# Determination of erosion thickness by numerical back analysis: The case study of Badenian clays in the Carpathian Foredeep, Czech Republic

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**Abstract:** The paper describes an application of the geotechnical numerical back analysis in estimating the thickness of eroded sedimentary overburden in shallow basinal sediments. The approach is based on the back-analysis of the coefficient of earth pressure at rest $K_0$ and on estimating the unloading from the obtained $K_0$ value. This approach is compared with the conventional methods represented by Baldwin–Butler's “compaction curves” and Casagrande´s concept of “preconsolidation stress”. The results of these two commonly used methods are incorrect if the sedimentary profile is affected by “ageing” effects, such as cementation, secondary compression etc. The method is demonstrated on the Lower Miocene marine clay, often called „Tegl” which was deposited in the Carpathian Foredeep in the vicinity of Brno, Czech Republic. The numerical back analysis was applied to galleries and adits opened during site investigation of the Královo Pole Tunnels in Brno. The application of Baldwin–Butler's equation suggested the erosion thickness of 180-270 m and Casagrande´s method of 100–800 m, while the numerical back analysis of 0–40 m.

**Key words:** ageing, coefficient of earth pressure at rest, compaction curve, erosion thickness, numerical back analysis, Tegl

## 1. Introduction

The thickness of erosion of sediments can be estimated using purely geological approach determining the altitudes of the current surface of the stratum and its denudation relics. An essential disadvantage of this approach is the fact that the result can be significantly affected by tectonic (vertical) movements. In order to avoid the problem, several techniques based on analyses of the mechanical properties of the soils have been developed. But it is well known that most mechanical properties of soils change during ageing (e.g., Chandler, 2010, Mesri and Hayat, 1993). The ageing effects are difficult, or impossible to quantify, and invalidate the estimates of the erosion thickness. This also disqualifies the two most common methods based on the analysis of mechanical properties: Baldwin–Butler's equation (1985) and Casagrande´s method (1936).

Determining the erosion thickness by the proposed geotechnical numerical back analysis does not have to consider the ageing effects, which would be necessary in both Casagrande´s and Baldwin - Butler´s method. On the other hand, the procedure assumes that there is no change in horizontal stress due to ageing. The literature review, however, revealed that the effects of ageing on the horizontal stress (and $K_0$) in clay massifs has not been solved to date. Nevertheless assuming constant horizontal stress seems to be plausible (Holtz and Jamiolkowski 1985, Gareau et al., 2006). In the following, the results of a numerical back analysis are compared with Baldwin–Butler's and Casagrande´s methods.

A soil affected by ageing had to be chosen for such a study. The Miocene clay of the town of Brno called “Tegl” seemed a good candidate for such an exercise: it had clearly been subjected to ageing since its sedimentation in the Carpathian Foredeep, and its thickness of erosion is still a matter of dispute. The estimated values vary from  tens to hundreds of metres (e.g., Boháč and Pavlová, 2012, Pavlík et al., 2009). Moreover, a well-documented geotechnical case-history was available for the study – the Královo Pole Tunnels project, during which exploratory adits, drifts, and final motorway tunnels were excavated in the Tegl strata (Pavlík et al. 2004, Svoboda et al., 2009; 2010).

## 2. Geological setting

The analyses were made on Middle Miocene, Early Badenian calcareous clayey sediment in the Carpathian Foredeep, further referred to Tegl. The Early Badenian (Moravian) sediments of the Carpathian Foredeep basin were deposited during a marine transgression from the ESE on the East margin of the Bohemian Massif. The lowermost units include the Iváň Beds and basal Brno sandstones and conglomerates with local maximum thickness of 190 m (Stráník et al., 2016). They are overlain by deepwater fine grained sediments described as Tegl. This unit without a formal lithostratigraphic name consists of blue-, brown- to green-grey massive calcareous clay with sandy laminae and horizons in the lower part. Frequent lenticular bodies are enriched in organic matter and fragments of molluscan shells. Tegl onlaps on the pre-Neogene units in the West and widely surpasses the regional extent of the basal clastics. This transgression is correlated with the eustatic sea level rise of the global ocean and the paleo-water depth is estimated to be as high as 100 m West of Brno city while 200-500 metres in upper bathyal setting in Brno-Královo Pole (Brzobohatý, 1982). Radiometric measurements of rhyodacite tuffs and tuffitic clays, which occur in local interlayers, provide age estimation of 16,2 ± 2,1 mil. years (Nehyba, 1997). The maximum known thickness of Tegl is more than 1000 m East of Ostrava city.

Tegl consist typically of quartz (ca 29%) and calcium carbonate (ca 31%). Smectite was detected only at small amounts (ca 3%). Gypsum and pyrite are also encountered in Tegl. Fe hydroxides in Tegl are products of the process of the gradual oxidation of pyrite. Calcium carbonate is present in form of the crystaline (calcite $CaCO_3$). The amorphous form of the calcium carbonate that could cause the cementation is not present, because the calcite precipitation from the solutions during post sedimentation process does not support formation of the amorphous form.

It is obvious that the top of the deepwater Early Badenian is erosional and that younger sediments which covered Tegl have been removed. The strata are influenced by ageing effects, such as secondary compression (Boháč and Pavlová, 2012) or tectonic movements (Pavlík 2004). The thickness of the eroded units has not been estimated in a satisfactory manner up to now.

## 3. Investigated sites

Samples and data from two sites in the area of Brno town were used. Data from Brno - Královo Pole (Královo Pole Tunnel project) were used in the numerical back analysis. After completion of Královo Pole tunnels, however, obtaining of new undisturbed samples of Tegl was impossible in the developed area. The samples for analyses according to Baldwin - Butler´s and Casagrande´s proposals were therefore taken  from Brno - Slatina (position of the V1 borehole in Fig.1). The thickness of erosion of Tegl for both site is assumed to be the same or very similar due to several reasons:

1) No significant tectonical movement has been identified between the areas.

2) The current head of the Tegl stratum is approximately at the same level at both sites (ca 230 - 245 m above sea level).

3) The coastline of the sea during deposing of the stratum is estimated to be 15 – 30 km from the investigated sites and that is why horizontal surface of Tegl after depositing is assumed for both sites.

