Green Golf Club Roof Structure on Undermined Territory

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Abstract: Floods and undermining have an analogous destructive effects on building construction and decline their lifetime. Load from constrained strain of historical concrete and masonry structures. Design of the FEM model for solving interaction between foundation and subsoil. Solution of soil – structure interaction on flooded and undermined areas. Influence of groundwater level changes on settlement and tensile stresses of structures. Settlement of structure with changes of mechanically - physical properties of subsoil. Arise a cracks in masonry and foundation structures. Elimination of cracks and tensile stresses by prestress and grouting.

Key-Words: Green roof structure, undermined areas, soil – structure interaction, slide joints, crack elimination

1 Deformation load

Deformation load generally causes major internal forces within the building structures. Below are most frequent deformation impacts:

- temperature
- creeping and shrinking
- undermining
- flooding
- prestress

Fluctuation of daily/yearly temperatures influences all building structures. Under normal conditions, if limit dimension of expansion units are followed, such fluctuation does not cause cracks to arise. Major impacts result from undermining and uneven subsidence caused by water-flooded underlying rock.

1.1. Undermining Impacts.

When extracting a seam by means of a reasonably wide working (working face), changes also occur in geostatic/tectonic state of stress within neighbouring rock massif. The changes are accompanied with deformation and shift of rock from the overlying rock into the mined-out area. In case of long mining workings such as adits or roads, the impacts are not in fact evident beginning from the low depth - thanks to arch action of the rock. In case of space mining workings, a subsidence through will appear after a certain time, depending on the excavation depth, geological structure of the overlying rock, seam thickness, and excavation method. The depth and layout of the subsidence trough depend mostly on the depth (h) and thickness (m) of the seam to be extracted, and limit angle (□) for surface extraction. In the Ostrava - Karviná coal basin, the limit angle is about 65 degrees. The volume of the subsidence trough depends also on the extraction method which is expressed by the extraction factor (a), being 0.8 to 0.9 if collapse extraction is used. The subsidence through depth (s) is greater with greater seam thickness and lower seam depth under the surface.

The subsidence trough consists of an internal quiet part, and boundary parts. The depth of the internal quiet part is almost identical as the subsidence depth (s), while the boundary parts are of vital importance for the proposed protective measures taken for the ground building.

In order to describe the landscape deformation intensity in the subsidence trough boundary parts, the mining industry uses following geometrical quantities

- s - subsidence [mm],
- v - horizontal shift [mm],
- i - inclination [rad],
- R - radius of bending [km]
- □ - horizontal relative deformation [-].

The inclination (i) and horizontal shift (v) have peaks in the subsidence line inflection point (s) above the working face edge. The horizontal relative deformation (□) and the landscape bending (□ = 1/R)reach the maximum at about + 0,4 r from the working face edge.

Most dangerous for the ground buildings are the horizontal relative landscape deformations ($\tilde{\varepsilon}$ They are positive, if above the subsidence line inflection point (landscape elongation), or negative, if under the subsidence line inflection point (landscape compression).

Table given in ČSN 73 0039 classifies the sites among groups I. to V., considering the expected landscape deformation intensity caused by the underground mining. In practical calculations, the landscape relative deformation ranges between

$$1.10^{-3} \le \varepsilon_{bu} \le 7.10^{-3}$$

Taking into account the current extraction intensity in the Czech Republic, typical deformation values are about the down limit, corresponding thus to the sites of group IV. or V.

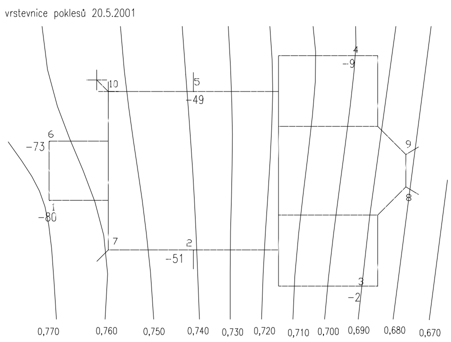

Figure 1. Subsidence Contours.

1.2. Flooding Impacts.

Major and destructive floods in 1997 and 2002 resulted in many static faults in buildings throughout the Czech Republic. Some structures were directly destroyed by water flow or drifted objects, while many other buildings were damaged by uneven subsidence after a rather long time following the floods.

Most frequent reasons for the subsidence include long-term changes in the groundwater table and changes in mechanical/physical properties of the cohesive soil.

In case of the non-cohesion soil which makes up generally good foundation soil, fine grain particles erode, as the groundwater flow is higher. The consequences include occurrence of cavities and/or loss of contact between the foundation and underlying rock.

The uneven subsidence results in stress changes in the foundation and the above ground buildings. Such changes can exceed the ultimate tensile strength of masonry, especially in old and historic buildings.

