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# Design of the Vienna metro shaft: probabilistic analysis and settlement forecasting of the soil-structure system via Monte Carlo simulation

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Article

**Keywords**
Probability approach
Reliability
Monte Carlo simulation
Reliability assessment
Regression analysis

**Abstract**

Reliability and safety assessments require optimizing decision-making throughout the lifecycle of systems such as the "soil mass – underground structure" system. Changes in the engineering-geological environment cause various geohazards that must be identified and evaluated during the project development stage, taking into account both natural and anthropogenic factors related to subway construction and operation in dense urban environments. This article presents an analysis of shaft structures for the proposed Vienna metro line using numerical and probabilistic modeling, with particular emphasis on the quality of input data, their probabilistic variability, and their influence on the stress-strain state of the structures. Finite element modeling was performed using the PLAXIS software package, employing advanced elastoplastic Hardening Soil (HS) and Hardening Soil Small (HSS) ground models. By repeatedly varying initial soil parameters surrounding reinforced concrete tunnel structures, variation coefficients for key loads and deformations in both the structures and soil mass were obtained, which can inform the design of similar structures in the future. A methodology and algorithm for assessing the reliability of the "soil mass – underground structure" system were proposed, based on the Monte Carlo statistical method. The evaluation criterion is the probability of exceeding threshold values of additional vertical settlement after tunnel excavation in unfinished sections of the Vienna metro. This approach enables the prediction of additional deformations affecting the foundations of existing buildings impacted by tunneling. Finally, regression analysis was conducted to assess the influence of varying factors on stress-strain parameters derived from FEM calculations, using both linear and multivariate models.

## 1. Introduction

The metro is a key element of the transport infrastructure in megacities, playing a crucial role in ensuring effective mobility and the sustainability of the urban environment. In conditions of high population density and limited space for building traditional transport networks, the metro becomes an important mechanism for addressing the issues of road congestion, reducing traffic jams, and improving the environmental situation. The role of the metro in stimulating economic development and enhancing the quality of life of citizens is extremely important. The metro contributes to improving the transport accessibility of various districts of the city, which in turn affects the optimization of human flows and fosters the harmonious development of the urban environment. An efficient transport network opens up opportunities for new residential, commercial, and business projects, ensuring a more even distribution of economic activity across the city.

## 2. Instances of significant accidents occurring during subway construction

The examples presented below underscore the critical necessity of rigorous engineering analysis and meticulous planning in the execution of construction projects, particularly in historically significant and densely populated areas, in order to minimize the risks posed to surrounding buildings and urban infrastructure.

### 2.1 Incidents during the construction of the Washington metro in the 1970s

In the 1970*s*, the construction of the Washington Metro was accompanied by several incidents leading to damage to buildings situated within the influence zone of the construction activities. One of the most significant cases occurred in 1973, when a ground subsidence at the intersection

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of Connecticut and M Streets NW prompted the evacuation of nearby buildings. However, no structural damage to the buildings themselves was observed in this instance. A more serious event transpired in 1975 in the 100 block of D Street SW, where two buildings, each over 100 years old, were at risk of collapse due to the effects of Metro construction. To mitigate the potential disaster, steel cables were employed to stabilize the structures and prevent further damage. The Washington Post newspaper wrote the following about this in 1975: "Metro has been plagued by cracks in walls and building owners' complaints (Washington Post, 1975). The Lincoln Room of the National Portrait Gallery suffered severe cracks during construction about four years ago and special shoring was necessary. Two years ago, there was a cave-in at Connecticut and M Streets NW in which nearby buildings were evacuated, although no structural damage occurred. The worst damage occurred in the 100 block of D Street SW, where two buildings more than 100 years old nearly collapsed and Metro had to use steel cables to hold them together."

### 2.2 The accident at the Hangzhou metro construction site in 2008

A comprehensive study by Qian & Lin (2016), focused on the systematization and management of risks arising throughout the entire lifecycle of underground engineering structures, provides a detailed analysis of the frequency and significance of accidents that occurred in China between 2001 and 2013 during the construction of subway systems. Specifically, on November 15, 2008, a major collapse occurred at the construction site of Line 1 of the Hangzhou Metro, resulting in a 75-m stretch of Fengqing Avenue sinking to a depth of 15 m, with 11 vehicles falling into the resulting pit. The collapse of the ground outside the foundation pit caused the loss of stability of the foundation walls, leading to the failure of the supporting system, and a large amount of contaminated water flooded into the pit. Around the construction site, four hazardous buildings had to be demolished, and residents within a 500-m radius were evacuated and relocated. Work on 35 subway stations under construction across the city was halted. According to Gong & Zhang (2012), the accident resulted in 21 fatalities and 24 injuries, making it one of the most serious catastrophes in the history of metro construction in China.

### 2.3 The accident at Pinheiros station (São Paulo) in 2007

The accident at Pinheiros Station (São Paulo), which occurred on January 12, 2007, stands as the most catastrophic incident in the history of underground construction in Brazil (Tunnels and Tunnelling, 2025). It resulted in substantial material damage to both the station and the surrounding infrastructure, along with partial destruction of local facilities. Tragically, the incident claimed the lives of seven individuals, as reported by Nieble et al. (2010). An expert commission appointed to investigate the event, as described by Barros et al. (2008), concluded that the primary causes of the accident were critical design flaws and significant errors in the construction process.

In each of the cases examined, the accidents were caused by the impact of metro construction activities on the surrounding environment and existing buildings. These activities affected both residential properties and elements of urban infrastructure, highlighting the high level of risk associated with metro construction in densely developed urban areas. All the incidents posed a threat to human life and health and required significant efforts to mitigate the consequences, underscoring the need for comprehensive safety measures.

An analysis of these events emphasizes the importance of meticulous oversight of construction projects in densely populated and rapidly developing cities. While the accidents in Washington and São Paulo were primarily related to design and geotechnical deficiencies, the incident in Hangzhou serves as an example of a catastrophe resulting from errors in the design and construction processes, with more extensive and devastating consequences. In Washington, evacuation and stabilization efforts affected only a limited number of buildings. In Hangzhou, the accident necessitated the demolition of several structures and the evacuation of residents within a 500-m radius of the disaster zone, leading to a more extensive intervention. In São Paulo, efforts were focused on the restoration of the damaged station and adjacent infrastructure.