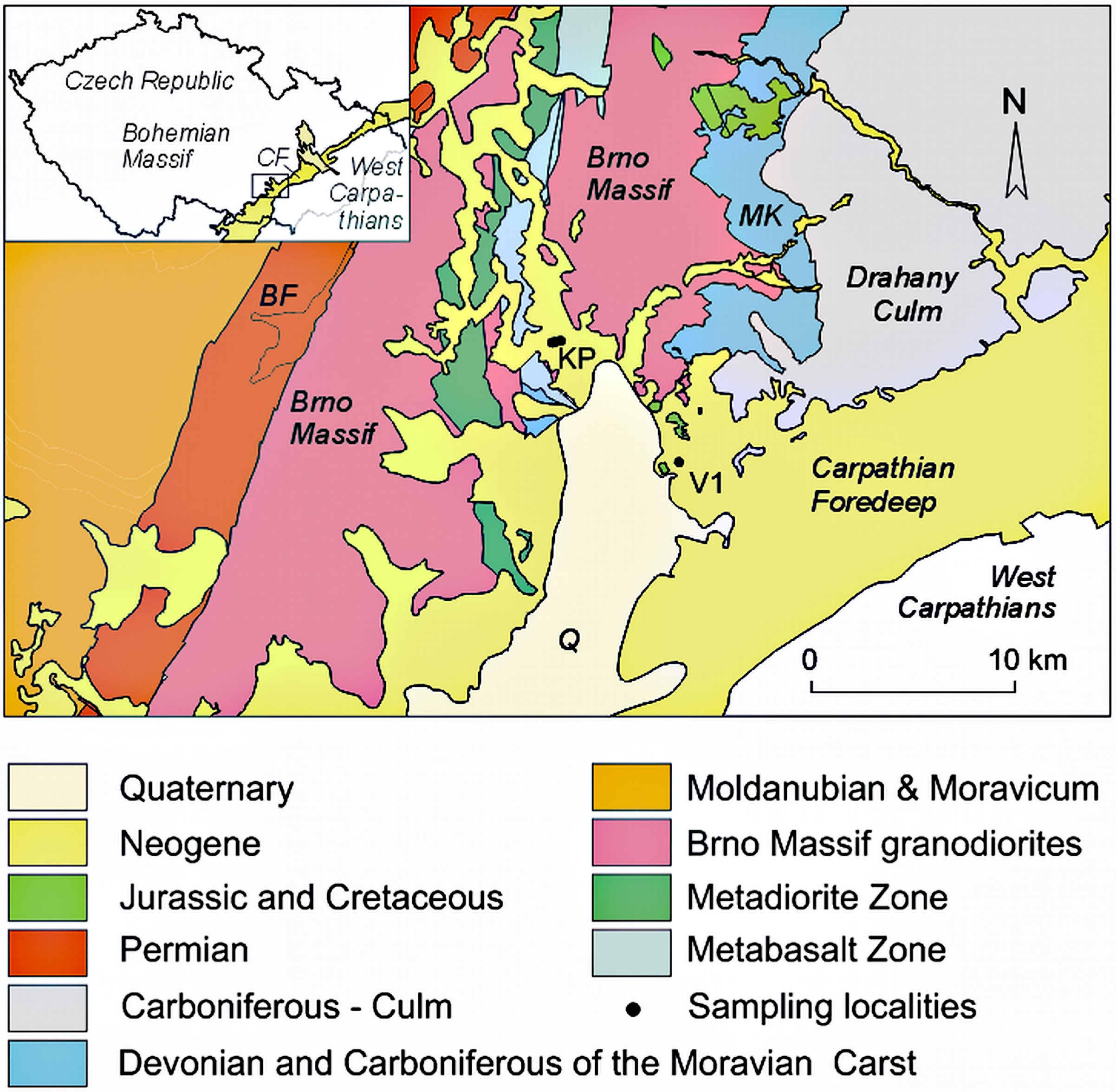


*Fig. 1. Locations of KP (Královo Pole tunnel) and V1 (Brno Slatina) boreholes. The yellow area shows the extension of the Carpathian Foredeep, adjacent geological units are shown for reference.*

### 3.1 V1 borehole

The borehole V1 was situated in Brno town between Drážní and Šmahova streets. Coordinates of the axis of the borehole are: 49.1709261N, 16.6826475E (WGS). The surrounding is flat and reaches approx. 250 m above sea level. Quaternary sediments are encountered in relatively thin layers (up to first metres) and thickness of Tegel is assumed several tens metres according to available archival data.

A spiral drillbit in dry drilling mode was used and the depth of 56.5 m was reached. Every ca three metres, undisturbed samples were taken using a pushed-in thin-walled steel sampler. The

V1 profile (Fig. 2) consists of anthropogenic debris at depth of 0-0.5 m and Quaternary eolian and deluvial silts and clays at 0.5-3.9 m. Early Badenian Tegl fills the 3.9-48 m interval, with grey-brown clay at depth of 3.9-13.8 m gradually changing into a layer of non-weathered and very stiff grey-green clay at 13.8-19.5 m. Basal Lower Badenian grey-green clayey sand and gravel were encountered at 48.0-56.5 m.

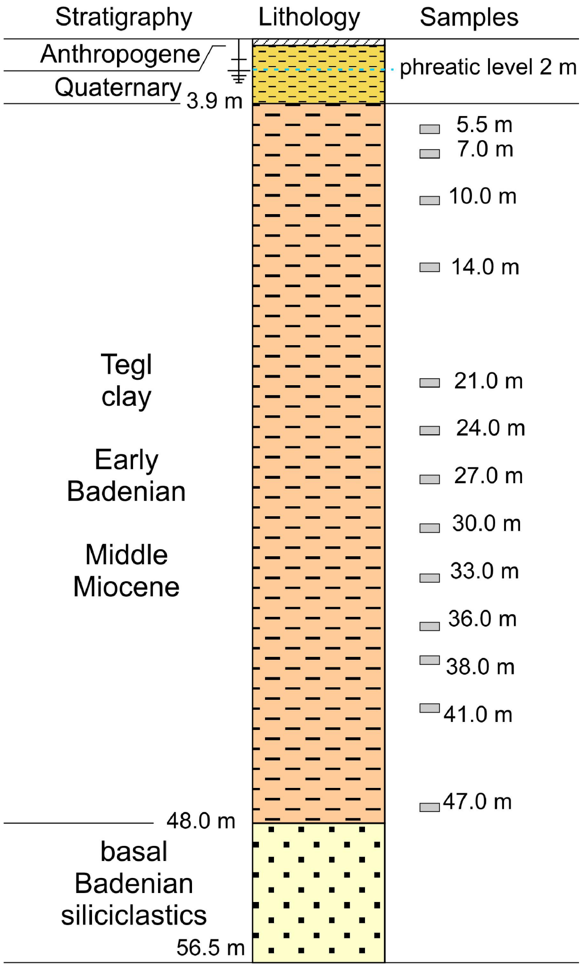


*Fig. 2. Schematic profile of V1 borehole with marked position of undisturbed samples.*

### 3.2 Královo Pole Tunnels

The two two-lane road tunnels of the Královo Pole project are situated in the north-western part of Brno. Three exploratory galleries and four adits were excavated, instrumented and monitored as a part of the site investigation  (Fig. 3). Geological setting around exploratory galleries is presented in Fig. 4. Head of Tegl stratum above the adits is approximately 230 m above sea level. Quaternary sediments above Tegl clay consist mainly of loess and fluvial sand and gravel (see Pavlík et al. 2004 for detailed information).

