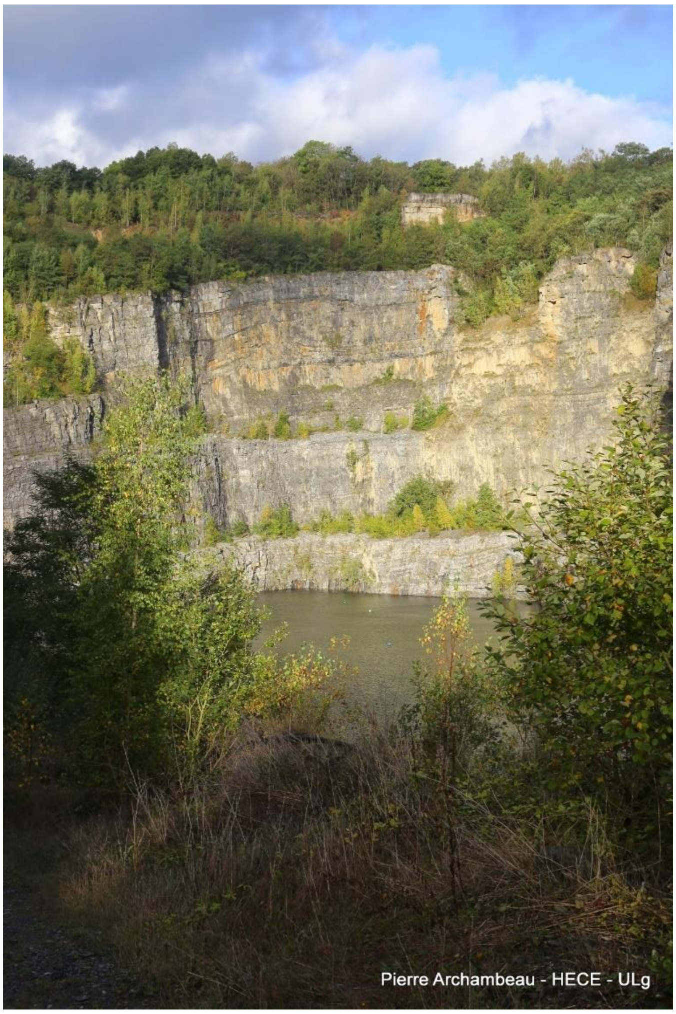
# Guidelines related to the use of an existing cavity (mine/quarry) as reservoir of a pumped storage hydroelectric facility

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# 1. Introduction

Underground mines/quarries or open pits are manmade earthworks resulting from previous extraction of natural resources. They were mostly abandoned after resources depletion but underground or surface excavations remain.

Pumped Storage Hydroelectricity (PSH) is one of the only efficient solutions for large scale energy storage. During peaks of energy production, water is pumped from a lower reservoir to an upper reservoir. On the other hand when demand peaks, energy is generated when water is transferred to the lower reservoir.

The objective of this work is to study the recovering of abandoned excavation volumes as lower reservoirs for pumped storage hydroelectricity, as depicted in Figure 1. Underground Pumped Storage Hydroelectricity (UPSH) is a particular case where at least one of the reservoir lies below the surface.

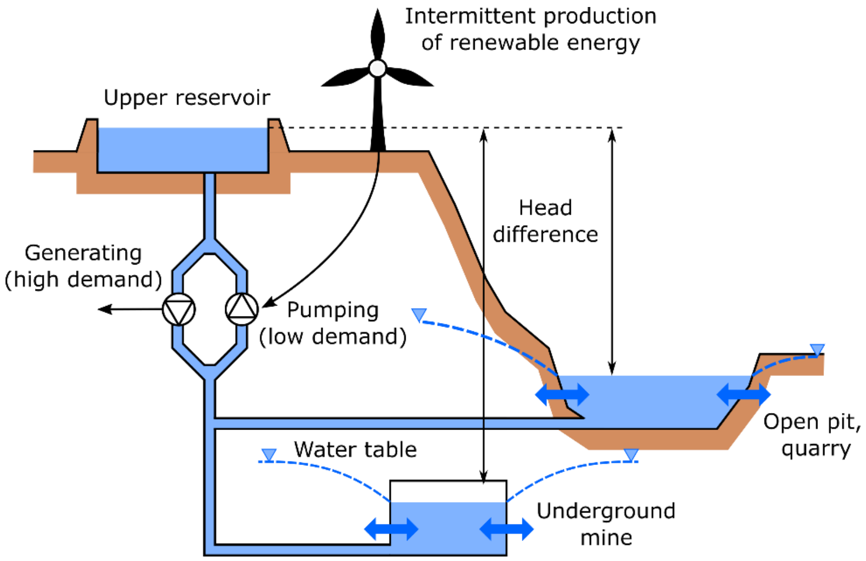


**Figure 1 Sketch of pumped storage hydroelectricity installations considered in this study**

Compare to classical PSH plants, using an existing cavity as a reservoir raises three main additional problems:

1) What are the water movements and thus discharge availability at the pump/turbine location depending on the reservoir (complex) geometry?
2) What are the exchanges between the reservoir (usually not watertight) and the surrounding medium?
3) How can the reservoir sides resist to cyclic loading imposed by the plant operation?

In the following, some simplified criteria and issues are provided to carry out feasibility assessments of such rehabilitation projects, from the existing reservoir point of view. These criteria do no replace a detailed and complete study but draws the attention on problems specific to PSH using existing cavities as reservoir. They do not concern problems existing for every PSH project, such as location of electricity transport lines, availability of water…

First, existing cavities typical of Wallonia are classified into six general categories (T1-T6). Then, for these categories, issues preventing or resulting from PSH operations are identified. The list included in this document is not exhaustive. These issues are studied from the geological (G), hydraulic (H), hydrogeological (HG) and geomechanical (GM) points of view.

Even if this document help in identifying some of the problems possibly impacting the operation of a PSH designed with an existing cavity used as a reservoir, it does not prevent from carrying on carefully detailed analysis of the project. Such analysis should be done using in particular numerical modelling in order to predict all the consequences of the plant operation on the cavity long term stability, the interaction with the surrounding porous medium and the fluids movement in the system. However, analytical solutions as the ones presented in this document may be of great interest during initial phases of future projects. These solutions, which allow computing some relevant aspects of the problem, can be used for screening purposes.

# 2. Context and data

## Typology of reservoirs

Potential reservoirs identified in Wallonia are separated into two main groups: underground and surface reservoirs as reported in Table 1. The main advantages of the first category reservoirs are: a high head difference and a low footprint. However most of these cavities are partially collapsed, under water and simply almost impossible to visit. On the contrary surface reservoirs allow a simpler characterisation of their geometries and general state but the head difference is limited and the footprint is more important.