## 3. The application of probabilistic approaches to reliability assessment in the context of objects interacting with the geological environment

The examples of major accidents in subway construction vividly demonstrate the critical importance of accounting for a multitude of risk factors in the design and construction of underground structures. Consequently, to enhance safety and reliability, a probabilistic approach is increasingly being employed, which enables a more accurate consideration of uncertainties and potential risks. The use of probabilistic analysis methods in design, including in the construction of underground facilities, has become a vital tool for minimizing errors and preventing accidents. This approach allows for a more effective assessment of potential risks and the adaptation of design solutions in the context of complex geological and construction conditions.

The presented examples of major accidents during subway construction vividly illustrate the critical importance of accounting for complex interactions among multiple risk factors under conditions of uncertainty. This necessity highlights the value of adopting fundamental approaches to structural reliability, which are thoroughly addressed in authoritative sources such as EN 1990 Eurocode and ISO 2394 (ISO, 2015), alongside theoretical advancements exemplified by the Probabilistic Model Code (JCSS, 2001) developed under the auspices of JCSS.

These key documents provide a systematic framework for understanding and applying the principles of structural reliability. EN 1990 Eurocode and ISO 2394, in particular, outline fundamental concepts and methodologies, while the "Probabilistic Model Code" emphasizes the use of probabilistic methods for evaluating structural reliability, offering detailed models for critical variables and probabilistic assessment techniques.

To determine the probability of failure of a structure (system), a limit state function is used, which describes the conditions under which the structure or its elements lose their functionality. This function is developed based on the established methodology for solving a specific deterministic problem within the framework of limit state theory. When solving reliability theory problems, it is particularly difficult to determine the risk of reaching a limit state based on solving the corresponding statistical dynamics problem. Currently, the following methods are used to solve such problems: numerical integration methods, Monte Carlo simulations, and First and Second Order Reliability Methods (FORM/SORM), among others. The Monte Carlo method (Monte Carlo Simulation) is a numerical technique for solving mathematical problems, based on the modeling of random variables and conducting a large number of statistical trials using random variables in accordance with known distribution laws. This method is quite universal and is used to assess the reliability and safety of building structures and foundations when solving reliability theory problems based on modeling random variables and constructing statistical estimates, as well as for developing possible (probable) processes or outcomes using various random number generation methods, which are then applied to a real model.

The application of the probabilistic-statistical approach in reliability calculations for complex technical systems requires a functional description and comparison of resistance and loading processes, resulting in the determination of an integrated operability function. In this context, loads and impacts are considered through the nature of random processes. In other words, probabilistic-statistical reliability calculations of systems are based not on specific (deterministic) values of loads, but on discrete or continuous probability distribution functions of random variables.

The Monte Carlo method has been extensively utilized in geotechnical design, with notable applications demonstrated in works by Baecher & Christian (2003), Chalermyanont & Benson (2004), Griffiths & Fenton (2007), Pereira & Caldeira (2011), among others. The comprehensive assessment of the reliability of both structural and geotechnical systems, as well as individual structures, necessitates the incorporation of the probabilistic nature of external loads and impacts, alongside the inherent variability of material properties and soil conditions. This is of paramount importance, particularly for structures situated in aggressive environments, such as underground structures exposed to moist soil and groundwater. The Monte Carlo method for statistical analysis has proven to be a robust and highly effective tool in the evaluation of structural element and foundation reliability and safety. Its application in geotechnical design is extensive, and it adeptly addresses uncertainties within input parameters, yielding more precise and reliable results. Thus, it has become an indispensable component of contemporary engineering analysis.

Reliability assessment methods have found increasingly broad application in tunnel engineering, facilitating the development of safe and economically justified design solutions. Foundational principles regarding safety factors and reliability in geotechnical engineering were established in the seminal work of Duncan (2000), which laid the groundwork for subsequent research in quantitative risk evaluation. A significant contribution to the understanding of the soil-structure interaction system was made by Wang et al. (2014), who analyzed the processes of soil swelling and shrinkage affecting tunnel settlement. Wang et al. (2023) provided a comprehensive review of contemporary approaches to reliability analysis of tunnel structures, encompassing the evaluation of support system behavior and potential directions for methodological advancement in engineering practice. The study by Sun & Dias (2022) emphasized the critical importance of accounting for geotechnical uncertainty, quantitatively assessing seismic-induced tunnel deformations under variable soil properties through stochastic modeling. Modern principles of reliability-based design and assessments of the effectiveness of design formats were addressed by Lin & Yuan (2023), while Zhang & Lin (2021) presented multi-objective optimization approaches aimed at mitigating tunnel-induced damages.

Despite considerable progress in numerical-probabilistic analysis and the formalization of engineering solutions, there remains a compelling need to develop integrated methodologies for reliability assessment of the coupled "soil mass – underground structure" system. This need is particularly pronounced in high-density urban environments, where geological heterogeneity combines with anthropogenic factors to substantially influence the stress-strain state of subterranean constructions.

The description, modeling, calculation, and analysis of the results of these processes is a complex task, the solution of which is impossible without the use of numerical methods and the construction of correct calculation schemes. For structures operating under significant ground contact conditions, the choice of the soil model must be substantiated to a sufficiently high degree. In turn, obtaining adequate results is impossible without conducting thorough geotechnical surveys and appropriate soil testing. This paper describes the experience of using probabilistic-statistical tools (Monte Carlo simulation) in determining and analyzing the stress-strain state (SSS) of

