

## **LOAD CALCULATIONS AND SELECTION OF THE POWERED SUPPORTS BASED ON ROCK MASS CLASSIFICATION AND OTHER FORMULAE FOR ABU-TARTUR LONGWALL PHOSPHATE MINING CONDITIONS**

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### **ABSTRACT**

The safe operations at longwall face depend on the type and capacity of the powered roof supports. At Abu-Tartur longwall phosphate mine, two types of powered roof support with various capacities were tried earlier. The two leg shield power supports was applied finally. The main problem at Abu-Tartur longwall mines is the high frequency of roof rock falls during face advance. Roof collapse is due to an inadequate capacity of the chosen powered supports. So, in this paper the load exerted on the shield support is calculated by different methods and taking into consideration the actual roof conditions by rock mass classification system to select the suitable type of the supports. From these calculations, it is found that the average maximum pressure on the supports is about 416t/m with the yield pressure on the shield support of a value of 520t/m. Different items are recommended such as; increase the rate of the advance, exploit ore in two consecutive shifts, decrease the period of the face stoppage and small thickness about 30cm from phosphate ore should be left in the roof during exploitation to ensure stability during face advance. The following shearer is recommended to increase rate of face advance. The specification of this shearer are model Cat EL 3000/2011 with typical length 15.2 m, seam thickness range from 2.5 to 5.5 m, cutting drum diameter up to 2.7 m, haulage speed up to 32 m per min, cutting drum speed 54.3 rpm and bits drum hardness up to 68.4 Mpa is to secure high rate of face advance. Shield support model Kottadih, CDFI, France, 2x470 is selected for Abu-Tartur mining conditions to support the face during working.

**Keywords:** Rock Mass Classification - Geological Strength Index (GSI) - Abu-Tartur longwall phosphate mine- Shield support- Shearers.

### **1. Introduction**

Determining the optimum capacity of the support and selecting the proper type of support are two major problems facing the selection of powered supports at longwall faces. The principal factors which influence the magnitude of the load on the support include rather than the geological formation of the roof and its rock mass classification, setting load density, height of the caving block, distance of fracture zone from the face, the overhang of goaf, the support yield characteristics, open width of panel, distribution of load, methods of extraction, face operations, and the mechanical strength of the debris above the canopy and below the support base. [1,2]

The features of a weak roof are very easily falling in an unsupported area between the canopy tip and faceline and tend to break into small sizes of rocks, which can easily enter into the working area and cause problems if a support without shield protection is used.

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However, under a strong roof condition, roof falls in an unsupported roof area is no longer a problem since a strong rock has a higher tensile strength, but it tends to overhang behind the support into the gob. [3]

The phenomena of rock pressure appear to have a dynamic nature even if the face remains stationary. Experience shows that in a stationary stope, the rocks as a rule become settled. This means that roof rocks continue slowly to sag, exfoliate and fracture, and fall down in small lumps. In the end, rocks in a stationary stope may become so loose that they will start to cave in. Stoppage of a face is particularly undesirable when the roof is made up of argillaceous rocks. Restarting operations in production stopes that have been inactive for a long time may lead to serious difficulties. Therefore, faces kept in reserve for a long time in conformity with operational plans of the mine or stopped temporarily for some reason should be periodically refreshed, that is advanced over a short distance so as to eliminate the hazards of rock settling. Hence the rapid advance of working faces, very advantageous in general and favoures in the case under discussion.[4]

The high rate of face advance prevents the formation of creep in the roof rocks, which causes an increase of roof deformation with time. This increase in the deformations leads finally to roof fracturing and falling. So, the high rates of face advance and exploit of ore in two consecutive shifts are highly recommended.

The previous studies for the calculation of load on the face support in Abu-Tartur phosphate mine don't take into considerations the effects of the rock mass classification. The aim of this research is to apply rock mass classification systems and other formulae to calculate the load on the shields and select the suitable powered supports for the conditions of Abu-Tartur phosphate mine. One of these systems is Geological Strength Index (GSI) method proposed by the Hoek and Brown [5] which determines rock mass properties for calculating load on the support. GSI values for immediate and main roof rocks are determined from geological conditions, as lithology, structure of the interlocking of rock blocks and the conditions of the surfaces between these blocks [6]. The data used in calculations are collected from geological reports of the company and from laboratory tests of phosphate ores and shale rocks in the roof.

