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CIVIL ENGINEERING

Assessment of safety of masonry buildings near deep excavations: impact of excavations on structures

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Abstract. The knowledge of the impact and the load-bearing capacity of unstrengthened/strengthened structures is a crucial source of information about the safety of masonry buildings near deep excavations, especially in dense urban areas. Incorrect calculations made for such designs can seriously affect not only an analyzed object but also the adjacent buildings. The safety of masonry buildings can be determined by many factors that are closely related to the hazards presented during the performance of deep excavations. These factors are at first identified and then prioritized. The AHP process in the multi-criteria analysis was used to support the decision-making process related to the verification of factors affecting the safety assessment of masonry buildings in the area of deep excavations. The proper design of building structures, including the verification of the structure strengthening near deep excavations, was found to be the most significant factor determining the safety of such buildings. The methodology for proceeding with the verification of ultimate (ULS) and serviceability (SLS) limit states in accordance with the literature data, current regulations, such as Eurocode 6 and other design standards, and the know-how of the authors, described in this paper was the next stage of the discussed analysis.

Keywords: structure safety; deep excavations; excavation impact on masonry structures; AHP; ULS; SLS.

1. INTRODUCTION

The need for areas in high-density housing results in building new civil structures near existing buildings, and underground and above-ground urban infrastructure [1, 2]. The design of building structures and the verification of limit states are crucial to the safety of unstrengthened/strengthened masonry buildings in the vicinity of deep excavations [3–5]. Masonry buildings near deep excavations are influenced by facility operations and the environment, including environmental impacts exerted by ground deformations. As they are subject to damage, the analyses performed on such structures near deep excavations require the complex management of georisk and safety assessment [5,6] This type of project requires precise methods of ground testing [7] adjusted to the urban fabric and the excavation method.

Many construction disasters related to the execution of deep excavations have occurred [8,9], causing not only material losses but also casualties. Therefore, the safety assessment of masonry buildings related to construction projects requiring the execution of deep excavations is an important research area in civil engineering and construction management. It should be emphasized that the implementation of such projects always requires the minimization of the construction impact on the infrastructure elements and buildings nearby. The engineering of deep foundations refers to projects consisting of the execution of excavations having a depth of three meters or greater. The main elements determining the safety of civil structures in the vicinity of deep excavations include dewatering, a structural design – verification of limit states, the execution of deep excavations considering the safety of existing civil structures, monitoring, and supervision [8, 9].

Identification of these factors is an important aspect of analyses concerning the safety of construction facilities near deep excavations. Such projects are characterized by the implementation of different types of construction methods and complex types of support systems for deep excavations. And their impact on the safety of the infrastructure of existing buildings is crucial. The probabilistic assessment can be applied to various factors affecting the safety of buildings near deep excavations. The papers [10, 11] proposed the probabilistic assessment of risks related to groundwater leakage. The probability of the leakage of the excavation support system was analyzed by modelling the major random variables (geometry, technology, and materials) affecting the stability of the excavation. This analysis led to the development of additional recommendations to improve the effectiveness of monitoring deep excavations. Such analyses can be also simplified. The detailed procedure presented in the

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paper [12] determined the safety based on the damage criteria, and the numerical FEM analyses were used to determine different limit values of deformations of foundations of reinforced concrete or masonry buildings. The structural safety of buildings is most often analyzed in the literature considering static interactions as non-uniform ground settlement. However, some dynamic or para-seismic effects can occur [13]. Such impacts should be analyzed with the associated ground displacements. The safety can be assessed by comparing the obtained results with the modified damage indicators. The paper [13] describes the methodology of calculating and strengthening the structure, and the procedure for taking preventive measures during emergencies that can occur during the execution of deep excavations in seismic areas. The complexity of geotechnical problems that need to be solved increases with the risk they pose [11]. The probabilistic method for assessing the risk related to groundwater is presented in the paper [11]. This groundwater impact is particularly important for the risk management for deep excavations in urban areas. As the problem of assessing the safety of masonry buildings near deep excavations is complex, this paper describes the multi-criteria analysis of factors determining the effective and safe execution of deep excavations, the most important information on the technology of executing and protecting deep excavations and their impact on adjacent structures. The major aim of this paper is to give an insight into issues related to the impact of deep excavations on masonry buildings and to highlight the most crucial factors determining structural safety.

2. ANALYSIS OF FACTORS DETERMINING THE EFFECTIVENESS OF WORK DURING THE EXECUTION OF DEEP EXCAVATIONS

The traditional methods for identifying the safety of civil structures, including masonry buildings in the area of deep excavation, usually focus on single factors associated with the implementation of such projects. The approach of traditional methods is usually one-sided and prone to subjectivity. Creating a comparison matrix, which includes the main factors affecting the safety assessment of masonry buildings in the region of deep excavations, and the determination of weights for individual factors makes it possible to classify them in terms of their relevance. Therefore, the use of multi-criteria analysis – the AHP method (analytic hierarchy process method), to identify and prioritize factors determining the safety assessment of masonry buildings in the region of deep excavations can significantly improve the accuracy of their safety estimation.

The AHP method was developed in the late 1970s and early 1980s by Saaty [14]. The method is based on the utility theory and is a widely used tool for making complex decisions based on accepted criteria. The AHP method allows for combining quantified criteria with non-quantified ones, and objectively measurable criteria with subjective ones. Theoretical bases for the method were described in the papers [14, 15], among others. The first step of the analysis is the identification of factors determining the safety of civil structures in the vicinity of deep excavations. The questionnaire sent in June-July 2023 was answered by 35 experts from different backgrounds: designers, contractors, and urban planners. The respondents were asked to create a comparison matrix for individual factors using Saaty's rating scale to determine their relative priority. Five factors were proposed to determine the safety of civil structures in the vicinity of deep excavations:

A – dewatering/ cutting off the water ingress to the excavation;
B – structural design – verifying limit states of construction;
C – executing deep excavations considering the safety of existing civil structures;
D – monitoring during construction work;
E – supervising.

To obtain the final comparison matrix, the average values of evaluations proposed by the experts were determined. The comparison matrix and weights of each factor are shown in Tables 1 and 2.

Table 1
Pairwise comparison matrix – M

Factor	A	B	C	D	E
A	1.000	0.333	5.000	6.000	5.000
B	3.000	1.000	6.000	7.000	6.000
C	0.200	0.167	1.000	3,000	1.000
D	0.167	0.143	0.333	1.000	0.250
E	0.200	0.167	1.000	4.000	1.000
Sum	4.567	1.810	13.333	21.000	13.250

Table 2
Standardization of matrix and weight values – w_i, for particular factors

Factor	A	B	C	D	E	Weight values (w_i)	Ranking
A	0.219	0.184	0.375	0.286	0.377	0.288	2
B	0.657	0.553	0.450	0.333	0.453	0.489	1
C	0.044	0.092	0.075	0.143	0.075	0.086	4
D	0.036	0.079	0.025	0.048	0.019	0.041	5
E	0.044	0.092	0.075	0.190	0.075	0.095	3
Sum	1.000	1.000	1.000	1.000	1,000	1.000	

The obtained values of the consistency index (CI) = 0.077 of the pairwise comparison matrix and the consistency ratio (CR) = 0.069 are less than 0.1, which indicates that the obtained weights are consistent and can be used in the process of evaluating and prioritizing the factors for determining the safety of masonry buildings in the vicinity of deep excavations.

According to the analysis performed with the defined factors affecting the safety of masonry buildings in the vicinity of deep excavations, the determining factor was found to be the professional structural design – verifying limit states of the structure, for which an obtained weight value was 0.489.

The structural design begins with the identification of the client's/owner's needs for a structure and the development of the structural design based on the identified performance criteria. From there the detailed design can start.

In large projects, owners often prefer to leave the detailed design to the contract track. This is established in the form of design-build contracts. To satisfy the specified needs, the structural design has to be performed thoroughly, since poorly developed performance criteria can lead to poor solutions [16].

