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The use of the GIBV method for monitoring the effects of urban excavations on built heritage

L. Piciullo, S. Ritter, A.O.K. Lysdahl, L. von der Tann, J. Langford & F. Nadim
Norwegian Geotechnical Institute, Oslo, Norway

ABSTRACT: This paper describes the application of the Ground-work Impact and Building Vulnerability methodology (GIBV) for the rapid assessment of damage caused on built heritage by an excavation. The GIBV damage assessment methodology is based on the combination of excavation-induced greenfield displacements and the vulnerability of buildings to subsidence. The combination of impact and vulnerability results in the expected building damage class. The GIBV has been implemented in a GIS tool to predict damage classes for buildings exposed to excavation-induced settlements.

This paper presents a further development of the GIBV methodology to include time dependence of consolidation settlements. The methodology is applied to assess a cultural heritage building in Oslo. The vulnerability has been assessed considering the following parameters: building length and shape for the geometrical characteristics; type of structure and foundation for the structural characteristics; and visible damages for the current condition of the building.

The results show the importance of considering long-term subsidence effects and building vulnerability when conducting a preliminary analysis of urban excavation effects in soft soils on built heritage. A comparison between predicted settlements and measured ones indicates that the conducted assessment results in reliable predictions.

1 INTRODUCTION

Monuments and historic sites are valuable assets for cultural heritage. Protecting the historic built environment from natural and human induced threats, thus, plays a vital role in sustainable development (IHBC 2020). The projected global trends of population growth and urbanization combined with climate change, however, put built heritage at risk.

Urban development requires underground construction activities that cause stress changes in the ground which often result in substantial ground displacements, especially in soft soils. In urban settings, such man-made subsidence can threaten existing historical structures (Burghignoli et al. 2013; Devriendt et al. 2013; Rampello et al. 2012). Especially in areas with subsidence prone clay deposits and sudden changes in the bedrock level, ground-water drawdown can cause substantial differential displacements and associated building damage (Langford et al. 2016b; Sundell et al. 2017). The associated consolidation process can result in subsidence continuing for decades. Reliably managing and safeguarding affected structures over long timespans can have severe economic implications. For example, in Oslo, Norway, it is projected that protecting vulnerable buildings on brick-type foundations will cost approximately € 6 billion (Venvik et al. 2018). There is therefore an urgent need to assess the potential risk of subsidence damage on the historic built environment and effectively communicate implications to decision makers.

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This paper introduces latest improvements in a recently developed assessment methodology, which is called the *Groundwork Impact and Building Vulnerability* methodology, GIBV (Piciullo et al. 2021). Specific focus is placed on an extension of the GIBV methodology which integrates time-dependent subsidence caused by pore pressure reduction and its impact on adjacent buildings. A case study approach was chosen to test and validate this latest development. The impact of a deep excavation on a historic structure, which is particularly vulnerable to ground movements, was studied. Monitoring data were explored to compare predictions with measured building behavior. Specific guidance on adopting the GIBV methodology to historic built environment is provided.

The paper is structured as follows: first, the GIBV methodology is briefly introduced; second, the time-dependent impact assessment is described in detail and evaluated by comparing results to more refined consolidation settlement calculations; third, the study area with specific focus on a historic building (i.e., Colosseum cinema) and the application of the GIBV methodology is described. The potential of adopting the GIBV methodology specifically to heritage structures is then discussed, after which conclusions are drawn.

2 THE GIBV METHOD FOR THE ASSESSMENT OF BUILDING DAMAGE DUE TO EXCAVATION

2.1 *Description of the method*

The GIBV methodology provides a framework for early assessment of potential building damage caused by excavations in densely built areas in clay or clayey soils. It assigns damage classes to buildings surrounding a planned excavation by combining (a) an evaluation of potential excavation-induced settlements (i.e., the impact of the excavation) with (b) a qualitative assessment of building vulnerability to settlement damage. To provide a practical tool for early-stage analysis of buildings exposed to excavation-induced displacements, the GIBV methodology has been programmed in Python and implemented as a tool into ArcGIS. A detailed description of the GIBV method and its validation using two case studies can be found in Piciullo et al. (2021).

Figure 1 provides a schematic overview of the methodology. The primary impact of an excavation on its surroundings are induced ground displacements. These settlements are implemented into the GIBV tool using empirical curves for excavations in typical Norwegian ground conditions for (a) settlements caused by horizontal displacement of excavation walls (i.e., so called short-term displacements, see Figure 1) and (b) pore water pressure reduction at bedrock level that leads to an increase of effective stresses and consequently consolidation settlements in the overlaying clay (i.e., long-term displacements, see Figure 1).

Figure 2 shows the short-term displacement curves as established by Langford et al. (2016a) and Piciullo et al. (2021). The curves are based on measured settlements with distance to the excavation for case studies in Norway and normalized with the depth of excavation, H. In the curves it is distinguished between excavations where the support wall (all projects are executed with conventional steel sheet pile wall structures) reaches the bedrock (blue curves) and floating walls (red curves). A difference is also made between excavations with a low safety factor (FS) against basal heave and/or a stiff wall with relatively small distance between strut levels (upper bound, dotted lines) and those with a high factor of safety against basal heave and/or a relatively flexible support wall and large distances between struts (lower bound, continuous lines). This grouping is coherent with other empirical studies of short-term excavation induced displacements where the factor of safety against basal heave, the shape of the excavation, the type of retaining structure, the geology and the depth to bedrock are mentioned as the main factors influencing greenfield displacements (e.g., Goldberg et al. 1976; Clough & O'Rourke 1990; Long 2001; Peck 1969).