## **2. Powered Roof Supports**

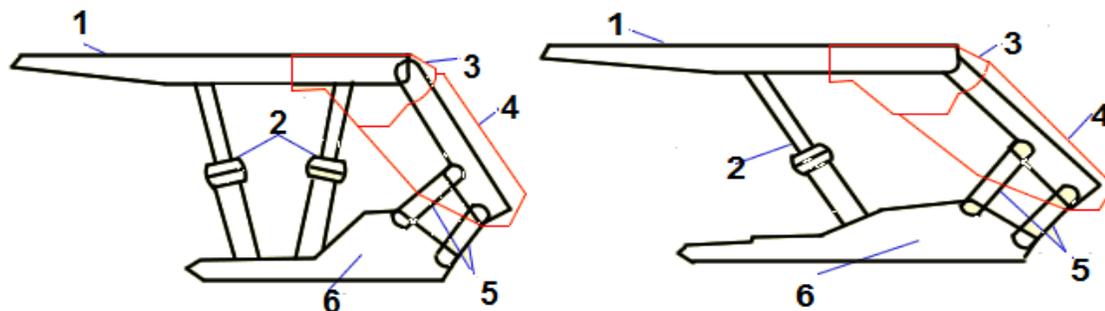
Powered supports have come after a long development of steel supports in longwall faces. A new system was developed, which is hydraulic in design, with props and caps incorporated into one unit and connected to the armoured face conveyors to advance regularly with the cutting at the face line which is suitable compared to the prop friction steel. This system has been further improved in different designs that make the back of the face safer with "shield" supports.[7]

### *2.1. Types of powered support*

- i- The hydraulic chock (Gullick Company of England)
- ii- The frame supports (Dowty Company of England)
- iii- The shield supports
- iv- The chock - shield supports

## 2.2. Description of powered support

All powered supports consist of a canopy, a base, hydraulic legs and control system as shown in Fig(1)[8]. Table (1), provides the dimensions and the operating data for each type of powered support, giving lower and upper limits.[7]



**Fig. 1.** The chock - shield supports & The 2-leg shield supports

1-canopy 2-hydraulic legs 3-side shield 4- goaf shield 5-lemniscate linkages 6-base

**Table 1.**

Dimensions and Operating Data for Powered Supports

type	Yield capacity (tons)	Dimension (cm)			Max. pressure(kg/cm <sup>2</sup> )		Clear working distance(cm)	Supported roof (%)	Hydraulic pressure(kg/cm <sup>2</sup> )	Canopy tip yield load (t)
		height	length	width	floor	roof				
frame	260	81.3	322.6	91.5	19	0.7	114.3	38	70.2	9.7
	1050	322.5	600	350	91.3	33.2	274.3	70	351	58
chock	150	71.1	284.5	81.2	15.5	1.5	91.5	85	101.8	8
	800	365.8	269	218.4	65.3	37.6	474.9	90	320	45
shield	115	61	330	130	4	3.5	160	100	140.5	
	600	460	518	150	77.2	21	287	100	35	
Chock-shield	320	94	450	140	16.2	7.4	203.2	100		8
shield	500	340	482.6	145	28	21	254	100		14.3

## 2.3. Design of powered supports

In designing powered supports there is no one established set of formulae or systems. Almost every country has established its own systems. Thus we will describe the systems classified by the country.[7]

## 2.4. Dimensions related to supporting

### 2.4.1. Yielding pressure.

There is a relation between the yielding and setting load or operating pressures as follows:[7]

$$P_y = 1.25P_i \quad (1)$$

$P_y$  = yielding pressure, t/m<sup>2</sup>

$P_i$  = operating pressure, t/m<sup>2</sup>.

#### 2.4.2. Unsupported face distance.

There is always a small distance between the face and the end of canopy. This distance increases as the wining machine cuts. It may change from 0.25 to 0.8 m according to the depth of cut.[7]

#### 2.4.3. Maximum and minimum heights of shield support.

“Maximum” and “Minimum” define the working heights of the supports according to the geological conditions and to the convergence evaluation of the face. Owing to changes of the seam thickness, some coal is left at the roof. The working heights are given by the following expression:[7]

$$\log \frac{h_{\max}}{1.1h_{\min}} = 1.704 \frac{m'}{m_{av}} \quad (2)$$

$$h_{\min} = m_{av} - m' - c.l \quad (3)$$

Where  $h_{\max}$  = maximum height, in meter       $h_{\min}$  = minimum height, in meters  
 $m_{av}$  = average thickness, in meters       $m'$  = geological deviations in thickness, in meters

$c$  = average convergence, in millimeters per meter     $l$  = width (supported span) of the face, meters

Table (2) is used to calculate minimum heights for powered supports for various face widths and different seam thicknesses.

**Table 2.**

Recommended minimum heights for powered supports for different seam thicknesses.