Underground and surface reservoirs are classified in three sub-categories mainly based on reservoir geometry. Each category gathers all examples sharing similar characteristics that could be studied by an integrated methodology covering geological, hydraulic, hydrogeological and geomechanical aspects.

| Underground | | | Surface | | |
|---|---|---|---|---|---|
| T1 | T2 | T3 | T4 | T5 | T6 |
| Rooms and pillars | Galleries | Mixed | Soil embankment | Rock wall | River |
| Ex. Slate quarry | Ex. Metallic Mine | Ex. Coal Mine | Ex. Artificial embankment | Ex. Chalk/limestone quarry | |

**Table 1 Typology of reservoirs and examples**

## Existing reservoir description

The first step of the feasibility analysis will be to describe thoroughly the reservoirs by collecting data about the mine or underground quarry. Mine or quarry operators should have all the needed data.

In the case of abandoned mines or quarries, this kind of data should be available in administration and municipality services or at state archives. The type of available data depends on the legal obligations imposed during the exploitation period. The method to process the data will depend on the available data type and format. For a regional screening of interesting sites, an important amount of paper-based ancient maps is difficult to process in a short time period. But for site specific study, data extracted from this ancient paper-based maps can be digitalized and georeferenced to analyze the location and environmental features of the study site.

*Ex: In Belgium for instance, reproductions of exploitation maps were delivered to the competent authorities and are still searchable at the State Archives. In that case, data about the location and distribution of the mine workings is available but shapes/volumes/sections of the different parts of the mine are not indicated. This type of data should still be used carefully. Indeed, georeferencing and reproduction errors are frequent on ancient exploitation maps since they were drawn over several years while reference systems changed. Moreover, preserved maps are seldom original maps but reproductions. One way to reduce this type of uncertainty is to localize remaining mine workings at the surface (e.g. shafts, drainage addits) on the field.*

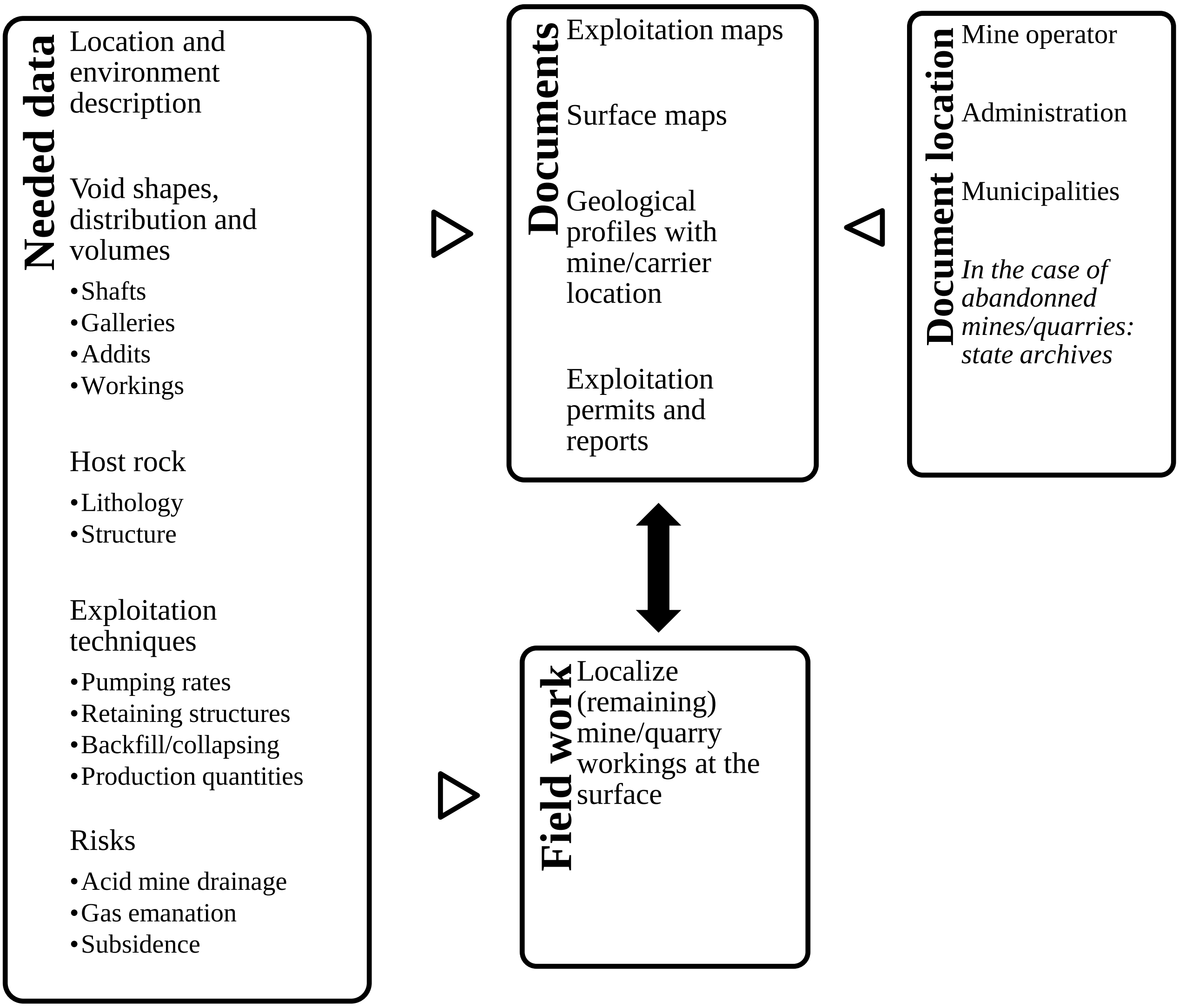


**Figure 2 data collection in abandoned mines/underground quarries**

Digitalized and georeferenced data from exploitation maps and profiles can be used to produce a 3D visualization or model of the mine/quarry but this is a time consuming task. Furthermore, volumes estimated with such models can largely overestimate existing residual volumes since remaining voids collapsed or were backfilled at the end of exploitation. Those rubble zones correspond to highly permeable areas where storage of water is still possible. Therefore, remaining volumes can be roughly estimated by different methods.

In the first method residual volumes $V_{res}$ [$m^3$] are considered to be proportional to extracted quantities *Ext* [t], a fixed bulk density $\rho$ [$t/m^3$] (ex: 1.25-1.7 $kg/dm^3$ for coal), and an estimated residual porosity $\phi_{res}$ [-] (ex. 0.2 in Van Tongeren and Dreesen, 2004)(Eq 1):

$$V_{res} = \frac{Ext * \rho}{\phi_{res}} \qquad \text{Equation 1}$$

The second proposed method requires more data (Eq. 2; in Thoraval, 1998). The extracted volume ($V_{ext}$; Eq. 3) can still be estimated by the extracted tonnage (*Ext*) and the density of the extracted rock ($\rho$). Backfill volumes ($V_{fill}$; Eq. 4) are estimated based on the wall volumes ($V_{wall}$) and a subsidence factor for backfill ($f_1$; 0.6-0.7 for horizontal walls; 0.45-0.5 for walls inclined with 35°). Subsidence volume can be estimated by comparing recent topographical maps with topographical maps produced before the underground mining. The comparison of both maps will help to determine subsidence surfaces and depths to calculate subsidence volumes.

$$V_{res} = V_{ext} - V_{fill} - V_{subs} \qquad \text{Equation 2}$$

$$V_{ext} = Ext * \rho \qquad \text{Equation 3}$$

$$V_{fill} = (1 - f_1) * V_{wall} \qquad \text{Equation 4}$$

The choice of the method should be based on the available data.

