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Orginal Article

Analysis of environmental consequences occurring in the life cycle of a retail facility

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Abstract: The increasing importance of environmental protection issues has recently forced a low–emission approach to investment processes. To accomplish the European Union's climate, energy and environmental goals, action is needed to achieve high levels of energy efficiency and low environmental damage. Among the energy–intensive sectors, construction deserves a distinction due to its leading share in gross energy consumption in developed countries. Therefore, it is necessary, and at the same time more and more popular, to analyse the environmental loads generated in individual phases and throughout the life cycle of building objects. This subject is also gaining importance in the context of the recent increases in the prices of energy carriers, which forces the search for new construction and exploitation solutions in line with the philosophy of sustainable development and the circular economy. The aim of the analysis was to assess the environmental consequences in the life cycle of a real commercial building located in Janikowo (Kuyavian–Pomeranian Voivodeship), which was carried out using the LCA (Life Cycle Assessment) methodology. The obtained results indicated the dominance of the facility exploitation phase in the level of cumulative environmental loads.

Keywords: retail facility, sustainable development, Life Cycle Assessment (LCA), Ecological Footprint

1. Introduction

As part of each type of economic activity, natural resources are used to obtain products or end objects [1], [2]. Manufacturing processes are sources of many different substances and wastes, which constitute burdens causing significant changes to the environment in a specific time perspective [3]–[5].

Nowadays, most countries and international organizations are intensifying activities aimed at mitigating the negative effects of this type of influence on the environment [2], [4], [6]. Indispensable for this are therefore appropriate assessment tools that allow taking into account emerging environmental problems [7]–[9]. The basic tool used for this type of assessment is the environmental analysis covering the full life cycle, i.e. LCA (Life Cycle Assessment) [10]. After the introduction of standards from the ISO 14000 family, the LCA methodology became more and more known in the world [11]–[13]. It is an evaluation tool ensuring obtaining comparable results of analyses while considering many different environmental problems [14]–[16].

From the environmental point of view, it is important to develop and implement new, effective technologies and products with good performance parameters, limiting the demand for non–renewable energy and primary raw materials. These technologies should promote the use of energy obtained from renewable sources as well as facilitate the use of recycled raw materials. The use of recyclable materials should, in addition to environmental benefits, also reduce the investment or operating costs of the building [17]. These trends correspond to the requirements of sustainable construction.

The previous LCA studies showed that the greatest environmental burden may be related to the exploitation phase of an analysed building (78% percent of all revenues, apart from generating solid waste) [18]. According to the authors, significant savings in the field of running water supply and sewage disposal (about 6%) can be achieved by reusing grey sewage and rainwater. Moreover, it was shown that the environmental costs related to the demolition and transportation of materials were only 0.3%, excluding material recycling. The GWP (Global Warming Potential) index was also estimated, where 84% of the index was related to the exploitation phase of the building. A comparable analysis conducted for a public utility building in Michigan, assuming a 75–year exploitation phase of the building, showed the percentage share of environmental impacts in this phase at 83% [19]. A similar analysis was carried out basing on the analysis of a public utility building with a concrete structure in [20]. The study showed that the environmental impact during the useful life cycle of the building reaches the highest value among all phases, due to the natural resources consumed and pollutants emitted, and the duration of this phase. The share of environmental impacts due to the exploitation phase was about 97%. Similar results were obtained in [21] for the assessment of environmental burdens with the IMPACT 2002+ method.

The subject of the assessment of environmental inputs in relation to construction objects using the LCA methodology has already been conducted to a limited extent [22]–[25]. However, there are still no effective solutions to reduce environmental outlays in commercial facilities [26]–[28]. Considering the existing legal conditions in the field of reducing energy consumption and environmental impacts, the aim of the work was a detailed analysis of the assessment of energy and environmental expenditure in the life cycle of a selected commercial building. The procedure was carried out using the LCA methodology [29] and the Ecological Footprint method [16], taking into account three basic phases in the life cycle of a commercial building [30], [31]: production, exploitation and post–use management in the form of landfill and recycling.