Average seam thickness(m)	Convergence (mm/m)	Geological deviations(m)	Minimum powered supported heights(m)		
			l=1.75m	l=2.5m	l=3.0m
2.20	80	0.20	2.26	2.20	2.16
2.40		0.20	2.46	2.40	2.36
2.60		0.20	2.66	2.60	2.56
3.00		0.20	3.06	3.00	2.96
3.20		0.25	3.26	3.20	3.16

### 3. Geology of Abu-Tartur plateau

The area of Abu-Tartur plateau is about 1200 km<sup>2</sup>. It is semi oval in shape opening towards the North West. In the south east and North West the plateau is limited by steep scarps.[9]

#### 3.1. Plateau formation (stratigraphy)

Rock properties and stratigraphic column along Abu-Tartur plateau are represented in Table (3)

**Table 3.**

Rock properties and stratigraphic column along Abu-Tartur plateau

	Column	Thickness, m	Description	Rock properties						
				Density, K N/m <sup>3</sup>	Compressive strength, Mpa	Tensile strength, Mpa	Modulus of elasticity, Gpa	Cohesion, Mpa	Bending strength kg/cm <sup>2</sup>	Angle of internal friction
Roof	Main2	20-120	Kurkur formation (limestone)	22.2	65.1	7.5	14.4	17.5	195	33.5°
		80-130	Dakhla formation (clayey-carbonate)	19.8	46.8	5.1	7.6	14.1	64.5	31.4°
		4.2-8	Phosphate-carbonate	21	43.4	3.1	6	11.2	130	30°
	Main1	6.6-16	Argillaceous sand	17	20.6	2.6	6.7	6.3	61.8	35°
		Imm.	7.5-30	Papery clay shale	21.4	14	4.2	7.2	6.6	53.3
Ore		0.75-7.3	phosphorite	20.6	70	3.1	8.2	11.2	12.8	34°
Floor		—	Nubia formation (variegated clay)	18.2	20.5	2.6	4.8	6.7	67.3	33.5°

3.2. For Abu-Tartur mining conditions; the available design data are as follows:

Phosphate ore (average thickness 3.5m and average volumetric weight 20.6 kN/m<sup>3</sup>)

Immediate roof (Papery clay shale with an average thickness of 17.5m, average volumetric weight 21.4 kN/m<sup>3</sup>, uniaxial compressive strength of 20.6 Mpa., Bending strength 53.3 kg/cm<sup>2</sup> and buckling factor 1.2).

Main roof (Argillaceous sand with average thickness of 11m, average volumetric weight 17 kN/m<sup>3</sup>, uniaxial compressive strength 14 Mpa. and Bending strength 61.8kg/cm<sup>2</sup>).

Panel width 100m (determined from the work [6] and recommended for use), average cover depth 200m, rate of advance 0.63 m per shift, the shield supports in use is the Joy mining machine 320 tonne two legs and extraction of ore is carried only in one shift/day.

#### 4. Methods of load calculations

There are three methods for the load calculation on the powered support with different in units (t, t/m and t/m<sup>2</sup>)

#### 4.1. Rock mass classification (russian system)

Estimations of the immediate roof rock properties.[7,10]

Geological Strength Index GSI value is determined based on geological descriptions of Abu-Tartur area, so the value of GSI will equal to 25 (GSI = 25)

The value of  $M_i$  (Hoek-Brown constant) = 6 (clastic sedimentary rock, shales)

$\sigma_{ci} = 14$  Mpa. From Table (3)                       $\sigma_{bi} = 53.3$  kg/cm<sup>2</sup>. From Table (3)

The value of modulus ratio (MR) = 200 (clastic sedimentary rock)

$s$ ,  $a$  and  $m_b$  constants can be calculated as follows:

$$s = \exp\left(\frac{GSI - 100}{9}\right) \quad \therefore s = \exp\left(\frac{25 - 100}{9}\right) = 2.404 \times 10^{-4}$$

$$a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right)$$

$$\therefore a = \frac{1}{2} + \frac{1}{6} \left( e^{-25/15} - e^{-20/3} \right) = 531.267 \times 10^{-3}$$

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right)$$

$$\therefore m_b = 6 \times \exp\left(\frac{25 - 100}{28}\right) = 411.967 \times 10^{-3}$$

