# Application of Predictive Relationships of Swelling Effects for Roadbeds

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## Abstract

Impact of swelling can be observed in a wide range of civil and engineering structures. In the case of swelling in the subsoil of roadbeds, the destruction of the road pavement in cuts or collapse of slopes of cut is a frequent occurrence. Problems with deformation from swelling can also occur in the case of bridge abutments based on pile trestles. The unpleasant fact is that the impact of swelling can occur even after a long time. In the case of roads, high swelling pressures can lead to total destruction of the structure of the pavement even after years of operation. The susceptibility of soil to swelling can be described using swelling parameters. These parameters can be measured directly in the laboratory and in situ or indirectly estimated from empirical correlations. The paper describes the prediction of swelling processes using indirect measurements based on the methodology "Identification and solution of problems of soils prone to swelling" certified by the Ministry of Environment of the Czech Republic.

*Keywords: swelling processes, indirect prediction, roadbeds, empirical correlations*

## Introduction

The term swelling includes several phenomena or physical quantities with differently understood contents. As part of the assessment of the swelling effects on the subsoil, we consider swelling of the soil as a process of increasing the volume of the soil connected with the water content increasing or decreasing suction in the pores. Swelling is a 3D problem and thus is similar to the effect of hydrostatic pressure. The swelling effects result in the form of deformation (displacement, uplift), pressure which limited swelling or most often both occur. The swelling ability or susceptibility to swelling of the soil are referred to as swelling potential. The possible range of swelling depends on the type of soil, its condition and the external tension in situ. Swelling process occurs only when the fine-grained soil absorbs from its surroundings water (or just moisture). It means that the swelling of the soil lasts until the soil is saturated to the final level. But this can happen some after a long time, even after several years.

There are three types of swelling in soils:

Swelling with limited deformation (when tested in an oedometer). This is the case where swelling takes place with the zero deformation. If the resistance in situ is greater than swelling pressure the effects of swelling will not be recorded (and the swelling is technically insignificant).

Resistance swelling, deformation also occurs here during the ongoing swelling process. In laboratory conditions, a calibrated dynamometer and a deflection gauge are inserted above the sample. In situ, the resistance is the surrounding ground or structure. If the resisting load of the swelling layer is greater than the swelling pressure, then this is swelling with restrained deformation.

Free swelling, the swelling process is not resisted by any back-pressure and the soil shows only deformation. This case occurs in the region of the subsurface part of the slopes or in the bottom of open cuts.

## Prediction of swelling processes

As part of the TAČR TA04021261 [1] project, a certified methodology entitled "Identification and solution of the problem of soils prone to swelling" was also created. This methodology provides an effective tool to minimize possible damage to structures occur as a result of soil swelling and applies only to soil swelling of natural origin. In some places, the methodology also includes a discussion on the correctness of selected generally accepted opinions, test procedures or their interpretation, including a description of methods that may indicate the risk of a problem with soil swelling. It is case of:

Mineralogical identification – the mineralogical composition has a considerable influence on the swelling of soils and is therefore important for the description of the swelling potential and the soils themselves, e.g. [2][3]. However, from the point of view of geotechnicians and engineering geologists, mineralogical identification methods are uneconomical and impractical.

Direct measurement of swelling soils - multiple methods are used here and generally require the use of special equipment. One of the first direct methods was published by Alpan [4], however the most widespread is the measurement of swelling potential under different conditions using an oedometer

Engineering-geological and special maps are the basic guide for identifying swelling soils [5], [6]. Their form can greatly vary depending on the size of the area, the details of the division or focus (risk for foundations construction, raw materials searching). In addition to the scale and details of the division, an important aspect in the map creation is the chosen criteria defining the individual sub-areas. In 2017, as a part of project TA04021261 [1] was created a special engineering-geological map of soil swelling potential for the area of the northern and western Bohemia parts (scale 1:50,000). This map is a well source of initial information about properties of soils in terms of swelling potential that can be expected in places of planned construction.

Indirect methods – they are mainly based on the results of index tests, moisture, grain size, bulk density and consistency limits. In the case of indirect measurements, the advantage is the possibility of using simple laboratory tests or just knowing the basic properties. Holtz and Gibbs [7] determined the swelling potential based on the plasticity number and moisture on the liquid limit. Their empirical sector chart was supplemented in 1981 with areas of the selected clay minerals occurrence [8] – Figure 1. Many empirical graphs showing the expected level of soil swelling susceptibility can be found in the literature (an overview is provided by e.g. [9]. In general, however, none is universally valid for all soils or localities. With the computer technology development and using methods such as nonlinear regression, neural networks, correlation analysis, regression analysis and sensitivity analysis, the original empirical relationships based on indirect measurement were supplemented by the influence of other variables. The advantage of using these new indirect methods is also the weight determination of individual parameters and the assessment of the reliability of the derived relationships.