The structure should be designed and executed so that the cost of the structure is as low as possible without losing the required quality. The cost of influencing the project grows as it proceeds. It is essential to have a well-organized plan from the beginning. Sometimes project designers favour certain solutions from the beginning. This may lead to disregard of the project cost. In such a case the project costs become very important and need to be precisely estimated. Margins for unforeseen effects and events shall be included to avoid under- or over-estimations. The standard PN-EN-1990 [17] states as a principle that structures shall be designed and executed economically. Cost efficiency is one of the functional criteria identified for the economy of construction works. Such costs should be kept as low as possible without losing the required quality. By estimating the total costs, the cost efficiency of structures can be evaluated. The estimation of the project total costs should be completed before construction work begins. The total costs should include capital costs for the construction work and also the operational and maintenance costs. From a geotechnical point of view, it will be economically feasible and profitable to perform relatively extensive probing in the form of field and laboratory testing. The depth and extent of the geotechnical investigation should be sufficient to identify all ground formations and layers affecting the construction, determine the relevant properties of the ground, and recognize the ground conditions [16]. To minimize the probability of errors and defects, the requirement for quality assurance must be fulfilled. The quality assurance plays an important role in the structural design. The term goes for all parts of the project, the construction as a whole, down to single details. The quality assurance should also help to verify the intended quality. Application rules for quality management can be found in PN-EN 1990 [17]. Two functional criteria have been identified for quality assurance which are design quality and execution quality. The design quality of civil structures should be assured to fulfil the desired properties. The design quality relies on careful planning and thorough investigations on-site to reach the intended quality of the structure. It can be considered advantageous to base the design procedures on established solutions. With established solutions, the uncertainties of the end product are minimized.

The design quality of civil structures can be verified with investigations, experience from earlier projects, and calculations. Special features shall include, where available, previous experience with designed and executed masonry buildings near deep excavations, the impact of excavations on structures, or underground works on or adjacent to the site. As you know, earthworks can cause ground movements. A lot of work is carried out in urban areas, where many structures and facilities already exist, close enough to affect the neighbouring structures. Therefore, it is important to understand how the ground movements caused by excavations affect the nearby structures and to determine the limit states, accordingly, as evidenced by the obtained weight value for the B parameter.

Therefore, the main part of this paper is focused on the methods of limiting displacement and protecting masonry buildings. An attempt was made to overview and present general aspects of the procedures in accordance with current standards [18] and draft Eurocode 6 [19].

3. TYPES OF SUPPORT SYSTEMS FOR DEEP EXCAVATIONS

There are the following types of support systems for deep excavations [20–22]:

- Diaphragm walls or slurry trench walls.
- Prefabricated diaphragm walls.
- Reinforced concrete (cast-in-situ or prefabricated) retaining walls.
- Soldier pile walls.
- Sheet pile walls.
- Pile walls (contiguous, secant).
- Jet-grouting and deep mixed walls.
- Soil nail walls.
- Combined technologies.

These technologies are applied to both the construction of transport infrastructure and general construction. The depth of the excavation in the transport infrastructure construction is determined by a profile of the infrastructure route and varies from more than 10 to 30–40 m. The excavation depth in the general construction sector depends on the number of underground floors and usually does not exceed 18 m. The stability of one of the above support systems for the excavation, except for soil nail walls, is provided with struts, ground anchors, or slabs of underground levels. Diaphragm walls, soldier pile walls, and sheet pile walls are the most economical and common types of support systems. Detailed information on the technology of executing the support systems for deep excavations is presented in the papers [21, 23].

4. PREDICTION OF GROUND DISPLACEMENT NEAR DEEP EXCAVATIONS

Deep excavations are always connected with the impact on the surroundings determined by many factors. These factors are, at first, horizontal displacements of the ground towards the excavation caused by ground removal beyond the support system. The appropriate design and performance of anchorage or strut systems, along with the consideration of slab floor action in the top-down method, effectively reduce horizontal displacements to safe levels [24, 25]. Horizontal displacements of the support walls during the excavation works also lead to vertical settlement of soil (directed downwards), which cumulates with the upward-directed displacement of soil caused by its heave, and further pressure as a result of building the structure [26, 27].

The excavation support system cannot be supported or behaves in a fixed-based mode during the initial phases of executing the excavation or when its depth does not exceed 4–5 m. Then, the soil settlement profile is convex (the curvature radius $R > 0$). Otherwise, the soil settlement profile near the excavation with the support system is concave ($R < 0$), and within some distance it again becomes convex. The maximum vertical displacements are comparable, but within various distances from the excavation – Fig. 1.

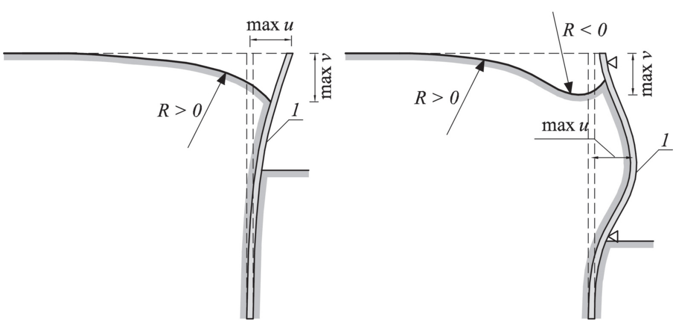

Fig. 1. Shape of soil settlement profile within deep excavation: a) cantilever type, b) spandrel type, 1 – excavation support system

If the initial stage of the excavation induces greater retaining wall deflection or the retaining wall deflects similar to a cantilever beam, then the settlement will be spandrel type and the maximum settlement will be close to the excavation area, as illustrated in Fig. 1a. Concave type settlement will occur provided that the wall has a deep inward movement as shown in Fig. 1b and the largest settlement magnitude will be positioned at a distance from the excavation.

It is the shape of the deformed ground surface and the related curvature, tilt, and deformations that change conditions and the behaviour of nearby structures [28, 29]. In the case of deep excavations, soil unloading is often greater than soil loading within the secondary range as the total weight of the building can be considerably smaller than the weight of soil removed from the excavation. Settlement is not observed in such situations, but soil unloading may occur after executing the excavation if the construction period is long and the structure is erected mainly on non-cohesive soil – Fig. 2. The mentioned soil displacement should also include settlement caused by a reduced level of groundwater outside the support system caused by dewatering of the excavation. Dewatering operations pose a serious risk to facilities surrounding the excavation caused by flow pressure, which washes out fine fractions of soil and is particularly important for non-cohesive fine soil.

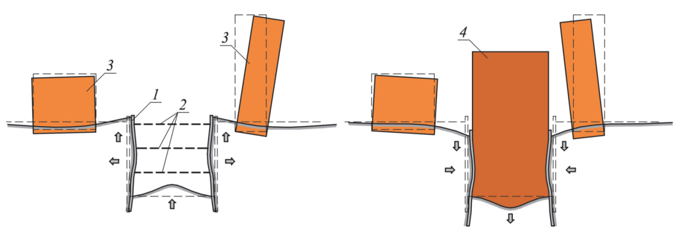

Fig. 2. Soil displacement during different phases of performing excavation in the support system: a) construction phase – performing deep excavation, b) final phase, 1 – excavation support system, 2 – elements supporting the excavation wall, 3 – nearby buildings, the constructed structure

The mentioned soil displacements are always observed, however, their percentage contribution to total displacements depends on both the type of the support system, the phase of the deep excavation, and possible mistakes made during the design or construction phase [27].

The known curve of subsoil deformation outside the support system is the base for analyzing the impact of deep excavations on the technical conditions of the building development. Knowing the tilting of the structure (non-uniform settlement or shear deformation) determined from the empirical curves of settlement, they can be compared with the values specified in the design guidelines [30] or relevant papers [25, 27, 31]. Values of shear deformations induced by deformable ground can be applied to buildings with masonry structural walls [32, 33]. Soil settlement outside the support system [34] can be calculated using the methods of Jen [35], Ilichev [36], and Michalak [26, 37].

