ZESZYTY NAUKOWE POLITECHNIKI ŚLĄSKIEJ 1990

Seria: GÓRNICTWO z. 175 Nr kol. 986

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STUDY OF THE FREEZING PRESSURE ACTING ON A SHAFT LINING

Summary. Carboniferous layer in Panji mining region is almost entirely covered with aquiferous overlays of different thickness. Overall thickness of the soil overlays amounts to over 200 m. For shaft sinking through this overlay the freezing method is used. In Panji shafts the reinforced concrete linings are used.

It has been found that the freezing pressure increases as the unevenly distributed soil depth does.

The freezing pressure resulted from the frozen wall's deformation may cause cracking of the reinforced concrete lining.

The freezing pressure has been measured in several shafts. It has been observed and described and a calculation method for the analysis of the shaft lining stresses under uneven freezing pressure has been suggested.

The carboniferous layer in Panji mining district is almost completely covered by waterbearing overburdens of varying thickness. The total thickness of soil overburdens is more than 200 m.

For the sinking of shafts through this overburden the freezing method is employed. The reinforced concrete lining has been applied in Panji shafts.

It is found that freezing pressure augments with the increase of depth of the soil, the distribution of which being uneven. The freezing pressure resulting from the deformation of the freezing wall is likely to give rise to the breaking of the reinforced concrete lining.

The freezing pressure is measured is several shafts. The behaviour of the freezing pressure is described and calculation method is proposed for stress analysis of the shaft lining under non-uniform freezing pressure.

Freezing Pressure in General

A freezing wall, cylindrical in shape, is artificially formed by freezing and frozen soil prior to the shaft sinking operation. Sinking process may be carried out safely within the wall without danger of influx of underground water. The freezing wall resists both water pressure and ground pressure applied by the water-bearing soil outside, ensures a temporary stabilization of the shaft site during the sinking operation and prevents the freezing tubes from being horizontally displaced too much from their initial positions.

The freezing pressure is the pressure of the freezing wall acting upon the shaft. How does this freezing pressure come about? About the cause of the freezing pressure there raged for many years a great controversy. However, it has generally been accepted that a relatively practical approach to this question would seem to be:

1. Prior to the formation of the freezing wall, the soil has already been under some kind of stress, the initial stress. After the freezing wall has been formed, an additional sterss, the temperature stress, results under the action of the temperature field.

From this is follows that the stress on every point is an overlap of these two stresses. In other words, prior to the excavation of the shaft, the freezing wall has accomulated a great amount of energy; and as soon as sinking operation begins, this energy tends to release, thereby resulting in a pressure acting on the shaft - the freezing pressure.

Considering that the initial stress is related to both the depth and the physico-mechanical behaviour of the soil layers, and that the temperature stress is related to temperature, it is inferred that the freezing pressure is not only a function of temperature but also a function of both the depth and the physico-mechanical behaviours of the soil. Accordingly, the actual freezing pressure varies with the depth of the soil.

Let P_f stand for the freezing pressure, P_s for the initial stress of the soil, and P_t for the temperature stress, we have

$$P_s = \gamma HK \tag{1}$$

where γ represents the average volume of the overlying soil; H the depth of the soil; and K the horizontal thrust coefficient of the soil.

$$P_t = \mu t_o \tag{2}$$

where to represents the average temperature of the frozen soil; and the temperature strese coefficient which means an increment of the frozen soil stress for every 1^oC increase of the average temperature of the soil. According to our definition, therefore, we obtain

$$P_f = P_s + P_t \tag{3}$$

or

$$P_f = \gamma HK + \mu t_o \tag{4}$$

For sandy loam, we may put K = 0.4-0.6; for sand layer, K = 0.4-0.6; for sandy clay, K = 0.4-0.5; for sandy gravel, K = 0.4-0.6. The temperature stress coefficient may be determined in this way: for sandy loam, = 0.2-0.4; for sand layer, = 0.1-0.2; for sandy clay, = 0.4-0.5; for clay layer, = 0.4-0.6; for sandy gravel $\mu = 0$.

Not only can the formulae listed above be used to explain the ground pressure found during the shaft sinking operation by the freezing process, but they can also be used to estimate the freezing pressure which will be of value for shafts to be designed and sunk by the freezing process.