*Ex: In Belgium, extracted quantities of each mining headquarter were communicated to the competent authorities since 1897 and are easily available in archives but no data concerning backfill is available. In such a case, only the first method can be applied.*

## Estimation of energy storage potential

The power *P* [MW] of a PSH can be evaluated based on the mean discharge *Q* [m³/s] and the mean available chute *H* [m]:

$$P = \rho g Q H / 10^6 \qquad \text{Equation 5}$$

with $\rho$ the water volumetric mass density [1000 kg/m³] and *g* is the gravity acceleration [9.81 m/s²].

Considering the above relation, the energy storage potential *E* [MWh] is linked to the total volume of the reservoirs *V* [m³]:

$$E = P * Time = \rho g V H / 10^6 / 3600 \qquad \text{Equation 6}$$

As a first approximation, the energy storage is not dependent of the instantaneously discharge. The effective power and energy depend on the installation efficiency $\eta$, which includes losses in the electric machines, the pipes…

$$E_{eff} = \eta * P * Time = \eta \rho g V H / 10^6 / 3600 \qquad \text{Equation 7}$$

Classical efficiency per cycle of energy storage in a PSH (storage then production) is 75%.

Due to the large variations of the water level in some configurations, the mean chute must be evaluated carefully to avoid overestimation.

## Geological context

As the lower reservoir of the PSH system is delimited by the host rock of the quarry/mine, the most representative lithology of the host rock needs to be characterized. Following characteristics of the rock needs to be evaluated:

- Competence
- Porosity (presence of open fractures, karsts,…)
- Structure: faults, folds, strike and dip of the geological layers

- Mineralogy
- Seismic risks (consider Eurocode 8)

## Hydrogeological context

During the whole life of the PSH installations water stored within the reservoir interact with the aquifer. It is essential to characterise the leakage flow rate and the zone of influence. Therefore it is necessary to assess:

- The water table level in the quarry/mine in natural conditions and the possible natural variations across the years
- The most representative rock type surrounding the quarry/mine and the inherent characteristics (porous, fractured, karstified, mixed)
- The mean hydraulic parameters values of the rock (hydraulic conductivity, specific yield)
- Existing pumping area or groundwater dependent ecosystems
- The geometry of the cavity

## Hydraulic context

Existing cavities are rarely large continuous volumes. Following characteristics needs to be evaluated to assess the water movements during PSH operation:

- 3D geometry and distribution of the available volume, including details of connections between adjacent large cavities
- Roughness of solid boundaries
- Presence of aeration systems
- Expected position of the pump/turbine unit

## Geomechanical hazard

Whatever the type of reservoir, stability of cavities and galleries must be ensured to preserve the total volume and free water flow. It is necessary to identify:

- Any existing slope instability along the quarry walls
- Possible ground surface instability around the quarry, including weathered rock area or karstified limestone
- Drainage, collapsed, load losses, strength
- Collapsed galleries or weakened pillars/walls

## Chemical and biological water quality

As natural stream flow is disturbed by the PSH system using existing a non-watertight cavity as reservoir, water quality changes because of the plant operation. The quality of water will depend on the source of water used and on the potential contacts of the water along its trajectory (with other water masses, air, different lithologies…). Water coming from coal and metal mines can for example be acidic and/or highly mineralized. Therefore, it is important to evaluate:

- Quality of potential water sources for the PSH system

- Quality of water masses in contact with the system (aquifer, rivers)
- Impact of changing turbidity and oxido-reduction conditions on the water quality

### Gas

Underground mines and quarries (T1-3) progressively accumulate gas desorbed from the host rock. Different issues can occur with the presence of gas like the loss of storage volume in the reservoir due to the presence of gas and the release of toxic, inflammable and/or greenhouse gas in the environment. Therefore, it is important to

- Check for the presence of gas
- Analyse gas composition, determine its compressibility and solubility
- Determine preferential flow paths for gas and specific conductance of the covering layers

# 3. Issues resulting from the use of an existing cavity as a PSH plant reservoir

## Leakage/inflow rate (T1-5)

### Description

Groundwater – cavity/quarry interactions could affect significantly the efficiency of the UPSH by reducing the expected water level fluctuations in the lower reservoir. Depending on the instantaneous water level in the lower reservoir, compared to the natural level, this effect could either increase or decrease the difference of elevation between the upper and lower reservoirs, compared to an equivalent impermeable case. Therefore, the instantaneous energy production (in turbine mode) and consumption (in pumping mode) are also potentially affected. This effect must be evaluated carefully to allow a correct design of the PSH plant.

As an example, Figure 1 displays the evolution of the water level along time in a hypothetic lower reservoir. The fluctuations are produced by PSH cycles. The water level in the lower reservoir oscillates around an average water level ($\bar{h}$) because of the pumpings/injections cycles. The amplitude of the fluctuations depends on the aquifer – cavity water exchanges. During early times, the average water level is lower than the initial level. It then tends to a dynamic steady-state corresponding to the initial water level in the lower reservoir. This figure was obtained from a synthetic operation scenario in which the water level in the lower reservoir is located at its natural depth before starting the activity of the PSH plant.