2. Materials and Methods

Life Cycle Assessment was used in this study to deeply analyse the impact of particular life stages on environmental burdens connected with commercial building, using the Ecological Footprint method. The research object is a commercial building located in Janikowo in central Poland , being in operation for already several years. The lifetime of the facility assumed by the operator is 40 years, after which the operator decides to demolish it, and build a new facility which meets the current requirements and expectations. The total area of the building is approximately 1000 m^2. The associated infrastructure (parking lots, access roads and sidewalks), the environmental impact of which has also been considered, has an area of approximately 2500m^2. The average annual consumption of energy and energy carriers related to the exploitation phase is as follows: natural gas about 11000 m^3 and electricity about 213 MWh. The first stage of the environmental analysis covering the full life cycle is in accordance with the ISO 14000 standards family, the definition of the system boundary, and next the identification of in– and output streams (Fig. 1). As input streams, the following were identified; energy resources, non–energy resources, water and land. While as output streams; pollution of the atmosphere, solid waste, pollution of water and soil and land degradation [32]. The Ecological Footprint method is based on the quantitative indicator of human impact on the environment. It illustrates the size of the biologically productive area (both lands, seas and oceans) necessary for the production of resources and products consumed by its user or users. The calculations also consider the area of land necessary to store the generated wastes and absorb the emitted pollutants. The Ecological Footprint (EF) is calculated for a specific period of time (usually one year) and for a specific population (on a global, regional or a single person scale). Since all impact categories are expressed in the same unit, a weighting factor of 1 is used for each of them [33].

The Ecological Footprint method can be successfully used in the environmental assessment of buildings in terms of energy and environmental optimization, which seems to be of a great importance for the performed study. The Ecological Footprint analysis of an academic building in India [34] revealed, that the replacement of energy from conventional sources with energy from PV panels would reduce the environmental burden by about 60%.

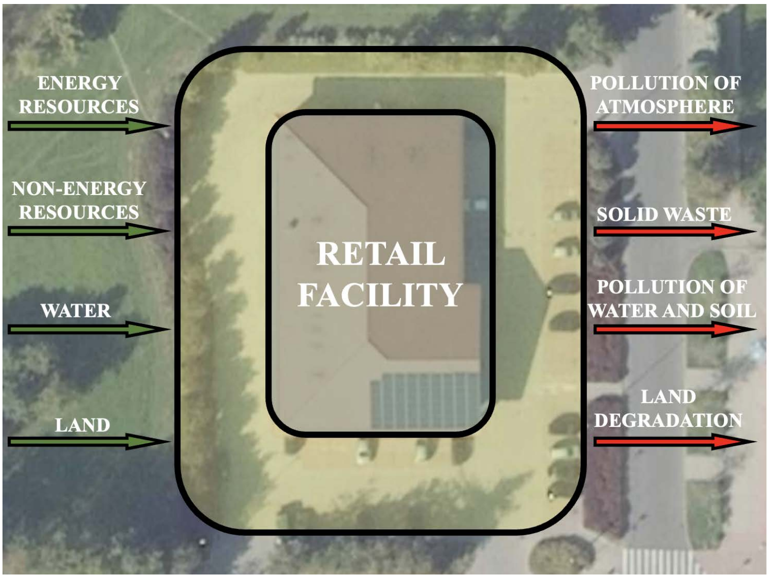

Fig. 1. Retail facility building as a subsystem in the eco–industrial facility system based on [10]

The cumulative environmental load over time is the sum of the loads from unit processes occurring at individual stages of the life cycle (Eq. 1) and is determined basing on the characteristics and exploitation plan of the building.

Knowledge about the amount of building materials and energy used, etc. allows to determine the total level of the cumulative environmental load over the life cycle of a commercial building (L_{RF}), which is the sum of:

$$L_{\mathrm{RF}} = Lc_{\mathrm{RF}} + Le_{\mathrm{RF}} + Lpc_{\mathrm{RF}} \tag{1}$$

where:

Lc_{RF} – cumulative environmental load in the production phase (incl. production of plastics, materials, elements and the construction of building objects);

Le_{RF} – cumulative environmental load in the exploitation phase, considering the consumption of energy and matter during exploitation and periodic repairs;

Lpc_{RF} – cumulative environmental load in the post–consumer phase, for example, landfilling or recycling [10].