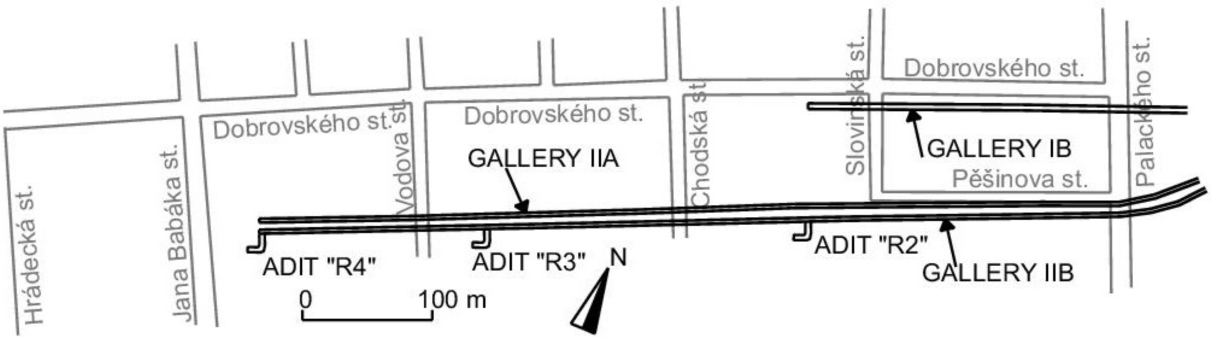


*Fig. 3.Underground works during Královo Pole Tunnels site investigations – Exploratory galleries and unsupported adits (Pavlík et al. 2004).*

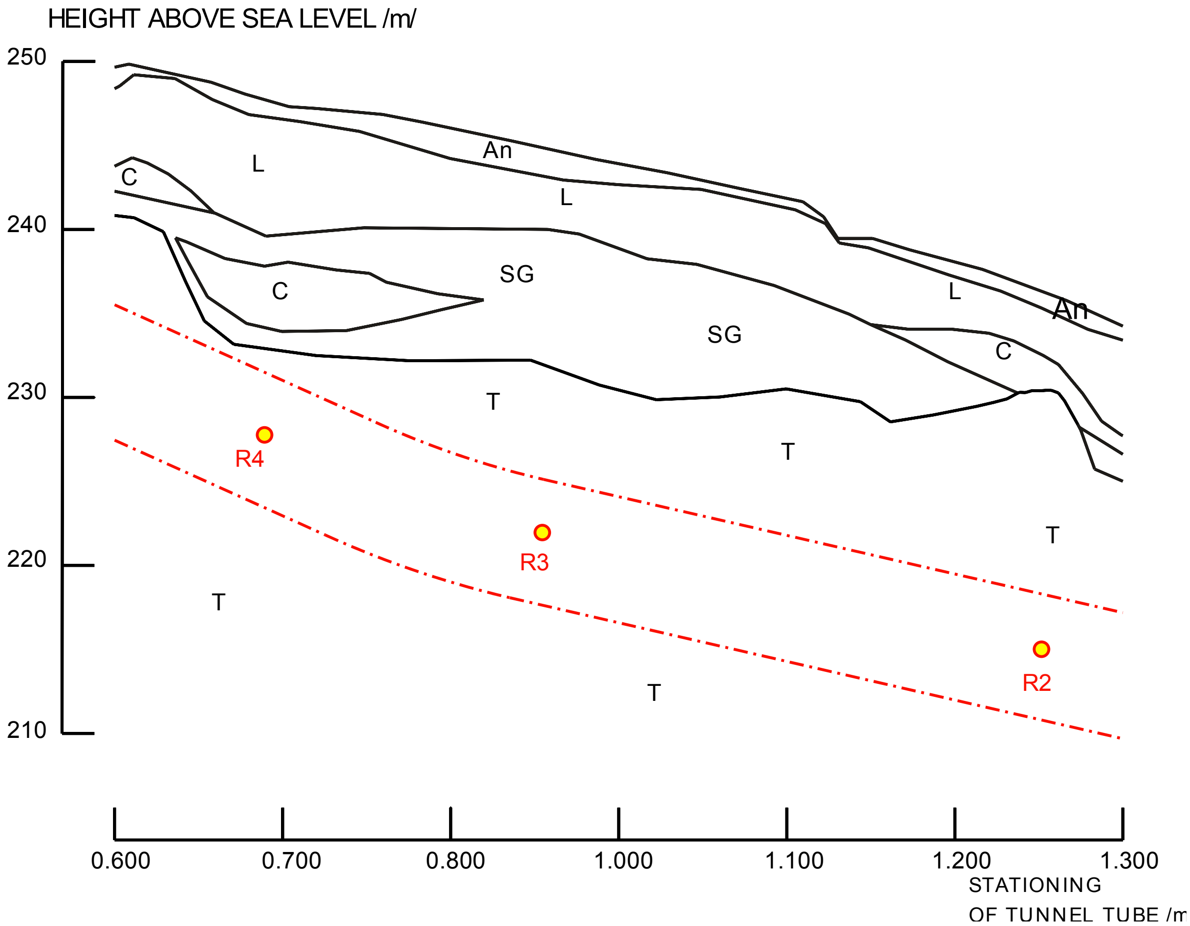


*Fig. 4 Geological setting aroud analyzed adits. T – Tegl clay, SG – sand with gravel, C – clayey silt, L – loess, An – anthropogenic sediments.Tunnel tube is marked with dot-and-dash lines.*

# 4. The thickness of erosion determined by conventional methods

## 4.1 Baldwin–Butler's equation

An empirical equation for the burial depth $b_d$ was proposed by Baldwin and Butler (1985) which is a regression curve of data collected mainly by Baldwin (1971) and several other autors. The equation is valid for argillaceous sediments.

$$b_d = 6.02\, S^{6.35}.1000 \text{ [m]} \tag{1}$$

where solidity *S [%]* is the volume of solid grains as a percent of total volume of sediment; it is a complementary value to porosity $n = (100\text{-}S)$ [%.]

The porosity (solidity) of a sample changes after removal from the *in-situ* stress conditions. In the laboratory, for estimation of solidity the samples have to be reconsolidated to the *in-situ* effective vertical stress $\sigma'_v$:

$$\sigma_v' = \gamma_{sat}.h - u \text{ [kPa]}$$

(2)

where

$\gamma_{sat}$ – [kN.m$^{-3}$] unit weight of fully saturated soil,

$h$ – [m] overburden height,

$u$ – [kPa] pore pressure *in-situ*

The unit weight $\gamma_{sat}$ of Tegl of 18.8 kN.m$^{-3}$ (Svoboda et al. 2009, 2010) was used.

The thickness of erosion is then calculated simply from the depth of burial and the depth of sampling under current surface:

$$E = 6.02S^{6.35}.1000 - h \tag{3}$$

$E$ – [m] thickness of erosion.

### 4.1.1 Results and discussion

The thickness of erosion calculated according to Baldwin-Butler's equation is summarized in Tab. 1. It is obvious that the values fluctuate around approximately 200 m, with the maximum and minimum values of 245 and 177 m, respectively. However, ageing is not accounted for in the Baldwin-Butler's equation. Moreover, the increase of porosity due to unloading by erosion is neglected.

