Crystal growth and geotechnics

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Summary

Crystal growth is a mechanism leading to extreme expansive phenomena and very high swelling pressures against engineering works. The lecture presents three cases which resulted in damage and expensive corrective measures in a tunnel excavated in anhydritic claystone, in a viaduct founded on piles and in the compacted access embankments to another viaduct. In the first two cases gypsum precipitation in rock discontinuities led to severe tunnel floor heaving, to very high pressures against tunnel lining and to the unexpected heave of the central pillars of the viaduct. The third case describes the massive growth of ettringite and thaumasite in embankments reinforced by Portland cement. As a result the bridge was axially compressed and damaged. In addition the heave rates of embankment surface required a full excavation of upper embankment layers and the construction of additional supporting structures. The paper also describes the fundamentals of gypsum crystal growth and presents a coupled Hydro Mechanical and Chemical model formulated in a porous media. The model was applied to interpret and simulate the heave experienced by the viaduct. Modelling capabilities were checked against the long term history of viaduct heave and also, against the foundation response when the vertical load from a surface embankment was added to counteract swelling.

1. Foreword

Signor Presidente dell’Associazione Geotecnica Italiana, cari colleghi e amici, Signore e Signori,

Un giorno, ormai lontano, del secolo scorso, pensai che la Geotecnica, in quanto scienza sperimentale e teorica, con la sua breve storia scientifica, la sua complessità, i suoi legami con il mondo infinitamente vario della Geologia, e con l’inevitabile necessità di astrazione che essa impone, avrebbe offerto sufficienti incentivi a un giovane ingegnere civile, convinto che tutto fosse possibile e che il tempo fosse illimitato...

In seguito, la realtà si presentò in tutta la sua crudezza. Sogni e ambizioni divennero percorsi impraticabili sotto lo sguardo impassibile di queste mete ideali, sempre lontane, sempre inaccessibili, sempre brillanti.

Fortunatamente il tempo e la generosità altrui ti conducono verso cammini impensabili che rendono gradevole il tuo lavoro. Il fatto stesso che io oggi sia qui è buon esempio di quanto dico.

Devo riconoscere la grande felicità che mi ha dato l’invito a presentare la decima Conferenza annuale “Arrigo Croce”. E non solo per la dose di “ego” che ciò ha significato a livello personale, ma anche per un sentimento di gratitudine e ammirazione nei confronti di questo grande paese.

Con riferimento alla Geotecnica italiana, ho sempre seguito con grande attenzione la forza e la brillantezza dei suoi contributi teorici, il suo genio costruttivo, pratico e innovativo, e la grande diversità e originalità delle sue scuole di pensiero. Tutto ciò in un contesto geologico di enorme varietà, dove coesistono l’alta montagna, le pianure alluvionali, i delta, i terreni vulcanici e i materiali di struttura complessa. Senza trascurare la necessità di prendersi cura di un patrimonio storico e artistico eccezionale.

Certamente il Professor Croce era assolutamente cosciente di questa diversità e della necessità di affrontare la geotecnica con creatività, senza per questo rinunciare ai principi fondamentali e alla base fisico-matematica che la caratterizza.

Ho letto alcune delle sue pubblicazioni e altri scritti raccolti nel lungo articolo del Professor Jappelli, dedicato alla sua vita e pubblicato sulla Rivista Italiana di Geotecnica.

Professor Croce was particularly gifted in extracting the nature of Geotechnics and its many aspects. The following brilliant sentence:

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1 Proceedings of a symposium on the “Geotechnics of Structurally Complex Formations”, 1977
2. Tunnels in anhydritic formations

Heave and structural damage in sulphated natural formations have often been associated with tunnelling. Triassic claystone formations in Central Europe (Baden-Württemberg, in Germany; Jura Mountains, in Switzerland) have been crossed, for decades, by tunnels. A significant proportion has experienced severe heave problems, which have been described by several authors [Kovari et al., 1988; Wittke, 1990, 2006; Wittke-Gattermann and Wittke, 2004; Madsen et al., 1995; Anagnostou, 1993, 2007; and Anagnostou et al., 2010].

The origin of the problem is often described as a transformation of anhydrite (CaSO₄) into gypsum (CaSO₄ × 2H₂O). The two added water molecules imply, at a molecular level, a theoretical increase of 62% in volume, which is then made responsible for the observed heave [Sahores, 1962; Einfeld and Götz, 1976; Einfeld, 1979, 1996; Wittke and Péra, 1979; Serrano et al., 1981; Zanbar and Arthur, 1986; Madsen and Nüesch, 1991; Kolumbas, 2005 and Wittke, 1990, 2006].

Anhydrite dissolution and subsequent crystallization of gypsum in the presence of clay minerals has also been suggested by Madsen et al. [1995] and Nüesch et al. [1995]. Expansive clay minerals (corrensite, smectite, chlorite) are also present in the Triassic sulphated formations. A given tunnel may experience swelling phenomena associated with the anhydritic gypsum presence and also, to the swelling of expansive clays. If available data on heave and swelling pressures is collected (Fig. 1) and classified on the basis of the type of rock involved, an interesting result emerges: the intensity of the swelling phenomena is significantly higher in sulphated formations when compared with clay or marl rocks. Heave, which is concentrated on tunnel floors, reaches values of 1 m in the case of sulphated rocks. Swelling pressures in excess of 5 MPa have been recorded in those formations. Heave tends to develop at constant rates in time and often, there are no signs if reaching limiting values (Fig. 2).

Calcium sulphate rocks also occur in extensive zones of the Iberian Peninsula and Balearic Islands as outcrops and buried formations. According to Ribas and Macau [1962] gypsiferous rocks outcrop in 7.2% of the Spanish territory and they are mainly located in the eastern part of the country (Fig. 3). This is, certainly, a very large area. Taking into account buried formations – particularly the materials from the Keuper –, the relevance of calcium sulphate rocks in the infrastructures of Spain increases substantially. In fact, tunnelling works in Spain affect sulphated formations relatively often. Therefore, a proper knowledge of main characteristics of these formations is essential to evaluate their behaviour in tunnels.

Anhydrite is sometimes reported to exist at depth and gypsum at the surface is explained by an exposure of anhydrite by erosion and tectonic process and a concurrent transformation of anhydrite into gypsum under isovolumetric conditions [Örtí et al., 1989]. According to these authors, this “secondary” gypsum is the dominant facies in most gypsum outcrops in Spain previous to the Pleistocene.

The groundwater in both Triassic and Tertiary formations in Spain is highly mineralized. Some cases from the Ebro and the Tajo basins in which deep excavations have been executed or tunnels are planned were analyzed in terms of saturation index for gypsum, SI, using data from different sources.

The results of these analyses are presented in figure 4. SI = 0 indicates equilibrium conditions, SI < 0 reflect subsaturation and SI > 0 reflect supersaturation. When equilibrium is not found SI indicates in which direction the process may go: for subsaturation dissolution is expected, and supersaturation suggests precipitation.

Then, precipitation of calcium sulphate, particularly of gypsum, due to supersaturation of natural sulphate-rich waters is possible as a result of either a fall in temperature – that causes a decrease in gypsum solubility –, or the exposition to a ventilated dry environment, which causes water evaporation and, therefore, an increase in calcium sulphate concentration.

Three tunnels excavated in the Ebro basin, Taragona province (Lilla, Camp Magré and Puig Cabrer tunnels) experienced severe heave problems immediately after construction. The three tunnels belong to the Madrid-Barcelona high speed railway line and are located at close distances to each other. Lilla crossed Eocene-Oligocene formations; Camp Magré is excavated in the same formation but in the proximity of limestones overlying the claystones; Puig Cabrer, however, crosses Triassic gypsiferous formations. Lilla experienced the strongest heave and, later, high swelling pressures against the lining, which exceeded 6 MPa in some points.

The plot of evolution in time of floor heave at some points in Lilla tunnel (Fig. 5) reproduces swelling patterns observed in tunnels in Central Europe. In Lilla, the heave rates are very high in some points (600 mm/yr!). A total of 1500 m out of the total tunnel length (2000 m) was severely affected by a rapid development of heave.

This phenomenon caused a major disruption in the tunnelling works and it led to a period of research on the causes of the observed behaviour and, also, on the necessary modification of the tunnel project to allow for a controlled and safe operation of the railway line.

The investigation followed initially a methodology derived from current knowledge on expansive materials. Samples were taken from the tunnel floor...


Fig. 3 – Schematic distribution of gypsiferous outcrops in Spain, including materials from the Triassic to the Neogene [Riba and Macau, 1962].

Fig. 4 – Saturation index for gypsum in natural waters from some Spanish evaporite deposits as a function of the temperature.

Fig. 4 – Indice di saturazione in gesso nelle acque naturali di alcuni depositi evaporitici spagnoli, in funzione della temperatura.
at various depths (0-10 m) and they were subjected to wetting under confining stresses. Oedometer and triaxial cells were used in these tests.

Some results are reviewed in the next section. In parallel, a heave scenario was developed, with the purpose of generating a geotechnical model capable of making predictions. In fact, the railway administration asked basically the following questions:

- Why did it happen? Explain the main phenomena
- Recommend the best remedial measures
- Design them
- Estimate the long term performance of the solution

A geotechnical model eventually evolving into a computational model could be of some value to answer these questions.

3. Laboratory observations

A large laboratory testing program was undertaken. Only a few relevant results are reported here. High suction values, in the range 20-40 MPa, were determined by means of a transistor psychrometer [Woodburn et al., 1993] on specimens from cores recovered in boreholes drilled from the floor of Lilla tunnel. It was originally thought that a possible explanation for the heave observed was the wetting of the rock formation under the tunnel floor. The hypothesis was that tunnel construction (by drill and blast) in a rock mass probably subjected to high horizontal stresses, had damaged the rock immediately under the horizontal floor. This damage would have resulted in the opening of discontinuities allowing groundwater to percolate, and, in so doing, wetting the claystone. It was then natural to perform wetting tests on undisturbed core specimens.

The results of one of these tests, performed in an oedometer cell, is shown in figure 6. A small confining stress was imposed. The specimen had a clay content of 55%, an anhydrite content of 42% and a gypsum content of 3%. The initial water content was 7%. The test lasted more than seven months. Heave progressed slowly and it apparently stopped after 5 months. The important information, however, is that measured heave strains were quite small. In fact, average swelling strains in the active zone, in areas of high recorded swelling were close to 15%. The maximum value observed in figure 6 is 0.6%, far from the values which explain the field records.

It was then decided to perform a different type of test, having this time in mind the idea of discovering basic mechanisms. The rock matrix was grinded to a fine powder and a reconstituted specimen was prepared by compacting the powder in five layers into an oedometer cell. A high static vertical stress (25 MPa) was applied to the five layers. The void ratio finally achieved was low (0.42). The degree of saturation was low (17%) and, therefore, the specimen was ready to absorb water. This powdered specimen had the following composition: Clay: 58%, Anhydrite: 25%, Gypsum: 17%. The test was carried out in this case in a suction control oedometer apparatus and the following test protocol was applied: 1) Wetting with a solution saturated in Gypsum (CaSO₄·2H₂O–2.15 g/l); 2) Drying with relative humidity control at RH = 60%; 3) Drying with relative humidity control at RH = 40%; 4) Wetting with a solution saturated in Epsomite (MgSO₄·7H₂O–710 g/l); 5) Drying with relative humidity control at RH = 70%; 6) Drying with relative humidity control at RH = 60%; 7) Drying with relative humidity control at RH = 43%. A small vertical stress (40 kPa) was applied to the specimen. The response of the specimen is given in figure 7. The testing steps can be followed in figure 7.
The first wetting-drying cycle induces a swelling-shrinkage response of the specimen. This is a “normal” behaviour. The intensity of shrinkage increases with the applied suction, again an expected result. The second wetting, this time with a solution of epsomite (a salt with a high concentration in saturated solutions) leads also to an expansion of the previously dried sample. Beyond this point drying the specimen does not lead to a new shrinkage. The specimen remains stable for RH = 70% and RH = 60%. When suction is increased again (RH = 43%), the specimen swells. This justifies the expression “swelling under drying” shown in the figure. Swelling proceeds for some time and it stops because there was a limited supply of liquid at the base of the specimen.

An examination of the specimen after the test provides an explanation for the observed swelling response: crystals grew at the specimen-porous disk interface (but not inside the specimen). The expansion associated with this phenomenon compensated the expected shrinkage during the final phases of increasing applied suction. The explanation given is that evaporation of water, forced by the dry atmosphere imposed on the upper face of the specimen, led to supersaturation of the liquid percolating upwards through the specimen and then conditions for salt crystallization were reached. Crystals grew at the interface because some open space was probably available at this location. This space was forced open by the accumulation of crystals. Crystals did not develop inside the specimen due, perhaps, to the small dimension of pores.

The third test reported here is shown in figure 8. Undisturbed cores (11 cm in diameter; heights in the order of 20 cm) recovered in boreholes performed from the tunnel floor were wetted at one end in the manner indicated in the sketch in figure 8. The upper face was directly exposed to the atmosphere of the laboratory (RH varying between 50 and 60%). The lateral cylindrical surface of the core was covered by an impervious plastic sheet in order to force a vertical flow through the sample. In fact, the applied flow gradient between the bottom of the specimen, maintained at zero suction, and the upper surface, maintained in a “dry” environment, was quite high. Different water compositions were applied at the bottom of the specimen, but all of them led to essentially similar phenomena. Tests lasted several months.

Cores experienced a continuous degradation. Fissures and cracks of increasing dimension were observed. A closer examination (Fig. 9) indicated that fissures were often filled with gypsum crystals. Degradation apparently progressed from the upper part of the cores, where water was being evaporated, towards the lower saturated bottom. It was interpreted that this migration of the observed degradation was following an evaporation front which displaced from the upper face towards inner positions within the specimen as fractures grew in size.