3. Results

The results of the conducted environmental analysis are presented with division into three sections, comparing the impact of post–consumer management in the form of landfill or recycling and presenting the results for individual life cycle stages (production, exploitation, landfill, recycling).

To obtain comparable results expressed in environmental points (Pt), the results were grouped and weighted in accordance with the Ecological Footprint procedure, and subsequently summed up. This made it possible to compare with each other the individual stages of the research object's life cycle. The largest number of negative environmental consequences in the life cycle of the analysed commercial building, both in the case of post–use management in the form of landfill and in the form of recycling, is related to the emission of carbon dioxide to the atmosphere ($2.43 \cdot 10^7$ and $2.41 \cdot 10^7$ Pt, respectively). The total level of impacts is higher in the case of post–use management in the form of landfilling ($2.50 \cdot 10^7$ Pt) than the analogous cycle ended with recycling ($2.47 \cdot 10^7$Pt) (Table 1).

Table 1. Results of grouping and weighing of the environmental consequences occurring in the life cycle of the analysed commercial building, considering the form of post–use developments

	Life cycle with post-use management in the form of landfill	Life cycle with post-use management in the form of recycling
Carbon dioxide emissions	2.43×10^7	2.41×10^7
Radioactive substances	5.91×10^5	5.00×10^5
Processes related to land use	1.59×10^5	1.33×10^5
Total	$\mathbf{2.50\times10^7}$	$\mathbf{2.47\times10^7}$

The comparison of the successive phases of the life cycle of the analysed object shows the dominant share of the exploitation phase in the cumulative level of negative environmental consequences. This is due to the high demand for energy throughout the exploitation phase (electricity and gas), which is necessary for the proper use of the building. Electricity in Poland is generated primarily in conventional processes, and as a result, a large amount of carbon dioxide is released into the atmosphere. It is also the reason for small differences in the cumulative level of environmental impacts of the building with post–use management in the form of landfilling and in the form of recycling, because the operating costs for both cases are comparable (Table 2).

Table 2. Results of grouping and weighing of the environmental consequences occurring at stages of the analysed commercial building's life cycle

	Manufacture	Exploitation	Landfill	Recycling
Carbon dioxide emissions	5.04×10^5	2.36×10^7	2.00×10^5	-
Radioactive substances	3.85×10^4	4.62×10^5	9.10×10^4	-
Processes related to land use	4.60×10^4	8.66×10^4	2.67×10^4	-
Total	$\mathbf{5.89\times10^5}$	$\mathbf{2.41\times10^7}$	$\mathbf{3.18\times10^5}$	**-**

The use of the conventional energy sources is the dominant cause contributing to the climate change aggravation, which globally is one of the key aspects in environmental protection. Utilities consumption in the exploitation phase of the analysed building is characterized by the highest emission level of compounds causing global warming ($2.40\cdot10^7$ Pt) (Table 3).

Table 3. Results of grouping and weighing of the environmental consequences of the compounds emissions causing global warming, occurring at the stage of exploitation of the analysed commercial building

	Construction works	Sanitary installations	Electrical installations	Roads and parking lots	Utility consumption
Carbon dioxide emissions	4.87×10^4	6.24×10^3	6.72×10^2	2.98×10^1	2.35×10^7
Radioactive substances	8.97×10^3	1.31×10^3	5.85×10^1	5.72×10^{-1}	4.51×10^5
Processes related to land use	1.63×10^3	6.57×10^2	8.44×10^1	1.07×10^{-1}	8.42×10^4
Total	$\mathbf{5.93\times10^4}$	$\mathbf{8.20\times10^3}$	$\mathbf{8.15\times10^2}$	$\mathbf{3.05\times10^1}$	$\mathbf{2.40\times10^7}$

4. Summary and Conclusions

In recent years, there has been a noticeable increase in importance of the energy and environmental efficiency issues of buildings in Europe. Legislators are increasingly strictening the requirements for environmental protection, which could potentially constitute a serious developments limitation for business entities that will not follow the ideology of sustainable development. For the above reason, it has become rational in recent years to carry out energy and environmental analyses for buildings.