*Tab. 1. Erosion thickness calculated using Baldwin–Butler's equation.*

| **Depth sampling (current burial) [m]** | **Solidity „S“ after reconsolidation [%]** | **Calculated erosion thickness [m]** |
|---|---|---|
| 7 | 59,4 | **214** |
| 14 | 59,7 | **213** |
| 21 | 67 | **231** |
| 24 | 59,9 | **208** |
| 27 | 61,2 | **240** |
| 30 | 61,3 | **240** |
| 33 | 59,7 | **193** |
| 36 | 59,9 | **195** |
| 38 | 59,2 | **177** |
| 41 | 61,9 | **245** |
| 47 | 61,5 | **227** |

## 4.2 Casagrande´s method

Casagrande (1936) stated that the largest overburden under which the soil (clay) had once been consolidated can be determined as the “pre-consolidation load” in the one-dimensinal compression test in the oedometer. Further he suggested a graphical method of determining the value of the preconsolidation pressure $\sigma'_{v\ max.}$ from the oedometer test results. An ideal compression curve of a preconsolidated (overconsolidated) sediment is shown in Figure 5A. For a real sediment however, with the compression curve not showing a clear kink of the $n$-$log\sigma_v'$ curve, the preconsolidation pressure is determined as shown in Fig. 5B. The method is often used to date, despite the fact that original Casagrande's geological interpretation must be in error due to ageing. However, neglecting ageing and provided  the weight of the sediment is known, it is tempting to determine erosion thickness for soils sedimented in water from

$$E = \frac{\sigma'_{vmax}}{(\gamma_{sat} - \gamma_w)} - h \text{ [m]} \quad (4)$$

$\sigma'_{v\ max}$ – [kPa] preconsolidation pressure

$\gamma_w$ – [kN.m$^{-3}$] unit weight of water

The undisturbed samples from the borehole V1 were subjected to one-dimensional oedometer compression. After reconsolidation to the estimated *in-situ* vertical stress the incremental step-wise loading was applied to the vertical stress of about 10 MPa (Fig. 7), which is substantially higher than the values suggested for overconsolidated and stiff clays in practice (Head and Epps, 2011). Despite the elevated stress levels the determination of preconsolidation pressure by Casagrande's method proved difficult. The results are in Table 2.

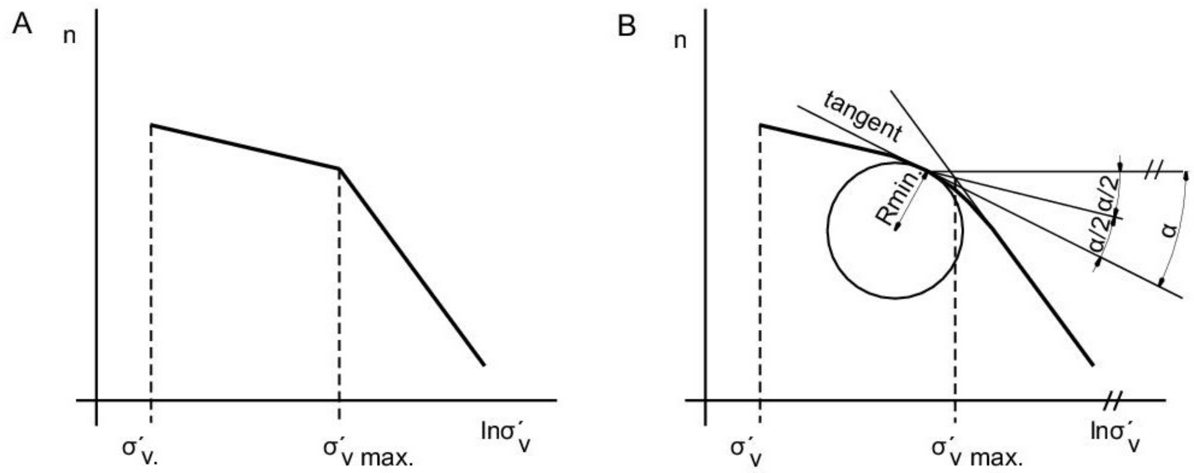


*Fig. 5. Semilogarithmic oedometer compression curve in the plane of porosity and vertical pressure $ln\sigma'_v$: A) An idealised oedometer compression curve; B) Determination of $\sigma'_{v\,max}$ proposed by Casagrande (1936).*

As suggested above, Casagrande´s geological interpretation of preconsolidation pressure is in error due to ageing, namely due to reduction of voids portion during time.

Decreasing of porosity without a change in vertical effective pressure is pronounced especially with clays. Due to secondary compression, oedometer tests carried out on good quality specimens of natural clay inevitably determine a “quasi-preconsolidation pressure” $\sigma'^*_{v\,max}$ instead of the “true” preconsolidation pressure $\sigma'_{v\,max}$ (Leonards and Altschaeffl, 1964; this is also included in Bjerrum's (1967) “delayed consolidation” and the concept of “time lines”). The quasi-preconsolidation pressure $\sigma'^*_{v\,max}$ determined by Casagrade's method can reach substantially higher value than the true preconsolidation pressure $\sigma'_{v\,max}$ (Fig. 6).

The change in porosity during secondary compression may be estimated using the secondary compression index $C_\alpha$ [-]. In geotechnical practice $C_\alpha$ is determined as the slope of the linear portion of the compression curve plotted as voids ratio vs logarithm of time (e.g., Head and Epps, 2011). However the extrapolation to geological times is a very crude and questionable estimate.

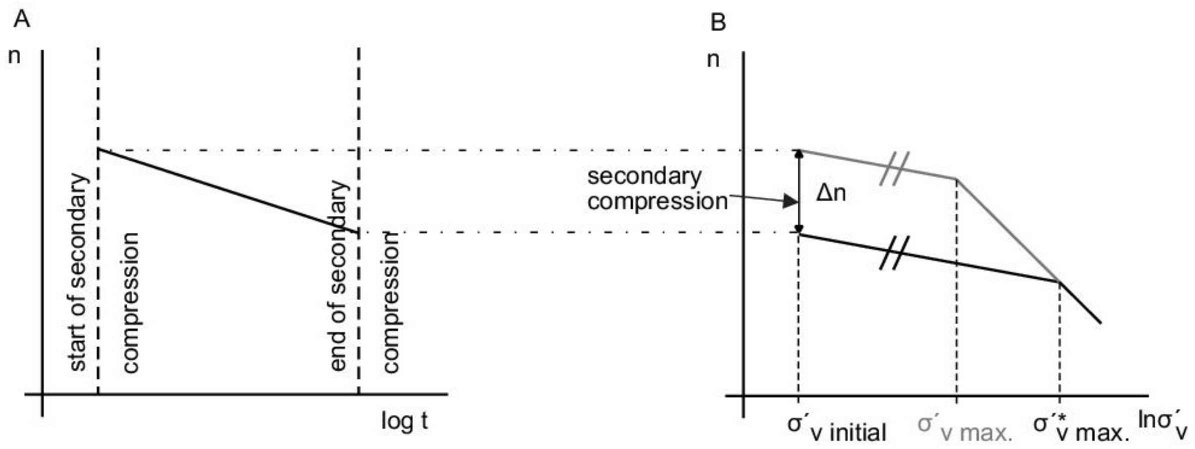


*Fig. 6. Definition of secondary compression (A), and its influence on the compression curve (B).*

Further, in order to estimate the change of porosity during secondary compression, it is necessary to determine its duration. The time needed for the sedimentation of the Tegl strata encountered in the V1 borehole was assessed by magnetostratigraphic measurements on the undisturbed samples (Bosák and Pruner, 2014), which showed that the sedimentation took place approximately between 14.8 and 14.24 Ma before present. The rate of sedimentation was so slow that the older part of the stratum was affected by secondary compression when the younger part of the stratum had not been sedimented yet. Hence, we decided to consider the mid-point of the sedimentation (14.52 Ma before present) for our calculation.

#### 4.2.1 Results and discussion

Applying the vaule of $C_\alpha = 0.016$ (Boháč and Pavlová, 2012) to the samples from the borehole V1, the decrements $\Delta n$ of porosity during secondary compression and consequently the corresponding thicknesses of erosion were calculated from the laboratory compression curves of Fig. 7 (Table 2). A test on reconstitued sample was carried out in order to evaluate the curves of undisturbed samples according to Casagrande´s approach (see fig. 7).