The freezing wall may be viewed as an elastoplastic mass which is in a plastic state under the action of the ground pressure and the water pressure from the outside, and whose deformation, under the stress of the frozen soil, develops as time goes by. This has been sufficiently proved by physicomechanical tests made on the behaviours of the frozen soil. Figure 1 shows the creep curves of the frozen soil. Obvious creep deformation can be seen before lining operation; and, after lining has been completed, this kind of deformation is retarded, thereby a freezing pressure acting on the lining occurs. This explanation for the freezing pressure is convincingly practical.

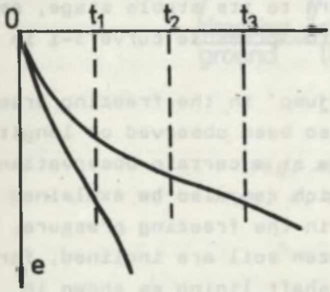

Fig. 1. Rheological curves for frozen soils

Rys. 1. Krzywe reologiczne dla gruntów zamrożonych

It would be lopsided regard the freezing pressure merely as a frost-heaving force. Nor would it be all-sided to think of the frost-heaving force resulting from refreezing (i.e., freezing again after thawing) as primary and decisive.

A Preliminary Analysis on the Anomaly in Freezing Pressure

Normally, freezing pressure has been found to behave like this: beginning with a sharp rise, followed by a stable stage, and ending with or without fall as temperature increases. Sometimes, however, during the transition from a sharp rise to the stable stage, as well as in the stable stage, there has been observed an abrupt rise followed by a fall in readings shown on some or all of the pressure gauges embedded at a certain observation level. This 'jump' has been directly related to change in soil state, upon which we speculate as follows:

1. The frozen soil about the shaft site wall, under the action of freezing wall's weight, the water and ground pressure from the outside, and the temperature stress, tends to transit to a plastic state. Spiral-shaped fissures form between the shaft lining and the freezing circle

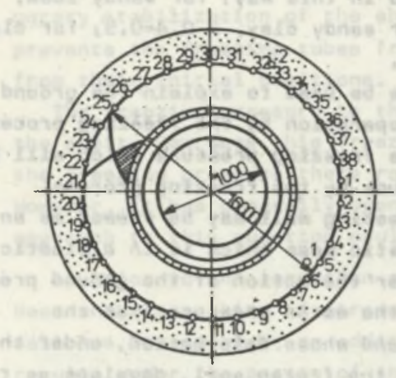

Fig. 2. Freezing ground failure around a shaft

Rys. 2. Pęknięcie zamrożonego gruntu wokół obudowy

when these fissures cross each other and from cone-shaped slides as shown in fig. 2, a 'jump' in freezing pressure in a certain locality will result. After these cone-shaped slides have been out of balance, the energy within the frozen soil is capable of selfad justing back to its orginal value, that is, freezing pressure will return to its stable stage, as shown in the pressure curve 3-1 in fig. 3.

2. A 'jump' in the freezing pressure has also been observed on longitudinal view at a certain observation level, which can also be explained as a 'jump' in the freezing pressure. In this case, however, the fissure in the frozen soil are inclined, forming a ring of conic cross-section round the shaft lining as shown in fig. 4. If, as is often the case, the ring moves inwards, a 'jump' in the freezing pressure will result on all the pressure gauges at a certain observation level, as can be seen from the abrupt rise and fall in the three pressure curves shown in fig. 5.