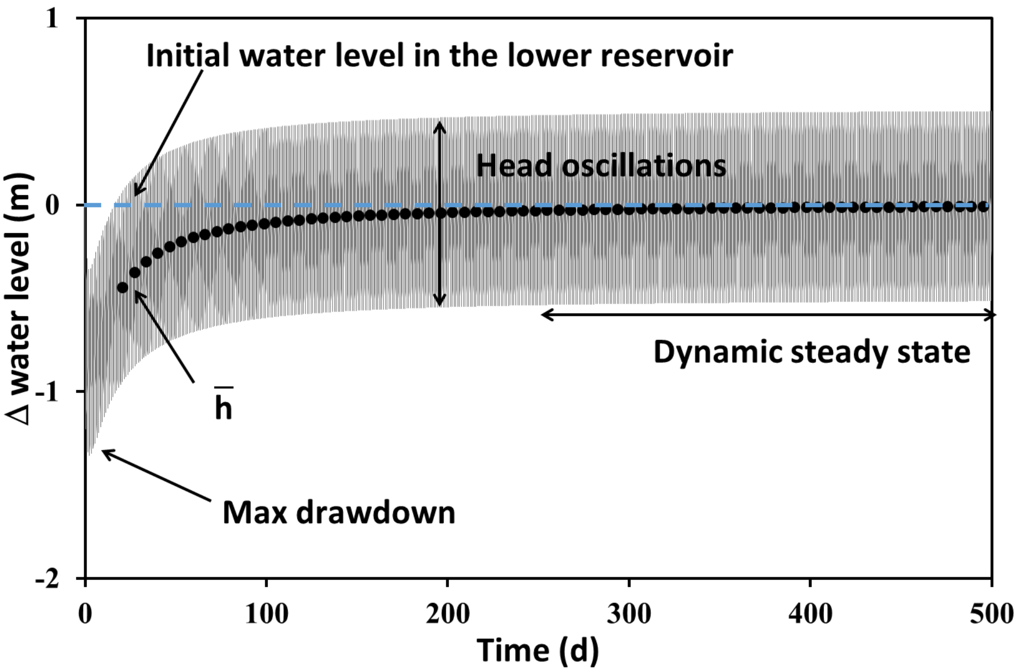


**Figure 1. Computed variation of the piezometric head in the surrounding porous medium for a synthetic scenario. It is considered that the water table is located at its natural depth before starting the activity of the UPSH plant. $\bar{h}$ is the mean water level during one cycle (modified from Pujades et al., 2016)**

### Experimental results/related parameters

Generally, groundwater fluxes exchanged between the cavity and the surrounding medium, and then the water level fluctuation, are function of the aquifer properties and the difference of water level between the cavity and the aquifer. More specifically, the water level fluctuations in the lower quarry/cavity during a pump-storage cycle depend on the hydraulic conductivity, the specific yield, the frequency of the pump-storage cycles, the pump-storage flow rate and the equivalent surface of the cavity. From a general perspective, the amplitude of the water level fluctuations in the lower reservoir decreases with increasing hydraulic conductivity and specific yield values. Classically, *in-situ* hydrogeological techniques such as pumping tests can be used to determine these parameters.

Numerical groundwater flow model can then be developed to better describe the groundwater – cavity/quarry. As an example, generic results are provided in Annex for an hypothetical lower reservoir located in an open pit quarry (from Poulain et al, 2018).

### Mitigation

Waterproofing of all the walls of the quarry or the mine is a very costly mitigation method. Concrete could be injected in localised areas of higher hydraulic conductivity such as open fractures to seal the fractures and limit the groundwater-cavity interactions.

## Influence zone (T1-5)

### Description

The induced water level fluctuation in the cavity/quarry propagates in the surrounding rock medium. Generally, the distance of propagation increases as the rock hydraulic conductivity is high and the specific yield is low. This distance of influence should be quantified to avoid interactions of the PHES station with problematic areas. As examples, any possible rock instability or pumping area in the area of influence have to be considered, and related risks must be assessed.

### Experimental results/related parameters

The distance of influence is higher when hydraulic conductivity increases and storage values decreases. Therefore, it would be important to determine these parameters before designing an UPSH plant. Classically, *in-situ* hydrogeological techniques such as pumping tests can be used to determine these parameters. Note that any rock heterogeneity such as discrete open fractures or conduits may significantly increase the distance of influence.

Analytical solutions may be of great interest during initial phases of future projects. These solutions, which allow computing some relevant aspects of the interaction, can be used with screening purposes. As an example, the analytical solutions presented in Section 4.1 could be used to compute as a first approximation, the drawdown around the cavity and also, the time needed to reach the dynamic steady state in a confined aquifer.

Numerical groundwater flow model can then also developed to better characterize the influence of the UPHS. As an example, generic results are provided in Annex for an hypothetical lower reservoir located in an open pit quarry (from Poulain et al, 2018).

### Mitigation

As the distance of propagation increases as the rock hydraulic conductivity is high, waterproofing of the walls could reduce distance of influence. As mentionned previously, waterproofing of all the walls of the quarry or the mine is however a very costly mitigation method. Concrete could be injected in localised areas of higher hydraulic conductivity such as open fractures to try to seal the fractured areas.

## Fatigue/Weathering of natural material (T1-5)

### Description

Transient variations (mostly cyclic) of the water level in underground or surface reservoirs leads to changes of the stress state within the rock material. The continuous change of principal stresses magnitude and orientation may lead to a premature failure of the earthwork. This fatigue of geomaterials involve many complex phenomena resulting in progressive degradation of the rock strength. Therefore the design load should not be obtained from monotonic tests but must be based on fatigue experiments.

Weathering of the rock material is mostly related to chemical weakening of the rock strength due to erosion or dissolution of the material. It mainly depends on the rock nature. However water movements in cracks and fractures may speed up chemical reactions.

Fatigue of geomaterials is most critical in underground cavities where variations of water level are higher. In addition the overburden increases with depth generating a more unfavourable initial stress state after excavation. Weathering may occur in both underground and surface reservoirs.

### Experimental results/related parameters

Fatigue experiments mostly consist in varying cyclically the load applied to a rock sample between a minimum and a maximum value. The number of cycles $N$ that could be applied to the sample before failure are reported for each constant amplitude ($\sigma_{max}/\sigma_{mon}$ where $\sigma_{max}$ is the maximum applied stress and $\sigma_{mon}$ is the monotonic strength). Summarising results for different amplitudes provides a S-N curve, relating the number of applied cycles of amplitude S. A fatigue strength is commonly identified when the number of cycles to failure becomes very high. This value mainly depends on the experimenter patience but is generally reported as 70% of the monotonic strength. Figure 3 reports several S-N curves based on different studies/materials/types of tests.

As a good practice rule, it is recommended to identify the number of cycles of the most damaging load variation over the operation life of the earthwork. This value corresponds to the minimum number of cycles that must be applied during fatigue tests. Uniaxial, triaxial or Brazilian tests can be carried out. However uniaxial tests are the most common and corresponds to the state of stresses around drifts.

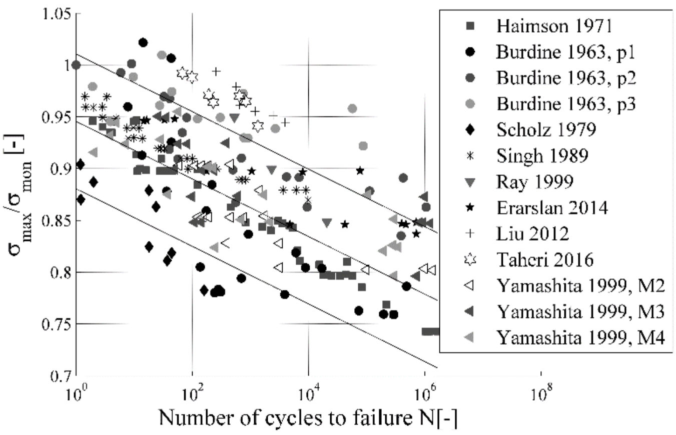


**Figure 3 S-N curve for different rock materials and types of tests at constant cyclic amplitude**

### Mitigation

Reinforcement of the whole reservoir is not an economical choice. It is recommended to limit the cyclic amplitude of variation and specially to avoid traction loading of the rock material. Critical zones where highest cyclic variations occur may be reinforced through appropriate counter-measures.