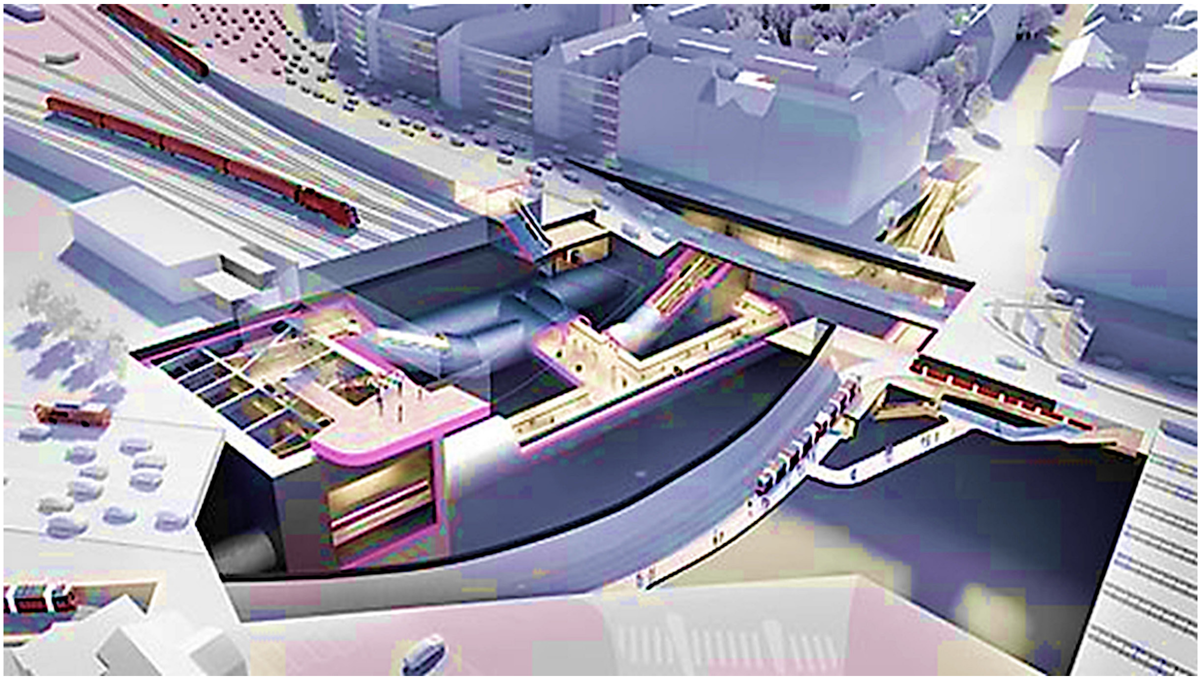


**Figure 1.** Visualization of Matzleinsdorfer Platz station (© Wiener Linien GmbH & KG. Used with permission).

the geotechnical system at the underground construction site in Vienna – Schacht Triester Straße (U2/18 Matzleinsdorfer Platz, Figure 1, 2).

# 4. Engineering-geological conditions in the area of the Vienna metro under construction

## 4.1 The role of metro systems in urban mobility and sustainable development: The case of the Vienna metro

The Vienna metro constitutes a fundamental element of the city's transportation infrastructure, serving as a key international transit hub and an essential component of urban mobility. As the capital of Austria, Vienna is a principal hub for tourism, education, and commerce, drawing millions of visitors and hosting a substantial student population. The metro system facilitates the efficient integration of the city's districts, significantly contributing to economic growth and enhancing societal well being.

Over the past 47 years, the Vienna metro has undergone five distinct phases of expansion. The current phase involves the construction of the U5 line and the modernization and extension of the U2 line. This project is characterized by its ambitious scope and technical complexity, as it is being developed within a highly urbanized area that presents challenging engineering and geological conditions (Vienna, 2025).

**Figure 2.** Reinforced concrete structures of the metro line under construction (photo by Julian Sigmund (TU Wien), 2023).

Construction methodologies are selected based on the geotechnical characteristics along the proposed alignment. In sections with favorable subsurface conditions, the open-cut excavation method is employed. However, in more geologically complex areas, the closed method is utilized, incorporating tunnel boring machines for efficient and safe excavation. Furthermore, the New Austrian Tunnelling Method (NATM) is implemented to optimize tunnelling processes, ensuring structural integrity and mitigating risks associated with underground construction in diverse geotechnical environments.

The territory of Vienna is situated on the eastern edge of the Alps, with the foothills of the Alps extending to the west and south, while the Pannonian Lowlands are located to the east and north. The absolute elevation of the city ranges from 151 m above sea level in Lobau to 542 m at Hermannskogel. To the north-west, west, and south-west of the city extends the Vienna Woods (Wienerwald), with its elevations.

The geomorphological structure of Vienna's territory is represented by a complex of Quaternary abrasion-erosion terraces that gradually descend toward the Danube River (Figure 3). These terraces are intersected by the valleys of streams and rivers, such as the Wien, Ottakringer, and Alserbach. The Vienna terraces consist not only of pure gravel but also contain sand, silt, and clay, often occurring as thin lenses that extend laterally for several hundred meters. The alluvial deposits of the Danube within the city area are significantly intermingled and fairly evenly distributed; however, to the south of the confluence of the Vienna and Danube rivers, there is a marked increase in sandy inclusions. This is particularly noticeable on the Stadt and Arsenal terraces, and to a lesser extent on the Wienerberg and Laaerberg terraces. The accumulation of sandy lenses results from the Vienna River transporting sandy erosional material from the western part of the Vienna Woods.

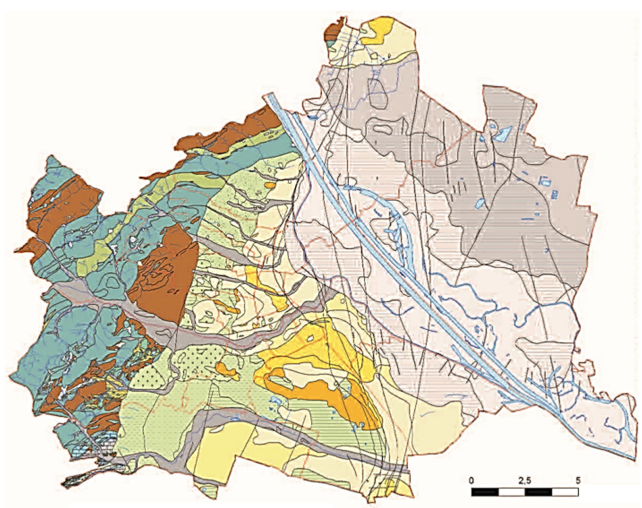
The geological structure of the city's territory is primarily composed of sedimentary deposits of Quaternary and Neogene age. The metro infrastructure is situated within the sedimentary layers of the Miocene epoch. The lower soil strata at the construction site consist of Neogene Miocene deposits, which are overlaid by Quaternary Danube alluvial deposits from the Pleistocene. Based on these characteristics, the following lithological units were distinguished, as shown in Krepper & Kronberger (2022), Figure 4:

- Danube quartz gravel (QG3);
- Miocene deposits (MZ).

The Miocene layer (MZ) is further subdivided into the following engineering-geological units:

- MZa (silt with a higher clay content);
- MZb (silt with a lower clay content);
- MZc (silty fine and medium sand);
- MZd (sand and gravel).