The obtained thicknesses of the eroded layer vary between 134 – 766 (approx. 100 – 800 m) Thus, the results cannot be considered reliable.

The problem of Casagrande's proposal to use the oedometer test in studying the overburden pressures in the geological history was well expressed by Mayne and Kulhawy (1982): “At present, however, there appears to be no known technique of determining $OCR_{max}$ ...“ (i.e. true preconsolidation pressure) “....for a specific soil deposit other than a good knowledge of local geology and stress history of the soil deposit.” The skepticism of the quotation is clearly confirmed by the discussion of our data above. Burland (1990) or Chandler (2010) also point out the problem explicitly, however, Casagrande´s (1936) approach is still the most used technique for *OCR* (or quasi - *OCR*) determination. It has to be mentioned that alternative methods for estimation of $\sigma'_{v\ max}$ from oedometer test has been suggested (e.g., Jefferies et al. 1987). Nevertheless, this is a technique for better determining of the „kink point“ and do not deal with the fact that the point does not respond to true overconsolidation pressure in the case of aged soils.

These facts served as motivation for proposing the new approach in the next chapter: combining a numerical geotechnical model with the well-established empirical relationship between in-situ stresses ($K_0$) and the true preconsolidation pressure (i.e. $OCR_{max}$) due to Mayne and Kulhawy (1982).

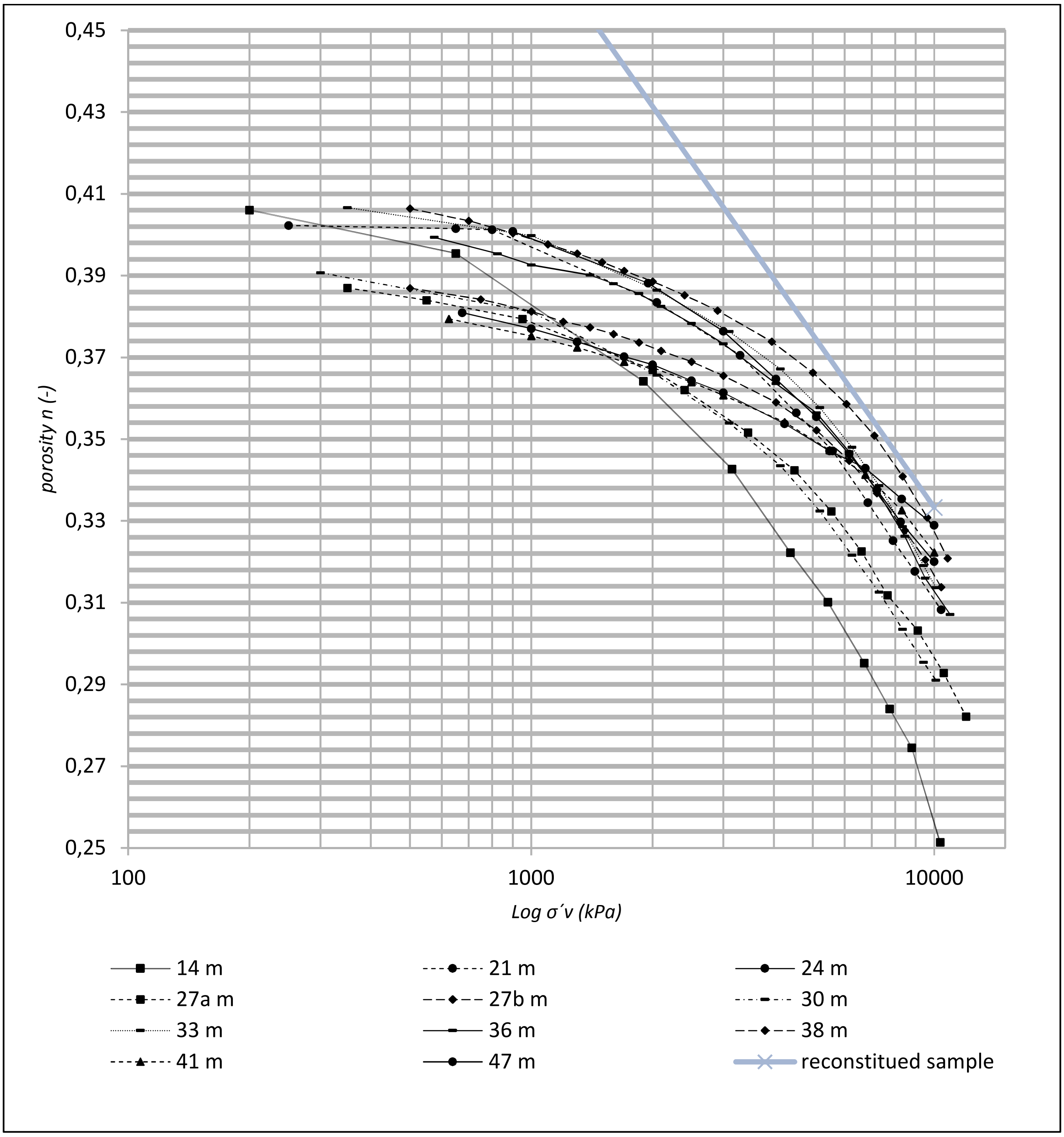


*Fig. 7. Compressibility curves from oedometer tests on undisturbed specimens of V1 borehole.*

*Tab. 2. The thickness of erosion of Tegl determined using oedometer tests.*

| Sample No. | depth of the samples [m] | $\sigma'_v$ [kPa] | $\Delta n$ [-] | $\sigma'^*_{v\,max}$ [kPa] | $\sigma'_{v\,max}$ [kPa] | thickness of erosion[m] |
|---|---|---|---|---|---|---|
| 4 | 14 | 124 | 066 | 2100 | 1300 | **134** |
| 5 | 21 | 185 | 053 | 3950 | 2600 | **274** |
| 6 | 24 | 211 | 047 | 5400 | 4050 | **436** |
| 7 | 27 | 238 | 035 | 7050 | 5550 | **604** |
| 8 | 30 | 264 | 055 | 3900 | 2300 | **231** |
| 9 | 33 | 290 | 050 | 5000 | 3450 | **359** |
| 10 | 36 | 317 | 048 | 4850 | 2700 | **271** |
| 11 | 38 | 334 | 046 | 5050 | 4000 | **417** |
| 12 | 41 | 361 | 031 | 8950 | 7100 | **766** |

# 5. The thickness of erosion estimated by geotechnical numerical back analysis

Our geotechnical numerical back analysis simulated the mechanical behaviour of the underground excavations carried out during the Královo Pole Tunnels project. As the project was relatively complex, the readers are referred to another publication for more details on the modelling precedure (Rott et al., 2015). The project consisted of two road tunnels, supported exploratory galleries of triangular cross-sections and unsupported four adits of circular cross-section, which all were thoroughly monitored. For our analysis, the unsupported adits were the most important. They were excavated from the exploratory gallery and they were intentionally left without any active support to make it possible to measure the convergence ("squeezing") of the cavities. Back-calculating the measured "squeezing" of the cavities by optimization of the value of the coefficient of earth pressure at rest $K_0$ using an appropriate advanced numerical model (Mašín, 2014; Rott and Mašín, 2014) allowed us to obtain the *in-situ* stress state in the clay massif prior to the excavation works. Knowing the *in-situ* stresses, which in the case of over-consolidated stiff clays are believed the most difficult parametres to be obtained (e.g., Hight et al., 2003), the depth of erosion could be calculated with a reasonable confidence. In the subsequent text, this procedure is explained in more detail.