As more and more subsidence occurred in the flood-affected buildings, the faults have been similar to those typical of the buildings which are located in undermined areas. It is however very problematic to forecast such subsidence, as the floods are of a random character.

Figure 2. Set of Golf Club Structures on Revitalised Undermining Territory

Figure 3. Green Roof Structure View

2. Subsoil model

The proposal of the European pre-standard P ENV 1997-1 provides a similar, though less suitable, definition of the deformation zone as ČSN 73 1001. The depth where the compressible soil layers still need to be taken into account depend on the size and shape of the foundation, changes in the soil compressibility, depth, and location of foundation components. This depth is usually the depth where the effective vertical tension caused by the foundation load achieves 20 per cent of the effective tension from the overlying rock. In accordance with ČSN 73 1001, the structural strength coefficient is m = 0.2 for this case. The informative attachment D

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to the standard P ENV 1997-1 provides also examples of possible methods used to subsidence evaluation. The so-called modified elasticity method employs in fact the current subsidence calculation method as given in ČSN 73 1001.

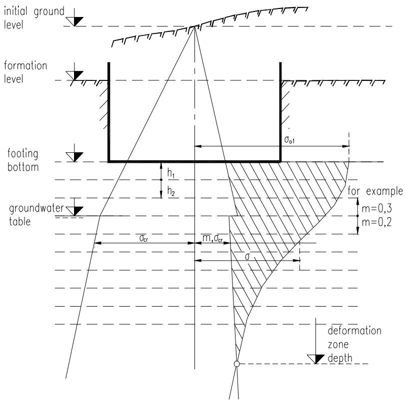

Fig. 3 Settlement Calculation Model Scheme and Active Zone Determination

3. interaction model

The Finite Element Method (FEM) is employed to calculate deformation and internal forces within the foundation. In the proposed solution, the foundation is modelled using iso-parametrical slab elements and Mindlin's theory. Shear has been also taken into account. The tension, subsidence and course of the contact tension have been calculated using a derived and general-purpose method based on transformation jacobian and numerical integration.

The state of stress and subsidence of the half space can be calculated for any shape and contact tension of the slab (1). This enable four-node and eight-node iso-parametrical elements with general node loading to be utilised. The numerical integration is applied to a selected number of integration points, using Gauss' quadrature formulae. For each node in the iso-parametrical slab element, depth of the deformation zone and subsidence are determined, and the values of the underlying rock contact functions are calculated.

$$\sigma_z = \sum_{p=1}^{n}\sum_{q=1}^{n} \alpha_p \cdot \alpha_q \frac{p_z(\xi_p,\eta_q)}{2\cdot\pi} \cdot \frac{3\cdot z^3}{r^5} \cdot det\left[J(\xi_p,\eta_q)\right] \tag{1}$$

In order to verify suitability of numeric integration for practical calculation of stress components caused by general loads from the half space surface, check examples have been calculated and results obtained have been compared with the known solution. Accuracy of □z stress as calculated has been checked for a various number of integration points and various z depths under the corner of a square, triangle and circular area of an elastic half space subject to an even load. Furthermore, the jacobian transformation has been verified.

In order to calculate the subsidence considering the structural strength pursuant to ČSN 73 1001, the vertical tension can be integrated down to the deformation tone depth, the interval thus being <0,zz>.

When determining the deformation zone depths, zz(x,y), it is assumed that the resulting vertical tension, □z(x,y,zz), is zero at the lower edge of the deformation zone. See Fig. 3. To solve this non-linear equation, the interval is to be halved numerically.

Underlying rock yielding parameters, C1z, are not constant within the element. When calculating the stiffness matrix for the underlying rock, the yielding parameters are approximated, using shape functions resulting from the shape values in individual node points

$$C_{1z} = \sum_{i=1}^{r} N_i(\xi,\eta).C_{1zi} \tag{2}$$

The shape function, Ni, can be used in a similar way to determine the course of the contact tension between the foundation and underlying rock.

$$p_z(x,y) \equiv \sigma_c(x,y) = \sum_{i=1}^{r} N_i(x,y)\cdot\sigma_{ci} \tag{3}$$

An iteration method is used to solve the non-linear interactive equation with the necessary solution accuracy or for a number of iteration steps. In the interaction calculation, it is also possible to consider unilateral bonds, poor stiffness of a slab with cracks, or loads from neighbouring structures. Occurrence of plastic areas under the foundation edge can be taken into account, when calculating the contact tension depending on mechanical and physical properties of the underlying rock.

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4. REHABILITATION OF CRACKS.

The masonry with cracks can be rehabilitated by injecting the polyurethane resin mass. The tensile strength of this mass exceeds considerably that of the masonry as well as that of the concrete. Consequently, other distortion, if any, arises out of the injected crack.