Bending rock mass strength ( $\sigma_{bm}$ ) can be calculated as follows:

$$\sigma_{bm} = \sigma_{ci} \frac{(m_b + 4s - a(m_b - 8s)) \left( \frac{m_b}{4} + s \right)^{(a-1)}}{2(1+a)(2+a)}$$

$$\therefore \sigma_{bm} = 53.3 \frac{(0.412 + 4 \times 2.404 \times 10^{-4} - 0.5313 (0.412 - 8 \times 2.404 \times 10^{-4})) \left( \frac{0.412}{4} + 2.404 \times 10^{-4} \right)^{(0.5313-1)}}{2(1+0.5313)(2+0.5313)}$$

$$\therefore \sigma_{bm} = 53.3 \frac{195.086 \times 10^{-3} \times 2.89906}{7.75209} = 3.887 \text{ kg/cm}^2 = 0.3887 \text{ Mpa.}$$

Intact rock modulus can be calculated as follows:

$$E_i = MR \times \sigma_{ci} \quad E_i = 200 \times 14 = 2800 \text{ Mpa.}$$

Deformation modulus of rock mass for compression

$$E_{rm} = E_i \left( 0.02 + \frac{1}{1 + e^{((60-GSI)/11)}} \right)$$

$$E_{rm} = 2800 \left( 0.02 + \frac{1}{1 + e^{((60-25)/11)}} \right) = 167.596 \text{ Mpa.}$$

Deformation modulus of rock mass for tension [11]

$$E_t = n E_c = 0.135 * 167.596 = 22.625 \text{ Mpa.}$$

As well as calculations for the main roof rock properties. (Main 1)

Geological Strength Index GSI value is determined based on geological descriptions of Abu-Tartur area, so the value of GSI will equal to 30 (GSI = 30)

The value of  $M_i$  (Hoek-Brown constant) = 7 (clastic sedimentary rock, siltstone)

$$\sigma_{ci} = 20.6 \text{ Mpa. From Table (3)} \quad \sigma_{bi} = 61.8 \text{ kg/cm}^2. \text{ From Table (3)}$$

The value of  $MR = 375$  (clastic sedimentary rock, siltstone)

$$s = \exp\left(\frac{GSI - 100}{9}\right) \quad \therefore s = \exp\left(\frac{30 - 100}{9}\right) = 4.19 \times 10^{-4}$$

$$a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right)$$

$$\therefore a = \frac{1}{2} + \frac{1}{6} \left( e^{-30/15} - e^{-20/3} \right) = 0.522$$

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right) \quad \therefore m_b = 7 \times \exp\left(\frac{30 - 100}{28}\right) = 0.575$$

$$\sigma_{bmi} = \sigma_{bi} \frac{(m_b + 4s - a(m_b - 8s)) \left( \frac{m_b}{4} + s \right)^{(a-1)}}{2(1+a)(2+a)}$$

$$\therefore \sigma_{bmi} = 61.8 \frac{(0.575 + 4 \times 4.19 \times 10^{-4} - 0.522(0.575 - 8 \times 2.404 \times 10^{-4})) \left( \frac{0.575}{4} + 4.19 \times 10^{-4} \right)^{(0.522-1)}}{2(1+0.522)(2+0.522)}$$

$$\therefore \sigma_{bmi} = 61.8 \frac{0.278 \times 2.524}{7.677} = 5.648 \text{ kg/cm}^2 = 0.5648 \text{ Mpa.}$$

$$E_i = MR \times \sigma_{ci}$$

$$E_i = 375 \times 20.6 = 7725 \text{ Mpa.}$$

Deformation modulus of the rock mass for compression

$$E_{rm} = E_i \left( 0.02 + \frac{1}{1 + e^{((60-GSI)/11)}} \right)$$

$$E_{rm} = 7725 \left( 0.02 + \frac{1}{1 + e^{((60-30)/11)}} \right) = 628.685 \text{ Mpa.}$$

Deformation modulus of the rock mass for tension

$$E_t = n E_c = 0.135 * 628.685 = 84.872 \text{ Mpa}$$

Data	$E_c$ , kg/cm <sup>2</sup>	$E_t$ , kg/cm <sup>2</sup>	$\sigma_{bm}$ , kg/cm <sup>2</sup>	Thickness, m	volumetric weight, t/m <sup>3</sup>
Immediate roof	1675.95	226.25	3.887	$h_1=17.5$	$\gamma_1=2.14$
Main roof	6286.85	848.72	5.648	$h_2=11$	$\gamma_2=1.7$

Pressure due to immediate roof (Q)

$$Q = \gamma_1 \times h_1 = 2.14 \times 17.5 = 37.45 \text{ t/m}^2$$

Deflection of the immediate roof

$$f_1 = \frac{\gamma_1 \times h_1 \times l_1^4}{2E_1I_1}$$

Where:  $E_1$  elasticity modulus in bending kg/cm<sup>2</sup>,  $I_1$  moment of inertia and  $l_1$  total distance for working and step of fractured, m.