5. ASSESSMENT OF EXCAVATION IMPACT ON BUILDINGS

Depending on the construction phase, buildings within the impact area of the excavation can heave, which is usually observed after soil unloading or settlement when the facility constructed in the excavation exerts pressure on the ground. Concerning the analysis of masonry buildings situated near deep excavations, as in the case of structures in the areas subjected to mining impact, the deformable ground can be characterized by vertical displacements v and the resulting non-uniform settlement Δs, vertical displacements u, horizontal displacements of the ground surface ε, the slope of the ground surface T, (convex or concave curvature) defined by the curvature radius R. Soil acting on the building can lead to tilt T_b, cracks characterized by their width w, shear deformation of walls and window openings Θ_i – Fig. 3.

Displacements are found at different stages of work which include performing the support system, deepening the excavation, performing under- and above-ground parts of a building, and exploiting the completed building. When deepening works are completed, the consolidation process is stopped, and the pore pressure also drops. Due to the effect of soil consolidation, the rate of an increasing settlement can last even a few years [38] after completing the building and is determined by the soil structure. Generally, after completing the building 70%–100% of maximum values are reached in cohesive and non-cohesive soils in a semi-dense consistency. For cohesive soils in stiff consistency 50–70% of maximum values are reached, whereas 30–50% of maximum values are reached for cohesive soils in soft consistency. The stabilization process for settlement in non-homogeneous soil can last from one to two years (and even up to five years) from the date of completing the structure and the application of the total operational load. Non-uniform settlements of the building foundations caused by vertical displacements are the main reason for observed damage.

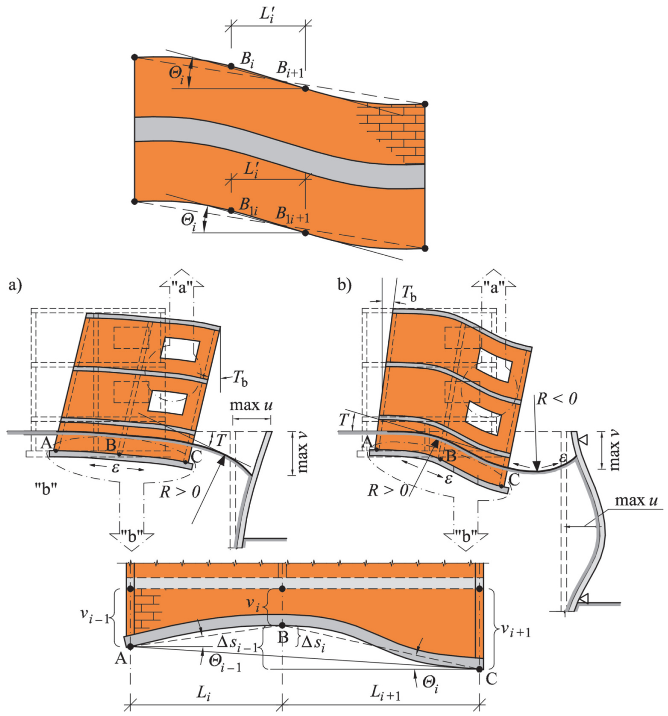

Fig. 3. Parameters typical for soil deformation near deep excavation for various schemes: a) the non-supported cantilever scheme, b) the supported scheme, "a" – shear strains of lintel or spandrel area, "b" – deformation at the level of foundation

Vertical displacement of soil v is the result of the superposition. The first recommendations for the non-uniform settlement of foundations were developed by Terzaghi [38] who defined them in the range of 1 inch (25 mm) for single foundations, 2 inches (50 mm) for foundation slabs, and ¾inches (18 mm) for differential settlements of the same foundation (Δs).

The standard PN-EN 1997-1:2008/Ap2 [39] recommends verifying both settlement ρ, differential settlement (non-uniform settlement) ρ_p, rotation θ, the tilt of the structure ω, relative deflection Δ and relative rotation β, angular strain α, horizontal displacement and vibration amplitude (Fig. 4).

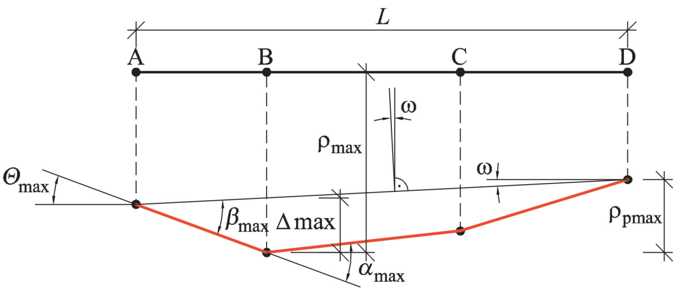

Fig. 4. Symbols for the displacement of foundations according to EC-7 [39]

For serviceability limit state (SLS), EC-7 [39] specifies that "The maximum acceptable relative rotations for open framed structures, infilled frames, and load bearing or continuous brick walls are unlikely to be the same but are likely to range from about 1/2000 to about 1/300, to prevent the occurrence of a serviceability limit state in the structure." Whereas for the ultimate limit state ULS, it is specified that "A maximum relative rotation of 1/500 is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150." When limit values of structure and foundation deformations are not specified, EC-7 refers to Annex H to determine serviceability limits states. According to the Polish National Annex, values of maximum settlements of foundations for the concave (niecka terenu) soil settlement profile were defined to be $\rho = 50$ mm, values for non-uniform settlement are $\Delta = 10$ mm, and for the convex (niecka terenu) soil settlement profile these values are reduced by half.

The horizontal displacement *u* in practice has no negative effect on detached buildings with a compact body. It is also significant to include displacements u of building units and (electric, water, gas, etc.) networks within the impact area of the excavation [40].

Horizontal deformations ε are connected with the horizontal displacement of soil, and they result in the soil impact on the foundations and walls of the basement floor. Deformations $\varepsilon > 0$ observed for convex soil settlement profile cause loosening of the soil and the occurrence of tensile forces in foundations and basement floor. In the case of the concave soil settlement profile, the observed deformations $\varepsilon < 0$ are accompanied by the additional earth pressure applied to walls set below grade and foundations. For soil deformations $\varepsilon > 0$ observed for the convex soil settlement profile, the papers [25, 41] described the relationship between the severity level of the structure distress and horizontal deformations of soil (Table 3).

Ground inclination *T* should be considered with reference to structural and functional aspects. At the forecast stage, the compatibility between the structure tilting and ground inclination is assumed to be $T_b = T$. Considering the structure conditions, an inclination results in the additional rotating moment of a component of the structure weight parallel to the inclined ground. The ground inclination can be accompanied by plastic strains in the ground near the building edges. Prior to soil deformation below the foundations, the primary soil reaction is uniform and then comes in trapezoidal shape as the effect of inclination caused by the bending moment of the horizontal component – Fig. 5.