In connection with the expansion of line U2 and the construction of the new U5 line, a thorough investigation of the physicochemical properties of the soil beneath the buildings was carried out prior to the commencement of




Applied Geological Legend:

clay over stream and river alluvium; loess, loess loam, clay;
gravel of the Prater Terrace; gravel at the level of the City Terrace;
gravel at the level of the Arsenal Terrace; Wienerberg Terrace;
Laaerberg Terrace; flysch – limestone; flysch – sandstone
tectonic lines

**Figure 3.** Applied Geological Map of Vienna (modified from Brix, 1972).
**Legend** reduced for clarity.

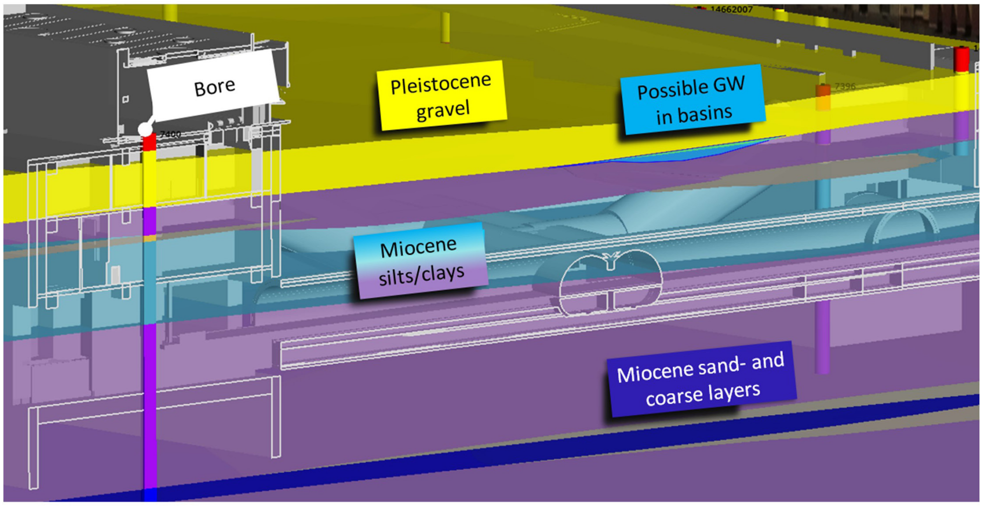



**Figure 4.** A subterranean model of the station Matzleinsdorfer Platz (Krepper & Kronberger, 2022).

construction works (U-Bahn Wien, 2025). This also involved the evaluation of their foundations, including the assessment of depth, material composition, and technical condition. In addition to the inspection of basements, an examination of the apartments in all buildings along the future metro route was conducted, during which their condition was documented to identify pre-existing damages as well as those occurring during the construction process (Mein Bezirk, 2017).

Throughout the course of the U2xU5 project, all the technical challenges posed by the Vienna soil in its various forms have been revealed: dense development with historical buildings on the surface, groundwater layers, and recurrent water ingress into tunnels and shafts (Wiener Linien, 2025). For instance, during the construction of the shaft for the future metro station on Reinprechtsdorfer Straße, additional measures were required, such as the installation of temporary foundations to counteract the forces exerted by the soil and provide further protection for the shaft. Moreover, the tunnel construction process turned out to be significantly slower than initially anticipated: the behavior of the surrounding soil during the excavation deviated from expectations, resulting, for example, in ground movements at the surface. To address these issues, further measures were necessary, which led to delays in the project schedule.

The construction of the metro line in the vicinity of Rathaus Square encountered considerable difficulties due to the presence of high groundwater levels. In order to maintain dry conditions within the tunnels during the construction phase, Wiener Linien implemented a large-scale groundwater drainage system extending from Rathaus to the Danube Canal, prior to the commencement of drilling activities in 2022.

Despite this preventive measure, unforeseen interactions with groundwater layers occurred during the construction process, including, for example, the inflow of 100,000 L of water from an aquifer into the tunnel. As a result, construction activities had to be temporarily suspended, additional pumping equipment was installed, and new wells were constructed to facilitate water extraction. The excavation in the water-saturated bedrock of the Danube led to additional soil deformations, which necessitated the implementation of supplementary safety measures and contributed to the slowing down of construction processes.

## 5. Modeling and calculations

### 5.1 Soil model

When analyzing the complex technical system of "soil mass – underground structure", it is necessary to address an integrated problem that includes evaluating the stiffness and strength of the foundation, the spatial stiffness and strength of the structure itself, its elements and joints, as well as their interaction and stress redistribution. This requires treating the foundation and the structure (building) as a unified spatial stochastic system and developing methods for its numerical analysis.

To automate the assessment of the stress-strain state in such complex systems, it is advisable to use the finite element method (FEM) in conjunction with probabilistic approaches. This approach enables the monitoring of reliability not only for individual system components but also for the entire structure as a whole. In this context, research in the field of numerical methods applied to probabilistic formulations for the "soil mass – underground structure" system remains a current and critical task in modern engineering practice.

In determining the key forces and deformations within the "soil mass – underground structure" system, the physical and mechanical properties of the soil play a pivotal role. These properties can significantly vary with depth and horizontal extent, even within a single soil layer. The variation in these properties is due to the diversity in the composition and structure of soils, their polydispersity, multiphase nature, polymineralit, and the characteristics of particle bonding, as well as the impact of geological and anthropogenic processes. Additionally, these parameters are influenced by factors such as stress magnitude, time, and engineering-geological processes that alter soil properties.

The correctness of the computational model of the "soil mass – underground structure" system, considered in this study, undoubtedly depends on the choice of an appropriate soil model. For numerical modeling, the Hardening Soil (HS) and Hardening Soil Small (HSS) models were selected, which are advanced elastoplastic models, implemented in the PLAXIS software suite.