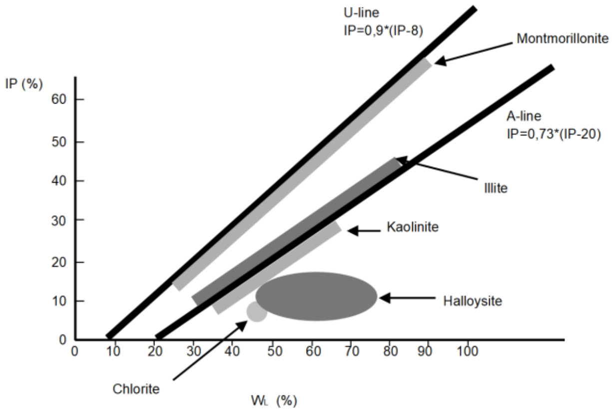


Fig. 1. Location of common clay minerals on Casagrande´s plasticity chart (after [8])

**Indirect prediction of swelling processes**

The methodology "Identification and solution of the problem of soils prone to swelling" also contains predictive relations for the calculation of individual parameters of swelling. The relationships were obtained by evaluating an extensive database of measurements carried out in the Czech and Slovak Republics (tuffitic clays, claystone, cretaceous marlstones) by the company of GeoTec-GS, a.s. and the Czech Technical University in Prague using neural networks, multiple correlation, regression analysis and sensitivity analysis with the professional statistical program QCExpert [10]. The prediction relationships are primarily designed for quick assessing whether the soil is or is not prone to swelling (without the necessity for time-consuming tests) and can only be used for soils in the natural environment without any additional additives (lime, cement, etc.).

Prediction of swelling pressure (swelling pressure with deformation prevented in oedometer) $\sigma_b$ (kPa):

$$\sigma_b = (K + 0{,}001)^{-0{,}048} \cdot (W_K - W_n)^{0{,}101} \cdot I_P^{1{,}443} \cdot I_A^{1{,}757} \cdot (1 + D_{05})^{-0{,}265} \tag{1}$$

where: $K$ – soil toughness ($K$ = 0) (mm/MN), $W_K$ – terminal water content of swelling (%), $W_n$ – initial water content (%), $IP$ – plasticity number (%), $I_A$ – index of colloidal aktivity (-), $D_{05}$ – proportion of inert (non-swelling) 0.5mm grains above the grading curve (%).

The value of relative deformation at swelling $\varepsilon_b$ (%) is based on the following prediction relationship:

$$\varepsilon_b = K^{0{,}0159} \cdot W^{-0{,}562} \cdot (W_K - W_n)^{0{,}805} \cdot I_P^{0{,}369} \cdot I_A^{0{,}167} \cdot (1 + D_{05})^{-0{,}503} \tag{2}$$

where: $K$ – soil toughness ($K$ = 1·1025) (mm/MN), $W_K$ – terminal water content of swelling (%), $W_n$ – initial water content (%), $IP$ – plasticity number (%), $I_A$ – index of colloidal aktivity (-), $D_{05}$ – proportion of inert (non-swelling) 0.5mm grains above the grading curve (%).

The prediction relationship for water content at shrinkage limit $W_s$ (%) is as follows:

$$W_S = (W_L + I_P)^{0{,}656} \cdot I_A^{-0{,}0338} \cdot (1 + D_{05})^{-0{,}131} \cdot (1 + V_{CA})^{-0{,}057} \tag{3}$$

where: $W_L$ – water content at liquid limit (%), $IP$ – plasticity number (%), $I_A$ – index of colloidal aktivity (-), $D_{05}$ – proportion of inert (non-swelling) 0.5mm grains above grading curve (%), $V_{CA}$ – content of calcium carbonate (%).

The prediction relationship for final water content $W_k$ (%):

$$W_K = (K + 0{,}001)^{0{,}0025} \cdot (W_L + I_P)^{0{,}774} \cdot I_A^{-0{,}464} \cdot (1 + D_{05})^{-0{,}114} \cdot (1 + V_{CA})^{-0{,}1041} \tag{4}$$

where: $K$ – soil toughness (mm/MN), $W_L$ – water content at liquid limit (%), $IP$ – plasticity number (%), $I_A$ – index of colloidal aktivity (-), $D_{05}$ – proportion of inert (non-swelling) 0.5mm grains above the grading curve (%) (if the grains above 0.5 mm are also swollen, the value of $D_{05}$ = 0), $V_{CA}$ – content of calcium carbonate (%).