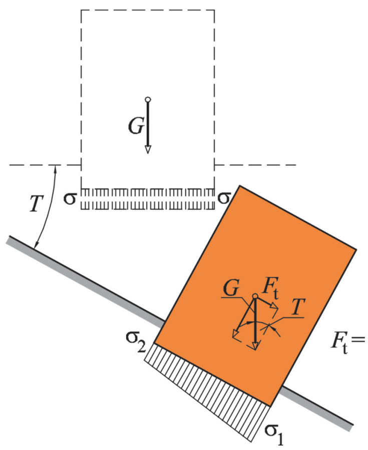

Fig. 5. Impact of ground inclination T on the building structure

Table 3
Relationship between severity level and limit deformations of soil [25,41]

Category no.	Severity level	Damage description	Repair works	Limiting values of deformations $\varepsilon_{\lim} > 0$, ‰	Approximated width of a crack w, mm
0	Non-significant	Surface crazing	–	0–0.5	< 0.1
1	Very minor	Cracks in interior walls, possible single cracks in walls	Wall finishing	0.5–0.75	1.0
2	Minor	Visible cracks Faulty door and window joinery	Filling cracks with mortar	0.75–1.5	5.0
3	Moderate	Serious cracking of walls, defects in the door and window joinery Damage to services	Rebuilding of cracks Adjustment of joinery	1.5–3	From 5 to 15 mm (a few cracks with a width > 3 mm)
4	Serious	Visible cracking and crevices Deformations of door and window joinery Broken services	Extensive repair works involving rebuilding of some walls near lintels and spandrel areas	> 3	From 15 mm to 25 mm, depending on the number of cracks
5	Very serious	Visible cracks and crevices, and deflection of walls can cause structural instability Cracking of joint elements	Required repair works for partial or complete reconstruction	21.000	Usually > 25 mm, depending on the number of cracks

When stresses at the most inclined edge reach the limit value, then strain softening of soil is observed, which leads to further settlement, and consequently to an increased deflection of the building.

Considering deep excavations, in-depth tests on the reactions of users to the impact on a building were not performed. However, a view of the reactions of the users to the construction project can be given by extensive tests performed on the mining areas in Upper Silesia [24]. The analysis of noticing ground deformation that leads to deflection of the building shows the following reactions of users:

$T \leq 5‰$ – unnoticeable reaction,
$5‰ < T \leq 10‰$ – noticeable reaction,
$10‰ < T \leq 15‰$ – noticeable or noxious reaction,
$T > 15‰$ – noxious reaction.

The results obtained are particularly significant at the stage of construction work and monitoring of the adjacent buildings. The building deflection by 5‰ always causes the intensified reaction of the building users who are affected by the work. It should be mentioned that the design values specified in the National Annex to EC-7 [39] are considered as the limit values of tilt ω in the range of up to 3‰ for the concave soil settlement profile, and 1.5‰ for the convex soil settlement profile.

Ground curvature K has the most negative impact on surrounding buildings by causing the formation of additional internal forces practically in the whole structure. In load-bearing wall structures, which can be classified as rigid structures not matching the ground curvature, significant forces are observed in connections between walls and in sections weakened by openings. Significant displacements and relatively small forces are expected in the flexible structures, which include elements of underground and above-ground infrastructure, which match ground deformations. Uniform loading from the walls (Fig. 6) is approximately balanced by the parabolic resistance of soil, with maximum values observed at points of contra flexure and minimum values at the outer edges of a building (even zero values where there is no contact between the building and the ground due to its curvature). Hence, the building is subjected to resultant loads being the sum of loads and soil reaction.

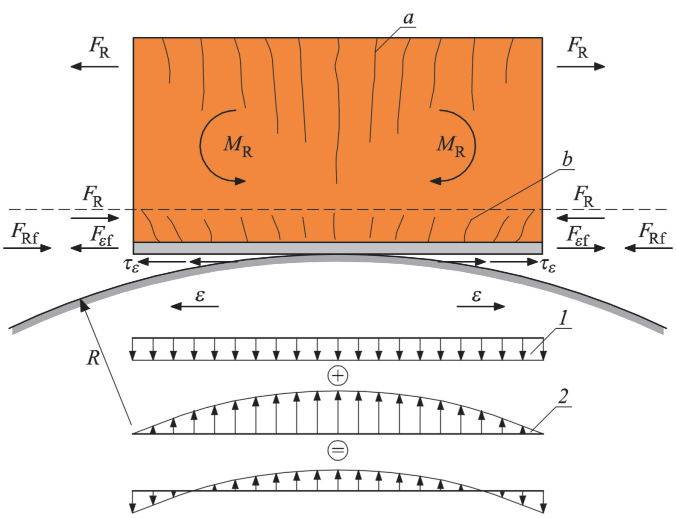

Fig. 6. Internal forces in the building with rigid structure at the convex curvature of ground: 1 – structure foundations subjected to loading, 2 – soil reaction, 3 – load resultants acting on the building, *a* – cracks caused by bending, *b* – diagonal cracks caused by shearing and tension

When the building is affected by the convex curvature of the ground ($R > 0$), foundations are subjected to compressive forces F_ε whereas tensile forces are exerted on the upper parts of the building and lead to vertical cracks (a). Simultaneous deformations ($\varepsilon > 0$) cause shearing in basement walls and tension in foundations $F_{\varepsilon f}$ which result in inclined cracks (b). The concave curvature of the ground ($R < 0$) produces tensile forces F_R in foundations and compressive forces in the vertical extension. Ground deformations ($\varepsilon < 0$) lead to compression of foundations. The combined effect of these two impacts is the result of the superposition of both states of loading.

The design of walls of the support system of deep excavation and construction work should be performed without exceeding allowable values of non-uniform settlement and radii of the ground curvature. Long-term observations of the behaviour of buildings were the base to specify limit values of soil parameters, which if exceeded, can cause serious damage creating risk to the building safety [27, 40]. These limit values are as follows depending on the dimensions of the building plan:

- Curvature radius R: (20–125)L, but no more than R = 2000 m for the convex curvature and R = 5000 m for the concave curvature.
- Non-uniform settlement Δs: (1/150–1/1000)L.

Resistance of the building is mainly determined by used materials and the structural system. Ranges of limit values of the curvature radius and non-uniform settlements are found to considerably vary, which significantly reduces their practical application.

Wall strain angle Θ is the result of shear stress τ caused by ground deformation. It usually leads to the formation of inclined cracks in walls. The shape, direction, and width (morphology) are mainly determined by standard vertical σ_y, and horizontal σ_x stresses that accompany shearing. For the existing buildings, values of stress components and properties of the masonry are unknown. Wall resistance to distortion is determined based on the analyzed conditions of deformation.

Apart from the ultimate limit state, both standards PN-B-03002:1999 [42] and PN-B-03002:2007 [43] also introduced the SLS deformation conditions expressed as:

$$\Theta_{\mathrm{Sk}} \leq \Theta_{\mathrm{adm}}, \tag{1}$$

where

Θ_{Sk} – the strain angle determined from the static and stress analysis (calculated for characteristic values of horizontal shear forces V_{Sk}) of the wall,

Θ_{adm} – the acceptable value of the strain angle presented in Table 4, which corresponds to the formation of cracks having the acceptable width $w = 0.1–0.3$ mm.

However, these standards do not specify important parameters for historic buildings, which require in-situ tests on specimens cut out from the structure. Table 5 presents values of strain angles and moduli of shear strain determined from the tests (performed in accordance with the standards [42, 43]) on wall fragments taken from historic buildings.

The Italian standard [47] specifies values of moduli of shear strain and angles of shear strain of masonry determined on their basis – Table 6.

Table 4
Acceptable values of strain angle Θ_{adm} [mrad = mm/m] specified in the standard PN-B-03002:2007 [43]

Group of masonry units	Cement mortar	Cement-lime mortar
Group 1 excluding autoclaved aerated concrete units	0.4	0.5
Group 2, 3 and 4	0.3	0.4
Autoclaved aerated concrete units	0.2	0.3

Table 5
Acceptable values of angle of shear strain Θ_{adm} [mrad = mm/m] specified in the standard PN-B-03002:2007 [43]

Wall type	Angle of shear strain Θ_{obs}, mrad	Modulus of shear strain G_{obs}, N/mm^2
Historic brick wall structure [44] (f_B, f_m – lack of data)	0.136	131
Brick walls [45] (f_B, f_m – lack of data)	0.47–0.29	173–333
Brick and sandstone walls [45] (f_B, f_m – lack of data)	0.51–0.64	195–220
Historic stone walls with a single layer of bricks [44] (f_B, f_m – lack of data)	0.791	30
Historic travertine wall structure [44] ($f_{\varphi 70/150}$ = 1.75–8.1 N/mm^2, f_m – lack of data)	0.942–0.370	19–60
Wall from Lisbon sandstone with weak lime mortar [46] (f_b – lack of data, f_m = 0.56 N/mm^2)	0.20–0.40	58–389
Walls from weak stone and weak brick [45] (f_B, f_m – lack of data)	0.33–0.81	249–290

Table 6
Values of shear strain angles according to various tests

Wall type	Strain angle mrad	Shear modulus N/mm^2
Irregular stone wall (fine stone and cobblestone)	0.13–0.14	230–350
Irregular stone walls used to build façade or infill walls	0.15–0.17	340–480
Cast-stone wall	0.16–0.18	500–660

The values presented in Tables 3 and 4 demonstrate that indicative values can be taken for historic walls to determine limit values of angles of shear strain:

- $\Theta_{\mathrm{Sk}} = 0.13–0.20$ mrad brick walls in satisfactory conditions (dampness and corrosion losses for bricks and mortar).
- $\Theta_{\mathrm{Sk}} = 0.20–0.30$ mrad brick walls, which do not exhibit any corrosion damage.