Measured radial strains in some of the tests performed are shown in figure 10. Volumetric strains accumulated at the end of the testing period (five months) are quite high (6%-17%). These values are consistent with the large average swelling strains which explain the recorded heave of the tunnel floor. It was also observed that the patterns of crystal growth within the cores essentially followed some initial discontinuities of the rock.

The set of experiments described seems to indicate that the heave observed in the tunnel could be attributed to the growth of gypsum crystals in discontinuities. Further observations in cores recovered within the “active” depths indicated the presence of crystals which, in occasions, tended to arrange in “rosettes” as shown in figure 11.

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**Fig. 7** – Free swelling test on powdered compacted specimen. Wetting and drying under relative humidity control.

**Fig. 7** – Prova di rigonfiamento libero su un provino di materiale polverizzato e ricompatto. Imbibizione ed essiccamento in condizioni di umidità relativa controllata.

**Fig. 8** – Test arrangement for free swelling of undisturbed unconfined cores.

**Fig. 8** – Schema di prova di rigonfiamento libero su carote indisturbate non confinate.
4. The initial conceptual representation and heave modelling

The preceding comments and experimental results suggest that the swelling observed in sulphate-bearing soft rocks is due to crystal growth, and, in particular, gypsum crystal growth. To be active, this mechanism requires two conditions: the supersaturation of the pore water and the presence of some open space to allow for the crystal development. One mechanism leading to water supersaturation in sulphate salts is evaporation. The second condition is probably met at discontinuities, interfaces and joints. Field and laboratory observations seem to support this conclusion.

Figure 12 illustrates these comments. A virtual “experiment” is suggested in the figure. A fissured piece of sulphate claystone is subjected to an upward flow from a saturated bottom boundary towards an upper evaporating surface, which will mimic the exposed tunnel floor.

The model reproducing the assumed mechanism of crystal growth was as follows: the fissured claystone is represented as a porous media. A formulation for a coupled flow-deformation analysis for saturated, as well as unsaturated conditions, is described in Olvela et al. [1994; 1996]. The model led to the Finite Element Program CODE_BRIGHT [DETCG, 2010] which is taken as a reference computational tool for the modelling of crystal growth. Flow will be induced by an evaporating boundary (the tunnel floor). Ground water has a given concentration of dissolved calcium sulphate. Field data shows that this concentration approaches saturation conditions, \( c_{sat} \). Whenever water evaporates, the mass of gypsum precipitating will be given by:

\[
dm = dV_w c_{sat}
\]

(1)

where \( dV_w \) is a water volume evaporating. Equation (1) is the key assumption of this scenario. A second important assumption is to accept that induced volumetric deformations will not correspond exactly to the precipitated gypsum mass, because of two possible mechanisms or circumstances: a “jacking” action of the precipitated crystal (it will increase the strict volume of gypsum mass) or a crystal filling an already open-space (it will reduce the swelling effect of the precipitation). Therefore, volumetric deformations are calculated through:

\[
\dot{\varepsilon}_v = \frac{\gamma}{\rho_c} m
\]

(2)

where \( \rho_c \) is the density of calcium sulphate and \( \gamma \) is a coefficient that was made dependent on confining stress. The entire formulation is described later in more detail, but within the context of a modified scenario for the reasons given below.
A discretized cross section of Lilla tunnel is given in figure 13. The initial state of stress is approximated by a $K_0$ value of 2. Tectonic considerations are behind this choice. A water table was assumed at the lower boundary of the discretization. The rock was simulated by means of an elastoplastic model. Embedded discontinuities in elements simulate the development of cracks when tensile strains develop (details of this formulation are given in OLIVELLA and ALONSO [2008]; and PLAZA [2008]). An elastic concrete lining was introduced. Tunnel construction is first simulated by removing stresses on the tunnel boundary. Evaporation towards the tunnel flow is then allowed. The tunnel atmosphere is characterized by a RH = 60%. Immediately after construction plastic deformations are calculated below the tunnel floor, as indicated in figure 14. Associated changes in fracture aperture result in a sudden modification of permeability, also indicated in figure 14.

Further insight into the phenomena occurring below the tunnel floor is given by the calculated changes in porosity (Fig. 15). Immediately after tunnel opening, changes in porosity reflect the plastic strains associated with the stress removal. After some time (2 yrs) porosity increase is localized in the position of crystal growth, because of the expansive volumetric strains associated with crystal development. Also indicated in figure 15 are two vertical sections where vertical strains have been calculated (Fig. 16).

The result under the tunnel axis is similar to the case of the half space. Volumetric strains mainly accumulate at the position of the evaporation front. The profile under the side wall is more complicated because the hydraulic regime, as well as the stress distribution, are now more complex. Note also that some compressive strains may now develop because of the confining effect introduced by the rigid lining and the rock cover. In fact, this type of behaviour was also detected by the “in situ” high precision extensometers. The deformed tunnel shape is shown in figure 17.
The model predicts that the development of heave is essentially linear in time (Fig. 18). This is also in accordance with the observations in the field. The calculated record in figure 18 corresponds to a total heave of 10 cm in two years. Some cross sections in Lilla tunnel had experienced similar heave values although larger displacements were observed in most locations.

The model seems to capture the observed swelling but it has an important limitation: the calculated volume of precipitated gypsum is very small. This is a consequence of the low solubility of gypsum in water. The $\gamma$ coefficient has to be increased to unrealistic very high values to reproduce the field swelling-time records.

On the other hand, in some of the swelling tests performed, crystals were shown to grow in fully submerged parts of samples. It was suspected that evaporation could not be the main reason behind crystal growth. Further evidence was gained when Lilla rock
fragments were submerged in distilled water inside hermetic containers. In one of the experiments, acicular crystals grew in the manner indicated in figure 19, one year after the initiation of the test. Evaporation was not possible in this experiment.

This evidence and the unique case described in the next section led to an alternative scenario for crystal growth and to a new formulation of a computational model.

Lilla tunnel was re-designed and a reinforced circular section was built. After an evaluation of the potential advantages of designing a “yielding” support, capable of reducing swelling pressures, it was decided to adopt a “full support” design. Expected short term swelling pressures are higher in this case but a yielding support may lead, in the long term, to an undesirable increased damage of the host anhydritic claystone.

5. Pont de Candí Bridge

5.1. Bridge performance

One of the main bridge structures of the recently built Madrid-Barcelona high speed railway in Spain is the Pont de Candí viaduct (Fig. 20). It is located between Camp Magré tunnel and Lilla tunnel. The bridge was built in the period 2001-2002. The construction of the viaduct finished in July 2002. The bridge deck was set in place by a pushing method. The double railway line is supported by a prestressed continuous trapezoidal concrete box girder, 413 m long, having 10 spans (35 m; 8 × 43 m and 34 m), which is anchored in one abutment (Fig. 21). In plan view the bridge has a constant curvature radius of 7.250 m and a longitudinal descendent constant gradient of 1.815% from abutment E1 to E2 (Fig. 21).

The upper deck is supported by long pillars (pillars P1 to P9). The height of the pillars varies from 11 m to a maximum height of 55.9 m in the centre of the valley (pillar P5). The pillars have a rectangular box cross section of 3.5 × 5.9 m at the top. Each one of the pillars is supported by a group of 3 × 3 large diameter (Ø = 1.65 m) bored piles, 20 m long in average, as shown in the longitudinal profile given in figure 22. In pillars P3 to P8 the pile cap thickness is 5.5 m.

Systematic levelling of the railway tracks, carried out by the railway administration immediately after construction, revealed the progressive development of vertical displacements of the viaduct central pillars, especially pillars P5 and P6. Heave profiles plotted in figure 23 for two dates show the pattern of heave, which mainly affected the central pillars. Heave accumulated at rates ranging from 5 to 10 mm/month.

The reason for this anomalous behaviour was not clear. The massive deep foundations of pillars could hardly experience significant vertical displacements induced by a shallow swelling layer. In fact, piles were socketed in a rigid stratum of anhydritic claystone, which could resist swelling strains possibly existing along the upper part of pile shafts.

Levelling records of the bridge deck indicated that pillars were also experiencing small rotations and not only a vertical displacement. Horizontal displacements in longitudinal and transversal direction were also measured at the top of central pillars (Tab. I). The deck was supported by means of bearings that were free to move in the longitudinal direction. The observed pillar rotation was a result of the rotation of the pile cap. Table I indicates that the distance between pillars P5 and P6, at the deck level, was opening at an average rate of 12 mm/year. However, the bearings restrained the motion in the transversal direction. As a result, the pillar rotations led to a relative motion between deck and pillar, which was resisted by the restraining system of the deck bearings. The support system was close to its strength limit in August 2008. The progressive heave was compensated at the bridge deck level by shortening the bearing supporting structures. There is a limit for this solution and it became clear...
that a geotechnical remedial measure had to be found.

5.2. Geological background

The valley crossed by the bridge is located in the eastern boundary of the large Ebro River depression. Sediments belong to old Tertiary formations. Piles were founded on the so-called “Red Formation”, which belongs to the lower-medium Eocene. Red claystones, with variable contents of gypsum and anhydrite, dominate the red formation. Thin levels of sandstone, poorly cemented, are also present at the bottom of this formation. At the top, white fibrous veins of gypsum exhibiting a high lateral continuity are found. Often these veins are arranged in a lattice, which is the result of tectonic thrust. Covering these materials and partially filling the paleorelief, alluvium and colluvium soils of moderate thickness are found. These Quaternary deposits are described as a mixture of a silty and sandy clay matrix and limestone gravels and boulders, which have not experienced long transport distances.

Structurally the valley is located between the Ebro River depression and a mountain range, sub-parallel to the Mediterranean coast, known as the “Prelitoral” chain. Horst (elevations) and graben (trenches) structures affect Tertiary formations, as well as more ancient rocks. The Pont de Candi val-
ley is the result of intense tectonic action, which resulted in faults crossing the valley and folded claystones.

The geological profile along the bridge, established with the help of data provided by the borings performed, will now be described in more detail (from top to bottom; refer to Fig. 22).

### 5.2.1. Quaternary sediments

A relatively thin mantle of colluvium soils (its average thickness is around 3 m) covers most of the valley slopes and bottom. Its maximum thickness is 6 m, between pillars P4 and P5. The colluvium is a mixture of a low plasticity silty clay matrix and subangular gravels and blocks whose size varies widely from a decimetre- to a meter-based scale. A stream of alluvium soils covers the bottom of the valley. Its composition is similar to the colluvium although blocks tend to be smaller and rounded; the sand/silt content of the fine matrix also increases.

### 5.2.2. Tertiary formations

An upper brown plastic clay layer follows the Quaternary soils. This unit increases in thickness from the centre of the valley (5-10 m) to the upper levels (up to 25 m at the position of pillars P3 and P8). The non-clay minerals of this clayey formation include gypsum, dolomite, calcite and quartz.

The profile in figure 22 indicates that this clay layer rests directly on the lower claystone substratum in the central part of the valley. However, an intermediate gypsum layer appears under the valley slopes suggesting a dissolution of gypsum under the valley bottom by running waters before the upper clay unit was deposited.

The gypsum layer, eroded and dissolved in the central part of the valley, has an average thickness of 15 m. Note the abrupt fossilised slopes in this formation between pillars P4 and P5 and between pillars P6 and P7. The gypsum layer is not homogeneous. Interbedded centimetric clay layers are often found. These clay layers exhibit a significant lateral continuity (tens of meters). X-ray diffraction analysis, as well as optical analysis of thin sheets of samples taken from the gypsum unit reveal also the presence of dolomite, calcite and, in some cases, anhydrite.

The lower hard and cemented substratum is the red claystone unit, which provides the name for the entire Tertiary formations. Cement is provided by sulphates and carbonates. Weathering intensity of upper levels is rated as II-III in a scale from I (unweathered) to VI (fully weathered). In a few cores recovered from the centre part of the valley a more intense weathering has locally been reported. Gypsum is also present in a network of crossing veins.

A continuous horizontal gypsum layer (0.6-2 m thick) at mean absolute elevation 263 m crosses the red claystones. This gypsum layer divides the claystone unit into an upper and a lower level. The gypsum layer was cut by all the borings performed in the area, during the successive geotechnical investigations, except for some borings performed on the central part of the valley between pillars P5 and P6. This feature is interpreted as an indication of past dissolution of gypsum by water infiltrating through a fracture or a fault.

The upper claystone level presents high gypsum content. Microscope analysis of thin sections revealed a secondary origin for the gypsum: it was the result of a previous dissolution of anhydrite and a subsequent gypsum precipitation. Anhydrite was not detected in this upper level. This upper claystone, which has higher water content (4%-9%) than the lower unit, will be referred also in figures as a ‘weathered’ claystone level or a gypsic claystone.

In sharp contrast, the lower claystone unit below the thin continuous gypsum layer has high anhydrite content, lower water content (1.2%-4%) and an increased strength. Pile’s tips of pillar P5 cross the dividing gypsum level and enter a few meters into the lower anhydritic claystone formation. Piles supporting pillar P6 reach the thin gypsum layer. The remaining pillars of the bridge are founded either on the upper weathered claystone formation (pillars P4 and P7) or in the upper gypsum layer (pillars P3 and P8).

### 5.3. Geotechnical properties

Preliminary geotechnical investigations aimed at defining the bridge foundation started in October 1999. The first two boreholes were located in both
bridge abutments and reached a shallow depth (10 m). It was initially suggested that the bridge pillars should be supported by means of spread footings. An additional site investigation was performed in 2001. Twelve boreholes (one per pillar – except for pillar P1, where two borings were drilled – plus two boreholes in abutments) were performed. Identification and strength tests (unconfined and triaxial undrained tests), as well as sulphate and carbonate content tests, were conducted on some of the samples recovered.

Reference was made in the geotechnical report to an intense network of fibrous gypsum precipitated in a system of fractures. These fractures were linked with the horst-graben tectonism mentioned before. It was estimated that the network of gypsum veins provided an increased strength of the claystone.