The relationships for selected parameters of this model are presented according to Schanz et al. (2019). The parameter $E_{50}$ is the confining stress-dependent stiffness modulus for primary loading which can be described in Equation 1.

$$E_{50} = E_{50}^{ref}\left(\frac{c\cdot cos\phi' - \sigma_3\cdot sin\phi'}{c\cdot cos\phi' - p^{ref}\cdot sin\phi'}\right)^m \quad (1)$$

For unloading and reloading, another stress-dependent stiffness modulus is defined as:

$$E_{ur} = E_{ur}^{ref}\left(\frac{c\cdot cos\phi' - \sigma_3\cdot sin\phi'}{c\cdot cos\phi' - p^{ref}\cdot sin\phi'}\right)^m \quad (2)$$

For oedometer conditions of stress and strain, the oedometer stiffness modulus $E_{oed}$ for primary loading is:

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cdot cos\phi' - \sigma_3\cdot sin\phi'}{c\cdot cos\phi' - p^{ref}\cdot sin\phi'}\right)^m \quad (3)$$

Where $p^{ref}$ is the reference pressure, $E_{50}^{ref}$ is a secant stiffness modulus defined for a reference stress; $m$ is the parameter that defines $p^{ref}$ and minor principal stress, $\sigma_3$, $E_{oed}^{ref}$ is the oedometer tangent stiffness modulus defined for a reference stress. $E_{ur}^{ref}$ is the Young's modulus for unloading and reloading defined for a reference stress, $\phi'$ – angle of internal friction.

Both $E_{50}^{ref}$ and $E_{oed}^{ref}$ are necessary for the formulation of hardening model. The $E_{50}^{ref}$ controls the magnitude of the plastic strains associated with $E_{50}^{ref}$ the shear yield surface. Similarly, $E_{oed}^{ref}$ is used to control the magnitude of plastic strains originating the yield cap.

The coefficients of variation for the varying characteristics were taken in accordance with the JCSS Probabilistic Model Code, Section 3.7: Soil Properties, as well as with the consideration of the research by Phoon & Kulhawy (1999), Griffiths & Fenton (2007).

## 5.2 Calculation scheme

The calculation model of the "soil mass – underground structure" system for the metro section was developed and analyzed using the PLAXIS program (Figure 5). The dimensions of the soil massif are 110 × 200 m.

The specified distributions for loads and materials are mainly taken from the Probabilistic Model Code, draft Eurocode 7 prEN 1997-1:2022 (E) (CEN, 2003), and from some other sources of scientific literature, including Griffiths & Fenton (2007), Misra & Roberts (2007). Geometric dimensions of building structures and their physical and mechanical characteristics are accepted in accordance with the design data.

Following a comprehensive series of calculations, the full set of parameters characterizing the stress-strain state of the "soil mass – underground structure" system within the

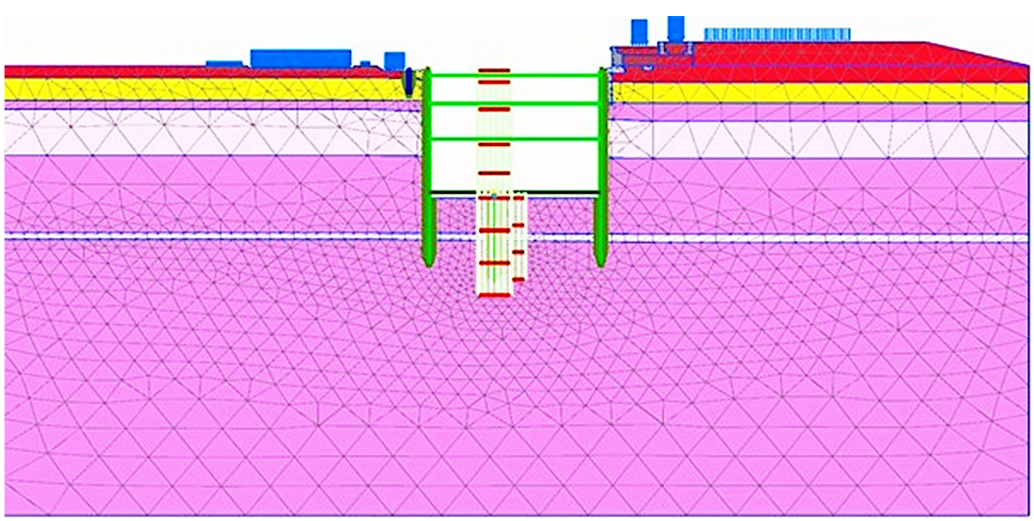


**Figure 5.** Calculation scheme.

metro construction site was determined. Figure 6 presents one of the possible distributions of vertical deformations in the analyzed system.

In the course of the calculations, empirical distributions of the parameters were modeled based on the initial probabilistic and statistical characteristics, which constituted the input data for the formulation and solution of this problem.

### 5.3 Calculation algorithm

The conducted numerical experiment determines all the parameters of the stress-strain state of the system using Finite Element method and Monte Carlo method in the following order. Values of some parameters of the soils used in the calculation ($\gamma$, $\phi'$, $c$, $E_{50}^{ref}$, $E_{oed}^{ref}$, $E_{ur}^{ref}$) and laws of distribution of values are shown in Table 1. The SSS random values of the system are fixed.

Since the calculation model includes not only the self-weight of the soil but also additional loads (from buildings, tram tracks, vehicles, and sidewalks with pedestrians) (see Figures 6), these loads were also subject to variation. The magnitudes of the loads and their coefficients of variation were determined in accordance with regulatory documents and European standards.

In the design and construction of underground structures in densely urbanized areas, one of the key requirements is to ensure the preservation of surrounding buildings. This requirement is more stringent than the conventional settlement limit (SLS), as the permissible deformations for existing buildings are always smaller than those for newly constructed structures. Therefore, when designing underground structures, it is essential to adopt structural solutions that ensure not only the reliability of the structure itself but also the safety of adjacent buildings. In this context, a quantitative risk criterion for existing buildings may involve limiting the additional settlements of urban buildings to an acceptable level: the magnitude of additional deformations must not exceed the allowable threshold. To assess the probability of exceeding these deformations, it is advisable to apply the statistical testing method (Monte Carlo method) based on calculations performed using the PLAXIS software.