### 5.1 Initial stresses in the numerical model

The initial *in-situ* stresses must be put into the advanced numerical models to simulate correctly excavations, typically using $K_0$

$$K_0 = \sigma'_h / \sigma'_v \ \text{[-]} \tag{5}$$

where

$\sigma'_v$ and $\sigma'_h$ are the vertical and horizontal effective stresses, respectively.

$K_0$ depends on the stress history of the soil, as was proved by a number of laboratory measurements (e.g. Brooker and Ireland, 1965, or Mayne and Kulhawy, 1982). The latter authors suggested an empirical equation, derived from laboratory data of loaded-unloaded-reloaded specimens of both sands and clays. This type of test is a laboratory simulation of sedimentation-erosion-resedimentation process:

$$K_0 = (1 - sin\phi')\left[\frac{OCR}{OCR_{max}^{(1-sin\phi')}} + \frac{3}{4}\left(1 - \frac{OCR}{OCR_{max}}\right)\right] \tag{6}$$

where $\Phi'$ is critical state friction angle of the soil (the critical state – see, e.g., Atkinson, 2007) and *OCR* is the overconsolidation ratio, defined as the ratio of the maximum and current vertical effective stresses:

$$OCR_{max} = \sigma'_{vmax.} / \sigma'_1 \ \text{[-]} \tag{7}$$

$$OCR = \sigma'_{vmax} / \sigma'_2 \ \text{[-]} \tag{8}$$

$\sigma'_1$ = stress after erosion; $\sigma'_2$ = stress after resedimentation

It is important to note that (6) is valid only for mechanical loading, unloading and re-loading of soils, i.e. true preconsolidation. It means that if $\sigma'^{*}_{v\,max}$ is used the result is not correct.

Therefore the equation (6) might not be used in estimating the stresses for Královo Pole Tunneling project directly from undisturbed oedometer specimens. However, if the stress state is reliably determined by another method – in our case by iterative back analysing the $K_0$ from the measured squeezing of the adits – the equation (6) can be used for calculating the thickness of erosion (under the assumption of constant $K_0$ during ageing, see the discussion later).

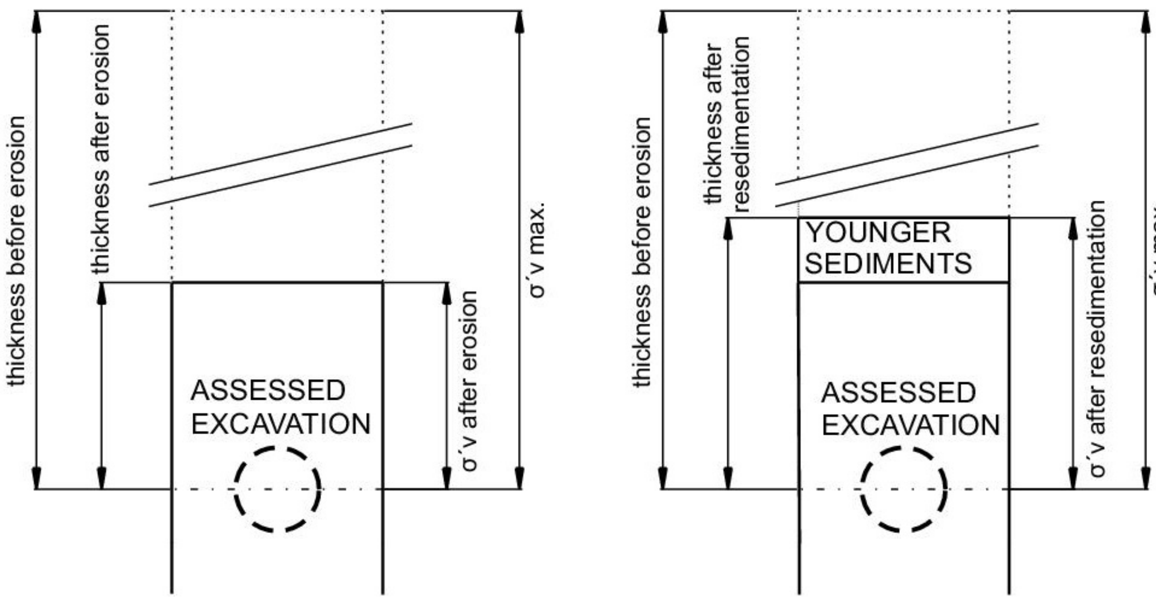


*Fig. 8. Effective vertical stress of equations (7) and (8).*

Thus, geotechnical numerical back analysis consist of several steps:

1) *K0* found by trial-and-error method in numerical model
2) Determining appropriate $\sigma'_{v\,max}$ in order to obtain *OCR* and $OCR_{max}$ which give the same value of $K_0$ as in the step 1) according to Eq (6)
3) Erosion thickness is computed using Eq (4).

### 5.2 Description of the model

The three modelled unsupported adits R2, R3 and R4 were excavated perpendicularly to the exploratory gallery IIB (Fig 3 and 4). They had a circular cross-section of approx. 2 metres in diameter. The steel frames (Fig. 9) were installed for the sake of security only, and they were not in contact with the face of excavation, and there was no need to consider them in the numerical model.

*Fig. 9. Lining with the offset of 50 mm from the soil (adit R2; photo by J. Pavlík).*

The hypoplastic constitutive model (Mašín, 2005) augmented with inherent anisotropy was used for Tegl. It allows for different stiffnesses in the horizontal and vertical directions (Mašín and Rott 2014, Mašín, 2014), which is a crucial requirement for simulating the squeezing of the adits properly.

The geological conditions of the three adits were determined by the geological and geotechnical site investigations for Královo Pole Tunnels (Pavlík et al., 2004), and they have been simplified for the purpose of the geotechnical model (see Fig. 10). The ground water level was situated at the interface of Quaternary sediments and Miocene Tegl.

Determination and calibration of the model parametres of the soils are out of the scope of the present paper, and the reader is referred to Rott et al. (2015).