A longitudinal geotechnical profile was developed and the pile foundation was defined. Files were conservatively designed on the basis of the undrained strength determined in tests. Unconfined compression strength of samples recovered in the claystone formation was often larger than 10 MPa. In a few cases values in the range 0.5 – 10 MPa were measured. It was decided to have the pile tip embedded on the gypsum layer (pillars P3, P8) or in the claystone formation (pillars P4 to P7). The base resistance of piles was defined accepting a conservative undrained strength $c_{ua} = 0.5$ MPa.

Once the levelling of tracks detected the progressive heave of central pillars, new geotechnical investigations were launched on May 2007. A campaign of levelling observations of pillars and readings in continuous vertical extensometers in a number of locations was carried out. The necessary borings provided also an opportunity to recover undisturbed samples and to perform some specific tests. Given the nature of the problem there was a specific interest in knowing in more detail the distribution of sulphated species in the successive layers. The degree of fracturing of the deep substratum was also soon identified as a necessary target of the new geotechnical campaign.

The presentation and discussion of results obtained are divided into two parts: the results of tests on recovered samples and cores and field monitoring results. The question of rock fracturing will be discussed below in connection with the development of a heave scenario.

Most of the new boreholes (Fig. 24) were performed in the vicinity of the central pillars of the viaduct. However, some borings were located upstream and downstream of the central pillars, following the direction of the valley, at distances of 100-130 m from the position of the pillars. In most of the boreholes a continuous core was recovered. Some of the cores were paraffin coated and sent to the laboratory for testing.

Index properties of the four main Tertiary geological units (upper brown clays; gypsum layer; gypsiferous claystone; anhydritic claystone) are given in figures 25, 26 and 27 and in table II. Laboratory results of different borings have been collected in plots showing the variation of a given index with depth.

Water content (Fig. 25) provides a good index to distinguish the different layers. The hard claystone substratum has low water contents, 4.2%, in average, for the weathered claystone and 2.7%, in average, for the anhydritic claystone. Water content is essentially nil in gypsum layers. The upper brown clay is a softer material and water content increases fast when moving upwards. These changes are also reflected in the measured dry unit weights shown in figure 26.

The degree of saturation was calculated from the measured values of natural unit weight, water content and solid unit weight. The results, which are collected in figure 27, show a significant scatter. Values in excess of 100% are an indication of testing errors. Values close to zero concentrate in elevations where gypsum layers are found. The upper brown clay layer is essentially saturated. However, degrees of saturation consistently lower than 100% are found in the lower claystone layers. This is a significant result which helps to define a scenario for the observed heave phenomena. Nevertheless, a word of caution should be given here because at low natural void ratios small changes in water content result in large changes in degree of saturation. These minor changes may result from specimen storage and handling in the laboratory.

Additional identification data is collected in table II. Mean values and the range of measured indices are given. Almost all of the Tertiary materials in the site exhibit a low plasticity. This fact rules out an explanation for the observed heave relying on the hydration of active clay minerals. The table reinforces also the lack of saturation of the lower claystone layers.

Mineralogical determinations, as given by quantitative X-ray diffraction analysis, are given in table III. Estimations based on thin section petrographic analysis were also performed. Observations in thin sections indicated that in the upper red claystone residual anhydrite takes the form of inclusions within the gypsum mineral. The gypsum itself was sometimes identified as secondary gypsum. However, gypsum found in fractures was primary precipitated gypsum mineral.

Anhydrite is absent in the upper brown clays and in the red gypsum-laden claystone (except for the inclusions mentioned). By contrast, it is the dominant mineral in the lower claystone.

There is a moderate content of clay minerals in all layers. Dolomite is also ubiquitous and it reaches the maximum concentration on the upper brown clays. The remaining minerals identified (quartz, calcite and feldspars) are present in relatively minor proportions.
Fig. 24 – Position of surveying boreholes and deep extensometers.

Fig. 24 – Posizione dei sondaggi strumentati e degli extensimetri profondi.

Fig. 25 – Variation of natural water content with absolute elevation.

Fig. 25 – Variazione del contenuto d’acqua naturale con la quota assoluta.

Tab. II – Average values and range of index tests for tertiary formations.

Tab. II – Valori medi e campo di variazione delle proprietà indice delle formazioni terziarie.

<table>
<thead>
<tr>
<th>Geological formation</th>
<th>Dry density (g/cm³)</th>
<th>Water content (w) (%)</th>
<th>Solid specific unit weight (γs/γw)</th>
<th>Degree of saturation (Sr) (%)</th>
<th>Atterberg limits</th>
<th>Particle size &lt; 2μm (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brown clays</td>
<td>1.80 (1.31–2.30)</td>
<td>19.8 (11.1–38.2)</td>
<td>2.65 (2.57–2.77)</td>
<td>~100 (90–100)</td>
<td>38.7 (30.4–61.1)</td>
<td>24.5 (19–40.8)</td>
</tr>
<tr>
<td>Gypsum level</td>
<td>2.41 (2.31–2.51)</td>
<td>–</td>
<td>2.63</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Upper red claystone</td>
<td>2.31 (2.01–2.48)</td>
<td>4.2 (0.3–9.2)</td>
<td>2.70 (2.60–2.75)</td>
<td>78 (5–100)</td>
<td>28.1 (22.1–42.7)</td>
<td>20.1 (14–34.5)</td>
</tr>
<tr>
<td>(gypsiferous)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower red claystone</td>
<td>2.4 (2.17–2.63)</td>
<td>2.7 (1.3–7.2)</td>
<td>2.70 (2.6–2.8)</td>
<td>68 (34–100)</td>
<td>30.5 (23.3–62.7)</td>
<td>22.4 (17.4–47.6)</td>
</tr>
<tr>
<td>(anhydritic)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
It turns out that the term “claystone” is not an accurate description in view of the mineral content although it will be maintained for simplicity. The soft rock is essentially a cemented aggregate of sulphate minerals and dolomite with a low proportion of clay minerals. Concerning the phyllosilicates the following minerals were identified: illite, smectite, chlorite and palygorskite.

5.4. Field observations

5.4.1. Piles and Pilecaps

In view of the difficulties to find an explanation for the observed upward displacements of the bridge deck, it was decided to investigate the foundation integrity. Attention was initially given to the state of the deep piles in an attempt to find a faulty pile construction. Vertical boreholes were drilled in the axis of piles. The entire pile length was drilled and concrete cores were recovered. Boreholes penetrated also a few meters into the natural ground below pile tips (Fig. 28). Twelve boreholes of this type were performed in pillars P4, P5, P6 and P7 (three boreholes per pillar).

No gaps or cavities were found in any of the boreholes. However, fractures were observed in the cores and also in the walls of the boreholes. Boreholes were inspected by means of a high resolution acoustic televiwer, a multi-arm caliper and an optical video camera. A crack was often observed at the pile cap contact plane in most of the cases. This observation was interpreted as an indication of the existence of swelling pressures acting against pile caps. The swelling pressures would explain the heave of the pillars, the gap

<table>
<thead>
<tr>
<th>Geological formation</th>
<th>Gypsum (%)</th>
<th>Anhydrite (%)</th>
<th>Phyllosilicates (%)</th>
<th>Quartz (%)</th>
<th>Feldspar (%)</th>
<th>Calcite (%)</th>
<th>Dolomite (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brown clays</td>
<td>14</td>
<td>0</td>
<td>12</td>
<td>8</td>
<td>1</td>
<td>14</td>
<td>51</td>
</tr>
<tr>
<td>Upper red claystone (gypsiferous)</td>
<td>39</td>
<td>0</td>
<td>13</td>
<td>9</td>
<td>1</td>
<td>1</td>
<td>36</td>
</tr>
<tr>
<td>Lower red claystone (anhydritic)</td>
<td>16</td>
<td>43</td>
<td>8</td>
<td>4</td>
<td>0</td>
<td>0</td>
<td>20</td>
</tr>
</tbody>
</table>

Table III – Summary of mineralogical analysis (X-ray diffraction and thin sections).

Tab. III – Sintesi delle analisi mineralogiche (diffrattometrie e sezioni sottili).

Fig. 26 – Variation of dry unit weight with absolute elevation.

Fig. 26 – Variazione del peso dell’unità di volume del secco con la quota assoluta.

Fig. 27 – Variation of degree of saturation with absolute elevation.

Fig. 27 – Variazione del grado di saturazione con la quota assoluta.
created at the pile cap contact and the tensile cracks observed in the pile shafts. The implicit assumption in this explanation, which is illustrated in figure 29a, is that the piles remained firmly anchored at depth where shafts were excavated in hard claystone.

The swelling pressure possibly acting against the pile’s cap was estimated. This pressure should be able to overcome the pillars vertical load and the tensile strength of piles. This condition leads to swelling pressures in the order of 0.7-0.8 MPa. However, such a high swelling pressure cannot be attributed to the low plasticity non-active brown clay layer directly in contact with the pile cap. There is an alternative explanation to the cap-pile cracks observed in borings, which is illustrated in figure 29b. If heave displacement of the piles originates below piles’ tip and it is not homogeneous, a single pile of a group, experiencing the highest heave, would generate a tensile stress in the remaining piles within the group.

Fig. 28 – Borings drilled through piles of pillar P5.

Fig. 28 – Sondaggi realizzati nei pali della pila P5.

Fig. 29 – Sketches showing two alternative interpretations of foundation-heave interaction. Also shown is the geological profile in the vicinity of pillar P6. (a) Swelling pressure against the pile cap induces tensile strains in piles and lifts the pillar. A crack develops at the pile-cap contact. (b) Piles are pushed upwards because of the heave at the deep active zone. Non uniform heave creates a tensile gap between cap and piles in most cases.

Fig. 29 – Schemi relativi a due interpretazioni alternative dell’interazione fondazione-terreno in sollevamento. A fianco la stratigrafia geologica in prossimità della pila P6. (a) La pressione di rigonfiamento sulla testa del palo induce deformazioni di estensione nei pali e solleva la pila. Una frattura si sviluppa al contatto. (b) I pali sono spinti verso l’alto a causa dell’espansione della zona attiva in profondità. Il sollevamento non uniforme crea, nella maggior parte dei casi, una discontinuità di trazione tra palo e plinto di collegamento.
5.4.2. WATER LEVELS

Boreholes drilled through the piles were initially filled with water coming from drilling operations. However, once emptied, they all recovered a water level located at approximately 7 m below the surface in piles of P6 and 16 m in pillar P5. Chemical analysis of the water filling again the boreholes, once emptied after drilling, confirmed that they contained natural ground water. This behaviour was an additional indication of the existence of fractures in the pile’s shaft. It was also observed that the speed of water level recovering in piles belonging to pillars P5 and P6 (those located in the bottom of the valley experiencing the highest heave) was higher than in pillars P4 and P7. These observations may be explained by a more severe cracking of piles in pillars P5 and P6. A higher permeability of the soil/rock around piles in pillars P5 and P6 would also explain the difference rate of water level recovery.

5.4.3. SURFACE MOVEMENTS

A network of surface topographic marks was installed on the ground surface. The network covers a corridor 200 meters wide, centred in the viaduct axis. Figure 30 provides the contours of equal vertical displacements at the ground surface measured from November 26th, 2007 to April 30th, 2008. A maximum value of 44.1 mm of accumulated heave was measured during the first 5 months of monitoring in a topographic mark located 30 meters upstream of viaduct axis, near pillar P5. It corresponds to a heave rate of 8.5 mm/month. During the first five monitoring months significant ground surface was measured (9–13mm) in points located 70–90 m far away from the axis of the bridge.

Figure 30 shows also the position of levelling marks and bridge pillars. The plotted contours were generated by an interpolating program. Contours of equal heave may be described as irregular ellipses whose major axis follows the direction of the valley crossed by the bridge. The centre of these ellipses is located 60 m upstream from the viaduct axis, near pillar P5. The ellipses indicate that the maximum values of accumulated heave are located near pillar P5 and are lower at points far away from the viaduct axis. The heave rate established in the period July 11th, 2008 to April 16th, 2009 is also reported in Figure 31.

Two open tube piezometers were located in a borehole drilled between pillars P5 and P6 at depths of 14.70 m and 26 m. Water levels in all the 12 boreholes drilled through the piles were also measured until stabilization. Water depth measured in a number of wells located in the bridge area provided also valuable data. In an excavated trench located in the valley bottom, upstream from the bridge alignment, a significant flow rate was observed to enter the trench at a depth of 1.60 m (a second trench, 5 m deep, located downstream from the bridge remained always dry). The set of observations available indicate that a relatively shallow phreatic level, 5 m deep, occupies the lower part of the valley crossed by the bridge. The deep piezometer also suggests that a deeper water level is found in the weathered claystone at a depth of approximately 14 m below the bottom of the valley.

Fig. 30 – Contours of surface heave during the period November 26th, 2007 to April 30th, 2008.
Fig. 30 – Curve di uguale sollevamento durante il periodo 26/11/2007 – 30/4/2008.

Fig. 31 – Contours of equal rate of heave displacement. Heave rate established in the period July 11th, 2008 to April 16th, 2009.
Fig. 31 – Curve di uguale velocità di sollevamento. Velocità calcolata nel periodo 26/7/2008 – 16/4/2009.
not located on the bridge axis but it is displaced in the direction of the slope of the valley bottom. This feature suggests that the natural flow of water in the valley controls to some extent the distribution of heave.

Heave contours passing through the position of pillars P3 and P8 mark the limits of observed heave. The geological profile along the bridge axis (Fig. 22) indicates that the deep foundation of pillars P3 and P8 does not reach the lower claystone formation. In fact, the tip of foundation piles of these pillars is located within the massive intermediate layer of gypsum. Maximum heave (pillars P5 and P6) is recorded when the piles’ tip reach the lower anhydritic claystone.

The closure of elliptical heave contours in figure 30 in areas with no direct field information (levelling marks were initially located in a corridor parallel to the bridge direction) is a non-realistic result associated with the “logic” of the interpolation program.