At the second stage, the rules and probabilistic parameters of two random variables are defined: 1) the maximum permissible values of additional vertical deformations

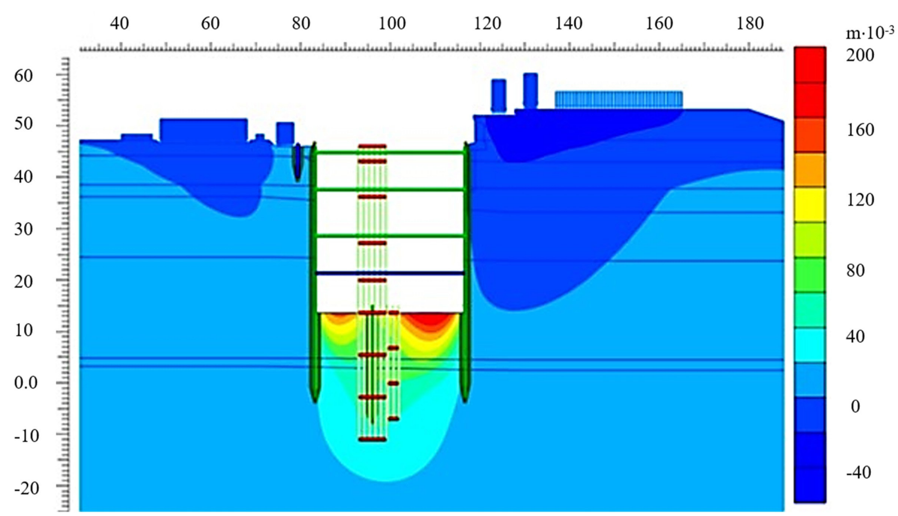



**Figure 6.** Distribution of vertical deformations as a result of one of the calculation variants.

**Table 1.** Parameters of soils accepted for calculation.

| Parameters | Designation | Distribution | *COV* | Mean (1 layer) | Mean (2 layer) | Mean (3 layer) | Mean (4 layer) | Mean (5 layer) |
|---|---|---|---|---|---|---|---|---|
| Weight density (kN/m3) | $\gamma$ | normal | 0.05 | 19 | 21 | 20 | 20 | 20 |
| Angle of internal friction (°) | $\phi'$ | normal | 0.1 | 25 | 35 | 25 | 25 | 25 |
| Cohesion (kPa) | $c$ | normal | 0.1 | 1 | 1 | 30 | 30 | 30 |
| Secant stiffness modulus (MPa) | $E_{50}^{ref}$ | log-normal | 0.2 | 10 | 60 | 20 | 20 | 20 |
| Oedometer modulus (MPa) | $E_{oed}^{ref}$ | log-normal | 0.2 | 10 | 60 | 25 | 25 | 25 |
| Secant stiffness modulus (MPa) | $E_{ur}^{ref}$ | log-normal | 0.2 | 40 | 240 | 100 | 100 | 100 |

$S_{add}$. 2) the calculated values of additional vertical deformations $s$, which are determined through Finite Element Method (FEM) calculations.

The following distribution laws are adopted for the input parameters:

- The distribution of the random variable $P_{S_{add}} = P_{S_{add}}(S_{add})$, representing the maximum additional vertical deformations, is assumed to follow a normal distribution, with a mean value $m_{S_{add}} = 0.03$ and a standard deviation $\sigma_{S_{add}} = 0.003$ m.
- The distribution of the random variable $P_s = P_s(s)$, representing the calculated deformations, is also assumed to follow a normal distribution, with parameters $m_S$ and $\sigma_S$, which are determined from the results of FEM calculations.
- Using the Monte Carlo statistical testing method, $10^5$ trials were conducted with the Mathcad software suite. For each trial, the condition $s \le S_{add}$ was checked. After performing all N trials, the probability of exceeding the additional deformations during the design service life was calculated as the ratio of the number of trials where $S_{add} - s < 0$ to the total number of trials. An analogous calculation is also carried out for the 50-year service period.
- According to EN0, the probability of reaching the limit state (probability of structural failure) $P_f$ can be calculated using the formula: $P_f = \Phi(-\beta)$, where Φ is the cumulative distribution function of the standardized normal distribution, and β is the reliability index.
- The failure probability was found to be $P_f = 0.002$ for a 1-year period and $P_f = 0.0055$ for a 50-year period. The reliability indices were $\beta_1 = 3$ and $\beta_{50} = 2.71$, which are acceptable values for the SLS state.

The value of $S_{add}$, set at 3 cm, was determined based on a comprehensive analysis of monitoring data for buildings and structures, as well as theoretical and applied research. This choice is grounded not only in empirical data but also in the practical consideration of allowable deformations that ensure the safety of existing buildings.

With high probability, it can be assumed that the surrounding construction primarily falls under consequence class CC2, for which the minimum value of the reliability index β is 2.9 for a 1-year period and 1.5 for a 50-year period (according to the EN 1990:2002). The three reliability classes − RC1, RC2, and RC3 − can be associated with the three consequence classes − CC1, CC2, and CC3. According to EN1990:2002 (CEN, 2003), the designations RC1, RC2, and RC3 represent reliability classes, which can be determined using the reliability index β. Table 2 also provides the recommended minimum values for the reliability index associated with each reliability class (in accordance with EN 1990:2002).

The calculation results are summarized in Table 3. Figures 7 and 8 depict, respectively, the probability density functions (PDFs) and cumulative distribution functions (CDFs) for three principal random variables employed in the conducted analysis. These variables comprise the calculated additional settlement ($s$, represented by the light gray curve), the additional settlement accumulated over a 50-year period ($s_T$, dark gray curve), and the maximum allowable additional settlement ($S_{add}$, black curve). The presented functions facilitate a quantitative assessment of the range of possible values of the investigated random variables, as well as the probability of exceeding the established serviceability limit within the designated design service life of the structure.

### 5.4 Analysis and results

Based on the results of finite element method (FEM) calculations with varying initial data, all characteristics of the stress-strain state of the “soil mass – underground structure”

**Table 2.** Reliability classes and recommended minimum values for reliability index β (CEN, 2003).

| Reliability Class | Minimum values for β | | | | | |
|---|---|---|---|---|---|---|
| | Ultimate limit states | | Fatigue | | Serviceability | |
| | 1 year reference period | 50 years reference period | 1 year reference period | 50 years reference period | 1 year reference period | 50 years reference period |
| RC1 | 5.2 | 4.3 | | | | |
| RC2 | 4.7 | 3.8 | | 1.5 to 3.8 | 2.9 | 1.5 |
| RC1 | 4.2 | 3.3 | | | | |

**Table 3.** Results of probabilistic calculations of exceeding for additional ultimate deformation of the base.

| Name of the values | Reference period | |
|---|---|---|
| | 1 year | 50-year |
| Probability of exceeding the additional limit deformation of the base | $2 \times 10^{-3}$ | $5.5 \times 10^{-3}$ |
| Recommended EN minimum values (SLS) (normal level) | $3.2 \times 10^{-3}$ | $8.1 \times 10^{-2}$ |
| Reliability index β | 3 | 2.71 |
| Recommended minimum values (SLS) (normal level) | 2.9 | 1.5 |

