grouting at this stage was to fill large voids and fractures in the KCC, to transform highly permeable rock into weakly permeable rocks, to ensure relatively stable borehole walls, and to reduce borehole cave-in accidents.

- Final water absorbance threshold for other primary boreholes was 0.005 L/min.m.m. At gradually increased grouting pressure, the medium-sized fractures were filled; the loose rocks in the KCC were reinforced; and the grout section was transformed into less permeable rock mass.
- For the late grouted three primary boreholes Z3-2, Z4-2, and Z5'-2, the water absorbance threshold was 0.005 L/min.m.m. After the large fracture passages and voids were filled in the KCC, the walls of these boreholes were relatively stable. The borehole collapse rarely occurred.

The water absorbance threshold for the secondary and tertiary grout boreholes was 0.00 l L/min.m.m. After a great amount of cement mud was injected into the primary boreholes and the cement mud was sufficiently hydrated and consolidated, the small and micro fractures were further filled and consolidated through the subsequent hydrodynamic grouting and reinforcement grouting. This strict threshold would ensure that the entire grouting section of the KCC became an impermeable or weakly permeable grout-plug.

4.5. Grout Intake Distribution

From August 13, 2006 to September 13, 2006, a trial injection was carried out in the upper section of borehole Z5-2, in which 3.5 million kilograms of cement was consumed. The formal injection of grout started on September 9, 2006 and ended on May 29, 2007. A total of 49,339 m^3 cement mud was used, including 41.6 million kilograms of #42.5 Portland cement, 8,000 kg triethanolamine, 80,000 kg industrial salt and 1,500 kg soluble silicate. Figure 6 shows the percentage distribution of injected cement in each borehole. The injected cement mixture at boreholes Zl and Z2' in the north of the KCC accounted for 24% and 30%, respectively; while the sum of the injected volumes at Z5 and Z5' accounted for 26%. These boreholes were the primary injection boreholes of the project and indicated that the strata in the south and the east of the KCC were fragmented and broken and consisted of voids and fractures. Coring of Z3 and Z3-J2 further confirmed that rocks at that part of the KCC were fragmented, and the core recovery was extremely low. The injected volume at borehole Z4 in northwest of the KCC accounted for only 6%, suggesting relatively competent rock masses at the location. In addition, Z4 was drilled after some of fractures may have been filled.

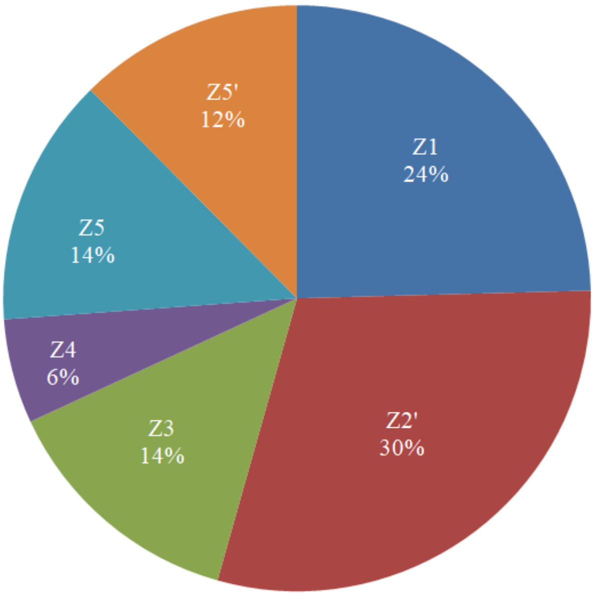

Fig6. *Grout intake distribution among grouting holes*

5. Evaluation Of Plug Effectiveness

5.1. Borehole Coring

Z4 and Z3 were used to check the quality of the constructed grout-plug. Boreholes Z4-J-1 and Z3-J2 were cored from 610m to the bottom of boreholes. The average core recovery at Z4-J1 was 37%. The borehole encountered normal formation from 610 to 657.4 m. Coring was relatively easy. Cement fills were observed in the core samples, as shown in Figure 7.

Fig7. *Core samples at select grout holes*

5.2. Water-Injection Test

Water-injection or water-pressing test was conducted at depths of 610 m, 664 m and 742 m at Z4-J1 and Z3-J2, respectively. The water absorption rate calculated from the water-injection test was smaller than the pre-determined threshold, i.e., 0.001 L/min.m.m.