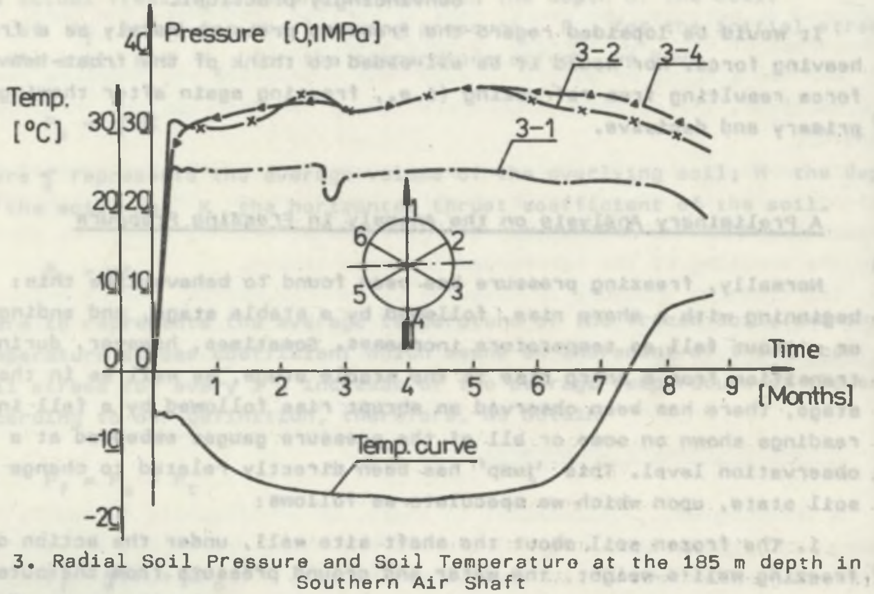

Fig. 3. Radial Soil Pressure and Soil Temperature at the 185 m depth in Southern Air Shaft

Rys. 3. Ciśnienie promieniowe w gruncie i temperatura gruntu na głębokości 185 m w południowym szybie wentylacyjnym

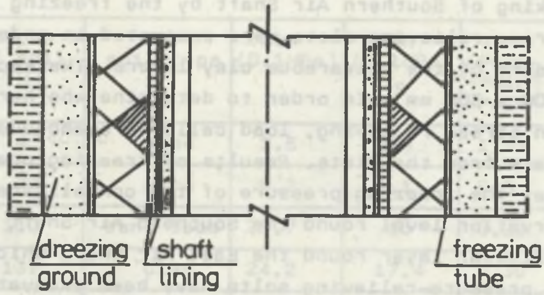

Fig. 4. Freezing ground failure

Rys. 4. Pęknięcie zamrożonego gruntu

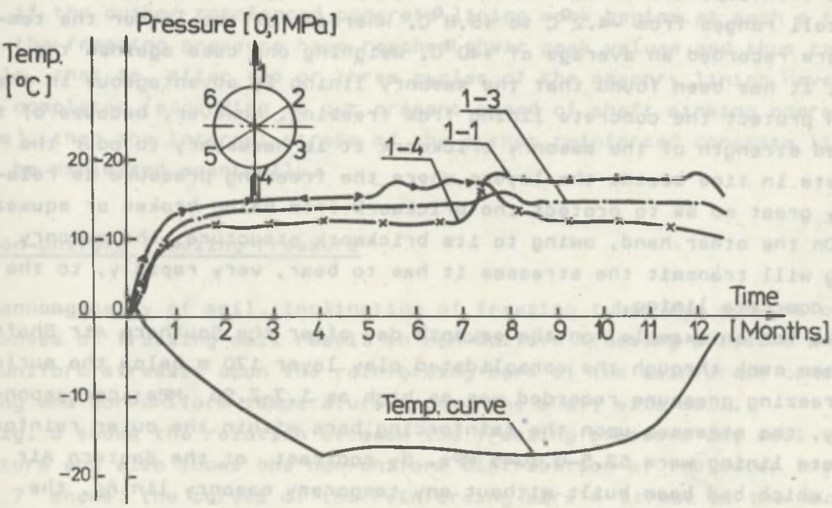

Fig. 5. Relation between Freezing Pressure and Ground Temperature at a Depth of 105 m, Southern Air Shaft

Rys. 5. Zależność pomiędzy ciśnieniem mrożenia oraz temperaturą gruntu na głębokości 105 m, południowy szyb wentylacyjny

The Effect of Stress-Relieving Slots

During the sinking of Southern Air Shaft by the freezing process, as many as twelve stress relieving slots were excavated in order to decrease the freezing pressure of the calcareous clay layers. The sectional area of the slots is 200 x 300 mm^2. In order to determine whether these solts can play a role in stress relieving, load cells were embedded at a distance of 30 centimeters from the slots. Results of freezing pressure measurements showed that the freezing pressure of the consolidated clay layer at the 130 m observation level round the Southern Air Shaft was not lower than that of the similar layer round the East Air Shaft which is 882 m away but where no pressure-relieving solts have been excavated. It is therefore doubtful whether such pressure-relieving solts will work.