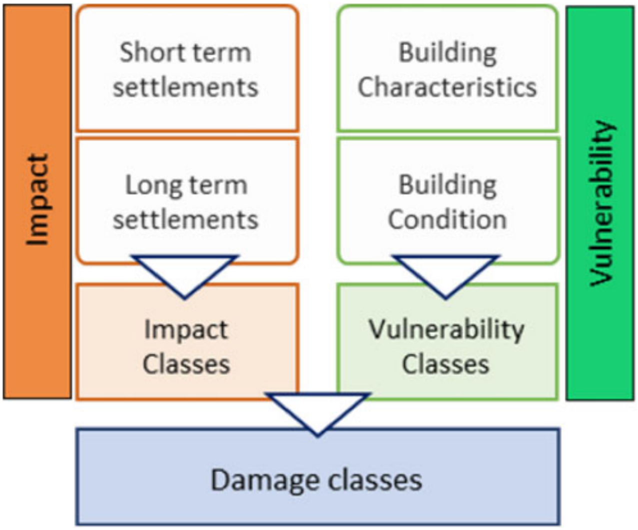

Figure 1. Ground Impact and Building Vulnerability (GIBV) methodology (Piciullo et al. 2021).

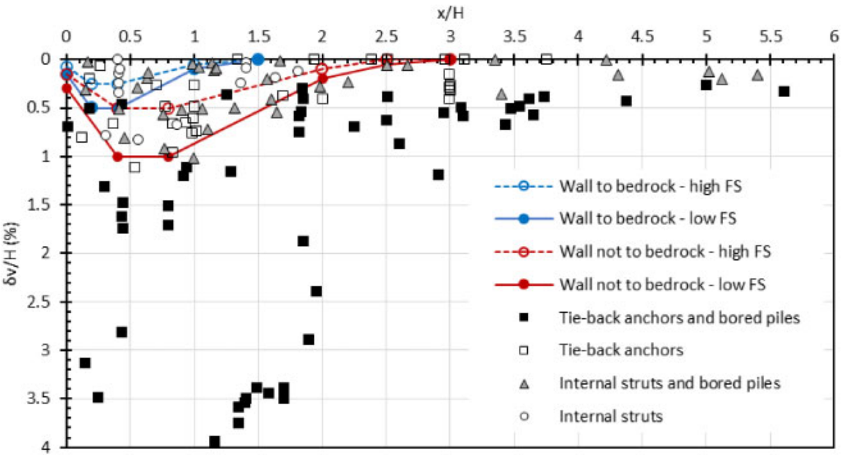

Figure 2. Expected normalized settlement, δv/H, versus normalized distance from wall, x/H in clay soils using different support methods.

The effect of pore water pressure reduction at bedrock level is in the Norwegian setting caused by leakage of water into the excavation through the interface between the soil and drilled elements such as the casings of tie-back anchors or piles, at the toe of the sheet pile wall or through uncovered bedrock. Without mitigation, a reduction of the pore water pressure in deeper permeable layers at the bedrock level can lead to long-term settlement of the overlying clay. The drawdown of pore water pressure can extend a considerable distance from the excavation. Figure 3 shows collected data of observed pore water pressure reduction at rock level from excavations in Norway as a function of the excavation depth below the groundwater level (H_{max}) and the distance from the excavation (Langford et al. 2016b). To estimate the pore water pressure reduction for ongoing or planned excavations, two curves are defined for excavations where mitigation measures such as grouting or infiltration wells are implemented and those with no such mitigation, respectively. In the GIBV methodology both curves from Figure 3 are implemented and can be chosen in accordance with the situation at hand. The consolidation settlements associated with this pore water pressure reduction are then calculated using Janbu's modulus concept (Janbu 1970). In the Nordic countries Janbu's modulus concept is widely recognized as a framework for interpretation of oedometer tests and settlement calculations of clays.

To assess the impact of the excavation on buildings in the vicinity, the short and long-term displacements are calculated and combined for the corner points of each building. For every building, the vertical settlement of the corner points and the rotation (or slope) of each wall are calculated. Using the four categories proposed by Rankin (1988) an impact class for each building is defined (Table 1).

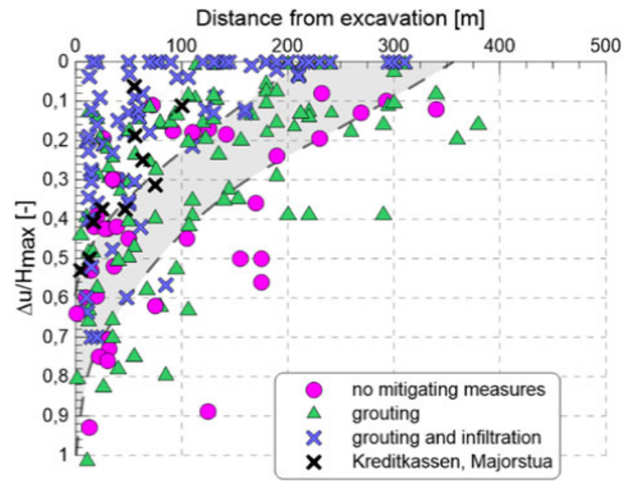

Figure 3. Database of pore water pressure reduction due to excavation works in Norway: Δu = pore water pressure reduction in m measured at the base of clay stratum, Hmax = depth of excavation base beneath initial ground water surface (adopted from Langford et al. 2016b).

Table 1. The maximum rotation and the maximum settlement are categorized in four level of impact (Adapted from Rankin 1988).

Impact level	**Maximum rotation (θmax)**	**Maximum settlement (δV,max)**
I1. Negligible	<1/500	<10 mm
I2. Slight	1/500–1/200	10–50 mm
I3. Moderate	1/200–1/50	50–75 mm
I4. High	> 1/500	>75 mm

The framework for assessing building vulnerability as implemented in the GIBV methodology is based on a qualitative evaluation of geometrical (length and shape) and structural (type of structure and foundation) characteristics and the current condition (see Table 2) of each building. For each parameter, a vulnerability grade (A-D) is assigned. The weighted sum of all parameters then provides an overall vulnerability score for the building. The weights were assigned to each parameter based on engineering judgement. For example, a higher weight was assigned to the foundation type given that the interaction between the soil and the foundation will likely govern the transfer of soil displacements to a building. To assign a final vulnerability index to the building, the derived vulnerability score is normalized by the sum of the maximum values available for each parameter. The vulnerability index ranges between 0 and 100 (Table 3). A detailed description of this rating method can be found in Piciullo et al. (2021).