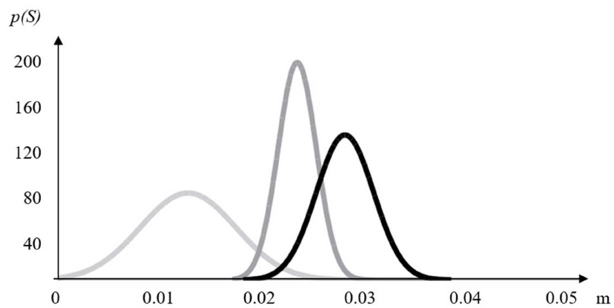



**Figure 7.** Probability density functions of $s$, $s_T$ and $S_{add}$.

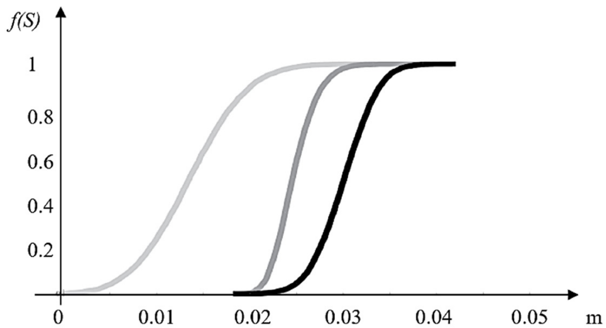



**Figure 8.** Cumulative distribution functions of $s$, $s_T$ and $S_{add}$.

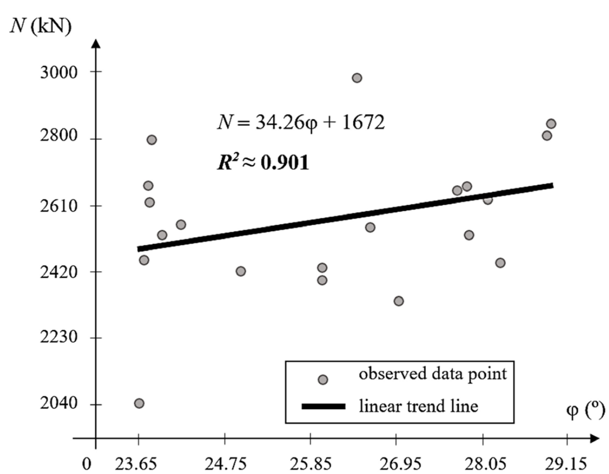



**Figure 9.** 90.1% of the variability in the longitudinal $N$ force values can be explained by a linear regression model.

system were derived, along with the coefficients of variation for these characteristics, which are as follows:

- for longitudinal forces $N$, $COV = 0.08$ (8%);
- for bending moments $M$, $COV = 0.09$ (9%);
- for shear forces $Q$, $COV = 0.11$ (11%);
- for vertical displacement $s$, $COV = 0.237$ (≈24%);
- for horizontal displacement, $COV = 0.2$ (20%).

Following the verification of the obtained values for all the aforementioned parameters using the agreement criterion at a significance level of 0.05, it was established that these parameters exhibit a normal distribution.

Probabilistic analyses conducted using the Mathcad Prime software package revealed that the probability of system failure, based on the criterion of additional vertical deformations, is $P_f = 0.002$, corresponding to a safety index $\beta = 3.0$, which is considered satisfactory and complies with the guidelines specified in the "Probabilistic Model Code."

### 5.5 Regression analysis

As part of this study, a regression analysis was conducted to evaluate the influence of the varying factors – $\gamma$, $\phi'$, $c$, $E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref}$ – on the primary parameters of the stress-strain state, which were derived from the calculations: $M$, $N$, $Q$, and the vertical and horizontal deformations of the soil mass. Prior to the commencement of the numerical calculations using the PLAXIS software, the values of the varying factors were averaged over the thickness of the soil layer involved in the collaborative interaction of the underground structure with the foundation. This was necessary due to the fact that the numerical model encompasses five distinct soil layers, each characterized by its unique properties. It should be noted that, when compared to the initial results, such averaging exerted a minimal effect on the magnitudes of forces and deformations, and can be employed, for instance, in preliminary (approximative) calculations.

Regression analysis constitutes a set of statistical methods used to evaluate the relationships between variables, and is applied to assess the degree of interdependence between variables as well as to model future dependencies. The regression analysis incorporates several models, the most prevalent of which are: linear, multilinear (multiple linear), and nonlinear models. In this study, the first two models were utilized, and the regression analysis was conducted based on the least squares method.

After an extensive evaluation of various alternatives, an intriguing pattern was identified: the values of the longitudinal force $N$ in the shaft wall can be satisfactorily represented by a linear regression model, as 90.1% of the variability in the longitudinal force values is accounted for by this model, as shown in Figure 9. In this two-factor experimental setup, the independent variables were the unit weight of the soil $\gamma$ and the angle of internal friction of the soil $\varphi$, while the dependent variable was the longitudinal force $N$. All computations were performed using the Mathcad software environment. Subsequently, an analysis was conducted to determine the relationship between the unit weight of the soil and the longitudinal force values, revealing that 86% of the variability in the longitudinal force can also be described by the linear regression model (Figure 10).

Furthermore, an experiment was conducted to establish the correlation between vertical deformation values and the