In order to establish better the limits of the heave phenomena in the direction of the valley, additional surface markers were installed in points which followed the direction of the valley, upstream and downstream of the bridge location.

An additional nine-month period of field measurements provided a more accurate picture of surface heave displacements. Figure 31 is a plot of surface heave rate measured between July 11th, 2008 and April 16th, 2009. Contours of equal heave rate are very similar to the initial heave displacement contour map given in figure 30.

A cross section of the heave contours given in figure 30, following the bridge direction is plotted in figure 32. Remarkably, the pattern of surface heave is similar to the vertical displacements measured at the bridge deck level (Fig. 32). The close agreement suggests that the massive pile foundation has no effect in restraining the surface heave. It is also an indication that the origin of heave is located below the level of piles’ tip. Deep extensometers reviewed below provided the necessary evidence in this regard.

Levelling of the pile caps of bridge pillars provided also time plots very similar to the heave records measured on levelling marks located on the ground surface, away from the foundation. This is shown in figure 33 which compares the heave displacements of the pile cap of pillar P6 and the heave recorded in a nearby levelling mark (levelling point 6i0, Fig. 30).

5.4.4. DEEP EXTENSOMETERS

The deformation of rock at depth in the vicinity of pillars P3 to P7 was investigated by means of rod and continuous extensometers installed in boreholes (magnetic incremental extensometers, rod extensometers and sliding micrometers [Kovári and Amstad, 1982]. Data were recorded on a weekly basis. The position of these extensometers is shown in figure 24.

Figure 34 shows the recorded vertical strains (mm/m) in extensometer IX–5, located in pillar P5. The borehole was drilled through the pile’s cap and penetrated a few meters below the tip of piles. Swelling strains concentrate below piles’ tips.
Fig. 34 – Measurements in extensometer IX–5 in the period September 4th, 2007 to January 29th, 2008, and profiles of anhydrite and gypsum content. Reference measurement from July 12th, 2007.

Fig. 34 – Misure nell’estensimetro IX-5 tra il 4/9/2007 ed il 29/1/2008 e verticali del contenuto in anidrite e gesso. Lettura di zero del 12/7/2007.

Fig. 35 – Measurements in extensometer IX–D in the period October 8th, 2007 to April 8th, 2008, and profiles of anhydrite and gypsum content. Reference measurement from June 20th, 2007.

Fig. 35 – Misure nell’estensimetro IX-D tra l’8/10/2007 e l’8/4/2008 e verticali del contenuto in anidrite e gesso. Lettura di zero del 20/6/2007.

Fig. 36 – Measurements in extensometer IX–B during the period October 4th 2007 to January 19th 2009 and profiles of anhydrite and gypsum content. Reference measurement from June 20th 2007.

Fig. 36 – Misure nell’estensimetro IX-B tra il 4/10/2007 e il 19/1/2009 e verticali del contenuto in anidrite e gesso. Lettura di zero del 20/6/2007.
and develop in time without changing the swelling strain pattern. Figure 35 shows the recorded swelling strains in extensometer IX–D, located upstream of pillar P5 (Fig. 24).

Continuous samples were recovered along some of the boreholes drilled for extensometer installation. Gypsum and anhydrite content were determined by means of quantitative X-ray diffraction analysis. The plot of the variation of gypsum and anhydrite content with depth provides an interesting information if it is compared with extensometer readings. This is done in figures 34 to 36 for extensometers IX–5, IX–D and IX–B. All of them indicate that the development of swelling strains starts when the anhydritic layer is crossed at absolute elevations around 261-265 m. Above this elevation, only gypsum is present. Note also that the gypsum content decreases suddenly within the anhydritic layer from high concentrations to very low values (a few percent units). Swelling is directly associated with the presence of anhydrite. No swelling is recorded if only gypsum is present.

It is clear that the extensometer lengths, shown in figures 34 to 36, were insufficient. Significant swelling strains were measured at the deepest levels of extensometers. In addition, heave displacements measured at the head of extensometers (ground surface), by means of conventional leveling, was higher than the integral of vertical relative displacements measured along the extensometer length. This is shown in figure 37 for extensometer IX–5. In view of this limitation and with the aim of identifying better the position and thickness of the deep active layer, a new sliding micrometer was installed in December 2007 between pillars P4 and P5 which reaches a depth of 58 m (SL-1; its position is shown in Fig. 24). Readings are shown in figure 38. This extensometer precisely detects an active layer, 9 m thick, between elevations 250 and 259. A small straining was also measured at deeper levels (elevations 237-243 m).

The set of continuous extensometers installed, even if the information they give is not precise concerning the lower boundary of the active zone, provide enough information to draw the approximate boundaries of the swelling zone (Fig. 39). Question marks and dotted lines indicate that measured heave at the top of the extensometer exceeds the integral of swelling strains measured by the extensometer. The uncertain position of the lower boundary is indicated in the figure.

Expansive strains concentrate in a horizontal band, entirely located within the anhydritic claystone. The upper boundary is approximately found centred at elevation 263, which is the position of the gypsum guide level. The thickness of this band varies between 9 and 15 m. An approximate lower boundary may be located at elevation 250.

Time records of heave displacement show linear trends when plotted in a natural time scale. This is shown in figure 40, which provides the measured vertical displacements of the extensometer surface points, recorded by topographic levelling. Similar linear trends have already been given in figures 33 and 37. Long term linear trends of heave are not expected if hydration of clay minerals is the underlying swelling mechanism. Time records of tunnel heave in sulphated formations exhibit similar long term linear trends as mentioned before.
5.5. Gypsum precipitation in the active layer

Cores recovered in borings crossing the active zone showed often the presence of gypsum crystals. Two morphologies were observed (Figs. 41 and 42). Crystals partially filling some open discontinuities of the claystone matrix grew as needles (gypsum crystallizes in the monoclinic system) oriented in a direction perpendicular to the plane of the discontinuity (Fig. 41). Some of the crystals (thin isolated needles) seemed to be very recent. The open discontinuities within the active zone offered an easy way for water flow. Crystal growth was still far from clogging the joint opening shown in figure 41.

The second morphology (Fig. 42) may be described as a set of thin warped layers of gypsum embedded into the rock matrix.

No direct indisputable evidence of recent crystal growth in discontinuities may be claimed, because the crystal growth process itself was not observed. However, the cores suggest that water flowing (mainly through discontinuities) is able to dissolve anhydrite and then precipitate gypsum in “open” spaces (the discontinuities). It is interpreted that the crystal growth acts as a local “jack”, capable of inducing swelling strains. This phenomenon will be reviewed and analysed in more detail below.

5.6. A classical explanation for heave in sulphated formations

It was already mentioned that a common explanation for the heave and swelling pressures observed is the transformation of anhydrite into gypsum, a reaction which is represented as:

\[
\text{CaSO}_4 + 2\text{H}_2\text{O} \rightarrow \text{CaSO}_4 \cdot 2\text{H}_2\text{O} \quad (2)
\]

A calculation of the change in volume when a given mass of anhydrite transforms into gypsum, comparing molar volumes of gypsum and anhydrite and taking the anhydrite value as a reference, provides a theoretical volume increase of 62%.

Fig. 41 – Gypsum crystal growth in needles in a recovered core from a borehole drilled for hydraulic cross-hole tests, at depths corresponding to the active layer. a) Image of a gypsum filled vein before opening it. b) Gypsum crystals form needles partially filling the open vein, once it is opened by hand.

Fig. 39 – Longitudinal section along the viaduct between pillars P3 and P8. The position of the sliding micrometers (SL) and the incremental extensometers (IX) is shown as well as the location of the active expanding layer.

Fig. 39 – Sezione longitudinale lungo il viadotto tra le pile P3 e P8. La posizione degli estensimetri tipo sliding micrometer (SL) e degli estensimetri incrementali (IX) è mostrata unitamente all’ubicazione degli strati con fenomeni espansivi in atto.
This direct transformation of anhydrite into gypsum seems to be more a simple concept than a realistic mechanism of gypsification of anhydrite. The theoretical volume increase of the gypsum molecule if compared with the “parent” anhydrite molecule has been challenged as a convincing explanation for the observed volume increase when gypsum precipitates. A number of authors [Holliday, 1970; Ortí, 1977; Madsen et al., 1995; Pimentel, 2003; Pina, 2009] concluded that the transformation represented by (2) is an isovolumetric process in which anhydrite dissolves and then precipitates as secondary gypsum. The excess of hydrated calcium sulphate can be transported in aqueous solution or precipitated in available open spaces of the host rock. An alternative to this explanation is the replacement of a host (anhydrite) by a guest (gypsum) crystal inside the rock matrix. These phenomena have been discussed by Nahon and Merino [1997], Fletcher and Merino [2001], and Banerjee and Merino [2011]. The replacement is explained as a simultaneous pressure dissolution of the host crystal and a growth of the guest crystal. This replacement preserves volume and shape in the examples given (which do not include gypsification of anhydrite). However, these transformations require high pressures and temperatures which are not present on the earth surface in connection with most civil engineering works.

It is concluded that the theoretical volume increase associated with reaction (2) is not a convincing explanation for the swelling observed in civil works when anhydrite and gypsum are present. Observations made in the interpretation of swelling experienced by the Lilla Tunnel suggest that the origin of the observed heave is the precipitation of gypsum crystals in discontinuities. This is a process which requires the presence of water in the host rock in contact with the anhydrite mineral. This understanding is the starting point for the analysis developed here.

5.7. Gypsum crystal growth

The case of Pont de Candí is a major obstacle to accept that evaporation is the driving mechanism leading to supersaturation. In fact, no evaporation may be assumed to act on the boundaries of the deep active layer precisely identified immediately below the tips of the foundation piles of pillars P5 and P6. A different mechanism has to be proposed. Consider in figure 43 the equilibrium concentrations of dissolved sulphate in water in the presence of anhydrite and gypsum. For a reference temperature, T = 15°C, water in contact with anhydrite is capable of dissolving 3.2 g/l of calcium sulphate. However, the equilibrium concentration in the presence of gypsum decreases to 2.0 g/l. Therefore, water dissolving anhydrite reaches supersaturation conditions, with respect to gypsum, and gypsum crystals may precipitate. In fact, there is no need for a pre-
existing gypsum crystal for further gypsum crystal growth, although the presence of gypsum crystals facilitates gypsum precipitation. 

Experiments reported by Kontrec et al. [2002] illustrate the transformation of anhydrite into gypsum, in aqueous media. They describe the spontaneous precipitation of gypsum from the supersaturated solutions in equilibrium with anhydrite. Crystal formation is enhanced by the presence of gypsum crystal seeds. This is a mechanism that may lead, in time, to the transformation of all available anhydrite into gypsum. It requires the presence of water and the transformation involves an intermediate step: the precipitation of gypsum crystals.

This mechanism is potentially more significant (and dangerous) for engineering works than the evaporation-based mechanism. Evaporation requires a boundary exposed to an atmosphere exhibiting a Relative Humidity (which is a measure of the water potential) lower than the water potential on the soil or rock. This is not the case of Pont de Candé deep foundations. In addition, evaporation rates, combined with the relatively low solubility of gypsum (or anhydrite) into water provide, in practice, a small mass of precipitated gypsum. This precipitation concentrates essentially on an evaporation surface and not in a rock volume.

The alternative mechanism (precipitation of gypsum via an aqueous solution in the presence of anhydrite) may occur in large volumes (this is the case of Pont de Candé), it does not require evaporation boundaries and the process continues in theory, until all the anhydrite precipitates as gypsum. Note that the size of molecules of gypsum and anhydrite are hardly relevant when attempting to calculate induced volumetric strains. Swelling will be controlled not only by the total mass of the precipitated gypsum (which may occur at a distance from the source of anhydrite dissolution) but also by the geometry of precipitated crystals and its interaction with the surrounding soil/rock. A further consideration is likely to have a major influence on crystal growth. Crystals precipitate more easily if some space is available for them. Pores of claystones provide an extremely reduced space and water flow is essentially restrained through them. But joints provide a more favourable environment for crystal growth: the “open” space increases dramatically, as well as the flow rate. This interpretation is illustrated in figure 44.

Experience in tunnels indicates that active zones concentrate under the invert. In horse-shoe tunnel cross sections, points under the tunnel floor experience the highest stress release and, therefore, the highest risk of opening new or existing discontinuities. In addition, massif water tends to drain towards the tunnel floor (where collector drains are usually placed).

These observations bring another difficulty to the observed phenomena in Pont de Candé. If a previous fissuring of the claystone rock is a requirement for crystal growth, it would be interesting to investigate if there was evidence of fracturing in the active zone identified below the piles tip. This question was answered by performing a hydraulic cross-hole test in the active band where heave develops. This is described in the next section.

5.8. Hydraulic cross hole tests

A hydraulic cross-hole campaign was set out in order to investigate the structure, fracturing and

Fig. 45 – Layout and instrumentation used during a hydraulic cross-hole test [Aitemin, 2009].

Fig. 45 – Schema e strumentazione usata per una prova idraulica cross-hole.
Hydraulic cross-hole tests are performed in boreholes. Boreholes are first divided into isolated sections by means of inflatable packers. During the test, water is pumped or injected in one isolated stretch of one borehole and water pressures are measured in the set of isolated sections of similar nature, both in the “emitting” and in the nearby boreholes. Figure 45 shows a scheme of the layout of a typical hydraulic cross-hole test.

Four hydraulic cross-hole tests were performed on three boreholes (S1, S2 and S3), 50 meters long, located downstream and close to pillar P5 (they span elevations 237 (end of boring) –287 m (surface)). Boreholes, 116 mm in diameter, were drilled with a double-wall core barrel. A continuous string of cores was recovered. Figure 46 shows the position of the testing boreholes.

Boreholes were located in an area affected by significant heave displacements according to the periodic topographic levelling carried out at the ground surface. The three drilled boreholes define two perpendicular directions, and two testing distances (4 and 8 m) (Fig. 46b). This layout allows obtaining the hydraulic characteristics of the rock mass in two directions.