$$E_1 = \frac{4E_c \times E_t}{(\sqrt{E_c} + \sqrt{E_t})^2} = \frac{4 \times 1675.95 \times 226.25}{(\sqrt{1675.95} + \sqrt{226.25})^2} = 483.999 \text{ kg/cm}^2$$

$$I_1 = \frac{b_o \times h_1^3}{12} + b_o \times h_1 \left( \frac{h_1 \times \sqrt{E_c}}{\sqrt{E_c} + \sqrt{E_t}} - \frac{h}{2} \right)^2$$

$$= \frac{1 \times 1750^3}{12} + 1 \times 1750 \left( \frac{1750 \times \sqrt{1675.95}}{\sqrt{1675.95} + \sqrt{226.25}} - \frac{1750}{2} \right)^2 = 733.349 \times 10^6 \text{ cm}^4$$

$$l_1 = b + l_f$$

Take working distance  $b = 2$  m

$$\text{Step of fractured } l_f = 2r = 2 \times 1.89 = 3.78 \text{ m} \quad \therefore l_1 = 2 + 3.78 = 5.78 \text{ m}$$

$$\therefore f_1 = \frac{0.00214 \times 1750 \times 578^4}{2 \times 483.999 \times 733.349 \times 10^6} = 0.589 \text{ cm}$$

Load from the immediate roof

$$R_1 = \frac{h_1 \times \gamma (3b^2 + 8b \times l_f + 6l_f^2)}{8b}$$

$$= \frac{17.5 \times 2.14 (3 \times 2^2 + 8 \times 2 \times 3.78 + 6 \times 3.78^2)}{8 \times 2} = 370.3 \text{ t/m}$$

Deflection of the main roof

$$f_2 = \frac{\gamma_2 \times h_2}{2E_2I_2} \left( \frac{l_1^2 \times l_2^2}{2} - \frac{l_2 \times l_1^3}{3} + \frac{l_1^4}{12} \right)$$

$$E_2 = \frac{4E_c \times E_t}{(\sqrt{E_c} + \sqrt{E_t})^2} = \frac{4 \times 6286.85 \times 848.72}{(\sqrt{6286.85} + \sqrt{848.72})^2} = 1815.595 \text{ kg/cm}^2$$

$$I_2 = \frac{b_o \times h_2^3}{12} + b_o \times h_2 \left( \frac{h_2 \times \sqrt{E_c}}{\sqrt{E_c} + \sqrt{E_t}} - \frac{h}{2} \right)^2$$

$$= \frac{1 \times 1100^3}{12} + 1 \times 1100 \left( \frac{1100 \times \sqrt{6286.85}}{\sqrt{6286.85} + \sqrt{848.72}} - \frac{1100}{2} \right)^2 = 182.127 \times 10^6 \text{ cm}^4$$

$$I_2 = \sqrt{\frac{h_2 \times \sigma_f}{3\gamma_2}} = \sqrt{\frac{1100 \times 5.648}{3 \times 0.0017}} = 1103.719 \text{ cm}$$

$$\therefore f_2 = \frac{0.0017 \times 1100}{2 \times 1815.595 \times 182.127 \times 10^6} \left( \frac{578^2 \times 1103.719^2}{2} - \frac{1103.719 \times 578^3}{3} + \frac{578^4}{12} \right) = 0.401 \text{ cm}$$

⊖  $f_2 < f_1$  So no effect for main roof

∴  $R = R_1 = 370.3 \text{ t/m}$

#### 4.2. Wilson formula

Peak abutment or yield stress is determined by this formula [12]

$$\sigma_y = C_o + bp$$

Where:  $C_o$  = uniaxial compressive strength for rock mass

$$C_o = \sigma_c = \sigma_{ci} \times s^a = 14 \times (2.404 \times 10^{-4})^{531.267 \times 10^{-3}} = 0.1673 \text{ Mpa.}$$

$$b = \frac{1 + \sin \phi}{1 - \sin \phi} = \frac{1 + \sin 30}{1 - \sin 30} = 3$$

$$p = \gamma \times H \quad p = 0.017 \times 17.5 = 0.2975 \text{ Mpa}$$

$$\sigma_y = 0.1673 + 3 \times 0.2975 = 1.0598 \text{ Mpa} = 105.98 \text{ t/m}^2$$