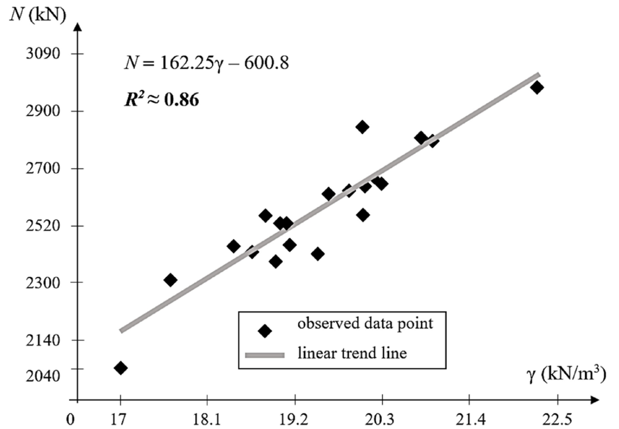



**Figure 10.** 86% of the variability in the longitudinal N force values can be explained by a linear regression model.

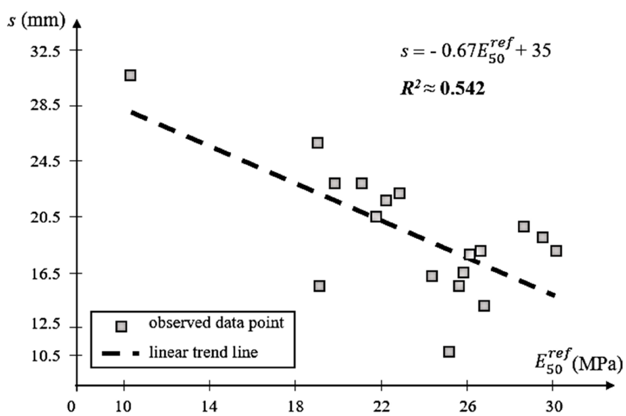



**Figure 11.** 54.2% of the variability in vertical deformation can be explained by a linear regression model.

unit weight of the soil γ as well as the modulus $E_{50}^{ref}$. It was found that the variability in vertical deformation values is also linearly dependent on these two independent variables; however, the linear model accounts for only 54.2% of the variability in the deformation values, as shown in Figure 11.

## 6. Conclusions

The modeling of empirical distributions for the parameters of the "underground structure – soil" system facilitated the construction of a more refined statistical model, which encompasses the full spectrum of parameters governing the stress-strain state of this system. The extensive variation analyses performed utilizing the PLAXIS software suite yielded coefficients of variation for the primary forces and deformations, which constitute a critical component in the reliability evaluation of structural integrity. These coefficients are instrumental not only in the design and strength assessment but also in the selection of cross-sectional profiles and reinforcement for reinforced concrete structures of comparable typologies.

The obtained range of displacement values for buildings during the construction process, alongside the reliability assessment conducted via Monte Carlo simulations, enabled an in-depth evaluation of the impact of additional settlement on surrounding structures. A reliability assessment algorithm for the "underground structure – soil" system was applied, based on the criterion of additional settlement of buildings, which was operationalized for the construction site located at Schachtes Triester Straße (Baulos U2/18 Matzleinsdorfer Platz). The system's failure probability was calculated to be 0.002 (for a period of 1 year) and 0.0055 (for a period of 50 years), values that are considered entirely acceptable within the framework of the Serviceability Limit State (SLS) condition. The reliability indices $\beta_1 = 3.0$ and $\beta_{50} = 2.71$ further substantiate the system's robustness under the specified loading conditions and operational parameters. Furthermore, the results derived from this analysis are applicable in forecasting potential settlements during metro construction in any geotechnical context, thereby enabling a proactive approach to assessing the impact of construction activities on pre-existing structures and mitigating the risk of induced damage.

A comprehensive regression analysis was undertaken to investigate the influence of varying soil characteristics on the principal parameters of the stress-strain state of the system, revealing a substantial impact of these characteristics on the computed results. Correlational dependencies among several parameters were established, facilitating a more precise characterization of the behavioral model of both the soil and the structures during their operational lifecycle. This insight is of paramount significance for further enhancing the precision of analytical models and advancing the design of infrastructure situated within regions characterized by complex geotechnical conditions.

## Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

## Authors' contributions

Oksana Kichaieva: conceptualization, data curation, methodology, investigation, visualization, writing – original draft. Dietmar Adam: conceptualization, supervision, validation, formal analysis, project administration, resources, software, writing – review and editing.

## Data availability

The data sets generated analyzed in the course of the current study are available from the corresponding author upon request. All data produced or examined in the course of the current study are included in this article.

## Declaration of use of generative artificial intelligence

Generative artificial intelligence (GenAI), specifically OpenAI's ChatGPT, was used solely to assist with minor language editing and grammar correction. The use of this tool was limited to improving linguistic clarity and did not contribute to the scientific content or interpretation of the work. All edits were reviewed and approved by the authors, who take full responsibility for the final content.

## List of symbols and abbreviations

| | |
|---|---|
| $c$ | Cohesion |
| $m$ | Parameter, that defines the reference pressure and the minor principal stress, σ3 |
| $m_s$ | Mean value of $s$ |
| $m_{S_{add}}$ | Mean value of $S_{add}$ |
| $P_f$ | Probability of failure |
| $p^{ref}$ | Reference pressure |
| $s$ | Vertical displacement |
| *CC1, CC2, CC3* | Consequence classes according to EN 1990:2002 (CEN, 2003) |
| *CDFs* | Cumulative distribution functions |
| *COV* | Coefficient of variation |
| $E_{50}$ | Secant modulus |
| $E_{50}^{ref}$ | Secant stiffness modulus for a reference stress |
| $E_{ur}$ | Stiffness modulus for unloading/reloading |
| $E_{ur}^{ref}$ | Secant stiffness modulus for unloading/reloading at a reference stress |
| $E_{oed}$ | Oedometric modulus |
| $E_{oed}^{ref}$ | Oedometric modulus for unloading/reloading at a reference stress |
| EN | European Norm |
| FEM | Finite Element Method |
| HS | Hardening Soil |
| HSS | Hardening Soil Small |
| ISO | International Organization for Standardization |
| JCSS | Joint Committee of Structural Safety |
| $M$ | Bending moment |
| MZ | Miocene deposits, Miocene layer |
| MZa | Silt with a higher clay content |
| MZb | Silt with a lower clay content |
| MZc | Silty fine and medium sand |
| MZd | Sand and gravel |
| $N$ | Longitudinal force |
| NATM | New Austrian Tunnelling Method |
| $P_s$ | Distribution of random variable $s$ (calculated deformations) |
| $P_{S_{add}}$ | Distribution of random variable $S_{add}$ |
| *PDFs* | Probability density functions |
| $Q$ | Shear force |
| QG3 | Danube quartz gravel |
| *RC1, RC2, RC3* | Reliability classes according to EN 1990:2002 (CEN, 2003) |
| $S_{add}$ | Additional vertical deformation |
| $S_T$ | Additional settlement accumulated over 50-year period |
| SLS | Serviceability limit state |
| *SSS* | Stress-strain state |
| β | Reliability index |
| $\beta_1$ | Reliability index for 1-year period |
| $\beta_{50}$ | Reliability index for 50-year period |
| γ | Weight density |
| $\sigma_3$ | Minor principal stress |
| $\sigma_s$ | Standard deviation of $s$ |
| $\sigma_{S_{add}}$ | Standard deviation of $S_{add}$ |
| $\phi'$ | Angle of shearing resistance in terms of effective stress (angle of internal friction) |

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