As a reference, continuous extensometers in this area indicate the existence of an active layer, 14 m thick, which extends from elevations 246 to 260 m.

Boreholes were lined in the upper 20 m. On each borehole two packers isolated a central section about 7 m long. Lower and upper sections were also defined, the lower one extending from the end of the borehole to the lower packer and the upper one extending from the foot of the casing to the upper packer. Four injection tests were performed by injecting water from the central section of borehole S2. The position of packers, always maintaining its relative distance, was lowered in steps for each successive test in order to investigate the response of the claystone at different depths.

A constant flow rate of water was injected in the central section of S2 and water levels were monitored in time in the 3×3=9 sections isolated in the three borings. Pressure-time records were interpreted by means of backanalysis procedures [AITEMIN, 2009]. The analysis provides an estimation of the soil hydraulic conductivity (by dividing the estimated transmissivity by the length of the corresponding borehole section).

Of particular relevance for the work reported here are the calculated permeability values. It was found that no hydraulic connection could be established in a vertical direction (pressure changes in boreholes S3 and S1 were essentially recorded in the central section). An average (horizontal) permeability of 2×10⁻⁶ m/s was calculated at depths varying between 22 and 34 m (elevations 265-245 m, where the active layer was independently located by extensometers data). Very similar permeability values were found on the two directions tested. The horizontal permeability decreased suddenly at higher depths. It was found to be 1–1.6×10⁻⁹ m/s at 33-39 m depth and 3–8×10⁻¹² m/s at depths of 39-50. The high horizontal permeability values measured in the position of the active layers cannot be explained as “matrix” permeability in a claystone of low porosity. The measured permeability reflects the existence of open horizontal joints.

The observations of the recovered cores from the boreholes indicated an irregular distribution of fractures. However, the cores presented an increased fracture density at depths of 22.50-35 m. At higher depths the cores were again massive.

Some of the fractures crossing the recovered core exhibited dense deposits of gypsum. One example is given in figure 47. Despite the accumulation of gypsum, channels can be identified. These joints explain a relatively high permeability.

The cross-hole testing campaign provided also a reliable information on the initial water levels within the claystone formation. Before cross-hole testing was started water level was measured in the three sections, separated by packers, described above. Once equilibrated, measured water levels were plotted against the elevation of each one of the sections, for the three boreholes. The results (Fig. 48) show a distinct downward vertical gradient in the three boreholes: 0.08 m/m in borehole S2, 0.57 m/m in
borehole S1 and 0.60 m/m in borehole S3. These are high values which consistently indicate a vertical flow from an upper aquifer to lower levels. This natural flow is even contrary to the expected “regular” behaviour in the bottom of a valley (an upward flow created by higher water heads on the valley slopes).

It is also revealing to check that the piezometric head was similar in all the borings at elevations 243-265 m (active layer), an additional indication of the high permeability of this zone. These are important observations which help to define the heave scenario described below.

The set of “in situ” observations described so far and laboratory investigations into the precipitation and growth of gypsum crystals indicates that the initiation of the mechanism requires a set of conditions:

- The presence of free water, able to dissolve anhydrite.
- Existence of open voids or fissures to allow the precipitation of gypsum (and the water circulation). Crystals are not expected to grow in the reduced space of claystone pores, having a low porosity, for at least two reasons: a very small free space and a low permeability.
- Pre-existence of some gypsum crystals, which act as seeds for further growth.

On the other hand, the scenario for the development of heave should explain also that the main triggering factor was the construction of the bridge.

The favoured explanation is that the lower level of fissured anhydritic claystone received a water inflow as a result of the bridge construction. Borings drilled at the location of each pillar with the purpose of designing the pile foundation connected the upper aquifer with the lower anhydritic layer in a few isolated locations. Later, pile construction, especially for pillars 5 and 6, crossed the reference gypsum layer separating the upper gypsum-claystone and the lower anhydrite-claystone. Even if the bored piles were filled with concrete, damage induced around the pile shaft during pile construction would allow a vertical transfer of water. Finally, the set of borings drilled later for monitoring and investigation purposes (most of them located in the vicinity of pillars P5 and P6) implied an additional downward water transfer. Figure 49 illustrates this scenario.

This hypothesis is also consistent with the heave contours plotted in figures 30 and 31. In fact, they show a displacement of the measured surface heave in the expected direction of natural flow in the valley. It also explains that the highest displacements are measured in pillars 5 and 6, whose pile foundations reached the anhydrite.

The next step will be the formulation of a working model at the scale of the field problem, which incorporates the key aspects described.
5.9. A model for gypsum precipitation and heave

The model is formulated within a general framework for hydro-mechanical analysis for saturated porous media. Some background references for the model, in its "standard" version, are Oliva et al., [1994; 1996]; DIT-UPC, [2002] and DETCG, [2010].

However, the material involved in gypsum crystal growth is not a standard medium. The claystone rock includes the clay minerals, not soluble, and two minerals (anhydrite and gypsum) that will dissolve and precipitate. Therefore the solid mass balance equation should include the three solid species. The mass balance equation of inert and salt species should be formulated as well as the mass balance of the liquid phase. In addition, the equilibrium equation should take into account that the crystallization of gypsum induces deformations. Also, the model has to keep track of the solute, which in this case is the calcium sulphate (\( \text{CaSO}_4 \)). Then, a solute balance equation has to be added for the transport of sulphates.

5.9.1. Mass and momentun equations

The solid skeleton of the soil considered in the present work has been represented in the phase diagram of figure 50. The volume fractions of the soluble species (anhydrite and gypsum) are denoted anhydrite porosity (\( \phi_{\text{anh}} \)) and gypsum porosity (\( \phi_{\text{gyp}} \)). Accessible voids are represented by the classical porosity, \( \phi \).

The mass balance has to include the three different species of the solid phase: the insoluble clay matrix, the soluble gypsum crystals and the soluble anhydrite crystals. The classical mass balance equation for standard soils transforms into three balance equations for the insoluble solids and for the soluble species, gypsum and anhydrite:

\[
\frac{d}{dt} \left( \rho_s (1-\phi-\phi_{\text{anh}}-\phi_{\text{gyp}}) \right) + V \cdot \left( \rho_s (1-\phi-\phi_{\text{anh}}-\phi_{\text{gyp}}) \frac{d\mathbf{u}}{dt} \right) = 0
\]

\[
\frac{d}{dt} \left( \rho_{\text{gyp}} \phi_{\text{gyp}} \right) + V \cdot \left( \rho_{\text{gyp}} \phi_{\text{gyp}} \frac{d\mathbf{u}}{dt} \right) = \frac{dm_{\text{gyp}}}{dt}
\]

\[
\frac{d}{dt} \left( \rho_{\text{anh}} \phi_{\text{anh}} \right) + V \cdot \left( \rho_{\text{anh}} \phi_{\text{anh}} \frac{d\mathbf{u}}{dt} \right) = \frac{dm_{\text{anh}}}{dt}
\]

The first equation states that the rate of change of mass of insoluble minerals, per unit volume, must be balanced by the net inflow rate of mass into the reference volume. \( \rho_s \), \( \rho_{\text{gyp}} \), \( \rho_{\text{anh}} \) are the densities of the insoluble minerals, gypsum and anhydrite, and \( \mathbf{u} \) is the displacement vector, that is accepted as a unique field for the porous medium. Acknowledging that gypsum and anhydrite may dissolve or precipitate, a third term, namely the mass rate of precipitation or dissolution, should be included in balance Eqs. (4) and (5).

Combining equations (3), (4) and (5), isolating the rate of change of "true" porosity, \( \phi \), and taking \( \rho_s \), \( \rho_{\text{gyp}} \), \( \rho_{\text{anh}} \) as constant values results in:

\[
\frac{D}{Dt} \phi = (1-\phi) \nabla \cdot \left( \frac{\mathbf{u}}{\rho_s} \right) - \frac{1}{\rho_{\text{gyp}}} \frac{dm_{\text{gyp}}}{dt} - \frac{1}{\rho_{\text{anh}}} \frac{dm_{\text{anh}}}{dt}
\]

where:

Fig. 50 – Phases and species of the anhydritic gypsiferous claystone porous media.

Fig. 50 – Ripartizione delle diverse fasi e dei tipi mineralogici in mezzi porosi costituiti da argilliti gessoso-anidritici.
is the material derivative.

The mass balance equation for the water can be written:

$$\frac{D}{Dt}(\rho_1 \frac{d}{dt} u) + \rho_1 \frac{D}{Dt}(\rho_1 \frac{d}{dt} u) + \nabla \cdot (\rho_1 q_1) = f^w \tag{7}$$

where, $\rho_1$ is the density of the water, $q_1$ is the flow rate of water and $f^w$ any sink or source of water. Gypsum dissolution or precipitation contributes to this term which is calculated through the mass rate of precipitation and dissolution of gypsum and the stoichiometric relationship (one kg of gypsum frees 0.21 kg of water when dissolved). However, the $f^w$ term is negligible in the open system analyzed here.

Consider now the mass balance equation of solute. Since both anhydrite and gypsum, when dissolved, originate calcium sulphate, only one solute will be considered. The balance equation reads:

$$\frac{D}{Dt}(\rho_1 o^m_l \frac{d}{dt} u) + \rho_1 \frac{D}{Dt}(\rho_1 o^m_l \frac{d}{dt} u) + \nabla \cdot (\rho_1 o^m_l q_1 - D \nabla o^m_l) =$$

$$= -\frac{d m_{gsp}}{dt} \left( 1 - \frac{\rho_1 o^m_l}{\rho_{gsp}} \right) - \frac{d m_{anh}}{dt} \left( 1 - \frac{\rho_1 o^m_l}{\rho_{anh}} \right) \tag{8}$$

In this equation $\omega^m_l$ is the mass fraction of dissolved sulphate in water (the product $\rho_1 \omega^m_l$ is the concentration of sulphate in water in units of mass/volume). The term $D \nabla \omega^m_l$ accounts for the diffusive rate of flow following a Fick’s law. $D$ is the diffusion coefficient.

Mass and water balance equations (Eqs. 6 and 7) may be combined into a single equation:

$$\phi \frac{D}{Dt}(\rho_1 l) + \nabla \cdot (\rho_1 q_1) = -\rho_1 \gamma \frac{d}{dt} u - \frac{1}{\rho_{gsp}} \frac{d m_{gsp}}{dt} - \frac{1}{\rho_{anh}} \frac{d m_{anh}}{dt} + f^w = 0 \tag{9}$$

where changes in the density of the solid species are neglected.

The equilibrium equation in terms of total stresses,

$$\nabla \cdot \sigma + b = 0 \tag{10}$$

where $\sigma$ is the stress tensor and $b$ is the vector of body forces, completes the set of equations to be solved.

5.10. Precipitation and dissolution of minerals

The rate of precipitation or dissolution of gypsum or anhydrite mass in equations (6) and (8) needs to be defined by some kinetic equations. Kinetic equations express the rate of mass change in terms of the “distance” between the current concentration of a given solute and the concentration for saturation conditions. In terms of mass fractions ($\omega^m_l$), the equations adopted in this work, adapted from LASAGA [1984], are:

$$\frac{d m_{gsp}}{dt} = \sigma \cdot \xi_{gsp} \phi_{gsp} \left( \frac{\omega^m_l}{\omega^m_{gsp}(T,p)} - 1 \right)^q \tag{11a}$$

$$\frac{d m_{anh}}{dt} = \sigma \cdot \xi_{anh} \phi_{anh} \left( \frac{\omega^m_l}{\omega^m_{anh}(T,p)} - 1 \right)^q \tag{11b}$$

where:

$$\xi_{gsp} = \frac{\omega^m_l - \omega^m_{gsp}}{\omega^m_{gsp}} \tag{12a}$$

$$\xi_{anh} = \frac{\omega^m_l - \omega^m_{anh}}{\omega^m_{anh}} \tag{12b}$$

In equation (11) the mass fractions for saturated solutions of gypsum and anhydrite are made theoretically dependent on pressure applied to the crystal,
dependence on temperature is well known (Fig. 43). The effect of stress acting on the crystals (gypsum and anhydrite) is described by Scherer [1999]:

$$\omega_{\text{sat}}^m = \omega_{\text{sat}}^m \exp \left( \frac{p \gamma}{R_g T} \right)$$

(13)

where, \(\omega_{\text{sat}}^m\) is the equilibrium mass fraction of dissolved sulphate in water, \(\gamma\) is the molar volume of the crystal, \(R_g\) is the ideal gas constant and \(T\) the absolute temperature. Temperature was constant in the case analyzed. This is probably the situation in the active region below the pile’s tips. The “pressure” on the crystals, \(\sigma_{\text{crystal}}\), will be made equal to the effective intergranular stress acting on the solid species:

$$p = \sigma_{\text{crystal}} = \frac{\sigma_i}{1 - \phi}$$

(14)

The effective vertical stress is selected in this case, where swelling strains are essentially vertical.

In equation (11), \(\sigma_i\) is the specific surface of the species (m² of crystal surface/m³ of medium) and \(\kappa\) is a constant controlling the rate of dissolution/precipitation (kg/s·m³ of crystal). Finally, the terms \(\xi\) (Eq. 12) provides a positive or negative sign to the mass rate of dissolution/precipitation. A negative sign implies dissolution \((\omega < \omega_{\text{sat}})\) and a positive sign \((\omega > \omega_{\text{sat}})\) implies precipitation. Exponents \(\theta\) and \(\eta\) were made equal to 1 in the calculations performed.