Table 2. Building vulnerability rating based on five parameters. Rating method adapted from Dzegniuk et al. (1997).

Grade vi								
Characteristic	**Parameter**	A [0]	B [5]	C [20]	D [50]	**Weight Pi**	**Max value**	**Relative weight**
Geometrical	Building length (m)	≤10	11–15	16–30	>30	0.75	37.5	30 %
	Building shape	>0.75	0.75–0.5	0.5–0.35	<0.35	0.75	37.5	
Structural	Structure type	Steel	Reinforced concrete	Wooden, Mixed	Masonry, special structure	1	50	50 %
	Foundation type	To bedrock, Piles	Raft	Strip	Wooden piles, isolated	1.5	75	
Condition	Visual damage	Excellent	Good	Medium	Bad	1	50	20 %

Table 3. Vulnerability classes defined according to the normalised vulnerability index value.

	Vulnerability Index of the building			
	V1 Negligible	V2 Low	V3 Medium	V4 High
Iv	0–25	25–50	50–75	75–100

As a final step in the GIBV methodology, the level of impact and the vulnerability index are combined in a risk matrix (Figure 4) to assess the damage of a building that could be caused by the excavation assessed. Here, five damage classes between negligible (D0) and severe/very severe (D4/D5) are proposed corresponding to damage classification as used by several authors (e.g., Burland et al. 1995). The configuration of the matrix can be adjusted in accordance with specific project requirements such as, for example, for areas with historical buildings.

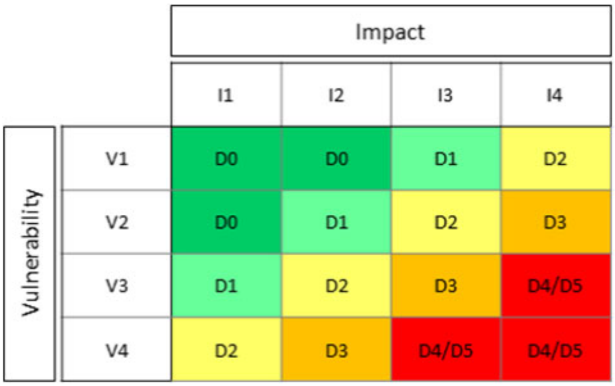

Figure 4. Expected damage classes and classification of the matrix cells. The 4x4 matrix plots the vulnerability against the impact classes.

3 TIME-DEPENDENT SETTLEMENT ANALYSIS

3.1 *Theoretical background*

The time dependent settlement analysis has been recently implemented in the impact assessment of the GIBV methodology. To calculate the settlement at the end of the consolidation phase, the theory proposed by Janbu (1970) has been applied, which describes effective stress changes in the vertical direction. The compression modulus M is expressed as:

$$M = \frac{d\sigma_v'}{d\varepsilon_v} \tag{1}$$

$$M = M^{OC} \quad \text{for } \sigma_{v0}' + \Delta\sigma_v' < \sigma_{vc}' \text{ for overconsolidated (OC) soils} \tag{2}$$

$$M = m(\sigma_{v0}' + \Delta\sigma_v' - p_r) \text{ for } \sigma_{v0}' + \Delta\sigma_v' > \sigma_{vc}' \text{ for normally consolidated (NC) soils} \tag{3}$$

where σ_v' and ε_v are, respectively, the vertical effective stress and vertical strain, σ_{v0}' is the initial vertical effective stress, p_r is the reference vertical stress for $M = 0$, m is the Janbu modulus number and σ_{vc}' is the vertical preconsolidation stress. Assuming an increment of the initial vertical effective stress, σ_{v0}', of a value $\Delta\sigma_v'$, the vertical strain increment is equal to:

$$\Delta\varepsilon_v = \int_{\sigma_{v0}'}^{\sigma_{v0}'+\Delta\sigma_v'} \frac{1}{M} d\sigma_v' \tag{4}$$

so that:

$$\Delta\varepsilon_v = \frac{\Delta\sigma_v'}{M^{OC}} \quad \text{for } \sigma_{v0}' + \Delta\sigma_v' < \sigma_{vc}' \tag{5}$$

$$\Delta\varepsilon_v = \frac{\sigma_{vc}' - \sigma_{v0}'}{M^{OC}} + \frac{1}{M} ln\left(\frac{\sigma_{v0}' + \Delta\sigma_v' - p_r}{\sigma_{vc}' - p_r}\right) \text{ for } \sigma_{v0}' + \Delta\sigma_v' > \sigma_{vc}' \tag{6}$$

The previous equations are used to compute the volumetric strain which can be used to compute vertical soil displacements. At an earlier stage of the consolidation process, the entire excess pore water pressure has not dissipated, and consequently smaller vertical settlements would be observed. To account for this time-dependent consolidation behavior, the one-dimensional consolidation theory, according to Terzaghi and Fröhlich (1936) has been considered, which describes the evolution of the excess porewater pressure $u(z, t)$ in time and space by the following partial differential equation:

$$C_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad C_v = \frac{kM}{\gamma_w} \tag{7}$$