## Walls flexion failure (T1 & T3)

### Description

This type of failure mode is specific to the exploitation of quarries with rooms and pillars (underground slate quarries for instance). In such cases, different rooms are separated by a very long continuous pillar, namely a wall. Figure 4 presents the cross-section of a quarry where successive rooms are separated by such a wall. All the rooms are connected by limited section galleries through the walls.

If water is pumped into a single room, variations of the water level are not identical in every rooms, leading to different water pressures on the left and on the right of each pillar as shown in Figure 5. This results in additional bending of the wall.

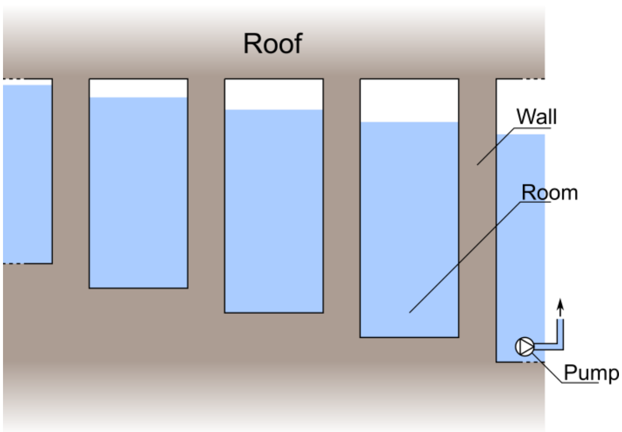


**Figure 4 Cross-section of walls and rooms**

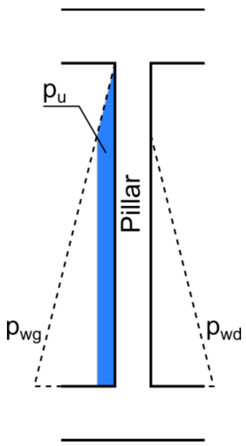


**Figure 5 Induced flexion within the wall**

### Experimental results/related parameters

A simple method based on a beam analogy is presented in Section 0. Distribution of normal stresses within the cross-sections of the pillars are estimated and compared to compression or traction strengths. Sufficient laboratory simple compression Brazilian or triaxial tests are required to estimate these resistances. Cohesion and friction angle of the material is another alternative.

Strength measured in laboratory is often quite different from strength of the rock mass due to the cracks and discontinuities. This scaling effect must be taken into account by an appropriate method. An estimation of the RMR (rock mass rating) is recommended.

### Mitigation

Reinforcement of the pillars is a very costly mitigation method. Reduction of the water level variations between the pillars is the simplest solution by:

- reducing head loss in inter-room connections;
- reducing flow rate.

## Backfill stability (T4)

### Description

Mining/quarry activities lead to the creation of backfills or tailings close to the extraction zone. They are not necessarily correctly compacted and may be composed of poor quality material. Variations of the water level in these backfills may lead to instabilities, due to decrease of the effective stress. In addition cyclic variations may degrade strength of the material and deformation of the backfill may accumulate cycle after cycle. Slope failure leads to a leakage of water out of the reservoir or damage to installations.

### Experimental results/related parameters

Friction angle, cohesion and compressibility of the material are relevant parameters. In situ tests such as CPT, SPT or pressiometer tests may be carried out to obtain strength parameters, depending on backfill granulometry. Laboratory tests such as oedometer, triaxial and permeability tests may provide additional information.

### Mitigation

In general it is recommended to avoid any variation of the water level within backfills or embankments. If one of the reservoir's wall is a soil embankment in contact with water, a sealing treatment (clay layer, asphalt overlay…) must be applied to limiter water flow. In addition safety against slope instability must be assessed through analytical/numerical simulations.

## Sediment transport (T 1-6)

### Description

Sediment accumulation can occur in the reservoirs for different reasons. If river water (T6) is used in the PSH system, natural sediment loads are transported. Heavier and coarse materials constitute bed loads that are transported on smaller distance by pump/turbine than fine materials suspended in the water. Similar processes can occur in abandoned quarries and mines by initiating fluxes, especially in the presence of waste embankments or loose materials (i.e. clay pits). Since the plant installation (artificial reservoir, transport network,…) can require soil stripping and compaction operations and/or embankment installations, soils surrounding the reservoirs become sensitive to erosion processes. Those eroded sediments can also be transported in the reservoir. The sediments transported cause abrasion of the turbines and storage volume reduction by sediment deposition in the reservoir.

### Experimental results/related parameters

Erosion sensitivity of an existing cavity will depend on various parameters such as cohesion and grain-size distribution of wall material, bottom slope, shear stress, flow depth and flow velocity.

### Mitigation

In a closed loop system such as most of PSH, the best way to avoid sediment transport is to remove the sediments from the reservoirs. If this is not possible, a dead volume should be considered in the reservoir (volume of water that is never removed from the reservoir during normal operation) in order to avoid the occurrence of flow conditions (velocity) prone to set sediment deposits into motion.

## Discharge availability (volume distribution) (T 1-6)

### Description

If the existing reservoir volume is not concentrated but distributed into several (inter)connected cavities or along a gallery network, the discharge availability at the pumping point and the discharge distribution from the injection point is possibly not controlled by the pump/turbine but by hydraulic capacity of the reservoir. Indeed, the reservoir geometry can limit the water movements and thus the instantaneous discharges at any location in the reservoir (Erpicum et al., 2017). In the specific T6 case where a river is used as a reservoir, environmental limitations may also exist during low flow period. A minimal discharge in the river must be preserved at any time and rapid flow variations have to be avoided or require adequate alert management.

To assess the discharge availability along time, it is necessary to develop a distributed hydraulic model of the reservoir and to solve the flow governing equations (mass and momentum conservation) at every point of the discretisation. Depending on the reservoir geometry, varied discretisation dimensions may be considered (lumped, 1D, 2D horizontal, 3D model).

In every case, inertia effects are usually not negligible. Special consideration has also to be paid to air movement in the case of underground reservoir, as emptying a cavity requires that air take the place of water. On the contrary, when filling an underground reservoir, air may limit the water volume if it is not able to escape.

### Experimental results/related parameters

Adequate geometry description of the reservoirs is necessary to perform any modelling of the reservoir. Even simplified analytical evaluation of the water movements, for instance considering local head losses and Bernoulli equation, requires a detailed knowledge the reservoir geometry.

### Mitigation

Limited geometry arrangement may help in improving significantly water movements in a reservoir (attenuation of local section reduction, bottom slope arrangement, local deepening of the reservoir close to the pump/turbine…). Drilling of new connections between adjacent cavities or aeration shaft may be necessary.