- $\mathit{\Theta}_{\mathrm{Sk}} = 0.30$–$0.50$ mrad stone walls with single layers of bricks.
- $\mathit{\Theta}_{\mathrm{Sk}} = 0.10$–$0.15$ mrad stone walls (irregular arrangement of stones – "random wall").
- $\mathit{\Theta}_{\mathrm{Sk}} = 0.15$–$0.30$ mrad cut stone walls (regular arrangement of stones – layered wall or wall made of rows).

The presented limit values of displacements or deformations of structures should be regarded as bottom envelopes of results from testing masonry buildings in the years 1950–1980 in Poland. The safety analyses require the investigation into the masonry properties and the uncertainties in the applied mechanical parameters. Additionally, the calculations should include uncertainties in the building models, which cover the most unfavourable static schemes.

The geometric method of determining strain angles for the wall subjected to the non-uniform displacement of the ground is presented in detail in the standard [43]. The reliable value of the strain angle $\mathit{\Theta}_{\mathrm{Sk}}$ to verify the condition (1) is recommended to be determined from the general relationship according to the model shown in Fig. 3:

– the wall with a length L_i:

$$\mathit{\Theta}_{\mathrm{Sd}} = \mathit{\Theta}_{i-1} = \frac{|v_i - v_{i-1}|}{L_i}, \tag{2}$$

– the wall with a length L_i:

$$\mathit{\Theta}_{\mathrm{Sd}} = \mathit{\Theta}_{i+1} = \frac{|v_i - v_{i+1}|}{L_{i+1}}, \tag{3}$$

where

v_{i-1}, v_i, v_{i+1} – values of vertical displacement determined at both ends of the analyzed section of the stiffening wall,

L_i, L_{i+1} – lengths of sections of the stiffening wall (a distance between intersecting walls or between openings).

According to the standard [43], the finite element method can be used to calculate deformations of walls. It is recommended to determine the strain angle from the following relationship:

$$\mathit{\Theta}_{\mathrm{Sd}} = \frac{|\Delta v_i|}{l_i}, \tag{4}$$

where

Δv_i – a difference in vertical displacements determined at both ends of the area (section) with the largest accumulation of deformations,

l_i – a length of the most deformed area (section) in a particular section of the analyzed wall.

6. SIMPLIFIED METHOD OF ASSESSING EXCAVATION IMPACT ON BUILDINGS

The simplified method of assessing the impact of excavations on buildings presented in the guidelines [30] can be applied in practice providing that their location with reference to the excavation edge meets the following condition:

$$d_{\min} > \Lambda H_w, \tag{5}$$

where

H_w – a depth of the excavation, m,

$\Lambda = -4$ – when the level of groundwater is not predicted to be reduced while performing the excavation,

$\Lambda = 5$ – when the level of groundwater is predicted to be reduced while performing the excavation.

If this condition is satisfied, then the excavation is considered to be located in the urban area. Otherwise, when buildings are within a greater distance, this area is regarded as an open area and the excavation effect can be neglected. Generally, this method is to verify whether the settlement of the structure does not exceed limit values. Ultimate and serviceability limit states are as follows:

$$\max s_k \leq [s_k]_u, \tag{6}$$

$$\gamma_f (\max s_k) \leq [s_k]_n, \tag{7}$$

where

$[s_k]_u$ – a limit value of the building displacement at the serviceability limit state, which signals possible cracks or excessive displacements,

$[s_k]_n$ – a limit value of the building displacement at the ultimate limit state, which signals the possibility of exceeding the ultimate capacity of individual structural members.

The limit values of the building displacement at the ultimate and serviceability limits states are provided in the guidelines [30] and Table 7.

It should be noted that values of non-uniform settlement at the serviceability limit state presented in Table 7, which refers to a building length, should not exceed values determined from angles of shear strain. Generally, the minimum value from the following values should be taken as the reliable value at the serviceability limit state:

$$[s_k]_u = \min \begin{cases} [s_k]_u & \text{– Guideline ITB 2020,} \\ \mathit{\Theta}_{\mathrm{adm}} l_i, & \end{cases} \tag{8}$$

Table 7
Limit values for building displacements expressed in [mm] [30]

Structure sensitivity	Type of structure	$[s_k]_u$	$[s_k]_n$	Relative rotation β	Tilting ω
Highest	Masonry buildings without ring beams, with timber or Klein-type floors	5–7	15–18	0.05%	0.1%
High	Masonry buildings with ribbed or reinforced concrete floors, prefabricated buildings	7–9	20–25	0.075%	0.2%
Standard	Buildings with a monolithic structure	9–11	25–35	0.15%	0.3%

where

Θ_{adm} – the acceptable value of the angle of shear strain specified in the standard [43] (Table 2) or presented in Tables 3 and 4,

l_i – the analyzed section of the wall or a length of the building, in which non-uniform settlement can be found.

Maximum displacements of the structure (Fig. 7) are determined in the following way:

- for the building with a foundation depth of $h_f \leq 2.5$ m

$$\max s_k \leq v_o \,, \tag{9}$$

- for the building with a foundation depth of $2.5 \text{ m} < h_f \leq 5.0$ m

$$\max s_k \leq v_o \frac{H_w - h_f}{H_w} \,, \tag{10}$$

where

v_o – displacement of the ground over the analysed section of a building, mm,

$d_{\min}$ – a distance from the excavation support system (wall),

H_w – a depth of the excavation, m.

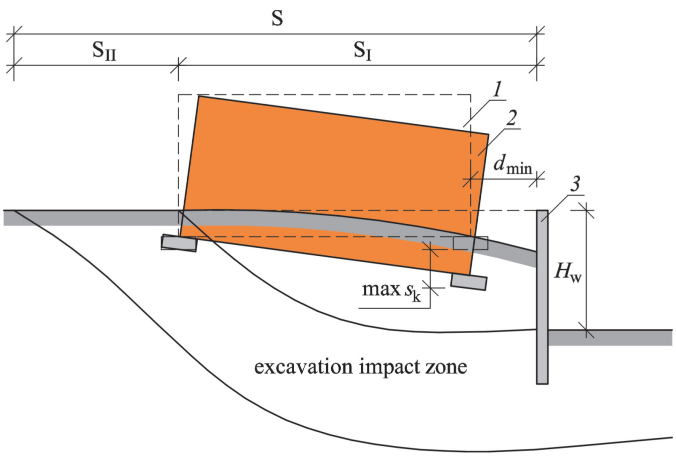

Fig. 7. Maximum displacement of the structure according to [30]: 1 – the state before the commencement of construction works, 2 – the state during construction works, 3 – the support system of the excavation, S – the impact area of the excavation, S_I – the area of the direct impact of the excavation, S_{II} – the area of the secondary impact

Values of ground displacement v_o can be determined with one of the previously described analytical methods (Jen, Iilichewa, Michalak). The guidelines [30] can be used to determine the simplified distribution of ground displacement based on the following parameters:

a) maximum displacements of the ground max v_0 in a direct vicinity of the excavation wall,
b) displacements v_{0I} at the boundary of the zones I and II,
c) zero displacements at the boundary of impact areas of the excavation S.