The Function of Temporary Masonry Lining

Field measurements hove shown that the temperature of the temporary masonry lining is much lower that that of the concrete lining. For the former the temperature recorde in the initial stage after embedding the load cell ranges from -4.2^{o}C to +6,8^{o}C. Whereas for the latter the temperature recorded an average of +40^{o}C. Weighing one case against the other, it has been found that the masonry lining is advantageous in that it can protect the concrete lining from freezing. However, because of the limited strength of the masonry brickwork it is necessary to pour the concrete in time beside the layers where the freezing pressure is relatively great so as to protect the brickwork from being broken or squeezed out. On the other hand, owing to its brickwork structure, the masonry lining will transmit the stresses it has to bear, very rapidly, to the outer concrete lining.

Take for an example, on the seventh day after the Southern Air Shaft had been sunk through the consolidated clay layer 170 m below the surface, the freezing pressure recorded was as high as 1.7-2.96 MPa: correspondingly, the stresses upon the reinforcing bars within the outer rainforced concrete lining were 52.5-0.1045 MPa. By contrast, at the Eastern Air Shaft which had been built without any temporary masonry lining, the stresses upon the reinforcing bars within the outer reinforced concrete lining were extensively found to be as high as 1.000-1.600 MPa, other things being equal. This testifies to the fact that the masonry lining plays some role in reducing the internal stress of the outer reinforced concrete lining.

Notwithstanding, there arises another problem that the increase in the strength of concrete with the pasage of time is still lagging behind the rapid rise of the freezing pressure.

Table 1

Results of in situ measurement at Eastern Air Shaft

Observation level's NO	Depth of soil (M)	soil type	Pmax (0.1MPa)	Pmin (0.1MPa)	ω (Pmax-Pmin) Pmin	Months after lining operation
1	40	sand	1.5	0.4	1.75	10
2	70	sand	9.2	6.1	0.5	17
3	110	sand loam	20.7	16	0.29	8
4	137	clay	24.2	17.4	0.36	3
5	170	sand loam	28,8	22.1	0.3	2
6	220	sand	-	-	-	-
7	278	gravel	32.0	19.7	0.6	6

In order to reduce the internal stress of the outher reinforced concrete lining, it is necessary to lengthen properly the time interval between masonry lining operation and concrete lining operation. To be more specific, if the outher reinforced concrete lining work begins at such a time when the freezing pressure have reached their peak values and thus remain stable, that is, after two or three cycles of the masonry lining have been completed (according to our present speed of shaft sinking operations), then the internal stress of the outher reinforced concrete lining will be minimized eventually.

Non-Uniform Freezing Pressure

Nonhomgeneity of soil, inclination of freezing tubes and non-uniform thickness of freezing wall result in non-uniform freezing pressure and non-uniform stresses upon the reinforcing bars of the reinforced concrete lining and non-uniform temperature around the shaft site wall.

Fig. 6 shows the relation between the freezing pressure and soil temperature and also shows the non-uniform distribution of the outer load. Fig. 7 shows the curves of the reinforcing bars's stress at the same observation level.

Let P max stand for the maximum freezing pressure at one observation level; P min for the minimum freezing pressure at the same level, for the non-uniformity ratio of external pressure; and ω = (Pmax-Pmin)Pmin. Table 1 shows certain results of in situ measurement and non-uniformity ratio of freezing pressure at the Eastern Air Shaft.

It is found that non-uniformity ratio varies with the type of soil. Freezing pressure at sand or gravel layers is more non-uniform than that clay and sand loam layers.

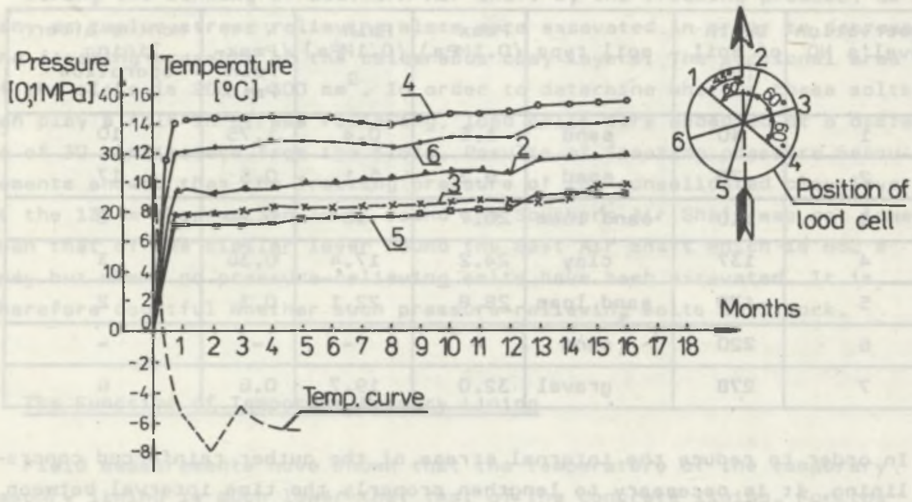

Fig. 6. Relation between soil pressure and soil temperature
Rys. 6. Zależność między ciśnieniem w gruncie a temperaturą gruntu