## Impact of water chemistry (T1-6)

### Description

The natural chemical composition of water in a reservoir varies with the composition of the surrounding lithologies and soil, the local fauna (and their uptake of nutrients), water movement (stagnant vs flowing water), external fluxes towards the reservoir (i.e. surface run-off, groundwater, river). As (U)PSH installation changes the natural flow regime, all other parameters will potentially be affected by this change.

In addition, the surface exposure of the pumped water will modify its chemical composition and will induce chemical reactions. In the same manner, reactions will occur when water stored in the upper reservoir is released into the underground one. This water will react with water filling the reservoir and with the surrounding medium. This may affect directly the resistance parameters of the rock material, for instance by dissolution.

Chemical reactions occurring in the upper and underground reservoirs will be relevant in terms of environmental impacts and efficiency (corrosions and incrustations may reduce the capacity and useful life of facilities). Reactive transport numerical models will be required to address this issue.

### Experimental results/related parameters

**Redox conditions** are affected by the contact of water with air. Pumping groundwater to the surface (T1-T3) will therefore generate oxidation processes in the empty cavity and in the water at the surface.

- Redox conditions will also impact the pH as lithologies containing metal sulphides (i.e. pyrite) produce acids once the sulphides are oxidised by oxygen. Precipitation of metal oxides (iron, lead, manganese, copper, silver, cadmium) will appear simultaneously. Typical reactions of pyrite oxidation producing acid mine drainage are described below:
- $2FeS_{2\,(s)} + 7O_2 + 2H_2O \leftrightarrow 2Fe^{2+} + 4SO_4{}^{2-} + 4H^+$

- $2Fe^{2+} + \frac{1}{2}O_2 + 2H^+ \leftrightarrow 2Fe^{3+} + H_2O$

  $Fe^{3+} + 3H_2O \leftrightarrow Fe(OH)_3 + 3H^+$

Acids produced in these reactions can also be buffered if the **alkalinity** of water is important. In that case, metal oxides co-precipitate with carbonates. The amount of precipitated metal oxides and the acidification after several pumping cycles depend the kinetics of the sulphides, the concentration of metals in the water and on the new inflow of fresh mine water in the reservoir (influence zone §3.2).

**Gas** present in the abandoned underground cavities (T1-T3) can dissolve in the water according to its solubility, which varies with its composition (i.e. higher for $CO_2$ than for $H_2S$), with temperature and pressure conditions. This process is described by Henry's law:

$$H = \frac{C_s}{p_i}$$

with $H$ as Henry's Law constant, $C_s$ as concentration of the solute at partial pressure $p_i$. Pumped water containing dissolved gas will release the gas at the surface if pressure decreases and temperature increases. Volumes of gas released at the surface vary with the water discharge and its gas concentration which depend on the quantity of gas present in the underground and the travel time of groundwater (i.e. on the influence zone of the UPHS §3.2).

### Mitigation

In case of acid mine water (T1-3), neutralization by lime should be included in the operation scheme of the PHS to neutralize acids and protect all pumps and installations from corrosion. If metal oxide precipitates are forming, equivalent solutions to sediment accumulations should be planned (i.e. flushing, dredging, …).

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# 5. Appendices

## Computation of normalized water table fluctuations in an open pit used as lower reservoir of a UPSH

As an illustration for a squared quarry, the figure below shows the simulated normalized water table fluctuations $\Delta h$ in an open pit, as a function of two non-dimensional parameters. These non-dimensional parameters are defined here below. Water table fluctuations are normalized by the expected fluctuations in an equivalent impermeable reservoir $\eta_c$ [m]. Values lower than 1 indicate than the fluctuations are attenuated compared to the expected fluctuations in an equivalent impermeable lower reservoir. **Erreur ! Source du renvoi introuvable.** enables to assess the amplitude of the water level fluctuations in the quarry, for any parameter values, controlling the non-dimensional parameters. Note that results of **Erreur ! Source du renvoi introuvable.** have been calculated considering a sinusoidal pump-storage flow rate and a generic quarry depth of 80 m (Poulain et al, 2018).

| | |
|---|---|
| $\overline{\Delta h} = \frac{\Delta h}{\eta_c}$ [-] : | Normalized water table fluctuations |
| $\eta_c = \frac{Q_{max}\,\tau}{S\pi}$ [m] : | Expected fluctuations in an equivalent impermeable reservoir |
| $\overline{D} = \frac{\tau K}{\eta_c S_y}$ [-] : | Non-dimensional diffusivity |
| $\overline{K} = \frac{K\tau\eta_c}{S}$ [-] : | Non-dimensional hydraulic conductivity |
| S [$m^2$] : | Surface of the open pit quarry |
| $\tau$ [s] : | Period of the PSH cycles |
| $Q_{max}$ [$m^3/s$] : | Maximum pumping/turbining flow rate of the PSH cycles |
| K [m/s] : | Hydraulic conductivity of the surrounding rock medium |
| $S_y$ [-] : | Specific yield of the surrounding rock medium |

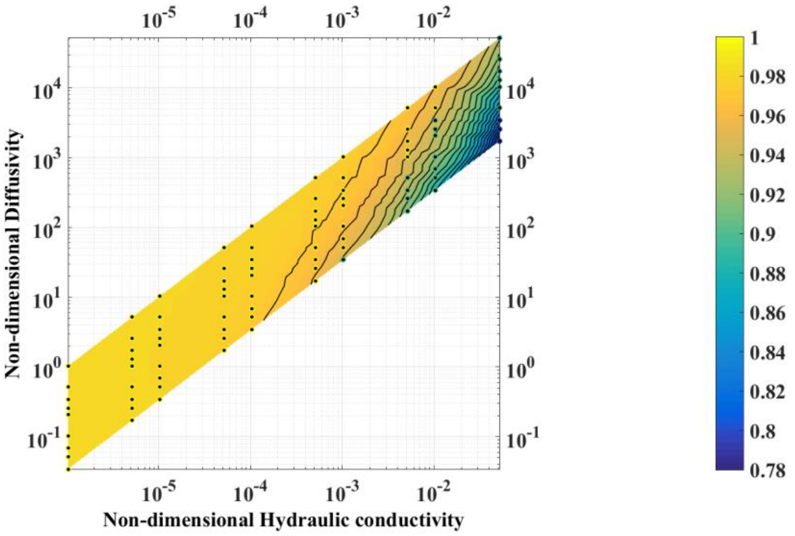


**Figure 6. 'Normalized amplitude' of the water level fluctuations in the quarry ($\overline{\Delta h_c}$), according to two non-dimensional parameters. Black dots correspond to couples of parameters values actually used for numerical simulation and interpolation (modified from Poulain et al, 2018).**

## Analytical solution to compute the drawdown around a UPHS and the time needed to reach a dynamic steady state in a confined aquifer