Maximum negative displacements observed near the support system of the excavation can significantly differ depending on the excavation depth, the type of soil, the type of wall structure, and the type of support. According to the authors [48] of the guidelines [30], average values of the settlement are estimated to be equal to 0.15% H_w, and maximum values are within a range of 0.3%–0.5% H_w. Maximum values of heaves are lower, approximately 0.10% H_w. The construction work performed in Warsaw (the first metro line and a few buildings with a great foundation depth) demonstrated that excavations performed in moraine soil (consolidated clay and clayey sand) were accompanied by the average settlement of approximately 8–12 mm (0.1%–0.12% H_w), and maximum values reached 50 mm and were related to mistakes made during the construction of excavation walls. And heave values did not exceed 10 mm.

The impact area of the excavation was determined by the properties of soil in the subsurface. When slightly deformable soils, such as non-cohesive sands and clays, predominate, displacements disappear within a distance of 1–1.5 H_w from the excavation edge. When deformable soils, such as stiff clays, predominate, the impact area is approx. 2–2.5 H_w. The impact area can even reach 4–6 H_w for highly deformable soils, such as plastic clays. Further tests conducted on deep excavations in Warsaw [27, 49, 50] showed that greater displacements were in the area having a width of 0.5–0.75 H_w, and settlement started to disappear within a distance of 2.0 H_w. Dewatering also has a certain influence on the impact area. It usually broadens the impact area. According to data presented in the papers [27, 49, 50], the application of dewatering wells caused the disappearance of settlement within a distance of 3–4 H_w. Displacements in the broadened impact area, caused by dewatering are small and do not have a significant impact on the state of facilities.

Figure 8 illustrates results from measuring displacements of buildings located near the construction site of underground stations of the first metro line in Warsaw. For buildings A-8 and A-10, the subsurface contained moraine soil, such as moderately compacted sand and stiff clay. However, the soil at station

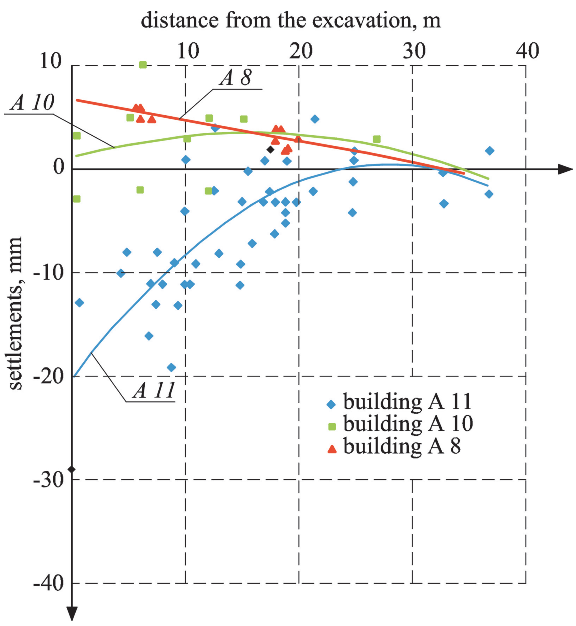

Fig. 8. Compared displacement of buildings located near three selected excavations with a depth H_w = 12 m (ITB analysis see [51])

A-11, below moraines characterized by the disturbed stratification, contained silty clays. Each time the excavation depth was approximately 12 m, and the level of groundwater was reduced below the excavation bottom.

Figure 9 shows results from measured settlements of some deep excavations performed in the USA and Norway, collected by Peck [52], while Fig. 10 presents measurements of displacements of three arrays of buildings, taken by the author during the execution of the excavation H_w = 12.7 m in cohesive soils. The simplified distributions of extreme vertical displacements of ground in the impact area of the excavation, adopted in the guidelines [30], are shown in Fig. 11.

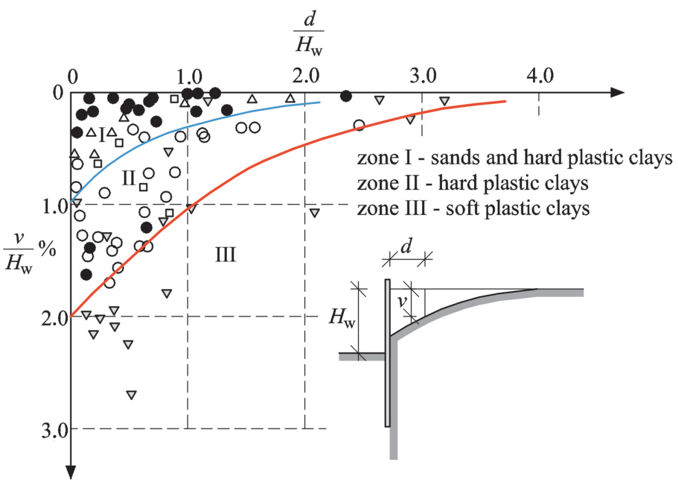

Fig. 9. Measurement results for ground settlement according to Peck [52]

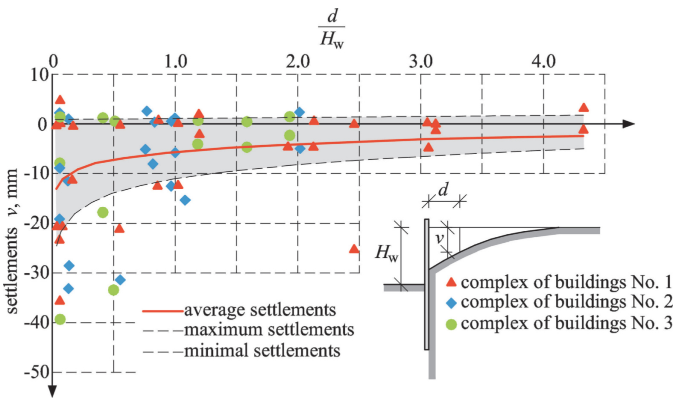

Fig. 10. Results of authors' measurements of ground settlement

The guidelines [30] can be applied to determine maximum negative displacements (settlements) from the following relationship:

$$\max v_0^{(-)} = v_i + v_u + v_w \,, \tag{11}$$

where

v_i – vertical displacements caused by the support system,

v_u – vertical displacements caused by vertical displacement of the wall,

v_w – vertical displacements caused by excavation dewatering.

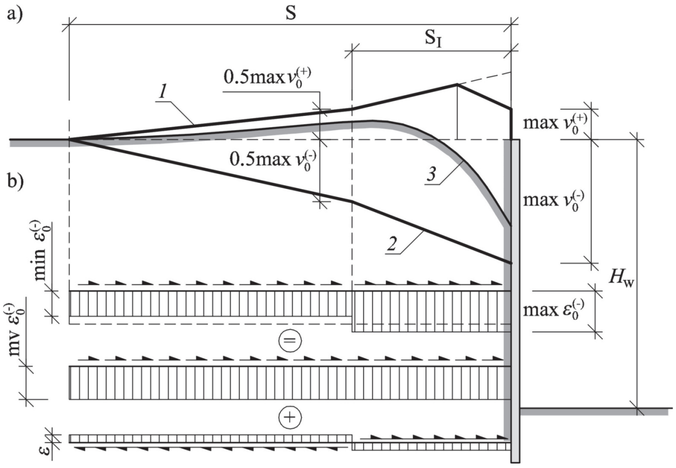

Fig. 11. Simplified distributions of vertical displacements and deformations of the ground near the deep excavation [30]: a) vertical displacements, b) ground deformations, 1 – extreme distribution of negative displacements, 2 – extreme distribution of positive displacements, 3 – average distribution of displacements

For the support system of the deep excavation performed as a diaphragm wall, vertical displacements caused by this support system are expressed by the following relationship:

$$v_i = \alpha\sqrt{H_w} \,, \tag{12}$$

where α – empirical coefficient.