A significant feature of stress-strain calculations is that precipitation of crystals induces deformations. A first approximation to calculate the imposed deformations is to assume that the precipitated mass results in a volumetric deformation equal to the crystal volume. However, observations in the field (see, in particular, Figs. 51 and 52) indicate that new crystals may either occupy part of the volume of an already existent open discontinuity or, else, generate additional porosity. A simple approach is to take these effects into account by means of a “swelling” parameter, \(\gamma_i\), introduced to calculate strain rates \(\varepsilon_i\) from precipitated gypsum mass as follows:

$$\frac{d \varepsilon_i}{dt} = \frac{\gamma_i}{\rho_{\text{gyp}}} \frac{d m_{\text{exp}}}{dt},$$

(15)

\(i = 1, 2, 3; \)

1 = Vertical (z);

2, 3 = Horizontal (h)

It is reasonable to accept that parameters \(\gamma_i\) depend on the applied effective stress on the direction \(i\). The higher the effective confining stress, \(\sigma_i\), the lower the expected volumetric deformation for a given mass of gypsum precipitated. On the other hand, the function \(\gamma_i(\sigma_i)\) should also include the possibility of a new crystal inducing additional porosity or partially reducing the existing one. In the second case, subsequent precipitation of gypsum crystals would eventually occupy all the available open space. Under general deformation conditions \(\gamma_i\) values should also account for the fact that a given volume of precipitated crystal deforms the rock or soil in three directions. The following equation is proposed:

$$\gamma_i = \gamma_{\text{max}} e^{-b_i} \text{ for } \sigma_i > 0; i = 1, 2, 3 \quad (16a)$$

$$\gamma_i = \gamma_{\text{max}} \text{ for } \sigma_i > 0; i = 1, 2, 3 \quad (16b)$$

Table IV – Model parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Parameter</th>
<th>Value</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial open porosity (\phi_{\text{initial}})</td>
<td>0.09</td>
<td>Elastic parameter (E)</td>
<td>1000 MPa</td>
<td>Initial gypsum volumetric fraction (\varphi_{\text{gyp}})</td>
<td>0.2</td>
</tr>
<tr>
<td>Intrinsic permeability (K)</td>
<td>2.10-13 m²</td>
<td>Poisson’s ratio (\nu)</td>
<td>0.2</td>
<td>Initial anhydrite volumetric fraction (\varphi_{\text{anh}})</td>
<td>0.15</td>
</tr>
<tr>
<td>Compound kinetic coefficient, (\alpha, \kappa)</td>
<td>0.76·10-4 kg/m³·s</td>
<td></td>
<td></td>
<td>Solid specific unit weight ((\gamma_i/\rho_{\text{gyp}}))</td>
<td>2.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gypsum density (\rho_{\text{gyp}})</td>
<td>2.3 Mg/m³</td>
</tr>
</tbody>
</table>

Anhydrite-gypsum transformation

Hydraulic

Mechanical

Tab. IV – Parametri del modello.
The maximum value $J_{\text{max}}$ is defined for zero confining stress. It may be suspected that parameters $J_{\text{max}}$ and $b$ will change with spatial direction. A constant value for model parameters $J_{\text{max}}$ and $b$ will be accepted here.

Deformation rates induced by gypsum precipitation are treated as imposed volumetric deformations. All these developments have been included into the CODE_BRIGHT Finite Element computer code for thermo-hydro-mechanical analysis of porous media [DETCG, 2010]. This program was used in the calculations reported below.

5.11. Heave calculations. Comparison with field data

The simulation performed concerns the central pillar 5 of the bridge, where maximum vertical displacements have been recorded (Fig. 53a). A column of foundation material, 15 m wide and 55 m long, at the position of pillar 5 was modelled under plain strain conditions. It was further assumed that the foundation material is laterally confined.

The geometrical model, sketched in figure 53b, includes the presence of the active layer, 15 m thick, and two stable layers, above and below. A phreatic level was located in the position shown in the figure. Following the scheme plotted in figure 49, a horizontal flow was forced in the active layer by adding two inert pervious layers on both sides of the central soil column (Fig. 53b). A small difference in piezometric head (1 m) between the two lateral layers forces an essentially horizontal flow within the active layer.

The model requires a number of parameters, which have been collected in table IV, under three headings: anhydritic-gypsum transformation, hydraulic parameters and mechanical parameters.

Mass fractions of gypsum and anhydrite for a saturated solution in water are based on a calculation performed with the computer program PHREEQC [PARKhurst and APPElo, 1999; PARKhurst, 1995] which simulates chemical reactions in aqueous solutions. Initial volumetric fractions (porosities) of gypsum and anhydrite in the active layer are approximate average values, which reflect core descriptions made during boring operations.

The kinetics represented by equation (11) has been investigated by a number of authors [BARTON and WILDE, 1971; JESCHKE et al., 2001; KONTREC et al., 2002]. Dissolution is controlled by two processes: A chemical reaction at the crystal surface and a molecular diffusion of dissolved ions through a boundary layer. Therefore, flow conditions around crystals control also the diffusion rate. Laboratory experiments described by the authors mentioned involve tests on crystals in suspension in an agitated aqueous solution or dissolution of crystals adhered to a surface which is rotated in a solution. Values of kinetic constants have been provided by JESCHKE et al. [2001] and BARTON and WILDE [1971] for gypsum and anhydrite.

Concerning precipitation, NancoLLAS et al. [1973] conclude that it is very difficult that anhydrite precipitates in nature due to the high activation energy required. Freyer and Voigt [2003] state that precipitation of anhydrite from aqueous solutions is the most difficult one of all phases of calcium sulphate (gypsum, two hemi-hydrates and three anhydrite phases). Below a temperature of 90ºC anhydrite does not precipitate spontaneously. KONTREC et al. [2002] indicate that the dominant precipitation of gypsum occurs spontaneously from highly supersaturated solutions. However, gypsum crystallization is possible from low super-saturation solutions provided there exists available gypsum crystal “seeds”. The second scenario is likely to prevail in the case described here.

It follows from the short review above that the process of anhydrite dissolution and gypsum precip-
Vertical effective stress

Fig. 54 – Variation of the $\gamma_z$ with the applied effective vertical stress.

Fig. 54 – Variazione di $\gamma_z$ con lo sforzo verticale efficace applicato.

Verticalization is a natural one under the conditions of low stresses and moderate temperatures. It is also consistent with “in situ” observations.

Note that equation (11) indicate that the rate of dissolution/precipitation is governed by the product $\kappa \sigma_z$. Uncertainties in $\kappa$ are high because of the significant differences between a fractured claystone and the laboratory experiments mentioned. The specific surface $\sigma_z$ (exposed to free water) is also difficult to estimate in a fractured rock mass. Therefore, the back-analysis of field heave records is better interpreted as providing an estimated value for the compound kinetic parameter $\kappa \sigma_z$. As a simple choice, a constant value was adopted for gypsum and anhydrite and a value $\kappa \sigma_z = 0.76 \times 10^{-3}$ kg/m$^3$-s was found by matching the field heave records. If further hypothesis are made, $\kappa$ and $\sigma_z$ could be isolated. For instance, if an anhydrite dissolution rate $\kappa_{anh} = 1.49 \times 10^{-5}$ Kg/m$^2$-s is accepted, an exposed specific surface for anhydrite, $\sigma_z = 5.1$ m$^2$/m$^3$ is calculated.

Very simple hydraulic and mechanical models were selected because the observed phenomena are not believed to be much affected by them in the observed field behaviour. Two parameters characterize the hydraulic behaviour of the active layer: porosity and permeability. An average value $\phi = 0.09$ was taken. This value is somewhat higher than values measured in some of the cores recovered to account for the open space provided by joints. In the active layer the horizontal intrinsic permeability found in the cross-hole tests described was imposed in the model ($K = 2 \times 10^{-13}$ m$^2$; above and below the active layer permeability was reduced to a lower value, $K = 1 \times 10^{-16}$ m$^2$).

The entire soil/rock column was assumed to be isotropic linear elastic. Initial stresses are given by a geostatic stress in the vertical direction and a horizontal stress given by an at rest pressure coefficient $K_0 = 2$. The side “auxiliary” columns in figure 53b were given a low elastic modulus to better approximate a free sliding condition in the vertical direction. The swelling phenomenon was activated only in the active layer.

Chemical analysis of water extracted from boreholes crossing the active layer indicated that it was saturated in calcium sulphate salts. This condition ($\omega_m = 2.028 \times 10^{-5}$) was imposed on the water filling the pores of the active layer.

The assumption made in the model was that (almost) pure water ($\omega_m = 1 \times 10^{-5}$) entered the active layer from its left boundary at the position of the active layer. This is an extreme case which may reproduce the inflow of water directly from surface running waters (see Fig. 49). However, the calculation shows that the water entering the active layer rapidly gets a supersaturated condition as anhydrite is dissolved. Almost no difference in results were calculated when the sulphate concentration of the incoming water was varied between extreme values.

The most uncertain parameters are probably those defining the coefficient $\gamma$ in equation (16a). The selected values $\gamma_{max} = 1$, $b = 2$ result in the $\gamma (\sigma_z$ crystal) relationship shown in figure 54. Selecting $\gamma_{max} = 1$ implies that no additional swelling, beyond the volume of the precipitated gypsum mass, is considered in the analysis.

Consider first the swelling displacements measured at the top of incremental extensometers IX–5 and IX–6 located in the vicinity of pillar P5 (the position of these instruments is given in figure 24. A 800 day history of heave displacements is compared with model calculations in figure 55a. The set of parameters leading to the good fit shown in figure 55a is given in table IV. This will be called “base case”. The swelling coefficient, $\gamma_z$ in equation (16), takes values varying from 0.11 (deepest point in the active layer) to 0.22 (upper point in the active layer). The range of $\gamma_z$ values within the active layer is marked in figure 54. These values suggest that a substantial part of precipitated mass of gypsum already occupied existing void space.

The evolution of calculated vertical strains is given in figure 56. Strains develop in a continuous manner in the active layer, a consequence of the porous model adopted for the claystone porosity.

The calculated heave plot in figure 55a shows an apparent linear trend of heave with time in natural scale. This is a consequence of the limited time span of the simulation performed. If time is increased (Fig. 57) there is a progressive decay in the heave rate that eventually stops. Heave development goes in parallel with the depletion of anhydrite content in the active layer. This is shown in figure 58. As the anhydrite content (volumetric fraction) decreases, the gypsum content increases in parallel. This is a long
term process. The model indicates that, if conditions are left unchanged, significant heave rates would develop during a period of 8 years. Once the anhydrite disappears, if hydraulic conditions are maintained, the model predicts a progressive dissolution of gypsum (Fig. 58).

The sustained heave rate of the central part of the viaduct experienced a significant reduction when a 29.5 m high embankment over the foundation of pillar P5 was built on the valley floor. Figure 55b shows the change in swelling rate at $t = 924$ days when the embankment construction started. The heave record in figure 55b was measured by surface levelling of the base of pillar P5 and also by levelling the top of the two extensometers located in the vicinity of pillar P5.

The model reacted in a manner very similar to the actual behaviour, predicting a new rate of swelling very close to the actual values. Note that model parameters were calibrated only on the first part of the swelling record (from $t = 0$ to $t = 800$ days). The model predicts also a transient settlement during the embankment ramp loading. This is a consequence of the stiffness assumed for the foundation materials. Field data is scarce during the embankment building period and this prevents a more accurate analysis of the immediate effect of embankment construction. However, the heave of the active layer remained at a reduced rate, well defined in subsequent measurements on the pillar surface.

5.12. Sensitivity analysis

Further insight into the phenomena and the characteristics of the model developed is gained through a sensitivity analysis on the relative importance of controlling parameters.

The results presented here maintain the parameters of the base case (Tab. IV) except for the parameters defining the effect being investigated. The analysis, in most cases, extends to the first 1000 days and, therefore, it incorporates the embankment loading (except for the cases concerning the effect of applied surface stress on heave development).

![Fig. 56 – Calculated vertical strains in the active layer.](image)

Fig. 56 – Deformazioni verticali calcolate nello strato espansivo attivo.

![Fig. 57 – Long term evolution of calculated heave in time.](image)

Fig. 57 – Evoluzione a lungo termine del sollevamento calcolato.

![Fig. 58 – Long term evolution of gypsum and anhydrite volume fractions.](image)

Fig. 58 – Evoluzione a lungo termine della frazione di gesso e anidrite in volume.
Porosity. Changing porosity modifies the water flow rate, which controls the sulphate concentration and, therefore, the rate of precipitation of gypsum and anhydrite dissolution. In addition, porosity variations change also the crystal effective stress which controls anhydrite and gypsum solubility as well as the swelling coefficient. Despite these cross effects it turns out that its effect is of relatively minor importance, as illustrated in figure 59. The uncertainty associated with the evaluation of the fissured volume or accessible porosity is therefore quite limited regarding its effect on swelling.

Initial volume fractions of gypsum and anhydrite. Volume fractions control rate effects (Eq. 11). For instance, equation (11a) tells that a higher gypsum content results in a higher rate of gypsum precipitation. In other words, increasing the exposed gypsum volume facilitates further gypsum crystallization. The effect of different initial volumetric fractions of gypsum and anhydrite is given in figures 60a and b. Gypsum volumetric fraction has a small effect (Fig. 60a). However, anhydrite content has a very significant effect on calculated swelling. This result stresses the need for a precise determination of anhydrite content in field investigations.

Equilibrium concentrations. Equilibrium concentrations at saturation are known to depend on temperature and crystal effective stress. The effect of temperature was illustrated in figure 43. The effect of a relatively small change in gypsum and anhydrite values ($\omega_{\text{gypsum}}^0$; from $2,028 \cdot 10^{-3}$ to $2,192 \cdot 10^{-3}$ and $\omega_{\text{anhydrite}}^0$ from $3,187 \cdot 10^{-3}$ to $2,848 \cdot 10^{-3}$) is shown in figure 61. These concentrations correspond to a change in temperature from 15°C to 40°C.