The coefficient of consolidation C_v indicates the speed with which consolidation takes place and is a function of the permeability k of the clay, the compression modulus M and the water weight density γ_w. The general solution of this equation is a sum of many solutions containing a sinusoidal function in space and a decaying exponential function in time. The contribution from the different solutions depend on the boundary drainage conditions and initial stress in the clay. In the GIBV tool, it has been assumed that the clay layer is bounded by an open drainage at the bottom, and by a closed drainage at the top (as the gradient for a pore pressure reduction at bedrock level will be downward). Then the solution becomes:

$$u(z,t) = \sum_{n=1}^{\infty} A_n e^{-\left(\frac{n\pi}{2}\right)^2 T} \sin\left(\frac{n\pi z}{2h}\right) \qquad T = \frac{Cvt}{h^2} \qquad A_n = \frac{1}{h}\int_0^h f(z) \sin\left(\frac{n\pi z}{2h}\right) dz$$

Here, T is a dimensionless time factor and h is the height of the clay layer. The coefficients A_n are dependent on the initial (time = 0) distribution of excess pore water pressure, called $f(z)$. For example, for a square isochrone, $f(z) = u_0 h$, the equation is:

$$u_{square}(z,t) = \frac{4}{\pi} u_0 \sum\nolimits_{n=1,3,5...}^{\infty} \frac{1}{n} e^{-\left(\frac{n\pi}{2}\right)^2 T} \sin\left(\frac{n\pi z}{2h}\right).$$

In order to obtain the total settlement, it is integrated over the total depth z. Terzaghi and Frölich (1936) showed that the average degree of consolidation U for the whole clay layer at time t can be expressed as:

$$U(t) = 1 - \frac{1}{u_0 h} \int_0^h u(z,t)dz$$

Inserting the expression for $u$and carrying out the integral

$$U_{square}(T) = 1 - \sum_{n=0}^{\infty} \frac{2}{N^2} e^{-TN^2} \quad N = \frac{\pi}{2}(2n+1)$$

In a similar way, it is possible to calculate the degree of consolidation for an initial triangular isochrone, both with the tip upwards, and with the tip downwards, as illustrated in Figure 5. The result is:

$$U_{\nabla}(T) = 1 - \sum_{n=0}^{\infty} \frac{2\,(-1)^{n+1}}{N^3} e^{-TN^2} \quad N = \frac{\pi}{2}(2n-1)$$

$$U_{\Delta}(T) = 2U_{square} - U_{\nabla}$$

The three curves U_{square}, U_{∇} and U_{Δ} are plotted in Figure 5 as a function of the time factor T. For small times, the settlement is almost zero, and for large times, it approaches the long-term settlement $\Delta\varepsilon_v$,at$U = 100\%$.

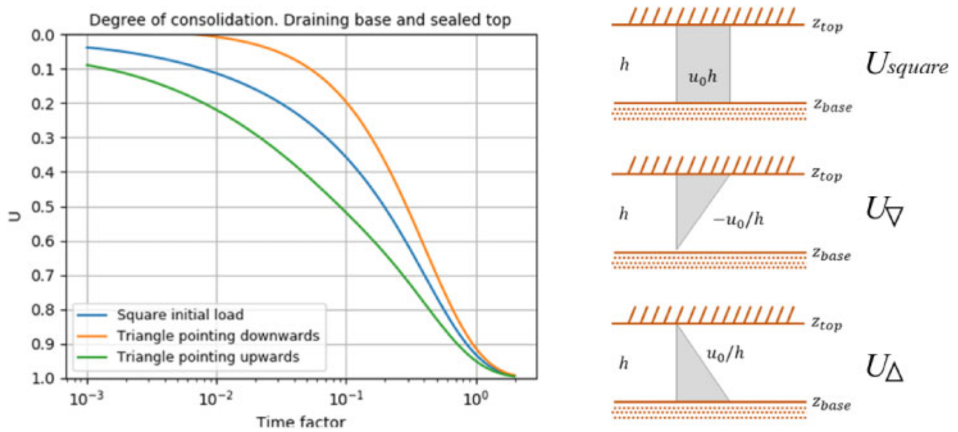

Figure 5. Triangular isochrone and corresponding U-T curve (from Terzaghi & Frohlich 1936)

In the GIBV tool, U_{Δ} has been implemented, to take into account the case of a permeable layer above the bedrock (i.e., porewater pressure reduction at bedrock level). The infinite sum in the equations above converges and could therefore be implemented in a computer program by using a for loop to a sufficient number of terms (n = 10). In the Terzaghi theory, the compression modulus M is assumed to be constant inside the clay layer. In reality, this is not the case, and for the calculation of the final settlement (i.e., end of consolidation) the clay has been discretized into layers of 1 m thickness and different M have been calculated with depth, but to calculate the settlement in time an average value of M for the clay layer has been considered to evaluate Cv. Different distributions of pore pressure reductions and boundary conditions can be implemented in the future.

3.2 *Comparison between settlements in time obtained with GeoSuite and GIBV tool*

To validate the results obtained by the described procedure, a comparative analysis was performed between the results of the time dependent settlement analysis obtained with the ArcGIS tool and the ones calculated by the commercial software Trimble Novapoint GeoSuite (https://civil.trimble.no/produkter/novapoint/novapoint-geosuite). This software is widely used in Scandinavia for slope stability and consolidation settlement analyses.

The analyses were carried out by considering an ideal simulation with a building located as close as possible to an excavation and a simple soil profile composed by a 7 m deep dry crust, a 2.5 m deep water table and 3 different clay thicknesses of 3, 13 and 23 m. The depth to bedrock for the three cases was thus 10, 20 and 30 m respectively. The geotechnical input data are summarized in Table 4. Two different over consolidation ratios (OCR) were considered: OCR = 1.0 to simulate a normally consolidated soil condition and OCR = 1.2 for a slightly over consolidated one, representing a typical range in the Oslo area. Three different scenarios for pore water pressure reductions at the bedrock level were investigated: 22.5, 50 and 100 kPa.

Table 4. The input parameters for the comparison study.