Values of this coefficient are presented in Table 8.

Table 8
Values of empirical α [30]

Subsurface composition or conditions for performing a tight diaphragm wall	s_k^u
Sandy clay, clayey sand, firm clay, semi-dense or stiff with moderately compacted interglacial sand	5–7
Difficult ground conditions posing a risk of collapsing in the crevice: – strongly compressible ground with deformation modulus of $E_0 \le 15$ N/mm^2 – strongly permeable ground, voids, and caverns which can cause sudden leakage of liquid which stabilized walls of the crevice	7–9

The ground displacement (at this stage of work) is usually smaller if the diaphragm wall is used in comparison to other types of support systems, such as sheet pile walls, pile walls, or soldier pile walls. For pile walls, it is safe to take values of the coefficient α for the equation (12), which are reduced even by 50%.

The second component describing vertical displacements of the ground, which are caused by vertical deformations of the excavation walls, is determined from the following relationship:

$$v_u = 0.75\,(\max u_k) \,, \tag{13}$$

where $\max u_k$ – the maximum horizontal displacement of a wall of the excavation.

Values of maximum displacements of the excavation walls (Fig. 12) should be determined at individual phases of construction works. Calculations should include displacements of the wall supports, which are caused by shortening, thermal effects, concrete shrinkage, and inaccurate assembly works. For reinforced concrete walls of the support system (diaphragm walls, pile walls), calculations of horizontal displacements should include changes in stiffness caused by section cracking. For the excavation walls in the form of soldier pile walls, horizontal displacements of walls should consider horizontal displacements of the ground caused by inaccurate adherence of lagging to the ground.

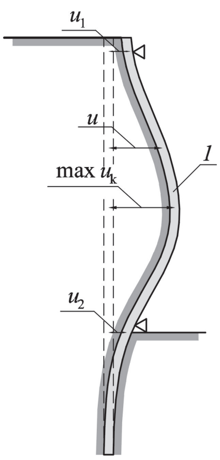

Fig. 12. Horizontal displacement of the support wall: 1 – excavation support system after deformation, u1, u2 – wall displacements caused by the installation of supports in stages

According to Polish test results [27, 49, 50], which include anchored soldier pile walls, pile walls, and anchored diaphragm walls, the empirical relationship was established as $v_u = (0.5–0.75) \max u_k$. For the diaphragm walls, this relationship in equation (12) was equal to 0.75, whereas, in the case of more flexible types of walls, etc. such as cut-off walls in equation (12), it is safer to take the coefficient equal to 0.5.

Based on measurements taken for more than 20 construction sites in other countries and Warsaw, the paper [25] presented values of the maximum horizontal displacement for different types of deep excavation walls. The obtained values are shown in Table 9.

The Russian recommendations [53] give values of maximum horizontal displacements of the fixed-based support system in the form of the diaphragm wall (with a thickness of 600 mm), (Larsen) sheet piling, and pipes with a diameter of 35 mm, which are expressed as nomographs. Recommendations were developed for sand, sandy clays, and clays. Regardless of the type of excavation support system, the greatest displacements were observed for sand, and the smallest for clays. For the excavation depth of 3 m, the displacement of the excavation support system at approx. the first level of anchorage or strutting was 40–15 mm for the pipes, 20–8 mm for Larsen sheet piling, and the smallest displacements of 15–5 mm were observed for the excavation support system made of diaphragm walls. Nomographs can be applied for the excavation depth of 6 m. Then, the observed maximum displacements were > 200−−150 mm for pipe excavation support system, 140–30 mm for sheet pile walls, and 40–10 mm for diaphragm walls.

Maximum values of ground heave (positive displacements) can be determined from the following relationship:

$$\max v_0^{(+t)} = \eta v_{\max}, \tag{14}$$

where
$v_{\max}$ – predicted maximum heave of excavation bottom,

Table 9
Maximum horizontal displacements of support system for deep excavation based on [25]

Author	$\max u_k$	Type of excavation support system
Burland J.B. *et al.*	10–40 mm	Lack of data
Simpson B. *et al.*	0.002 – 0.004 H_w	Lack of data
Breymann H. *et al.*	0.002 H_w	Lack of data
Long M.	0.0005–0.0025 H_w (maximum 0.007 H_w)	Anchored walls, strutted, and performed with the top-down construction method
Long M.	0.001–0.02 H_w (0.003 H_w on average)	Sheet pile walls
Siemińska–Lewandowska A. *et al.*	0.0018–0.002 H_w	Anchored diaphragm walls
Siemińska–Lewandowska A. *et al.*	0.0005–0.001 H_w	Braced diaphragm walls
Siemińska–Lewandowska A. *et al.*	0.0008 H_w	Diaphragm walls performed with the top-down construction method
Kotlicki, Wysokiński L.	0.003–0.005 H_w	Lack of data
Smoltczyk U.	0.01 H_w	Sheet pile walls
Smoltczyk U.	etc. 0.001 H_w	The strutted walls, included in the design project due to load imposed by active earth pressure, are built in cohesive and non-cohesive soil in both stiff and dense consistency

η – reduction factor determined by the foundation depth of the excavation support system below the excavation bottom, equal to:

- 0.3 – for excavation support system set below the excavation depth, at a depth of at least 3 m,
- 0.6 – in other cases.

The final component of the equation (11) expresses the effect of displacements caused by dewatering of the excavation. If the excavation is in low deformability soils ($E_0 \geq 40$ MPa), then the effect of displacements caused by the lowered level of groundwater can be ignored. However, in other cases, maximum vertical displacement caused by the reduced level of groundwater outside the excavation support system is expressed by the following relationship:

$$v_w = \vartheta v_{(\mathrm{w,max})}, \tag{15}$$

where

$v_{w,\max}$ – maximum displacement of the ground caused by lowered water level,

ϑ – the coefficient that includes generally more desirable for buildings and more "gentle" distribution of displacements v_w when compared to the distribution of v_i and v_u.

When deformable soil is in the subsurface, the value $v_{w,\max}$ can be calculated as the settlement rate as a result of the increased weight density of dewatered soil. The coefficient ϑ can be determined from the following equation:

$$\vartheta = \frac{L}{R}, \tag{16}$$

where

L – a building length or width perpendicular to the excavation,

R – an area of the depression cone.

In the most common situations with predominating moraine soil, a reduction of the water table by 1 m also lowered the ground by approximately 1 mm.

The value of displacements v_{0I} at the boundary of zones I and II should be taken as half of the displacements at the wall edge:

$$v_{0\mathrm{I}} = 0.5\left\{\max v_0^{(+)} \quad \text{or} \quad \max v_0^{(-)}\right\}. \tag{17}$$

The tests conducted in Poland and the results from the published papers were used to specify two impact zones in the guidelines [30]. They are:

Zone I – the zone of direct impacts, in which displacements hazardous to the safety of the structure can occur in critical situations, such as design flaws or faulty construction works.

Zone II – the zone, in which displacements can lead to damage noticeable in the building elements, but not hazardous to the safety of the structure.

This classification is reasonable as different types of risk for buildings occur in particular zones. The direct impact zone, that is, the area of most likely failure wedge, requires structural surveys, which provide information on the building conditions, and particularly their strength to predict displacement. Non-uniform settlement beyond the impact area does not pose a danger to buildings; however, some damage is possible – slight architectural damage. Depending on the type of soil in the subsurface, the area of impact zones is determined in the following way – Fig. 13, Table 10.