#### 4.3. German system

Maximum carrying capacity of chock support is determined as follows.[7]

$$F_{\max} = 5nm = 5 * 2 * 3.5 = 35 \text{ t/m}^2$$

Where:  $F_{\max}$  = maximum carrying capacity of chock support, in t/m<sup>2</sup>

m = seam thickness, in meters

n = factor of safety, in general taken 2

#### 4.4. English system

Minimum capacity of support is determined as follows. [7]

$$F_{\min} = \gamma \times h_{im} = \gamma \frac{m}{k-1} = 2.14 \frac{3.5}{1.2-1} = 37.45 \text{ t/m}^2$$

Where:  $F_{\min}$  = minimum capacity of support, in tonnes per square meter

$\gamma$  = density of immediate roof, in tonnes per cubic meter

$h_{im}$  = immediate roof height, m

$m$  = thickness of the seam, m

$k$  = factor of expansion of the immediate roof; may be taken as 1.2

#### 4.5. Austrian systems

Minimum bearing capacity of one hydraulic unit support is determined as follows. [7]

$$R_0 \geq \frac{3}{4} \frac{\beta}{n} \frac{(d+e)^2}{k-1} \frac{m}{d} \gamma = \frac{3}{4} \times \frac{0.9}{1} \times \frac{(3+0)^2}{1.2-1} \times \frac{3.5}{3} \times 2.14 = 75.84 \quad t$$

For two legs (two hydraulic units) = 2 \* 75.84 = 151.68 t

Where:

$R_0$  = minimum bearing capacity of one hydraulic unit, in tonnes

$\beta$  = diminishing factor, usually taken as 0.9

$n$  = number of units of power support frame (or chock) per linear meter of face

$d$  = the distance between the back of support and face line in meters

$e$  = the distance between back of support and uncaved roof, m

$m$  = seam thickness, in meters

$k$  = expansion factor, usually taken as 1.2

$\gamma$  = density of immediate roof, in tonnes per cubic meters

#### 4.6. French system

Load carrying capacity of support is determined as follows. [7]

$$CvT = (q m)^{3/4} H^{-1/4} \left( \frac{6800}{P_a} + 66 \right)$$

The minimum convergence rate can be expressed as follows [13]

$$CvT = 30 + 10m = 30 + 10 * 3.5 = 65 \text{ mm/m}$$

$$65 = (1 \times 3.5)^{3/4} \times 200^{-1/4} \left( \frac{6800}{P_a} + 66 \right) \quad \therefore p_a = 230t/m$$

Where:  $CvT$  = convergence at the face, in millimeters per meter of advance

$W$  = thickness of the seam, in meters

$q$  = subsidence factor: 1 for caving; 0.6 pneumatic stowing; 0.15 hydraulic stowing

$H$  = depth below surface, in meters (between 100 and 1000 m)

$P_a$  = load carrying capacity of support, in tonnes per meter of face length.

#### 4.7. Polish system

Average carrying capacity is determined as follows. [7]

$$P_o = \frac{P_1 + P_2 + P_3}{F} n$$

Where:  $P_o$  = average carrying capacity, in tonnes per square meter

$P_1$  = nominal load of one unit, in tonnes

$P_2$  = load on the unit when advancing, in tonnes, taken as zero

$P_3$  = carrying load of the unit just set, in tonnes

F = the area of the face covered by three supports, in square meters

n = efficiency factor of supports, taken around 0.8

Taken the area covered by three supports placed at 1.4 m intervals, the width of the face (0.0m unsupported at the back + 3.65m supported by the canopies + 0.35m unsupported at the front), all totaling 4 m, the nominal carrying load is  $p_1 = 70$  t per leg and set support carrying load  $p_3 = 23$ t per leg. Each support is equipped by four legs. Then

$$p_1 = 4 * 70 = 280 \text{ t}$$

$$p_2 = 0$$

$$p_3 = 4 * 23 = 92 \text{ t}$$

$$F = 4 * (3 * 1.4) = 16.8 \text{ m}^2$$

$$\therefore P_o = \frac{280 + 0 + 92}{16.8} \times 0.8 = 17.71 \text{ t/m}^2$$

#### 4.8. American system

Load of the immediate roof to be supported is determined as follows. [7]

$$W = LS_wH$$

Where: W= weight of the immediate roof to be supported

L = length of the beam

S = average spacing between the supports

support

w = average weight density of the roof rock

H = thickness of the immediate roof

roof

$$\therefore W = 3 * 1.5 * 2.14 * 17.5 = 168.53 \text{ t}$$

#### 4.9. China system

Load carrying capacity of support is determined as follows.[13]

$$R = \gamma h l_s \frac{[(1 + k_d(1 + \tan \delta)) / \tan(\alpha - \delta)]}{[1 + (F_d / \tan(\alpha - \delta))]}, \text{ kN/m}$$