Any proposal for injection must be however supported by a detailed analysis of reasons and correction actions. Otherwise, the cracks will continue appearing in other cross-sections.

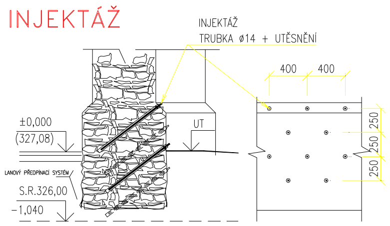

Figure 4. Combination of Original and New Foundations.

A number of different polyurethane injection systems can be used. The trade names include 3P, Ongropur, or Bevedan – Bevedol. A particular resin and resin type must be proposed considering the injection area, viscosity, necessary strength, ageing resistance, setting time, environment temperature, oil resistance, and chemical resistance. The resulting mechanical and physical properties of the injection mass or injection environment often depend on the foam level as achieved.

If the load-carrying capacity of the underlying rock is not sufficient, the deformation properties of the rock can be increased by the injection. It is however essential to keep in mind that the increase in the underlying rock stiffness results also in re-distribution of the internal forces within the foundation - even with the same load.

The underlying rock injection can not only increase the foundation soil deformation module, but also fill in the cavities between the foundation and the underlying rock. It is however necessary to monitor the injection pressure in order to avoid undesirable deformation of the structure under rehabilitation. Otherwise, it is possible that the components of the impaired structures with poor load-carrying capacity, such as floors or week masonry, will be destroyed.

The injection mass must be selected, considering its suitability in terms of environment resistance and environment durability. The quality of the mechanical and physical properties of the injection mass should be better than those of the structure under rehabilitation.

5. INCREASE IN STRUCTURE STIFFNESS.

When rehabilitating existing cracks or eliminating possible impacts of mining activities or floods, it is essential to increase the space stiffness of the building. Pursuant to many recommendations and comments to ČSN 73 0039, it is desirable to employ the stiff structure systems for following buildings, especially:

- the buildings which are sensitive to changes in the structure's geometrical shape caused by bending impacts and horizontal landscape deformation - if additional correction or step-by-step rectification is not possible.
- the buildings the layout of which enable the undermining force impacts rather than the deformation impacts to be transferred more easily - it would be more difficult to size these buildings in order to "release" such impacts. Typical examples include massive cast-in-place foundation structures, slab blocks, masonry, other tower-like building, cast-in-place or fabricated carrying structures for major effective load, or closed underground structures for sumps, channels, or collectors.

5.1. Horizontal Reinforcement of Buildings.

The additional horizontal reinforcement of walls in a masonry building ranks among most typical protections against horizontal landscape deformation or bending impacts. For that purpose, following solutions are available: reinforced-concrete rings with the closed layout, steel tie members, or prestress ropes or cables.

The reinforced concrete rings are usually used in ceiling structures. First, a groove is cut out into the masonry and stud connectors are installed. The longitudinal and transversal tensile forces are then transferred through the reinforcement of this ring. An disadvantage consists in partial weakening of the wall, this being the case of subtle masonry components mostly.

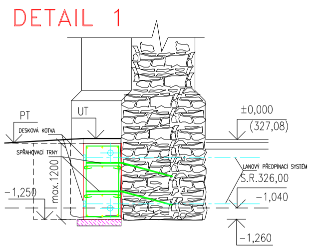

Figure 5. Location of Prestressing Cables in New Concrete Foundation which is Combined with Original Structure.

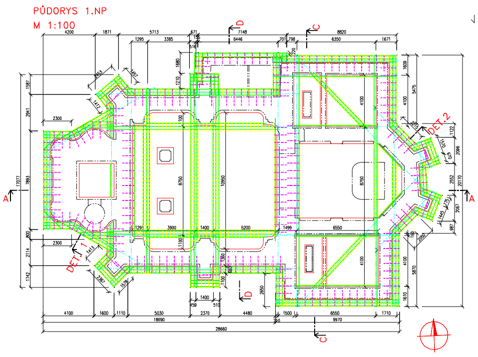

Figure 6. Layout Extension, Reinforcement, and Combination of Original and New Foundations.

The steel tie members are often installed onto the outside/inside face of the enclosing walls. Advantages include easy erection, while disadvantages consist in poor efficiency (in case of decaying impacts - too extensive prolongation of the tie members), necessity to provide corrosion protection, and the appearance.

The prestress which arises in the building under rehabilitation using the prestress reinforcement is highly effective - from the point of view of the influence, and from the method itself. The prestress cables are generally used within the building at the level of the ceiling structure. At the level of the foundations, the cables are laid in an additionally extended continuous footing which is combined with the original one.