The prediction of relative linear deformation at shrinkage $\varepsilon_S$ (%) is based on the following relationship:

$$\varepsilon_S = W_S^{0{,}171} \cdot (W_n + W_S)^{0{,}752} \cdot I_P^{0{,}0544} \cdot I_A^{0{,}0676} \cdot (1 + D_{05})^{-0{,}426} \tag{5}$$

where: $W_S$ – water content at shrinkage limit (%) (possibly humidity corrected for shrinkage $W_S$* (%) ), $Wn$ – initial water content (%), $IP$ – plasticity number (%), $I_A$ – index of colloidal aktivity (%), $D_{05}$ – proportion of inert (non-swelling) 0.5mm grains above grading curve (%).

**Requirements for laboratory tests**

The following properties are determined in laboratory on specimens:water content at liquid limit $W_L$ (%),

- water content at plastic limit $W_P$ (%)
- percentage share of grains over 0.5 mm $D_{05}$ (%) (for assessing the swelling capacity of larger grains);
- percentage share of grains 0.002 mm $D_{002}$ (%);
- calcium carbonate content $V_{CA}$ (%)

The plasticity number $I_P = W_L - W_P$ and the index of colloidal activity $I_A = I_P / D_{002}$ are calculated from the above-mentioned data. For that reason the grading curve of the soil being assessed is necessary for the determination of the index of colloidal activity. If only fine-grained soil with the absence of fractions over 0.5mm is in question, the actual value $D_{002}$ is taken into consideration. It is necessary to assess whether grains above 0.5 mm ($D_{05}$) can also be swelling or are inert (non-swelling), for example clayey sands, etc. For non-swelling grains above 0.5 mm, correction of the grain size curve to 0.002 mm is necessary.

**Assessment of the effects of swelling under the road**

This part shows the practical use of prediction relations for determining the swelling pressure of a pavement with a cement concrete cover. The schematic profile of the layer under the road is shown in Figure 2. and in Table 1 are given the input data for assessing the swelling position in the subgrade.

Tab. 1. Input data for the assessment of the swelling position in the bedrock

| $W_n$ | $W_L$ | $W_P$ | $D_{002}$ | $D_{05}$ | $V_{ca}$ | $I_P$ | $I_A$ |
|---|---|---|---|---|---|---|---|
| 10 % | 58% | 25% | 17 | 0,0 | 0,0 | 33% | 1,94 |

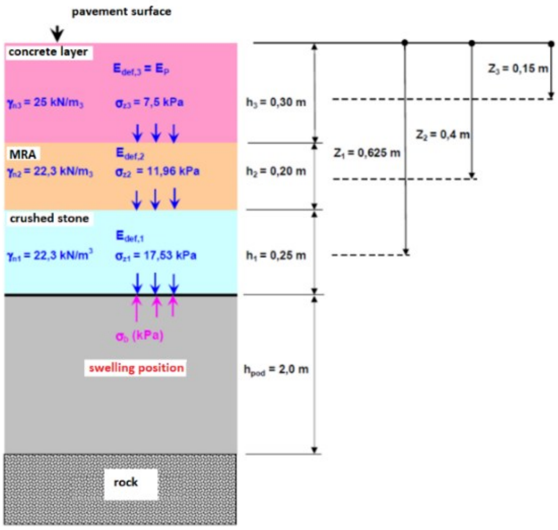


Fig. 2. Schematic profile of the roads

According to prediction relation (4), the final humidity $W_K$ is 23.7%, the swelling pressure value $\sigma_b$ given by relation (1) is 903 kPa. Furthermore, a plate load test on 0/32 crushed stone and on mechanically reinforced aggregate (MRA) was evaluated, the equivalent modulus of elasticity (for the relevant stress $\sigma$ and deformation y) and the modulus of elasticity $E_{def1,i}$ of the given layer were calculated depending on the stress σ (Figure 3).