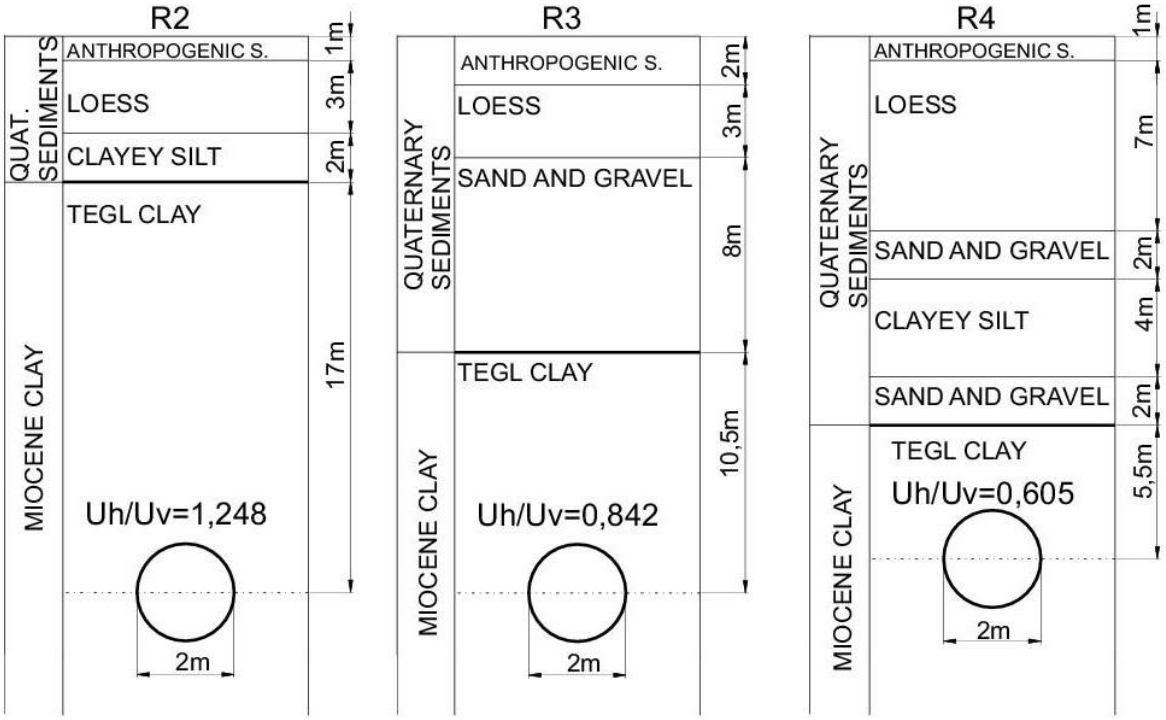


*Fig. 10. Geological conditions of the adits and the ratio of horizontal (Uh) and vertical (Uv) deformations measured in the modelled convergence profiles.*

The adits were modelled in three dimensions by finite elements (FEM) using the commercially available geotechnical software Plaxis 3D. The model was 55 m wide, 37 m deep and 36 m long. Since the studied "convergence profiles" in the adits were just a few metres from the exploratory gallery, each model contained both the adit and the gallery (Figs. 11 and 12).

The primary lining of the gallery has been composed of two components: the shotcrete and a massive steel support. The lining has been modelled using shell elements characterised by a single parameter set obtained using homogenisation procedure described in Rott (2014).

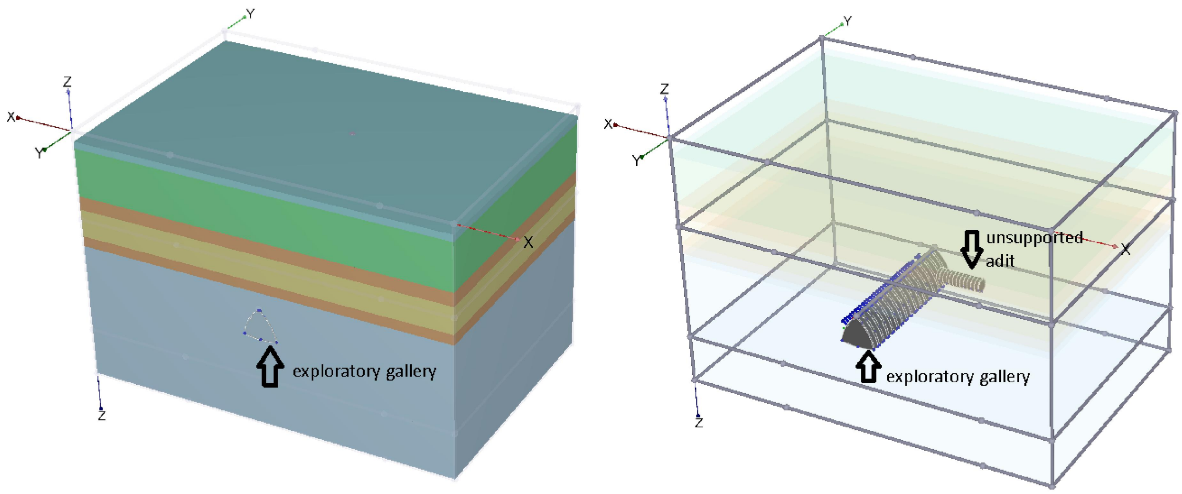


*Fig. 11. 3D model of the unsupported adit and the exploratory gallery in Plaxis software.*

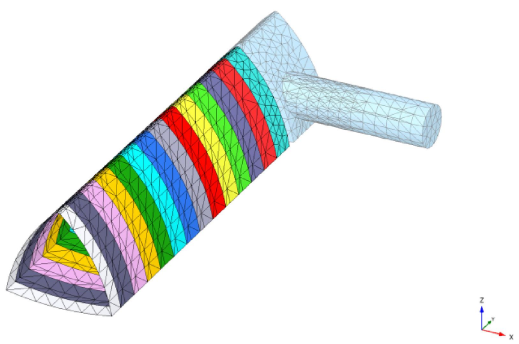

*Fig. 12. Juction of the exploratory gallery and the unsupported adit. Different colours indicate excavation steps, each associated with different time-dependent lining stiffness.*

As suggested above, the aim of the numerical modelling was to obtain by $K_0$ optimization the same ratio of horizontal ($U_h$) and vertical ($U_v$) displacements, which was measured *in-situ* for 3 adits R2, R3, R4. $K_0$ was changed repeatedly by trial-and-error until the model convergence approximately equalled the monitored values (Boháč et al., 2013). In the cavity R2 the ratio of the deformations was $U_h/U_v = 1.248$ In the case of the R3 and R4 cavities the ratios were $U_h/U_v = 0.842$ and $U_h/U_v = 0.605$, respectively (Table 3).

In the final step, Equation (6) was used to find the overburden stresses, and thus the depths of erosion, corresponding to the values of $K_0$ determined by the 3D numerical modelling. For this purpose, the geological development of the investigated site was simplified in the following way: the sedimentation of Tegl took place under water, and all the time during erosion the water level coincided with the surface of Tegl. Second, the Quaternary sediments of constant (current) thickness were considered. Last, similarly to Casagrande´s method, constant unit weight of Tegl of 18.8 kNm$^{-3}$ was assumed, regardless of the depth below the surface.

## 5.4 Results and discussion

Tab. 3 shows the deformation ratios measured *in-situ,* the deformations obtained by the numerical model after the optimisation procedure, and the corresponding $K_0$ values of the 3D model for the individual adits.

*Tab. 3. Convergence ratios and corresponding values of $K_0$.*

| **Adit** | **$U_h/U_v$ measured *in-situ*** | **$U_h/U_v$ from the models after $K_0$ optimization** | **Corresponding $K_0$** |
|---|---|---|---|
| R2 | 1.248 | 1,250 | **0.75** |
| R3 | 0.842 | 0.842 | **0.58** |
| R4 | 0.605 | 0.601 | **0.60** |

Tab. 4 shows the computed $K_0$ coefficients with respect to the current thicknesses of Tegl from the centres of the adits to the top of Tegl (base of Quaternary). Furthermore, Tab. 4 shows $K_0$ values given by the equation (6) for arbitrarily chosen thickness of erosion.