Stress effects. Stress effects are substantial in the model developed for two reasons: They modify the concentration of saturated solutions (Eq. 13) and they are expected to modify the swelling coefficient $\gamma_i$ (Eq. 16). The combined effect is shown in figure 62. Increasing the stress increases the solubility of anhydrite and, therefore, an increased mass of dissolved sulphate is available for gypsum precipitation. In addition, the direct mechanical effect on $\gamma_i$ reduces further the expected heave. However, the swelling coefficient dominates this effect.
Other effects. Parameters controlling water flow in the active layer (induced gradient, permeability) had a minor effect. Gradient changes result in limited changes in vertical effective stress. Permeability dictates the flow rate, which has not a direct influence. It affects the time required for the incoming water to reach a saturated concentration of sulphate. But this process is rapid, at least for acceptable permeability values of the anhydritic formation, and the incoming water saturates in calcium sulphate shortly after entering the active layer, even if pure water is injected.

The effect of the initial state of stress ($K_0$) depends on some modelling hypothesis. If only swelling strains in the vertical direction are activated (a hypothesis favoured by the horizontal pattern of discontinuities), the vertical stress controls the calculated swelling and therefore $K_0$ has no effect. In the model, $K_0$ and Poisson's ratio have a definite effect. A change in swelling stresses $\sigma_h$ implies a change in $J_h$ which in turns modifies the calculated horizontal strains needed to compensate the imposed horizontal displacements (zero at the boundary). The calculated horizontal strains result, through the Poisson's effect, in vertical strains which modify the vertical swelling. This effect is small, however. A constant Poisson ratio $\nu = 0.2$ was adopted in all calculations.

5.13. Remedial measures

Remedial measures carried out were inspired by the belief that adding weight would reduce the rate of heave and eventually this added stress would be able to eliminate heaving. An embankment partially filling the valley was therefore designed. It was decided to build the embankment in two stages. The first one would reach a height of 33 m over the lowest elevation of the valley. In the case that the embankment weight is not able to stop displacements, the embankment height will be increased to a maximum height of 48 m over the valley centre.

The first stage of embankment construction (Fig. 63) was actually built in the period October 2009 - August 2010. Pillars and the original pile foundation were protected in the manner sketched in the figure. Pillars were embedded in a fill of compacted non active material, which is bounded by a “ring” of loosely compacted soil to reduce the effect of the embankment deformations on the concrete pillar shafts. In addition, a protective cap founded on deep bored piles installed around the existing original foundation was built to avoid a direct action of the embankment on the original piles. The new piles reached depths similar to the original ones and therefore their tips are located above the active layer. The new pile cap is not structurally connected to the pillars (a gap of 20 cm was left). Also, pillars are protected by a double sheet of polyethylene membrane to minimise the friction between pillars and surrounding compacted soil. Surface runoff waters were collected and drained away to limit water content changes of the embankment.

The construction of the embankment slowed the heave rate of the bridge deck (Fig. 64). However, the expansive activity at depth has not ceased completely. This is shown in figure 65, which shows the strain variation with depth measured in sliding micrometer SL-S5bis (see location in Fig. 24). The upper part of the micrometer experiences a compression, as a result of the embankment loading. However, the length of the extensometer crossing the active layer reveals that some swelling strains still develop. Compressive strains dominate expansions measured in the recording period shown in Figure 65. Compressive strains of the layers above the active band will eventually vanish sometime after the application of the embankment loading and the evolution in time of surface (and pillars) displacements will be again dictated by the expansion of the active layer. This is shown in figure 66, which shows the displacement measured by the sliding micrometer SL–5 (Fig. 24).
The time plot of surface heave (measured total displacement) shows an initial sudden reversal of swelling and a subsequent net settlement which gradually levels off. Eventually, heave resumes although the heaving rate is now reduced to 1 mm/month, substantially lower than the rate measured before embankment construction (about 7 mm/month in average). The plot shows also that the accumulated heave on the active layer (integrating strains on the depth interval 25 m to 40 m) increases at the rate observed on the surface once the transient settlement associated with the embankment construction has ended.

Heave rate measured on pillar P5 has been related with the vertical effective stress acting on the centre of the active layer under pillar P5. Maximum and minimum heave rate values (Fig. 67) as well as a weighted average (weighting reflects the time interval of each of the measuring records selected) decrease with applied effective stress. The plot suggests that heave rate may eventually stop if an effective vertical stress of 1.25 MPa is acting on the mid level of the active layer. Reaching this condition would require a small increase of embankment height (approximately 4 m).

6. Pallaresos embankments

6.1. Introduction

Not far from the location of Pont the Candí, a 196 m-long bridge, Pallaresos Bridge, which also belongs to the high speed railway link between Madrid and Barcelona, was built in 2004. Figure 68 shows the plan view and a longitudinal section of the bridge. The two abutments, of similar design and dimensions, were directly founded on a hard marl of tertiary age. They limit the two embankments shown in the figure, which reached a maximum height of 18 m in the proximity of the abutments. The thickness of the approaching embankment decreased progressively away from the bridge abutments. The internal design of the embankments is shown, in longitudinal section, in figure 69. Transition wedges of increasing stiffness were designed to ensure a smooth transition from the compacted soil to the rigid bridge structure. The wedge closest to the abutment was specified as a cement-soil mixture. The embankment material was previously excavated in a Miocene natural formation: a sequence of claystones with some proportion of gypsum veins and interstratified sandstone layers. Some selected soil was used also as a rail track sub-base.
Once the railway line was in operation, rail levelling started on a routine basis. Results of this levelling for two dates: December 13th, 2005 and April 3rd, 2006 are shown in figure 70. The levelling performed in April 2006 shows two distinct swelling peaks in the position of the two abutments. No significant settlement or heave was detected along the bridge itself. Data in Figure 70 show a maximum heave of 12-16 mm with respect to the initial levelling. Maximum heave rate, comparing the two successive levellings, is about 4.0-4.5 mm/month.

Countermeasures were adopted and the rail tracks were levelled periodically by adjusting the thickness of the ballast layer. Since no signs of a reduction in heave rate were noticed, geotechnical investigations were commissioned. A difficulty to all the subsequent field activities was the need to maintain the railway line in full operation.

Samples taken in the first borings drilled through the two embankments indicated that the cement-treated soil was rather weak and prone to disintegration. It was also difficult to identify the expected geometry of wedges as shown in figure 69. These findings were probably the reason behind the decision to reinforce again the embankments, in October 2006, by means of 1.5 m diameter jet-grouting columns, which were arranged in the manner shown in figure 71.

The central part of the embankment was treated. Columns reached a depth of 10 m in the vicinity of the abutment. The length and the density of columns were reduced as the distance to the abutment increased. The transition zone was extended in both embankments to an overall length of 30 m.

One year after finishing jet-grouting treatment, at the end of 2007, embankment heave was again detected and track adjustment, by modifying the ballast cushion, had to start again. In May 2008 a surface topographic surveying was providing precise information on the evolution of heave. It was found that the embankment heave had resumed and meas-

Fig. 68 – Longitudinal profile and plan view of Pallares sos bridge.

Fig. 68 – Sezione longitudinale e planimetria del ponte Pallaresos.

Fig. 69 – Design of the embankment.

Fig. 69 – Progetto del rilevato.

Fig. 70 – Rail levelling after construction.

Fig. 70 – Livellazione della ferrovia dopo la costruzione.

Fig. 71 – Jet grouting treatment of embankments. (a) Position of columns in plan view. (b) Cross sections.

Fig. 71 – Trattamento di jet grouting dei rilevati. (a) planimetria con ubicazione delle colonne. (b) sezione trasversale.
ured heave rates (4.2 mm/month; 5.7 mm/month; 6.5 mm/month in different positions) were even higher than the values first observed when the problem was initially detected. Field monitoring and soil testing was thereafter increased.

6.2. Field observations

Additional surface topographic marks were installed on the top of the embankments. Figure 72 provides the topographic measurements of horizontal and vertical surface movements in one of the abutments from March 2nd, 2009 to April 19th, 2010. The records indicated that significant horizontal movements were developing in the transversal direction. An accumulated horizontal transversal movement of 150 mm was measured during the first 17.8 months of monitoring in the topographic mark PR-1.5 located 10 meters away from the Lleida abutment structure. An accumulated heave of 59 mm was measured at PR-1.5 during the same period.

The distribution of surface heave along distance is shown in figure 73 for Abutment 1. The maximum heave occurs at a distance of 10-13 meters from the abutments. At further distances the displacement decreases progressively. At distances in excess of 30 meters to the abutment no movements were detected. The transversal horizontal movements follow the same pattern. The maximum transversal horizontal movements were measured at the same points where the maximum heave was recorded.

Both embankments exhibited a similar behaviour. The topographic monitoring also allowed to measure horizontal movements along the longitudinal direction of the embankments. Points displaced towards the bridge abutment. Longitudinal displacements were substantially lower than the displacements recorded in the other two directions. However, displacements reaching 22 mm towards the abutment structure were measured during 17.8 months in topographic marks installed along the first 10 meters from the abutment. The smaller displacements recorded in the longitudinal direction is explained by the confinement applied by the abutment and the bridge structure on one side, and by the rest of the embankment on the opposite side.

Topographic stations and levelling marks were also installed outside the embankments, on the natural ground, but no movements were recorded.

The evolution of vertical displacements in time at some points of both embankments is shown in figure 74. The rate of vertical displacements has not been constant in time. From November 2008 to February 2009 a high heave rate up to 7.5 mm/month was measured. Figure 74 also provides the total ac-

Fig. 72 – Evolution in time of movements measured on the embankments surface. The movements along three perpendicular directions were monitored by topographic surveillance.

Fig. 72 – Evoluzione dei movimenti misurati sulla superficie del rilevato. I movimenti sono stati misurati lungo tre direzioni perpendicolari dalla sorveglianza topografica.
cumulated rainfall. A relation between the evolution of heave and the rainfall events can be identified. Heave rate accelerates in periods immediately following significant rainfall events.

Vertical deformation of embankments was investigated by means of continuous extensometers [sliding micrometers, KÖVÁRI and AMSTAD, 1982] installed in boreholes. Data were recorded on a monthly basis. Continuous extensometers having lengths varying from 18 to 48 meters were installed in each embankment at distances of 8, 13 and 40 meters from abutments. Measured swelling strains concentrated at the first 8 meters. Smaller compressive strains were recorded in the lower part of the embankments (Fig. 75). Strain records maintained in time the pattern of vertical variations (Fig. 75). The upper “active” level did not progress downwards. Micrometers installed at a distance of 40 m from abutments only recorded a small compression.

The integral of strains along depth, measured in micrometers, was found consistently to be very close to the surface displacements measured in topographic marks. A continuous extensometer was also installed in natural ground at the centre of the valley to check if other source of movements, other than the deformation of the embankments, was present in this case. No vertical displacements were measured in this instrument. The substratum was also shown unstrained in all sliding micrometers installed. Inclinometers records showed that horizontal movements were developing along the first 8-10 meters of boring.

The results described indicate the three-dimensional nature of the deformation of embankments as a result of an internal volumetric swelling. A significant result was the reduced longitudinal deformation, which implied that high horizontal loads could be acting against the abutments and therefore,

![Figure 73](image1.png)

**Fig. 73** – Distribution of heave magnitude, measured by topographic levelling, on the embankment surface versus the distance between the levelling mark to the abutment. Initial measurement: May 26th, 2008.

![Figure 74](image2.png)

**Fig. 74** – Evolution of vertical displacements in time measured at some levelling marks of both embankments and total accumulated rainfall recorded near the bridge.

**Fig. 74** – Evoluzione nel tempo degli spostamenti verticali misurati in corrispondenza di alcuni punti di livellazione di entrambi i rilevati e del totale della pioggia accumulata, rilevata nei pressi del ponte.
against the bridge itself. These forces were probably symmetrical, acting on both sides of the bridge. In fact, an inspection of the bridge structure revealed the existence of fissures and spalling damage at the contact between the abutment and bridge structural elements (Fig. 76). The development of swelling in the embankment induced also damage on the communications and drainage conduits on the top of embankments, near the abutments. A displacement of the abutment structure towards the bridge was noticed “de visu”. This unforeseen horizontal loading against the bridge prompted a thorough inspection of the structure and the adoption of a number of repairs which are not described here. An interesting observation, which shows the effect of two opposite forces acting on both sides of the bridge, is the pattern of bending induced cracking on the lower part of pillars, which is sketched in figure 77.

There was also a concern about the possibility of strong passive stresses developing on the upper part of the embankments, in the longitudinal direction. A passive failure could result in a risk of rail distortions. An analysis of the stress state inside the embankments, reported below, was performed. Firstly, the geotechnical properties of the embankment materials will be described.

6.3. Geotechnical data

Pallaresos Bridge crosses a small valley in the eastern part of the Ebro basin, which is filled by Tertiary deposits of Miocene age. The observation of the surface of a cut through the upper part of the embankments showed the jet-grouting columns, a cement treated soil in the lower part and the reddish compacted fill with an abundant presence of gravels and small boulders.

Samples taken in some borings were tested and vertical profiles of some identification properties were obtained. The fine fraction is low plasticity clay. Relevant information is the variation with depth of soluble sulphates in the soil, given in figure 78. It reaches values in the range 2.0-2.5% in the upper 8 m. At lower levels, the sulphate content drops to less than 0.5%. Sulphate attack to cement is likely to develop in the upper 8 m of the embankment.

6.4. Swelling tests

The soil material recovered in boreholes was homogenized in lengths of 1.20 m and compacted to reproduce the conditions in the embankment. Four samples were prepared from the embankment material from the surface to a depth of 4.80m. Samples were tested in the plastic mould used for compaction (diameter: 120 mm; height: 160 mm). Silicon grease
was previously applied to the mould inner surface. The unloaded samples were placed in a closed chamber at a constant temperature (8ºC) and the lower 20-30 mm of each sample were kept in a bath under water. Water could migrate upwards by capillary gradients. Vertical displacements were measured on the sample top.