VARIABLE	UNIT	VALUES
Dry crust thickness	[m]	7
Groundwater depth	[m]	2.5
Soil unit weight	[kN/m3]	18.5
Overconsolidated ratio	[–]	1–1.2
Janbu's modulus number	[–]	15
Pore water pressure reduction at the bedrock	[kPa]	22.5–50–100

Figure 6 shows the comparison between the results obtained with the ARCGIS tool and the Geosuite software for the two different OCRs and for three different pore water pressure reductions at bedrock. The bias, i.e., the difference between the settlement values, calculated with the GIBV tool and Geosuite were plotted in Figure 7. It shows that the settlement values at the end of the consolidation phase were almost identical for the cases. This was expected as both, the GIBV tool and the Geosuite software compute the settlements at the end of the consolidation phase considering a variable value of the modulus (M) (i.e., resistance against deformation Janbu, 1970), with depth. However, small differences were observed when increasing the clay thickness and the excess pore water pressure. The maximum difference was approximately 1 cm. It can be followed that the GIBV tool is to a very small degree (i.e., less than 8% for the considered scenarios) underestimating the settlements at the end of the consolidation rate. The mismatch can be ascribed by the way the calculation has been carried out with the GIBV tool, since the formulation employed by the tool and Geosuite software is the same. The computed points have a small distance from the defined excavation. This distance, even if very small (around 1 meter), still influences the pore water pressure reduction value, that, according to Figure 3, will result in a lower value than the three values defined for this comparison (see Table 4).

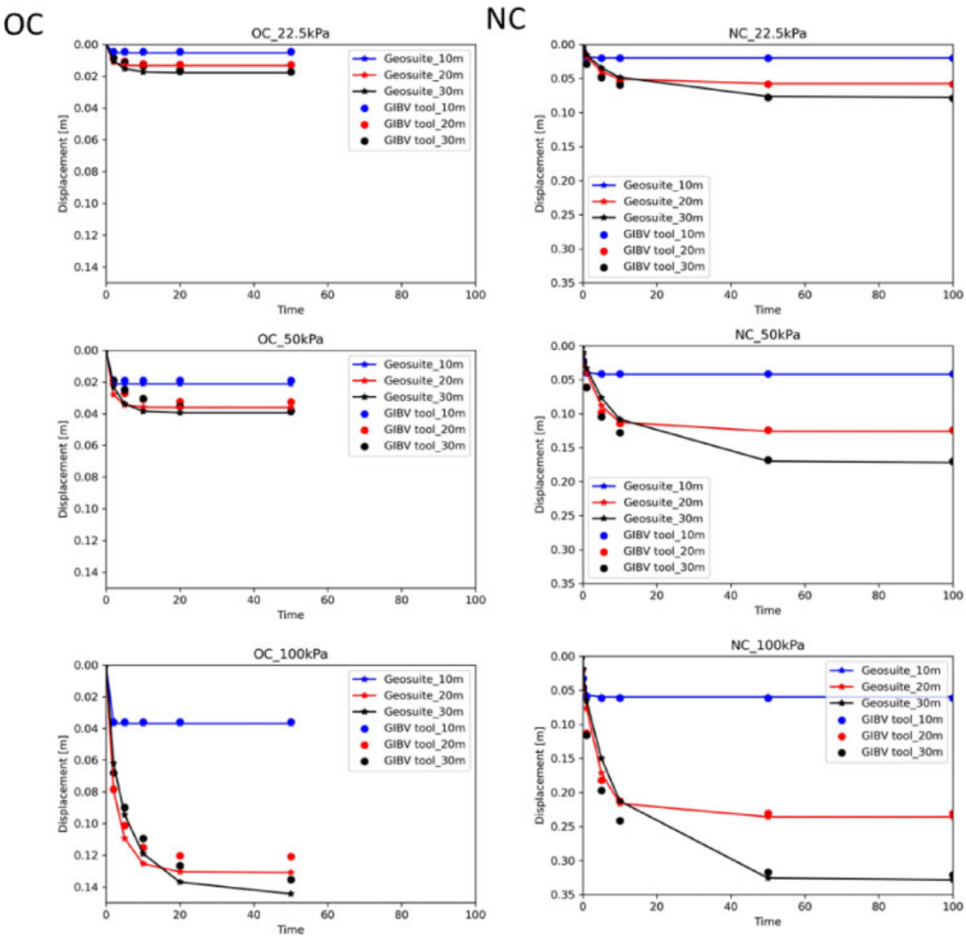

Figure 6. Comparison between consolidation settlement analysis implemented in the GIBV methodology and Geosuite reference calculations for overconsolidated clay (OC, left column) and normally consolidated clay (NC, right column). Three scenarios of porewater pressure reduction (22.5, 50 and 100 kPa) and three clay thickness (10, 20 and 30 m) were investigated.

Greater differences have been observed for the settlements in time. In this case, the formulation defined in the GIBV methodology and implemented in the ArcGIS tool (see Section 3.1) is different from how it is computed in Geosuite. The latter computes time-dependent consolidation settlements considering a consolidation coefficient varying as a function of the compression modulus, while the GIBV methodology uses a constant consolidation coefficient with time for the clay layer.

Figures 6 and 7 indicate that the solution implemented in the GIBV methodology likely results in an overestimation of the settlements at the early stage of the consolidation process (smaller degree of consolidation). As expected, the effect increases with greater clay thickness and larger pore water pressure reduction. The maximum mismatch of 5 cm after 1 year of consolidation was observed for the NC case, with 30 m bedrock depth and 100 kPa of pore water pressure reduction (Figure 7, bottom right). However, the accuracy is considered sufficient for the purpose of the tool, which is to obtain an early assessment of possible effects of deep excavations on existing buildings.