Analytical solutions (Pujades *et al*, 2016) were derived from the Papadopulos and Cooper (1967) and Boulton and Streltsova (1976) equations for large diameter wells. The Papadopulos-Cooper (1967) exact analytical solution allows computing drawdown (s) in a confined aquifer:

$$s = \frac{Q}{4\pi Kb} F\left(u, \alpha_{\mathrm{W}}, r_0/r_{\mathrm{ew}}\right) \tag{1}$$

where $b$ is the aquifer thickness [L], $Q$ is the pumping rate [L3T$^{-1}$], $r_{\mathrm{ew}}$ is the radius of the screened well [L], and $r_{\mathrm{o}}$ is the distance from the observation point to the centre of the well [L]. $\alpha_{\mathrm{W}} = r_{\mathrm{ew}} S / r_{\mathrm{c}}$ , where $r_{\mathrm{c}}$ is the radius of the unscreened part of the well [L], and $u = r_0^2 S / 4Kbt$ , where *t* is the pumping time [T]. It is considered that $\alpha_{\mathrm{W}} = S$ because $r_{\mathrm{c}} = r_{\mathrm{ew}}$. Values of the function *F* have been previously tabulated (Kruseman and de Ridder, 1994).

If the aquifer boundaries are far enough and their influence is negligible in the interest zone, the maximum head oscillation ( $\Delta s$ ) can be approximated as:

$$\Delta s = \left(\frac{Q}{4\pi T}\right)\left[\Delta F_{\left[\langle t_0 \rangle \text{ to } \langle t_F \rangle\right]}\right] \tag{2}$$

where $t_0$ is the initial time of a pumping or an injection (i.e., 0 days) and $t_F$ id the final time of the pumping or the injection. This solution can be used to compute the head oscillations in the reservoir or in the surrounding medium. The main limitation is related with the available tabulated values of the function *F*.

The time to reach the dynamic steady state ( $t_{\mathrm{SS}}$ ) can be determined by plotting the tabulated values of the function *F* versus 1/*u* and identifying the point from which the slope of *F* does not vary or its change is negligible. However, this procedure is too arbitrary. Therefore, it is proposed to determine $t_{\mathrm{SS}}$ from the derivative of *F* with respect to the logarithm of $1/u$ . Flow behaviour is totally radial and dynamic steady state is completely reached when $dF/d\ln(1/u) = 1$ (= 2.3 if the derivative is computed with respect to $\log_{10}(1/u)$ ). For practical purposes, it is considered that dynamic steady state is completely reached when $dF/d\ln(1/u) < 1.1$. However, dynamic steady state is apparently reached when the radial component of the flow exceeds the linear one because more than 90% of $\overline{h}$ is recovered when that occurs. The time when dynamic steady state is apparently reached can be easily determined from the evolution of $dF/d\ln(1/u)$ because its value decreases. Figure 2 shows $dF/d\ln(1/u)$ versus $1/u$ considering a synthetic scenario for a piezometer located at 50 m from the underground reservoir (values of *F* and *u* are tabulated in Kruseman and de Ridder, 1994). Flow behaviour is totally radial ($dF/d\ln(1/u)<1.1$) for $1/u>500$, and the percentage of radial flow exceeds the linear one for $1/u\approx 50$. More precision is not possible because there are no more available values of *F*. Actual times are calculated by applying $t=r_0^2S/4Kbu$.

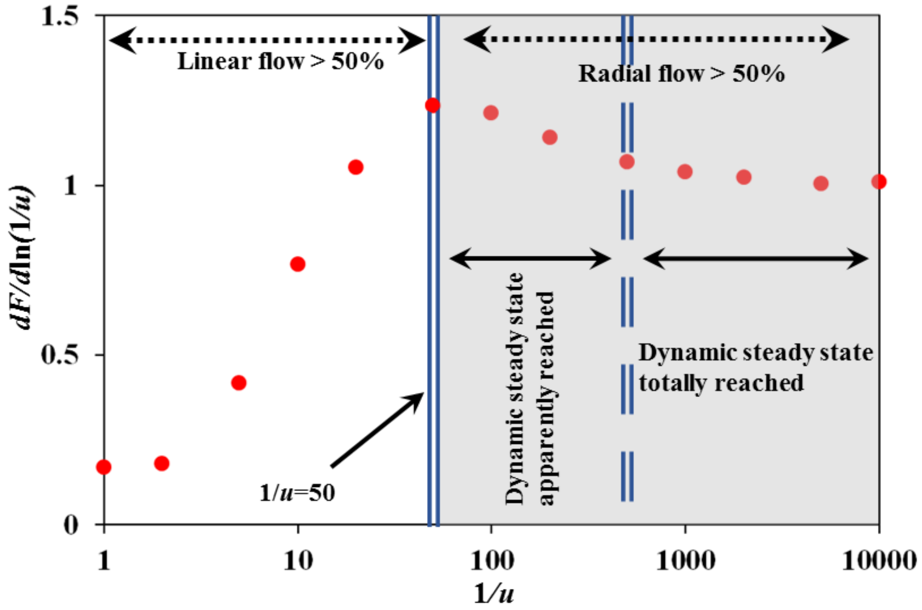


**Figure 7**. $dF/d\ln(1/u)<1.1$ **versus** $1/u$ **for a piezometer located at 50 m from an underground reservoir (Pujades et al., 2016).**

Note that, if the observation point is too far (more than 10 times the radius of the underground reservoir) from the underground reservoir, the slope of *F* is constant from early times and values of $dF/d\ln(1/u)$ do not decrease with time. In these cases, the piezometric head oscillates around the initial one from the beginning.

This procedure to calculate $t_{SS}$ is only useful if the aquifer boundaries are far enough away so that they do not affect the observation point before the groundwater flow behaves radially. If the boundaries are closer, dynamic steady state is reached when their effect reaches the observation point. This time ($t_{BSS}$) can be calculated from Eq. 16

$$t_{BSS}=\frac{\left[L+(L-r_0)\right]^2 S}{T} \qquad \text{(X)}$$

where $L$ is the distance from the underground reservoir to the boundaries [L]. More information about the analytical methods can be found in Pujades et al., 2016.