Results are shown in figure 79. The initial response reflects a “standard” swelling associated with suction reduction and clay minerals hydration, but the long-term swelling observed in all samples cannot be explained by those mechanisms. In addition, the strong swelling measured in some samples, especially in one of them (1.20-2.40 m), is not expected in a low plasticity soil compacted at a Standard Proctor density. The significant result is that all tests exhibited a long term swelling which, despite their variation from test to test, was very relevant. These tests are also an indication of the heterogeneity of fill characteristics. The sample taken from 1.20-2.40 m, which developed the strongest swelling, was dismantled after 150 days of testing and smaller portions were subjected to X-ray diffraction and SEM observations. Ettringite and thaumasite crystals were identified. They provided also strong peaks in the X-ray diffractograms. Samples collected in the embankments were also analyzed as described below.

6.5. Mineral growth and chemical reactions

Samples for mineralogical observations were taken from several positions in the field (on the surface and in depth).

They were subjected to X-ray diffraction analysis and to SEM-EDS (Scanning Electron Microscopy-Energy Dispersed Spectrometry) observations. An optical microscope having a magnifying power 100× was also employed.

Ettringite and, most notably, thaumasite were always found. SEM photograph in figure 80 shows the two minerals. Calcite, gypsum, quartz, dolomite, illite (sericite) and kaolinite were detected in the reddish clay matrix.

Chemical formulae for ettringite and thaumasite are:

- **Ettringite:**
  \[ \text{Ca}_6[\text{Al(OH)}_6]_2(\text{SO}_4)_2\cdot 26\text{H}_2\text{O} \]

- **Thaumasite:**
  \[ \text{Ca}_6[\text{Si(OH)}_6]_2(\text{CO}_3)_2(\text{SO}_4)_3\cdot 24\text{H}_2\text{O} \]

In both minerals, the presence of water is remarkable. Thaumasite growth is regarded as a secondary process once ettringite has crystallized. Comparing the two atomic compositions, thaumasite im-
plies the substitution of Si by Al and the presence of a carbonate component. The development of both minerals follows a complex process which has been described by Hunter [1988], Mitchell and Dermatas [1992] and Mohamed [2000]. In lime or cement stabilized soils, the process starts by the hydration of lime and ionization of the calcium hydroxide. The highly basic environment (pH > 12) dissolves clay minerals, which provide a source for Al and Si. High pH also favours the dissolution of sulphate minerals, which provides Ca²⁺ and SO₄²⁻ ions. Ettringite precipitates when aluminum released from clays, calcium from cement or lime and sulphates combine with water molecules. Crystals develop in the pore solution [Deng and Tang, 1994; Mohamed, 2000].

Carbonic acid, present in the pore water, and the dissolution of calcite leads to precipitation of thaumasite, once ettringite is present. The dominant presence of thaumasite in the analyzed samples from the embankment suggests an advanced state of sulphate attack. It has been reported that thaumasite crystallizes only at temperatures not reaching 15°C, a result that was recently challenged by Blanco-Var-ela et al. [2006], who found that thaumasite may develop at temperatures as high as 25°C. Rajasekaran [(2005) reports that ettringite appears to be more stable above 15°C. The climate in Pallaresos (a marked two-season Mediterranean environment) may explain the development of thaumasite as well as ettringite at all times throughout the year.

6.6. Modelling embankment swelling

Measured swelling strains and surface heave provided data to perform a stress analysis of the embankment. A suspected passive state in the upper part of the embankment caused some concern because of a possible instability disrupting the rail tracks. Also, there was an interest of structural engineers in charge of bridge rehabilitation in estimating the existing longitudinal forces against the bridge.

A plane strain analysis was conducted (Fig. 81). The embankment material was simulated as a Mohr-Coulomb model having parameters estimated from available design and construction data (E = 67MPa, ν =0.3, c' =5 kPa, φ' = 30° ). Elastic moduli were measured in loading-reloading branches of plate loading tests performed during embankment construction. Seven tests were performed and the chosen value is close to the lower limit. Strength parameters are a conservative estimate of the compacted low plasticity soil. A zero dilatancy angle was also imposed.

Swelling was modelled by imposing a volumetric deformation distributed in the volume indicated in figure 81. This active zone was divided in sectors following the data provided by the continuous extensometers. The imposed strains were guided by two

![Fig. 81 – Geometry of finite element model.](image)

Figure 82 shows the comparison of measured and calculated heave in the period May 26th, 2008 to December 9th, 2009. The calculation provided an estimated total force of 2.32 MN/m in the transversal direction against the bridge abutment. A discontinuity is calculated at the boundary between the swelling layer and the lower non-active soil.

Calculated stresses and forces against the bridge are most probably a lower limit to the actual values because of the conservative estimate of friction and effective cohesion, having in mind that the fill remains unsaturated. Increased strength would not avoid reaching a passive state because the actual swelling experienced by the fill since the first warnings in 2006 is substantially higher than the modelled heave recorded in the shorter period analyzed.

It was concluded that, in addition to the necessity of providing a new and stable support for the rails, the stresses against the bridge abutment should also be reduced substantially. On the other hand, there was no hope of a reduced swelling rate for the immediate future.

6.7. Remedial measures

Despite the signs of a mature state of the sulphate attack (hydro-chemical calculations described; dominant presence of thaumasite) the field swelling records suggested that heave of the treated embank-
Sulphate attack is a well known mechanism of degradation of concrete and mortars made of Portland cement. The phenomenon is well-known in general terms. Portland cement has a dominant content (60-70%) of calcium oxide (CaO), a significant proportion (20-25%) of silica (SiO₂), a small proportion (2-6%) of aluminum and iron oxide (Al₂O₃, Fe₂O₃) and sulphate (in the form of gypsum: CaSO₄ 2H₂O) (1-5%). Gypsum is added to retard the paste setting.

Sulphate attack of a hardened cement paste leads to the development of ettringite, a hydrated sulphate of calcium and aluminium. This mineral crystallises in bundles of elongated filaments which retain a high proportion of water molecules in its crystalline structure. The development of ettringite implies a destruction of the strength of the cement paste and a substantial swelling. Aluminium oxides are reduced to a minimum in sulphate resistant cements. In the absence of aluminium ettringite does not develop. Another mineral, thaumasite, develops also as a consequence of sulphate attack. It is a hydrated sulphate of calcium and silicon. It may crystallize from ettringite by means of an isomorphous substitution of aluminium by silicon.

Cement and lime treated soils are often used to stabilise road bases and sub-bases. When the soil has some proportion of gypsum, or the treated soil is exposed to sulphated waters, a similar attack resulting in loss of strength and significant heave has often been reported [Sherwood, 1962; Mitchell and Dermatas, 1992; Snedker, 1996; Rajasekaran et al., 1997; Puppala et al., 2003; Rajasekaran, 2005]. Some of these studies discuss the minimum sulphate content, which triggers the attack. Most of the papers point out that sulphate contents in excess of 0.5-1% (concentration of soluble sulphate in water, by weight) result in ettringite formation and soil swelling.

However, lower threshold values (0.3%) have also been identified [Mitchell and Dermatas, 1992; Snedker, 1996]. It appears that the loss of strength of the treated soil and associated swell is related to the sulphate content. Sherwood [1962] described an unconfined compressive strength reduction of 24% of treated soil when the sulphate content was as low as 0.25%.

Unlike “pure” cement mixtures or concrete, treated soils usually contain a certain proportion of clay minerals, which are a source of aluminium and silicon ions. In fact, the highly basic environment (pH in excess of 12) created by the hydration of cement’s calcium oxide is capable of dissolving the clay minerals and releasing Al and Si atoms, ready to be integrated in ettringite and thaumasite molecules. The implication is that sulphate resistant cements, low in aluminium oxides are not necessarily a guarantee to prevent sulphate attack in treated soils. However, Puppala et al. [2003] report the good performance of sulphate resistant cement to stabilise soft and expansive clays with high sulphate content.

Most of the geotechnical literature on sulphate attack concerns the stabilisation of compacted road...
bases and sub-bases. The treatment is applied to relatively thin layers and the sulphate attack results in surface heave and reduction of soil strength. In contrast, the case described here concerns an entire embankment. The induced swelling strains not only resulted in a surface heave but in very high and totally unexpected forces against bridge abutments, which caused significant structural damage.

The case is probably an extreme case of sulphate attack to a large engineering structure and highlights the severity of the phenomenon, which may develop when a number of contributive factors cooperate to create a critical and dangerous situation.

7. Concluding remarks

The lecture describes three exceptional cases of damage induced by crystal growth associated with the presence of sulphated minerals. Sulphates are common in some geological levels such as Triassic and Mesozoic fine sediments in evaporative environments.

Swelling in tunnels crossing Triassic deposits in Central Europe is well documented. The first case history described (Lilla tunnel) served the purpose of introducing the subject and provided direct evidence of the swelling mechanism. Gypsum crystals precipitate from sulphate supersaturated groundwater. Water evaporation towards the tunnel was initially regarded as the mechanism leading to supersaturated groundwater. However, some laboratory observations and further field evidence, notably the second case history described, ruled out this hypothesis.

The heave of central pillars of the Pont de Candí Bridge is believed to be rather unique and no similar cases have been found in the literature. The singularity of the case described lies in the position of a very thick active layer deeply buried under a cover in excess of 25 m and the limited stress changes introduced by the pile foundation.

Heave of the central pillars of Pont de Candí Bridge, which are founded on massive deep foundations socketed in sound claystone is a unique case that required the combination of a few main circumstances: the presence of anhydrite under the piles tips, the existence of a network of fissures within the anhydritic claystone and a modification of the hydraulic regime of the entire foundation area. The third circumstance is associated with construction works and it is believed to be a consequence of the connection of an upper aquifer with the fissured claystone at depth. This connection was made possible by a number of reconnaissance borings but also by the pile construction itself.

The circumstances described allowed the circulation of water through the anhydrite level, its dissolution and the precipitation of gypsum crystals in the discontinuities.

Water super-saturation is a consequence of the presence of anhydrite. In fact, water in chemical equilibrium with anhydrite reaches concentrations that exceed the sulphate saturation conditions for sulphate precipitation. It has also been argued that the initial presence of sulphate crystals may play the role of seeds that facilitate the initiation and development of the phenomena.

The model developed remains within a framework of flow-deformation analysis of porous media. The porous media was assumed to be integrated by an insoluble species (inert rock matrix), two soluble species (anhydrite and gypsum) and water. Mass conservation equations have been formulated. They require the knowledge of kinetic equations providing dissolution and precipitation rates of soluble species. One important aspect of the model, insufficiently known, is the relationship between precipitated gypsum mass and strain development.

Uncertainties remain in some of the assumptions made, especially when trying to relate the basic crystal growth process and its mechanical implications. The dependence of kinetic equations for gypsum and anhydrite precipitation and dissolution on applied stress is not known with certainty.

The entire heave process develops within a fractured soft rock. This medium has been approximated by a porous medium in the model. Probably this is not a serious limitation in the case described but may prove to be too simplified in other cases.

Despite these uncertainties, the model is able to integrate observed experimental information from laboratory and field in a calculation procedure, which does not enter into major inconsistencies.

The third case describes a more familiar "sulphate attack" to cementitious materials. But the scale of the reaction, which implied a phenomenal swelling of two embankments providing access to a railway bridge is also considered to be quite unique.

Sliding micrometer data was conclusive in showing the extent of the swelling strains within the embankment. The 8 m had a sulphate content (2.5%) sufficiently high to explain the attack. However, solid gypsum gravels were scattered throughout the soil mass. They constitute "de facto" a substantial increase in gypsum content, which is not accounted for in the chemical analysis of the fine fraction of the soil. The presence of cement treated transition wedges and, specially, the installation of jet-grouted columns within the embankment triggered the attack. The growth of ettringite and thaumasite requires the presence of sulphates, clay, a basic environment provided by cement hydration and water. All of these conditions were readily available in the embankments.

Pressure developed against the bridge abutments on its upper 8 m was capable of damaging seriously the bridge. In addition, the vertical heave and a state of passive stresses, eventually leading to a
shearing failure of the upper meters of the embankments posed an additional threat to the operation of the railway line.

The lecture outlines also the corrective solutions adopted: A circular highly reinforced lining was built in Lilla tunnel. An embankment was built under the Pont the Candí viaduct to counteract, with an additional vertical stress, the deep swelling. The attacked Pallarressos embankments were excavated in part and a new structure founded on piles was executed.

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Accrescimento di cristalli in Geotecnica

Sommario

L’accrescimento dei cristalli è un meccanismo che porta a fenomeni espansivi estremi e a pressioni di rigonfiamento molto elevate per le opere di ingegneria. Nel corso della conferenza sono presentati tre casi in cui si sono avuti danneggiamenti e costosi interventi correttivi: una galleria scavata in argilliti anidritiche, un viadotto fondato su pali e su un rilevato compattato per l’accesso ad un altro viadotto. Nei primi due casi la precipitazione di cristalli di gesso nelle discontinuità della roccia ha indotto un notevole sollevamento dell’arco rovescio e forti pressioni sul rivestimento della galleria, nonché un inatteso sollevamento dei pilastri centrali del viadotto. Il terzo caso illustra invece l’accrescimento massivo di cristalli di ettringite e thaumasite in viadotti rinforzati con cemento Portland, che ha assoggettato il ponte ad uno stato di compressione assiale con conseguente danneggiamento. Oltre a questi inconvenienti si è reso necessario lo scavo degli strati sommitali del rilevato e la costruzione di strutture di sostegno aggiuntive.

Nell’articolo sono descritti inoltre i principi dell’accrescimento dei cristalli di gesso e viene formulato un modello costitutivo accoppiato idromeccanico-chimico per un mezzo poroso. Il modello è stato applicato per interpretare e riprodurre il rigonfiamento manifestato dal viadotto.

La capacità di modellazione sono state confrontate e verificate con l’andamento del sollevamento del viadotto nel lungo periodo e con la risposta della fondazione ai carichi applicati sulla superficie del rilevato che è stato sovrapposto all’impalcato del viadotto per contrastare il fenomeno di rigonfiamento.