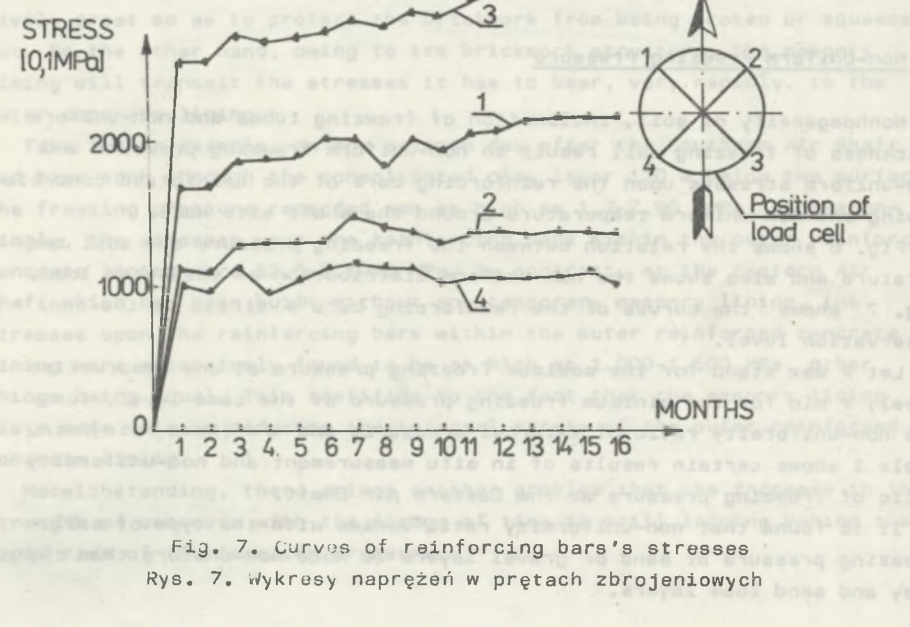

Fig. 7. Curves of reinforcing bars's stresses
Rys. 7. Wykresy naprężeń w prętach zbrojeniowych

Stress Analysis of Shaft Lining under the Action of Non-uniform External Ground Pressure

The distribution of non-uniform ground pressure around a shaft is most frequently specified in the form as shown in fig. 8. Ground pressure is broken down into two components: a uniform pressure over the whole lining and a non-uniform component which varies from zero to a maximum in each quadrant.

The ground pressure P at any point on the lining can be determined as follows:

$$P = P_o + \frac{P_1}{2}(1 + \cos 2\Theta) \tag{3}$$

where:

P - external pressure acting at any point on the lining,

P_o - uniform external pressure component,

P_1 - non-uniform external pressure component,

Θ - angle measured from major pressure axes.

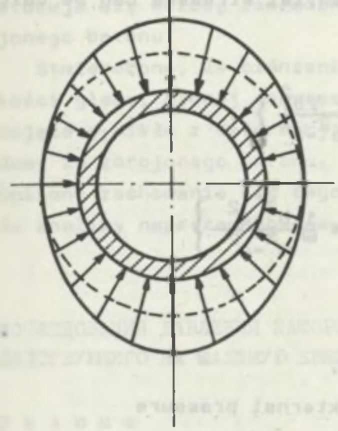

Fig. 8. Unequal load on shaft lining

Rys. 8. Nierównomierne obciążenie obudowy szybu

The stresses at any point on the circular lining is determined by formule based on elastic theory.