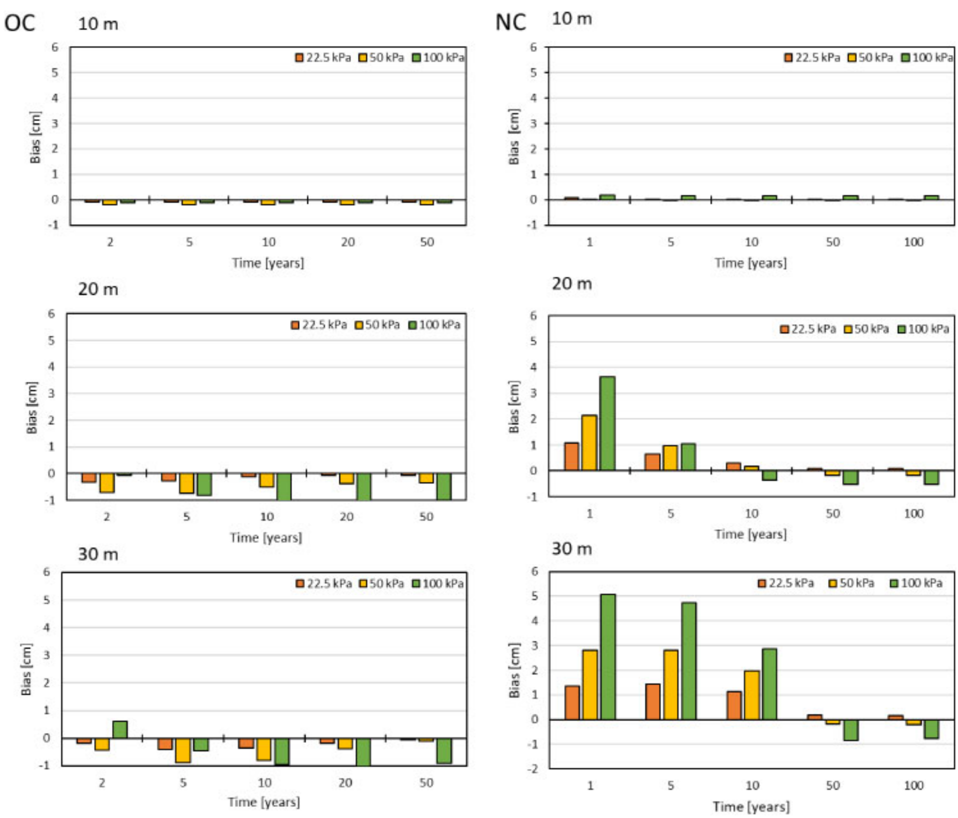

Figure 7. Differences (bias) between settlements computed by the GIBV methodology and Geosuite calculations for the scenarios described in Figure 6.

4 CASE STUDY: THE IMPACT OF AN EXCAVATION ON A SURROUNDING HERITAGE STRUCTURE

4.1 *The Colosseum cinema*

The newly introduced features of the GIBV methodology were tested on the Colosseum cinema (Figure 8), which is a historic building in the city of Oslo, Norway. The Colosseum building is one of Europe's largest cinemas. Built in 1926, the original building was a wooden structure. The building is listed as cultural heritage on municipal level. This means it is not protected as national heritage but a permission by the planning authorities is required for changes to the exterior of the building. Following a fire in 1963 in that the dome of the building collapsed, the cinema had to be rebuilt. The main hall of the cinema integrates structural elements from both, the original and the new supporting structure. The new construction reused the existing strip foundations, but additional foundations were needed to support the dome, which consists of a 8 cm think concrete shell structure. All strip foundations are founded directly on clay and the building has been subjected to settlements since 1964. The cause of the settlements is a combination of consolidation settlements due to the weight of the building as well as slow ongoing creep settlements of the marine clay. In addition, the area has been affected by underground construction of sewage tunnels in the 70s, which has resulted in pore pressure reduction and consolidation settlements. In total the settlements amount to up to 70 cm since systematic monitoring started in the 90s.

In 1997, the cinema was extended towards the South-West. At that point, it was reported that the floor of the main hall showed an inclination towards Essensdropsgate of approximately 25 cm. The extension of Colosseum cinema itself, that is built on piles reaching the bedrock, is only one of many construction projects that have taken place in the vicinity of the cinema since 1970. The location of several construction projects is illustrated in Figure 8. The VEAS sewage tunnel was constructed between 1977 and 1979 in bedrock but without pre-excavation grouting, leading to a drop of porewater pressures at bedrock level causing increased settlements in the area. For the Colosseum cinema, settlements between 10 mm/year at the most northern corner towards Fritjof Nansen's road and 17 mm/year towards Essensdrops gate were reported between 1970 and 1985. To mitigate these settlements, infiltration wells were installed in the area around New Year 1992 and additional ones in 2002. In 2002, leakage into the tunnel was also reduced through sealing by lining the bedrock tunnel. The last infiltration well was phased out in 2016.

Due to extensive construction activities in the area, the Colosseum building has continued to experience large settlements in recent years. In particular, the construction of the new building for "Kredittkassen" between 1990 and 1992 lead to immediate settlements of up to 10 cm and continuous settlements after 1992 (Figure 10). This historic construction activity influencing the Colosseum building is the focus of the GIBV application presented in this paper.

Figure 8. Colosseum cinema in 1933 (on the left, Photo by Christoffersen). Building activity around the Colosseum cinema since the 1990th (on the right).

4.2 *Kredittkassen excavation in 1990*

This excavation in Oslo had a depth of 16 m over an area of around 150 m × 100 m (see Kredittkassen area in Figure 8). The excavation was supported by a sheet pile wall installed to bedrock, supported by five levels of tie-back anchors drilled to bedrock. The soil conditions consisted of 1–2 m dry crust clay over normally consolidated soft clay. Beneath 8 m depth, the clay was quick. The depth to bedrock in that area varies from 10 to 30 m. Undrained shear strength, s_u, in clay is generally between 15 and 40 kPa. The natural water content is around 30–40%, and soil unit weight is 19 kN/m^3. The groundwater level is at approximately 2–3 m below terrain.