Where:  $\gamma$  = rock weight per unit volume

h = height of immediate roof, m

l<sub>s</sub> = length of roof beam, m

F<sub>d</sub> = coefficient of friction between fractured rock

fractured rock

$\delta$  =  $\tan^{-1}(F_d)$  angle of friction between fractured rock

$\alpha$  = angle of fractured rock in immediate roof

k<sub>d</sub> = constant due to passive or weighting load by the main roof. Normally 1.2-1.8 as determined by the force equilibrium conditions

$$\therefore R = 21.4 \times 17.5 \times 4 \times \frac{[(1 + 1.5(1 + \tan 30)) / \tan(34 - 30)]}{[1 + (0.58 / \tan(34 - 30))]} = 5818.70 \text{ kN/m} = 581.87 \text{ t/m}$$

#### 4.10. Terzaghi formula

Pressure on face supports is calculated by Terzaghi formula [17], this formula will be:

$$\sigma_t = \frac{\gamma B}{K \tan \phi}$$

Where:  $\sigma_t$  = pressure on face supports, t/m<sup>2</sup>

$\gamma$  = density of immediate roof, t/m<sup>3</sup>

B = half width of the panel subjected to loading, m

B1 = half actual width of the panel subjected to loading, m

m = seam thickness, m

$\phi$  = angle of internal friction of roof rock, in degree

K = an empirical coefficient, taken as K = 1

$$\Theta B = B1 + m \tan (45 - \phi / 2) \quad \therefore B = 50 + 3.5 \tan (45 - 15) = 52.02 \text{ m}$$

$$\therefore \sigma_t = \frac{2.14 \times 52.02}{1 \times \tan 30} = 192.82 \text{ t/m}^2$$

#### 4.11. Yehia formula

Bearing capacity of powered support is calculated by Yehia formula [18], this formula will be:

$$P_s = 0.95 \left( \frac{Q \cdot W \cdot M \cdot L^{0.75}}{C \cdot N_b^{0.125}} \right)^{1.5} \text{ , t / m}$$

Where: PS = Bearing capacity of powered support required in front of a longwall face, t/m

Q = Filling coefficient depends on the method of strata control and equals to (0.9 - 1.0) in caving method

W = The thickness of the seam, m

M = Constant depends on the rock type of the mine roof, up to 10m and equal to 140 for weak rock

L = Width of the working zone, evaluated by the distance between longwall face and gob end of support canopy, m.

C = The convergence of the immediate roof taken as average value of 17.5 mm/m in weak rock

Nb = The number of overlying strata of immediate roof, up to 10m

$$\therefore P_s = 0.95 \left( \frac{0.9 \times 3.5 \times 140 \times 3^{0.75}}{17.5 \times 1^{0.125}} \right)^{1.5} = 413.61 \text{ t / m}$$

All previous calculations can be summarized as shown in Table (4). The units of load on the support are first calculated by units (t, t/m and t/m<sup>2</sup>). The units are unified to get pressure on support by unit t/m to compare and select the suitable values.

**Table 4.**

Load on powered supports by different formulae.

The formula	Reference	Load on the supports		Remarks
		Calculated	Unified unit ,t/m	
RMC <sup>*</sup> (Russian)	[10]	370 t/m	370	medium
Wilson	[12]	106 t/m <sup>2</sup>	318	medium
French	[7]	230 t/m	230	medium
China	[13]	582 t/m	582	high
Terzaghi	[17]	193 t/m <sup>2</sup>	579	high
Yehia	[18]	414 t/m	414	medium
German	[7]	35 t/m <sup>2</sup>	105	Small- neglect
English	[7]	37 t/m <sup>2</sup>	111	Small- neglect
Austrian	[7]	152 t	101	Small- neglect
American	[7]	169 t	113	Small- neglect
Polish	[7]	18 t/m <sup>2</sup>	54	Small- neglect

\*RMC (Rock Mass Classification)

The average value between medium and higher load pressures on the support taken as (370,318, 230,582, 579 and 414) t/m as shown in Table (4) is 416 t/m, which it used for support selection.

## 5. Design and selection of powered supports

**Yielding Pressure.** From equation (1)

$$P_y = 1.25P_i = 1.25 \times 416 = 520 \text{ t/m}$$

**Maximum and Minimum Heights.**

Minimum height  $h_{\min} = 3.16\text{m}$  from Table (2)

Maximum height from equation (3)

$$\log \frac{h_{\max}}{1.1h_{\min}} = 1.704 \frac{m'}{m_{av}} \quad \log \frac{h_{\max}}{1.1 \times 3.16} = 1.704 \frac{0.25}{3.5}$$

$$\therefore h_{\max} = 4.6\text{m}$$

The suitable type of the shield support for Abu-Tartur mines conditions is Kottadih with a capacity of 2x470 (940t), two leg-shield support as shown in Table (5).[1]

**Table 5.**

Specifications of the power supports.