*Tab. 4. Comparison of the $K_0$ values of the 3D model analyses and those calculated using equation (6)*

| **Adit** | **Thickness of the Tegl strata/m/** | **Thickness of Quaternary strata /m/** | **$K_0$ from numerical back analysis /-/** | **$K_0$ /-/ from Eq. (6) for erosion:** | | |
|---|---|---|---|---|---|---|
| | | | | **0 m** | **20 m** | **40 m** |
| R2 | 17 | 6,0 | 75 | 0.63 | 0.68 | 0.76 |
| R3 | 15 | 13.0 | 58 | 0.63 | 0.63 | 0.70 |
| R4 | 5.5 | 16.0 | 60 | 0.63 | 0.63 | 0.69 |

The numerical back analysis combined with the empirical equation for $K_0$ suggested that the erosion of Tegl at Královo Pole was between 0 to 40 metres. However, the $K_0$ values obtained by the advanced numerical analyses for the adits R3 and R4 are slightly lower than for the normally consolidated (0 m erosion) Tegl. The discrepancy could be caused by several reasons, which are discussed in the following.

**Uncertainty in input parametres**

The presented numerical back analysis depends on many geotechnical parametres and state variables. The most important mechanical phenomena influencing the results however are the inherent anisotropy (Mašín and Rott, 2014; through its effect on the results of the numerical back-analysis) and the soil strength (through its effect on estimating the erosion from the value of $K_0$).

The critical state friction angle is a function of mineralogy and grading. The Tegl strata are not consisting of pure clay fraction, and locations with, for example, a substantial amount of sand particles occur. They have an inevitable influence on the critical state strength expressed by $\phi'$. The previously published values for Tegl from several locations ranged between approx. 19 and 27 degrees (Svoboda, 2009, Boháč, 1999). The significant role of the friction angle in estimating the erosion thickness is clear when Eq. (6) is inspected. The coefficient (1-sin$\phi'$) in Eq. (6) is 0.67 for $\phi'$=19° and 0.55 for $\phi'$=27°. We adopted $\phi'$=22° as the most credible value, determined by triaxial test on specimens from the vicinity of the adits.

The inherent anisotropy was expressed as the ratio "$\alpha_G$" of horizontal and vertical moduli of the soil and in the model the ratio was assumed a constant: $\alpha_G = 1.45$. For adit R2 the dependence of $K_0$ on $\alpha_G$ was evaluated by Rott et al. (2015), and the result is presented in Fig.13. *In-situ*, soils may be influenced by several other factors, for example tectonic movements, diagenesis etc., which can cause spatial inhomogeneity of stiffness. This has already been proved for Tegl by *in-situ* measurement (Malát et al., 2015).

Considering the inherent uncertainties in the parametres and of the model, and the relatively small differences in $K_0$ values obtained (Table 4) the thickness of erosion of 20 to 40 metres is believed to be a plausible estimate.

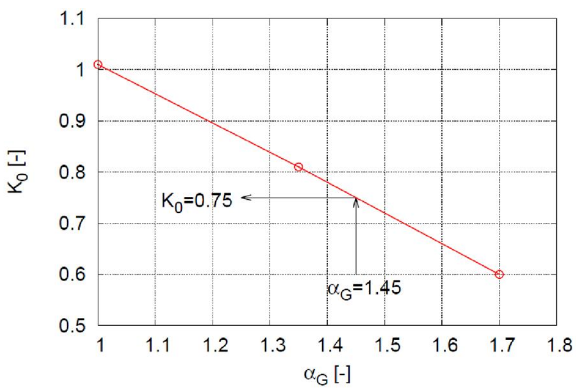


Fig. 13. *$K_0$-$\alpha_G$ relationship for R2 adit (Rott et al. 2015).*

**Simplification of geological development**

Closer to the surface $K_0$ is relatively more responsive to change in effective stress since related *OCR* change is higher in comparison with deeper part of a stratum. Cyclic erosion and re-deposition of Quaternary sediments or ground water level fluctuation even in range of a few metres affect $K_0$ significantly in upper part of Tegl clay. Moreover, the upper part of Tegl strata can be remoulded by climatic influences which can result in "loss" of overconsolidation. It is impossible to reconstruct the evolution of the geological basin completely. Hence, it seems plausible to consider the model and result of adit R2 more relevant, since the Tegl overburden is the thickest.

Assumption of constant $K_0$ during ageing

The numerical back analysis was based on the assumption that $K_0$ does not change during ageing. However the influence of ageing on $K_0$ is still a matter of dispute in the geotechnical literature.

The most discussed phenomena is the influence of the secondary compression on $K_0$. Several laboratory studies have been carried out with different results (Mesri and Hayat, 1993, Kavazanjian and Mitchell 1984, Gareau et al., 2006). The discrepancy is caused probably by the problem that during the tests zero horizontal strain has to be kept. Some of the older laboratory test supported the opinion that $K_0$ due to secondary compression gradually converges towards the value of 1.0 (e.g. Kavazanjian and Mitchell, 1984) but their conclusions have been criticized for unacceptable strains compensated during the tests (Holtz and Jamiolkowski, 1985). Others showed (e.g. Mesri and Hayat, 1993) that *K0* during secondary compression increases. Some newer laboratory data, however, that the $K_0$ was constant with time (Gareau et al., 2006). Construction of their device allowed the smallest strains of a soil in comparison with the older devices. Hence, in our model we assumed that $K_0$ is constant.

Apart from secondary compression other ageing effects are difficult to quantify. Nevertheless, while Tegl can be considered to be uncemented and non-expanding soil allows the assumption that K0 is not affected.

## 6. Conclusions

The thickness of erosion of Brno Tegl was determined using three independent methods: Baldwin – Butler's equation, Casagrande's method and the geotechnical numerical back analysis.

1. According to Baldwin–Butler's equation the thickness of erosion corresponds to approx. 180 – 270 m. The interval stems from the range of porosities (solidities) of undisturbed specimens as obtained from the oedometer tests.

2. Baldwin–Butler's equation has several deficiencies. Changes in porosity (solidity) due to unloading by erosion and/or reloading by further overburden are not captured. More importantly, it cannot allow for secondary compression, or the influence of other ageing effects. Moreover, Eq. (1) is created as a regression curve of collected data but these are in significant scatter and the equation is inaccurate in principle. These facts invalidate the estimations of the thickness of eroded layer using this method.

3. Casagrande´s method yielded the thickness of erosion in the interval from 130 to 770 m. The effect of ageing was tackled using the coefficient of secondary compression, which however must be based on laboratory data at the time scale completely different from the geological times. However, further effects of ageing, probably related to chemo-structural changes, are also likely to affect the compressibility of the clay in the laboratory oedometer tests needed for Casagrande's method. It is possible to conclude that the method is not capable of determining the erosion thickness of the overconsolidated Brno Tegl.

4. The geotechnical numerical back analysis combined with the empirical dependence of horizontal *in-situ* stress (or $K_0$) on *OCR* (true overconsolidation ratio) has the advantage of not having to deal with ageing.

5. The numerical model and the monitored mechanical behaviour around unsupported circular adits were found by $K_0$ optimisation. From the resulting value of the earth pressure coefficient at rest K the thickness of erosion was estimated to be in the range from 0 to 40 metres, the most probable values being 20 to 40 metres.

6. The proposed combination of a numerical model of a thoroughly monitored excavation and the empirical equation by Mayne and Kulhawy (1982) proved a suitable tool for studying the erosion of aged sediments elsewhere in the Carpathian Foredeep, or at other sedimentary basins.

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