Ongoing settlements of 20 mm/year were registered at the time of construction, caused by drainage to existing tunnels in the area that caused a pore water pressure reduction at the bedrock of around 10–35 kPa. The bedrock beneath the sheet pile wall was grouted to a depth of 10–15 m below the bedrock surface. The rock grouting was drilled vertically from the terrain level. Six infiltration wells were used to maintain pore pressure levels during construction. The pore pressure was monitored at bedrock and in the clay with several piezometers.

4.3 *Impact in time*

In the GIBV methodology, buildings are schematized into corner points and line walls. The vertical settlement (δv) and the slope of walls (θ) are calculated respectively for every corner point and wall line (connecting two corner points), for both short- and long-term settlements. These parameters are then classified according to the four categories proposed by Rankin (1988) in the so-called impact classes (Table 1). The use of maximum vertical settlement to classify the impact, generally represents a more conservative approach than the slope because it neglects differential displacements. In this specific case study, a large difference can be seen among the two approaches (Figure 9). This difference was expected for a building which predominantly moves as a rigid body with less pronounced building distortions (i.e., bending or shear deformations). Consequently, the classification based on two settlement parameters resulted in two different impact classes after 5 years: I1 (green) when considering the slope (θ) and, I4 (red) when considering the vertical settlement (δv).

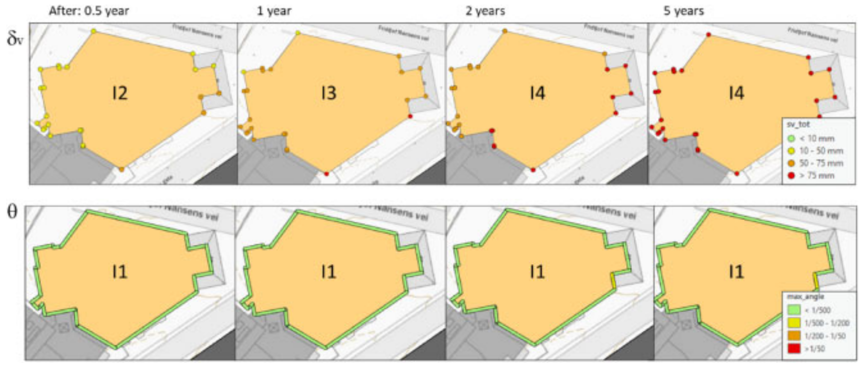

Figure 9. Classification of the groundwork impact according to Rankin 1998 (see Table 1). Top row: vertical settlements of the corner points. Bottom row: rotation (or slope) of the building walls.

For this case study, the use of the vertical settlement is likely more appropriate, since vertical settlements of up to 25 cm have been observed and the corner points of the analyzed line walls are not showing high differential settlements. The evaluation of the impact in time has been carried out evaluating the settlements for the corner points of Colosseum cinema. Figure 10 shows the comparison between measured and computed settlements using the GIBV methodology. Six corner points have been considered for the comparison: 8-1, 8-3, 8-4, 8-6, 8-7, 8-9. The comparison between measured and computed settlements shows an underestimation of the settlements with the GIBV tool. The underestimation is particularly emphasized for corner points 8-1, and 8-3 and the mismatch increased with time. This can be partially explained by the presence of ongoing settlements in the area due to previous projects (e.g., VEAS tunnel) that are not connected to the excavation of the Kredittkassen building. In Figure 10, it is possible to observe ongoing settlements before the sheet pile wall installation and excavation started, of approximately 10 to 17 mm/year. Over the period shown in Figure 10Figure 8 (i.e., 8 years), these ongoing settlements, which are not considered in the GIBV predictions, accumulate to approximately 80 to 136 mm over 5 years. Reducing the measured settlement data by these values would result in a reasonable agreement between the measurements and the predictions.

4.4 *The influence of vulnerability in the damage assessment*

For this case study the slope shows an underestimation of the impact category (see Figure 9). The following analysis thus considers the impact class defined with the maximum vertical settlement (*δv,max*) experienced by the corner points of Colosseum cinema. The impact classes in time are shown in Figure 11a. Two different vulnerability classes have been defined for the building: V2 and V3. The classifications are obtained considering two different building foundation (parameter "Foundation type" in Table 5) to consider the foundation improvement carried out in the 60s (Section 4.1): "Raft" and "Strip". Table 5 summarizes the entire characteristics considered to evaluate the vulnerability of the Colosseum cinema.

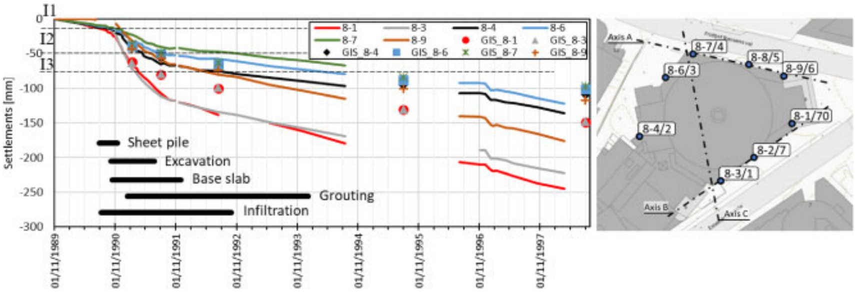

Figure 10. Comparison between predicted and measured settlements. Continuous line: measured settlements of the Colosseum cinema between 1990 and 1999. Point: settlements of the corners point of Colosseum cinema at 4 different time steps: 0.1, 1, 2, 5 years, calculated with the GIBV tool. Position of settlement bolts shown on the right.

Table 5. The characteristics considered to evaluate the vulnerability of the Colosseum cinema. Two vulnerability classes have been considered for the analyses: V2 and V3.