5.3. Water Release Test

The quality of the grout-plug was defined by the water-sealing effect of the plug, the recharge inflow of the KCC, and the spatial hydraulic conductivity in the upper section of the plug. It was quantitatively evaluated by conducting an underground aquifer test in which 242 m^3 of water was drained. The drainage borehole was in the -480 m drift with a total depth of 341.6 m. Small-scale faults were intercepted at the depth of 289.6 m. The borehole intercepted the KCC at 308 m, and a water inrush occurred at 310 m with the maximum inflow of 14.2 m^3/h. The inflow was stabilized at 11.8 m^3/h.

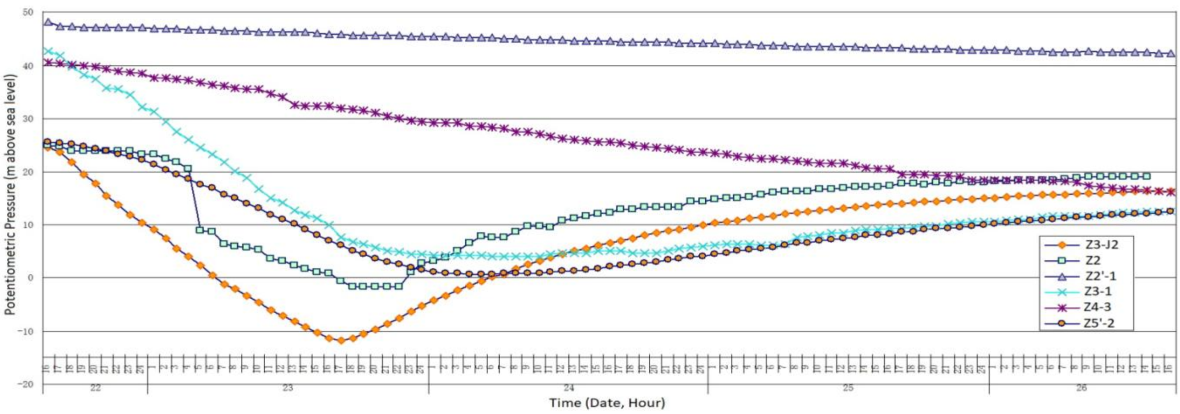

Fig8. *Potentiometric pressure measurements at observation wells in response to water release test*

There were six surface observation boreholes Z3-J2, Z2, Z2'-1, Z3-1, Z4-3 and Z5'-2 (Fig. 8). Of them, Z3-J2 was the principal observation borehole and was drilled into the center of the upper section of the KCC. The maximum drawdown at Z3-J2 was 36.3 m. The water level was recovered to the level of 16.4 m asl. During the water drainage test, the water levels at Z2'-l, Z4-3, and Zl-1 did not change, which suggested a lack of connection between these three boreholes and the drainage borehole. The water in these boreholes were confined and isolated with no hydraulic connection to either the Ordovician limestone or the voids in the upper part of the KCC.

In summary, the results from the core samples, the water-injection test, and the water drainage test showed clearly that the constructed grout-plug had successfully cut off the hydraulic connection between the coal seam and the lower Ordovician limestone.

6. CONCLUSIONS

Design and construction of watertight plugs in underground mines is a complicated process, and the design equations available for plug length estimate can give results over a wide range. Aside from following best practices in mining engineering and civil engineering, the practitioners' knowledge and experience are critical in their design and implementation. In response to a water inrush in which pressurized groundwater in the underlying limestone suddenly flowed into the mine through a KCC in Dongpang Mine, China, a vertical watertight plug was successfully constructed within the KCC to restore the flooded mine to full operation. The grouting was conducted on the surface by drilling boreholes and pumping grout into the collapse column, while monitoring of the effectiveness of the plug was carried out underground by artesian flow tests through directionally drilled wells. The completed watertight plug is 73 m high at depths from 664 to 737 m below groundwater surface. Construction of such a large in-situ plug in a concealed collapse feature overcame several engineering challenges. A key component to the success of this project was that all the grout holes including primary, secondary and tertiary ones were directed precisely to the designed locations. In addition, the technique, quality control, grouting stages, and completion criteria of each stage were tailored to the site-specific conditions. Core samples collected at boreholes and underground water-injection and water drainage tests verified effectiveness of the completed plug.

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