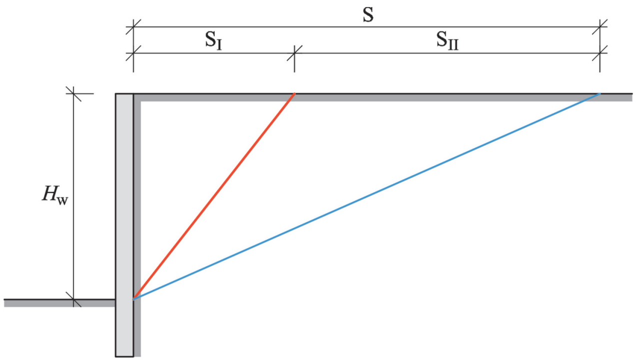

Fig. 13. The area of impact zones of the excavations S_I and S [30]

Table 10

Impact zones of the excavation [30]

Soil type	S_I	S
Excavation in sand	0.5 H_w	2.0 H_w
Excavation in loam	0.75 H_w	2.5 H_w
Excavation in clay	1.0 H_w	3.5 H_w
No data on soil	50 m	100 m

The area of impact zones of the excavation S can be reduced by 20% if no reduction of the groundwater table is predicted. However, the area of impact zones specified in Table 8 should be expanded ("slightly" [30]) by approximately 5%–10% when the dimensions of the excavation plan exceed 60 m. The area of the impact zone of the excavation S is also expanded for the mean modulus $E_0 < 15$ MPa. Then, the value $S > 2.5 H_w$ should be taken (as for clay). A designer of the excavation support system is obliged to define the impact zones of the excavation. The defined areas of impact zones should be included in the building plan as a map – Fig. 14. This map should also present the location of the excavation and buildings, which are completely or partially situated in the direct impact zone. All buildings within the impact zone of the excavation (zone S) should be monitored during construction works. After conducting the in-depth analysis, some facilities should be also strengthened considering the effect of non-uniform settlement.

Individual interactions, including local conditions and experience, should be considered for the determination of the range of impact zones and the range of probable vertical displacements of soil beyond the excavation support system (particularly positive displacements). To estimate possible displacements with FEM, we should determine reliable geotechnical parameters of the ground, adopt the relevant model of the ground, the reliable software, and experience in such calculations, supported with results from measuring the completed objects. Investigation performed by Italian researchers from the University of Salerno is an example of successful research and development work [10–13].

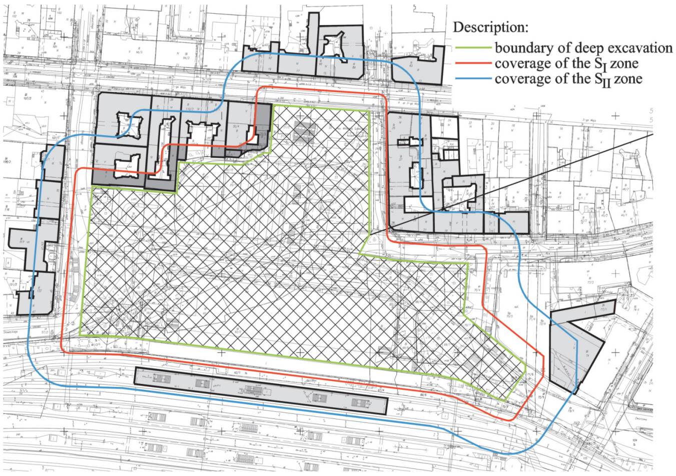

Fig. 14. Impact zones of the excavation

The paper [13] proposed, inter alia, the design procedure for deep excavations based on single- or multi-criteria probabilistic analyses for reinforced concrete and masonry buildings. Ground deformations were modelled with the advanced material model implemented into PLAXIS software (developed by Plaxis bv), which also covered geometry and non-linear behaviour of the soil (H-S (hardening soil) model with Mohr-Coulomb criterion), sequences of (static and operational) loads, boundary conditions, and building-soil interactions. This procedure was applied to a large open-pit excavation of the design new subway line in the city of Naples (Italy) [11–13]. The damage function, proposed by Burland [54], of horizontal deformations and the deflection ratio [10–12] was used for excavation-induced damage to masonry structures. Two-dimensional probabilistic analyses were conducted on a typical building with a height $h = 20$ m and a width $L = 30$ m located within a different distance from the excavation edge. The probability of serious damage was demonstrated to be the highest when a building is situated within a distance smaller than $1.5\,H_w$ (H_w – excavation depth) and decreases with the increasing distance. On the other hand, the probability of moderate and minor damage increases with the growing distance of the building from the excavation edge. This behaviour is due to the fact that horizontal deformations cease to exist at a much slower pace than vertical displacements, which are the most critical near the excavation edge. Areas that cause minor damage to buildings may be observed in the most distant zones from the excavation edge. The analyses described in the papers [11–13] are convergent with Polish know-how [27,49,50] and design recommendations [30] (cf. items 4–5).

7. NOTES ON DESIGNING DEEP EXCAVATIONS IN PORTUGAL

The Polish know-how in the field of deep excavations dates back to the early 1990s [55]. The Portuguese know-how is much richer and dates back over six decades (the first implementation was in 1950). Due to the construction of this type of facility in the vicinity of historic masonry buildings, top-down methods [56] and the technology of diaphragm walls with various complex shapes [57] were developed. In Portugal, the method of protecting objects according to standards or guidelines is not used, but deformations of the support system of excavations and the ground are each time analyzed in detail. This method is, of course, more accurate, but it requires a very accurate identification of the ground and surrounding buildings and the use of modern computational methods.

8. SUMMARY

This paper presents the identified parameters that determine the safety of masonry buildings near deep excavations. The following five factors were specified: dewatering/cutting off the water ingress to the excavation, structural designing – verifying limit states of the structure, performing deep excavations considering the safety of existing civil structures, monitoring during construction works, and supervising. They were prioritized by the AHP method. The structural project and verification of limit states were the parameters with the greatest weight, which has a significant meaning according to the authors. While conducting structural designs, the majority of designers often forget that Eurocodes are the harmonized standards that should be used comprehensively, and not selectively, and the design quality is precisely defined and described in PN-EN 1990 [17]. This paper thoroughly describes serviceability limits states with reference not only to standard recommendations but also to technical guidelines. Performance of deep excavations near civil structures is unavoidable due to location, economic, architectural, and structural restrictions as well as legal requirements concerning obligatory parking space.

Such undertakings require the technology that provides safe construction and minimizes its impact on elements of adjacent infrastructure and buildings. Displacement of the support system of the excavation during its deepening each time causes deformation of the adjacent area and consequently can damage adjacent masonry buildings (usually erected in the traditional technology, and often in poor condition), which are very vulnerable to all external impacts. The impact of deep excavations on the surrounding urban structure can be minimized by a series of diagnostics work, repairs, and strengthening of the nearby building executed with the greatest care at the design and building permit design stage after conducting necessary analyses of the effects of ground deformation. The variant solution for securing deep excavations and choosing the optimum solution considering technical and economic aspects, minimizing the impact of the construction project seems to be the best solution.

The most crucial parameters that should be considered for the project in the vicinity of deep excavations are mainly vertical displacement of ground causing bending and deflection of buildings, and horizontal displacement which leads to great tensile forces in underground parts of buildings. The number of displacements mainly depends on the ground structure, the support system of the deep excavation, and the implemented method of securing the excavation in the form of strut systems, ground anchors, or basement floors. The number of displacements can change in particular phases of executing the deep excavation with a change in the structural system of the excavation walls and a change in loading. The Polish experience from executing deep excavations with the use of diaphragm walls, which are strutted or supported with underground slabs shows that the greatest displacements are observed within a distance of approx. $1.3\,H_w$ and are decreasing with an increasing distance from the excavation edge. The total area of impact zones depends on the geological structure of the ground but is within a range of 2.8–5.4 H_w. As a general rule, all buildings located in this area should be regarded as exposed to the impact of performed work. A developer/designer should be aware that all facilities located in the direct vicinity of the excavation (within a distance of approximately $1.3\,H_w$) will be displaced during construction works, and hence they will require an in-depth analysis, and sometimes even strengthening.

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