$$\sigma_r = \frac{P}{2}\,\frac{b^2}{b^2-a^2}\left[1-a^2r^{-2} + \frac{1}{(b^2-a^2)^2}\left\{b^4+b^2a^2+2a^4-2a^2(2b^4+b^2a^2+a^4)r^{-2} + b^2a^4(3b^2+a^2)r^{-4}\right\}\cos 2\Theta\right] + P_a\,\frac{b^2}{b^2-a^2}(1-a^2r^{-2})$$

$$\sigma = \frac{P_1}{2}\,\frac{b^2}{b^2-a^2}\left[1+a^2r^{-2} + \frac{1}{(b^2-a^2)^2}\left\{2(b^2+3a^2)r^2-(b^4+b^2a^2+2a^4) - b^2a^4(3b^2+a^2)r^{-4}\right\}\cos 2\Theta\right] + P_o\,\frac{b^2}{b^2-a^2}(1+a^2r^{-2})$$

$$\tau_r = \frac{P_1}{2}\,\frac{b^2}{(b^2-a^2)^3}\left\{(b^2+3a^2)r^2-(b^4+b^2a^2+2a^4)-a^2(2b^4+b^2a^2+a^4)r^{-2} + b^2a^4(3b^2+a^2)r^{-4}\sin 2\Theta\right\}$$

Let = P_1/P_o, where denotes non-uniformity ratio of ground pressure distribution, and let r = a, r = b, the tangential stresses can be calculated as follows:

$$\underset{v=a}{\sigma_\Theta} = P_1\left\{\frac{b^2}{b^2-a^2} - \frac{2b^2(b^2+a^2)}{(b^2-a^2)^2}\cos 2\Theta + \frac{1}{\omega}\,\frac{2b^2}{b^2-a^2}\right\}$$

$$\underset{v=b}{\sigma_\Theta} = P_1\left\{\frac{b^2+a^2}{2(b^2-a^2)} + \frac{b^4+6a^2b^2+a^4}{2(b^2-a^2)^2}\cos 2\Theta + \frac{1}{\omega}\,\frac{b^2+a^2}{b^2-a^2}\right\}$$

vhere:

P_1 denotes non-uniform external pressure,

= P_1/P_o denotes non-uniformity ratio of external pressure

a denotes internal radius of the lining,

b denotes external radius of the lining.

REFERENCES

[1] Tsytovich N.A.: The Mechanics of Frozen Ground, McGrow-Hill Book Company.

[2] Chou Wanxi: Raport on in Situ Measurment of Ground Pressure at the Southern Air Shaft of No. 2 Panji Colliery, Proceeding of Huainan Mining College, No. 1, 1918.

[3] Chou et ol. Engineering Handbook of Mining Construction, Chinese Coal Mining Company, 1980.

Recenzent: Prof. dr hab. inż. Mirosław Chudek

Wpłynęło do Redakcji w czerwcu 1988 r.

STUDIUM CIŚNIENIA ZAMRAŻAJĄCEGO DZIAŁAJĄCEGO NA OBUDOWY SZYBU

S t r e s z c z e n i e

Warstwa karbońska w okręgu górniczym Panji jest niemal całkowicie pokryta wodonośnymi nadkładami o różnej grubości. Całkowita grubość nakładów gleby wynosi ponad 200 m. Do głębienia szybów poprzez ten nadkład stosuje się metodę zamrażania. W szybach Panji stosuje się obudowę z zbrojonego betonu.

Stwierdzono, że ciśnienie zamrażające zwiększa się ze wzrostem głębokości gleby, której rozmieszczenie jest nierównomierne. Ciśnienie zamrażające wynikłe z deformacji ściany zamrożonej może powodować pękanie obudowy ze zbrojonego betonu. Ciśnienie zamrażania mierzono w kilku szybach. Opisano zachowanie się tego ciśnienia i zaproponowano metodę obliczeniową do analizy naprężeń obudowy szybu pod niejednolitym ciśnieniem zamrażania.

ИССЛЕДОВАНИЯ ДАВЛЕНИЯ ЗАМОРАЖИВАНИЯ, ДЕЙСТВУЮЩЕГО НА ШАХТНУЮ КРЕПЬ

Р е з ю м е

Карбонский слой в горном районе Панйи почти целиком покрыт водоносными наносами различной толщины. Общая толщина наносов почвы составляет более 200 метров. Для проходки стволов в этих наносах применяется метод замораживания. В шахтах Панйи применяются крепи из железобетона.

Подтверждено, что давление замораживания увеличивается с возрастанием глубины почвы, размещение которой неравномерно. Давление замораживания, являющееся результатом деформации замороженной стены, может быть причиной растрескивания крепи из железобетона.

Давление замораживания измерялось в нескольких стволах. Описано поведение этого давления и предложен метод расчётов для анализа напряжения в шахтной крепи при неравномерном давлении замораживания.