Name of Project	Make	Support Capacity (Tonnes)&Type	Working Distance Range(m)	Depth of Working(m)
Sheetalpur	Gullick, UK	4x240, Chock Shield	1.40 - 2.09	420 - 450
Dhemomain	Gullick, UK	4x360, Chock Shield	2.02 - 3.20	300
Dhemomain& Jhanjra	Jessop/Gullick	4x550, Chock Shield	1.70 - 3.05	40 - 100
Jhanjra	KM-130,USSR	2x320, Chock	2.50 - 4.10	40 - 90
Churcha & Jhanjra	Joy	4x680, Chock Shield	1.65 - 3.60	90 - 200
<b>Kottadih</b>	<b>CDFI,France</b>	<b>2x470, Shield</b>	<b>2.20 - 4.70</b>	<b>180 - 220</b>
Pathakera	MAMC, Dowty	6x240, Chock Shield	1.11 - 1.74	110

## 6. Conclusions

From this study, the following conclusions can be drawn:

- 1- The average maximum calculated pressure on support is 416 t/m.
- 2- The yielding pressure (capacity for the needed support) equals 520 t/m , Maximum and Minimum Heights ( $h_{max}=4.6m, h_{min}=3.16m$ )
- 3- The suitable type of the shield support for Abu-Tartur mines conditions is Kottadih, CDFI, France with a support capacity of 2x470 (940t), two leg-shield support, working distance ranges (2.2-4.7)m and depth of the working (180-220)m.

## 7. Recommendations

- 1- Increase the rate of the advance and extract ore in two consecutive shifts.
- 2- Decrease the period of the face stoppage
- 3- Shearer machine of model Cat EL 3000/2011 with typical length 15.2 m, seam ranges 2.5 - 5.5 m, cutting drum diameter up to 2.7 m, haulage speed up to 32 m per min, cutting drum speed 54.3 rpm and bits drum hardness up to 68.4 Mpa is to secure high rate of face advance.[15,16]
- 4- Small thickness about 30cm from phosphate ore can be left in the roof during working to ensure roof stability during face advance.

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## حسابات الأحمال واختيار نوع الدعامات المتحركة بناءً على تصنيفات الكتل الصخرية وبعض المعادلات الأخرى الملائمة لظروف التشغيل في مناجم فوسفات أبوظرطور

### الملخص العربي

تعتمد العمليات الآمنة في واجهة الحش على نوع وقدرة الدعامات المتحركة في مقاومة الحمل الواقع عليها من صخور سقف المنجم، وفي مناجم فوسفات أبوظرطور تم تجريب نوعين من الدعامات وتم تطبيق النوع ثنائي الأرجل، وتتلخص المشكلة الرئيسية في مناجم فوسفات أبوظرطور في التساقط المتكرر لصخور السقف أثناء عملية الحش مما يؤدي الي تعطيل الإنتاج بالمنجم ، وتساقط السقف يعتمد أساسا على اختيار الدعامات المناسبة لتحمل الضغوط العالية أثناء فترة الحش وقد تم في هذا البحث استخدام تصنيفات الكتل الصخرية وغيرها من الطرق لحساب الأحمال المتوقعة علي الدعامات أثناء فترة الاستخراج، وقد تم اختيار القيمة المتوسطة للحمل من عدد ست قيم محسوبة للحمل مع استبعاد عدد خمس قيم محسوبة لهذا الحمل نظرا لاختلافها الواضح عن القيم المأخوذة في الاعتبار وقد وجد أن متوسط الحمل علي الدعامات هو 416 طن/متر وقد تم حساب الخضوع علي الدعامات بالمقدار 520 طن/متر. وفي النهاية تم عمل التوصيات و الاستنتاجات التالية وهي: زيادة سرعة تقدم الواجهة أثناء الحش وتشغيل المنجم لعدد وريدين وأن تقلل فترات توقف المنجم مع الأخذ في الاعتبار ترك سمك من الخام الفوسفات مقداره 30 سنتيمتر في السقف لزيادة صلابته مع التوصية باستخدام هذه المعدات:

1- ماكينة الحش من الطراز (Cat EL 3000/2011)

2- نوع التدعيم (Shield support model Kottadih, CDFI, France, 2x470 two-leg)