5.2. Reinforcement and Prestressing of Foundation Structure.

Reinforcement of the foundation structures and additional prestressing of the bearing walls in their foundations is of key importance for elimination of tensile stress in the carrying system and creation of other cracks, if any. Before prestressing, it is however essential to reinforce the existing foundation made from the rubble masonry. For that purpose, the injection is placed into the joint between the rubble masonry, and the new concrete foundation ring is combined, using the stud connectors.

The purpose of the backfilling of the side surfaces of the foundations with materials with lower strength/deformation parameters is to restrict impacts on foundation side walls.

If no detailed data about the strength/deformation parameters are available for the backfilling materials, values listed in ČSN 73 0039 should be used.

Use of piles with the aim to increase the load-carrying capacity of the foundation on the undermined area entail other little-investigated-into risks:

- positive horizontal deformation of the landscape (elongation) results in partial loss of the skin friction
- vertical relative deformation in both directions have been measured in past years in the underlying rock of the buildings. The deformation of this kind is more pronounced that the horizontal relative deformation. The vertical deformation creates negative skin friction, damaging possibly the designed function of such pile.

If the combined prestress continuous footing is used, it is not necessary to provide pile foundations generally, or to increase the load-carrying capacity of the underlying rock by injection. The extended continuous footing decreases considerably the contact stress in the footing bottom, and, in turn, the friction between the foundation and underlying rock.

If the flood-affected buildings are to be reinforced, the piles represent most suitable and effective tool, increasing the resistance of the foundation structure against erosion of fine-grain articles.

5.3. Space Stiffness.

As the buildings are frequently subject to unfavourable torsion loads caused by the uneven subsidence, it is impossible to reach the required stiffness by typical rehabilitation, this means by

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mere reinforcement of the foundations. During the torsion, the walls are shear loaded. This results in typical vertical cracks along the wall thickness in regular distances. The resistance of the existing enclosing walls against the shear forces can be increased by prestressing the walls and foundations and by injecting the damaged cracks.

6. PROPOSED PROTECTION OF CHURCH.

The principles above have been used in practice, when evaluating and proposing rehabilitation of the historic St.Cross Church in Staříč close to Frýdek-Místek (Czech Republic).

The current conditions of the church - cracks in the walls and vault chords - have been caused by the undermining activities under the territory concerned. It is probable that the mining extraction will continue in the future, deteriorating thus the current condition of the building (increasing inclination and transverse torsion).

It follows from the analysis of past as well as expected mining impacts, that the building under evaluation have been and will be influenced by the mining activities. It is evident from the technical/geological survey, site survey and masonry strength tests that the building can be protected against the undermining impacts by prestressing the reinforced foundations and head-pieces of walls under cornices, by providing the reinforcing rings, and injecting the cracks.

For detailed procedure of the building works, it is necessary to prepare construction design documents, carry out survey and preparatory works, and provide analyses of structure for the design reinforcement.

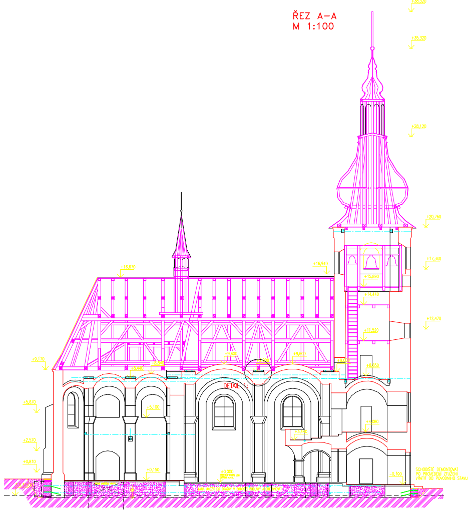

Figure 7. Longitudinal Section - Church Protection.

Based on these input values, the general static protection of this ecclesiastical architecture monument can be proposed and carried out, with no danger of damage to the structures in consequence of rehabilitation works.

10 Conclusion

Based on previous relationships the computer program NONSTAC was compiled [11], which allows to effectively solve the one-dimensional transient temperature field of multilayer plane structures with different material properties.

For the numerical solution of this modified differential equation Euler and Runge - Kutt method with a variable length of the integration step is used (*Nevřiva, 1975,* [14]). Accuracy of the solution is checked for during the calculation and if inadequate error is made an integration step is either shortened or lengthened. Conditions, error estimate, and details of the solution can be found in [14].

The advantage over commercial software products is the ability to define in the calculation a number of the temperature courses in the external environment, thermally nonlinear material properties of the individual material layers and heat transfer coefficients. Internal sources of heat, which are usually represented by the development of hydration heat during the casting, can be defined in terms of standard curves or laboratory-measured dependencies. A computer program has been successfully used in solving the casting method of solid foundation slabs [4] and the fire resistance of reinforced concrete flat structures see [5] to [10]. Preferably, it is also used for determining the temperature of sliding joints [12].

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