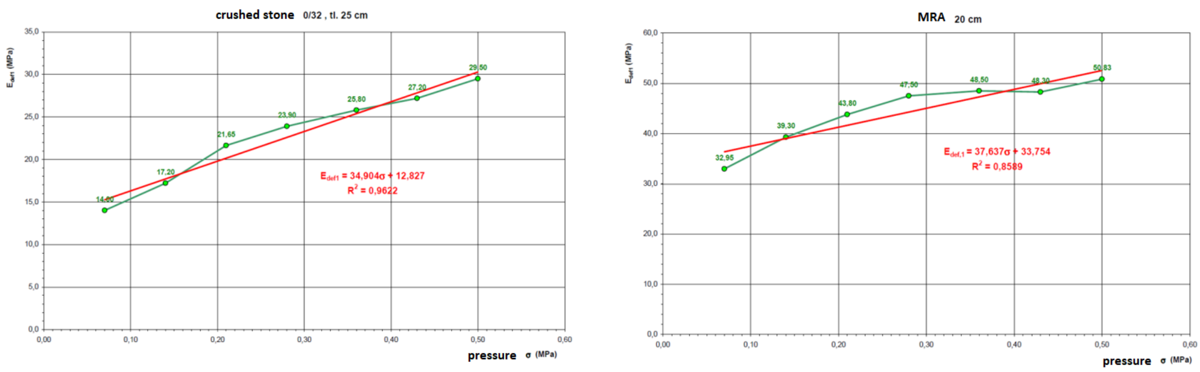


Fig. 3. Linearized dependence of $E_{def1,i}$ on stress σ for gravel pit and mechanically hardened aggregates

The weighted average determines the characteristic value of the deformability modulus $E_{def,ch}$ of resisting layers (gravel, mechanically reinforced aggregate, concrete layer):

$$E_{def,ch} = \frac{E_{def,1} \cdot h_1 + E_{def,2} \cdot h_2 + E_{def,3} \cdot h_3}{h_1 \cdot z_1 + h_2 \cdot z_2 + h_3 \cdot z_3} = 548 \text{ MPa} \tag{6}$$

where: $E_{defl,i}$ modulus of elasticity of the given layer Pa), $h_i$ thickness of i-th layer (mm).

Next, the deformation of the layer $\Delta h$ is determined:

$$\Delta h = \frac{\sigma_z}{E_{def,ch}} h = 0{,}00239 \text{ mm} \tag{7}$$

where: $\sigma_z$ vertical geostatic stress from the overburden (kPa), $h$ thickness of resisting layer (position) (mm), $E_{def,ch}$ deformability modulus of resisting layers (Pa)

The stiffness of the entire overburden (resisting layers: gravel, MRA, concrete layer) given by relation (8) is introduced into the prediction relations for swelling pressure (1) and deformation (2). Substantioal is the swelling pressure which is used for further assessment - for example, in FEM modeling.

$$K = \frac{\Delta h}{0{,}001 \cdot \sigma_z \cdot S} = 0{,}136 \text{ mm/MN} \tag{8}$$

where: $\Delta h$ layer deformation (mm), $\sigma_z$ vertical geostatic stress derived from overburden (kPa), $S$ sample area (m2), here contact area 1.0 $m^2$

The swelling pressure $\sigma b$ is given by relation (1) and for stiffness $K$ = 0.136 mm/MN it is 714 kPa. Furthermore, it is necessary to assess the condition before and after swelling (for example using FEM. If we know from the prediction that the in situ can swell, the area prone to swelling should be predicted. In practice, it is very difficult and it is necessary to assess local areas gradually. Given that there are countless possible combinations, it is advisable to choose the extreme part of the subsoil under the road for the first assessment. Based on the assessment for a swelling pressure of 714 kPa, damage will occur to the concrete road surface with technically significant uplifts.

**Conclusion**

If, on the basis of the predicted swelling pressure σb, it is found that the concrete pavement will be broken for the given values (i.e. the swelling problem is current), it is necessary to propose appropriate measures. One possible solution is to use an asphalt surface instead of a cement concrete surface. The structural layers should have an increased thickness so that after milling the undulating asphalt surface, the strength of the structural layers (even if deformed) is preserved, and the final surface is made only after the swelling process has taken place (the swelling process stops when the final moisture content of the swelling soil is reached).

The relationships presented above are for linear swelling. In situ, however, the swelling process does not take place linearly, but spatially (volumetrically). For swelling pressure, the effect of neglecting the 3D effect is insignificant. This is no longer the case with proportional deformation. The biggest problem, however, arises in determining the thickness of the layer that can be carbonated or where the possible irrigation reaches. Carbonation in situ (in the field) rarely occurs continuously (as in laboratory conditions) and depends on precipitation. Sometimes, when the morphology is changed, for example by building a road cut, there will also be a change in the hydrogeological conditions. Determining the thickness of the layer that can be carbonated or where the possible dewatering reaches can be determined, for example, by the finite element method (FEM) in the stationary flow regime in the given environment.

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