## Computation of normalized distance of influence in an open pit used as lower reservoir of a UPSH

As an illustration for a squared quarry, **Erreur ! Source du renvoi introuvable.** shows the normalized distance of influence $\bar{d}$, as a function of two non-dimensional parameters. These non-dimensional parameters are defined here below. The distance of influence $d$ is defined as the distance between the quarry walls and the point in the rock domain where the amplitude of the hydraulic head fluctuation is reduced by 99%, compared to the theoretical amplitude of the water level fluctuations in the quarry, in absence of any water exchange between the quarry and the surrounding medium. $\bar{d}$ is the distance of influence, normalized by $\eta_c$. Note that results of **Erreur ! Source du renvoi introuvable.** have been calculated considering a sinusoidal pump-storage flow rate and a generic quarry depth of 80 m (Poulain et al, 2018).

| | |
|---|---|
| $\bar{d} = \frac{d}{\eta_c}$ [-] : | Normalized distance of influence |
| $\eta_c = \frac{Q_{max}\,\tau}{S\pi}$ [m] : | Expected fluctuations in an equivalent impermeable reservoir |
| $\bar{D} = \frac{\tau K}{\eta_c S_y}$ [-] : | Non-dimensional diffusivity |
| $\bar{K} = \frac{K\tau\eta_c}{S}$ [-] : | Non-dimensional hydraulic conductivity |
| S [m$^2$] : | Surface of the open pit quarry |
| $\tau$ [s] : | Period of the PSH cycles |
| $Q_{max}$ [m$^3$/s] : | Maximum pumping/turbining flow rate of the PSH cycles |
| K [m/s] : | Hydraulic conductivity of the surrounding rock medium |
| $S_y$ [-] : | Specific yield of the surrounding rock medium |

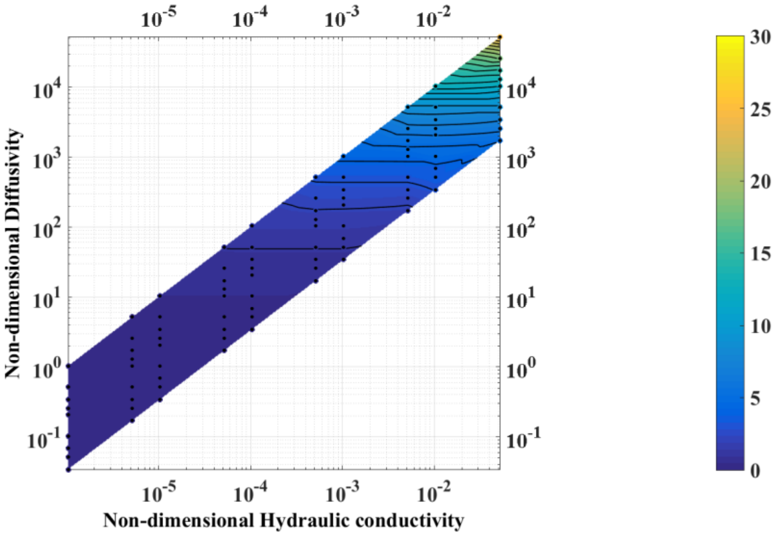


**Figure 8. Normalized 'distance of influence' $\bar{d}$ in the rock media, according to two non-dimensional parameters. Black dots correspond to couples of parameters values actually used for numerical simulation and interpolation. (Modified from Poulain et al, 2018)**

## Estimation of walls strength

Flexion generated within the pillar by the uniform load applied over a part of its height may be approximated by a beam analogy. The following hypotheses are assumed

- the pillar is represented by a doubly clamped Euler-Bernoulli beam;
- the loading is assumed constant over the length **a** of the beam, as shown in Figure 9;
- the section is homogeneous and constant over the length;
- there is no second order effects.

The uniform load is computed as

$$q = \Delta h \, \gamma_w,$$

where $\Delta h$ is the difference of water level between a pillar and $\gamma_w$ the weight of water. The distribution of shear and moment over the beam is given in

| | $x \leq a$ | $x > a$ |
|---|---|---|
| T(x) | $T = T_1 - q\,x$ | $T = T_1 - q\,a$ |
| M(x) | $M = T_1 x - M_1 - q\dfrac{x^2}{2}$ | $M = T_1 x - M_1 - qa\left(x - \dfrac{a}{2}\right)$ |
| $T_1$ | $= qa\left[1 + \dfrac{a^3}{2L^3} - \dfrac{a^2}{L^2}\right]$ | |
| $M_1$ | $= \dfrac{qa^2}{12L^2}[3a^2 - 8aL + 6L^2]$ | |

where $T_1$ and $M_1$ are respectively shear and moment load at the bottom.

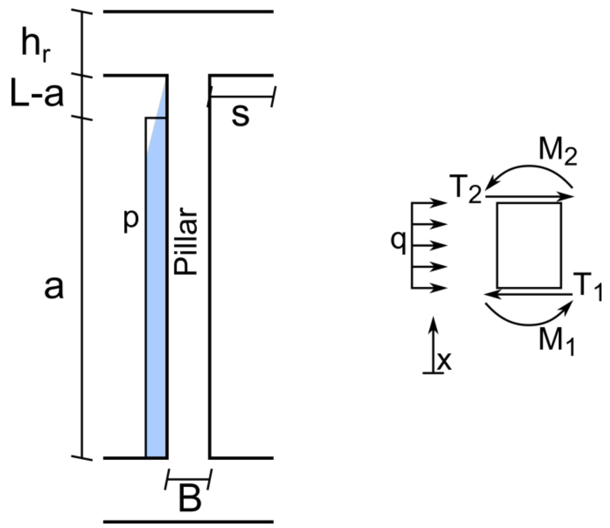


**Figure 9 Equivalent beam, the origin of x axis is in the middle of the column base**

The normal load applied at the top of the wall depending on the room's span is computed according to

$$N_0 = (B + 2s)h_r\,\gamma$$

where $\gamma$ is the rock weight, $h_r$ the thickness of the roof and s the half span of the room. Therefore the distribution of normal load is derived from

$$N(x) = N_0 + B\,\gamma(L - x).$$

Maximum normal stresses (normal in compression) are computed on both sides of the beam ($\sigma_{max}$ compression or $\sigma_{min}$ traction),

$$\sigma_{max}(x) = \frac{N(x)}{B}\left(1 + \frac{6|e(x)|}{B}\right),$$

$$\sigma_{min}(x) = \frac{N(x)}{B}\left(1 - \frac{6|e(x)|}{B}\right).$$

where $e(x) = M(x)/N(x)$ is the eccentricity. In each cross-section, maximum and minimum normal stress may be compared to different criteria.

Criterion in compression:

$$\sigma_{max} < \sigma_y$$

where $\sigma_y$ is the design simple compression strength. This criterion assumes simple compression conditions.

Criterion in traction:

$$\sigma_{min} > 0 \text{ or } \sigma_{min} > \sigma_{y,t}$$

where $\sigma_{y,t}$ is the traction strength of the material. The first criterion ensures there is no traction at all within the material. The second one allows traction to be generated. The traction strength may be derived from the classical Mohr-Coulomb or Hoek-Brown criteria.

Some additional refinements may be added to this simple model:

- a base displacement may be imposed to take into account varying level of the floor;
- a Timoshenko beam may be considered to compute deformation and load distribution (greater importance when L/B decreases);
- the criterion in compression or traction may take into account the normal shear stress interaction;
- the influence of discontinuities is only partly taken into account by reducing the homogeneous cohesion;
- the doubly clamped beam overestimate stiffness of the roof and the floors, linear springs may replace these conditions;
- the unsaturated apparent strength may be considered instead of a drained simple compression resistance.