Characteristic	Parameter		
Geometrical	Building length (m)	>30	>30
	Building shape[1]	>0.75	>0.75
Structural	Structure type	Special structure	Special structure
	Foundation type	Raft	Strip
Condition	Visual damage	Medium	Medium
Vulnerability		**V2**	**V3**

[1] is a number representing the geometric squareness or complexity of a building polygon (see Piciullo et al. 2021).

The damage class was evaluated by combining impact and vulnerability classes employing the matrix shown in Figure 4. Figures 11b and c show the damage classes assigned to the Colosseum

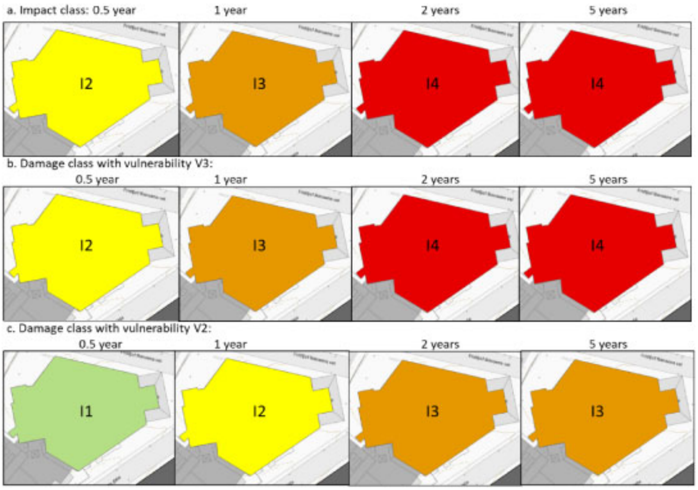

Figure 11. (a) Impact class in time considering the maximum vertical settlement ($\delta v,max$) experienced by the corner points of Colosseum cinema. Damage class in time with a vulnerability class V3 (b) and damage class in time with a vulnerability class V2 (c) for the Colosseum cinema due to the excavation.

cinema as a function of the different vulnerability classes (i.e., V2 and V3). The damage class increased as a function of the impact, which is a result of the time-dependent settlements. The vulnerability class once defined, was kept constant in the conducted analyses. A vulnerability class V3 resulted in an overall damage class D2 after 6 months from the excavation excecution and D3 after 1 year and D4/D5 after 2 years. Considering a vulnerability class V2, the damage classes of the presented time steps reduced by one class compared to V3 (Figure 11).

5 DISCUSSION AND CONCLUSION

The GIBV methodology has been implemented in a tool in the commercial software ArcGIS. The tool has recently been further developed by implementing the empirical 1D-consolidation curves by Terzaghi to compute settlements at different time steps. The results have been compared with the commercial software GeoSuite. The final, full primary, consolidation results in very good agreement (maximum difference 1 cm, circa 8%). However, the settlements at different degrees of consolidation differ more notably, with a maximum difference of 5 cm after 1 year for a normally consolidated clay. The greater the clay thickness and porewater pressure reduction at bedrock level, the greater was the bias between the settlements computed with GeoSuite and the GIBV tool. The differences between the two calculation methods are that the GIBV methodology followed the Terzaghi and Fröhlich (1936) solution while GeoSuite uses a more advanced numerical approach considering, for example, a time dependent variation of the consolidation coefficient.

The GIBV tool has been employed to evaluate the damage classes in time of an historical cinema in the city of Oslo, Norway, after the execution of an adjacent excavation. The settlements in time have been evaluated and compared with the measured data. A satisfactory match was observed up to 5 years from the excavation for four corner points (8-4, 8-6, 8-7, 8-9). The settlements of two additional corner points (8-1 and 8-3) were underestimated by the GIBV tool, especially after 6 months from the excavation. This underestimation can be explained, in this specific case study, by observed ongoing settlements in the area prior to the excavation (see Figure 10); approximately 10 to 17 mm/year were reported. One more important aspect to highlight is that, in this case study, the impact class evaluated considering the slope, instead of the vertical settlement, would lead to a lower impact class and damage class (see Section 4.1). For a more direct comparison with in-situ measurements, the impact has been classified according to the vertical settlement experienced by the corner points of the Colosseum cinema. Section 4.2 shows that the definition of the vulnerability class for the building is an important part of the assessment, specifically for cultural heritage buildings, leading to a different damage assessment.

Moreover, different risk matrix classifications, like the ones proposed in Figure 12, could be adopted by decision makers when computing the damage class for cultural heritage buildings, allowing for a more conservative approach. Figure 12a is keeping the same shape of the matrix shown in Figure 4 but equally expanding the D4/D5 class along columns and rows. Figures 4b and c represent matrix configurations allowing the highest damage class (i.e., D4/D5) with either low vulnerability (V2, Figure 4b) or low impact classes (I2, Figure 4c). The choice of the matrix

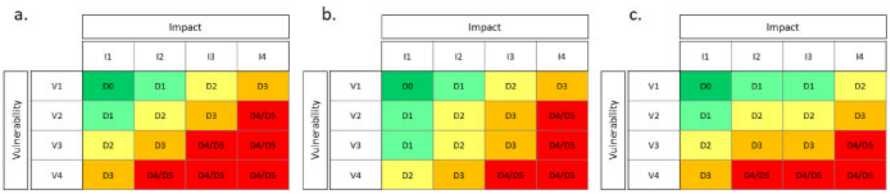

Figure 12. Proposed variations of risk matrices that could be adopted for heritage buildings: a) very conservative (very low willingness to accept damage), b) and c) configurations with low willingness to accept damage oriented respectively to impact and vulnerability.

configuration depends on the willingness to accept damages and on the weight assigned to Impact and Vulnerability by decision makers.

This study focused on a single excavation pit affecting a historical structure. Making use of historic monitoring data of a heritage structure in Oslo, it was shown that in urban settings multiple construction activities can impact a building. The GIBV methodology is currently limited to evaluate the effects of a single groundwork at a time over time. Further work needs to be conducted to better account for multiple groundworks affecting their surroundings simultaneously.

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