Considering the previously presented results of LCA, it has to be noted that the highest amount of environmental burdens, as well as greenhouse gas emissions has been recorded for the exploitation stage of the analysed building. This is due to the high level of conventional energy sources used for energy production at this stage of life cycle. These outcomes are similar to the previously performed LCA studies [17]–[19].

The results of the conducted analysis indicated the exploitation phase as the main cause of environmental burdens at the total level of 96%. The above-mentioned results are consistent with literature studies, which have been reviewed in chapter 2 of this paper, e.q. [19] and [21].

The life cycle of commercial buildings is characterized by a high level of utilities consumption, which is related to emissions of harmful substances. Additionally, the amount of energy accumulated in building materials is large and may range between 5.5–6.5 $GJ\times Mg^{-1}$.

Due to very energy–consuming production process, cement is the material that significantly increases the level of accumulated energy. The other essential material influencing the amount of energy consumption is the reinforcing and structural steel. Increasing the use of lightweight concretes and insulating materials makes it possible to significantly reduce the energy demand in the production phase. In addition, their use is also of key importance in reducing exploitation energy consumption.

Keeping in mind the analysis which has been carried out, and also the facts described above, it should be noted that the reuse of building materials after the end of their life cycle, to the extent that technical and economical possibilities, as well as the knowledge in the field of recycling currently allow, has a positive effect on the amount of environmental burdens. An aspect not covered by the classic LCA analysis is the lack of need to obtain raw materials for materials that can be recovered in the recycling process and reused. On the one hand, it is associated with a relatively large energy expenditure related to transport, possibly steel melting or crushing concrete, while the savings resulting from the lack of use of raw materials are significant. Such a procedure is in line with the idea of sustainable development.

There are various methods in the field of science and civil engineering aiming at reducing the environmental loads of buildings caused during the exploitation phase. The environmental burden can be influenced already at the building design stage by appropriate selection of the facility area and volume in reference to the needs. An important aspect is to design the building in a compact form to reduce its energy consumption. Efforts to reduce energy consumption from conventional sources are also important, e.g. by using renewable energy sources (photovoltaic panels, heat pump) and ensuring an appropriate operation plan and the high efficiency of heating and air-conditioning installations.

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Orginal Article

Comparison of approaches to reliability verification of existing steel structures

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Abstract: Many existing steel structures are exposed to degradation due to corrosion or fatigue and to increasing loads. Their reliability assessment is then needed. The key question is whether a particular structure can be preserved ‘*as it is*’, or needs to be strengthened, or whether it needs to be replaced. Unnecessary replacements of existing structures may be avoided and the remaining service life of existing steel structures may be authorized by: using advanced reliability verification techniques, optimizing target reliability, and obtaining data for a specific site or structure. In this contribution, the application of advanced reliability approaches is illustrated by the assessment of an existing steel structure. The case study demonstrates that such approaches may significantly improve assessment and allow to increase the load-bearing capacity of the structure (in the case under investigation by 10 to 20%). Improvements in reliability assessment are attributed to the use of an optimal target reliability level, case-specific statistical parameters and probabilistic distributions of the basic variables, and adjusted partial factors.

Keywords: Existing structures, adjusted partial factors, probabilistic approaches, reliability

1. Introduction

Existing structures represent a large volume of structures and they can be exposed to degradation due to corrosion or fatigue or to increasing loads. Their reliability assessment is then needed. The key question is whether a particular structure can be preserved ‘as it is’, needs to be strengthened, or whether it needs to be replaced. Unnecessary replacements of existing structures may be avoided and the remaining service life of existing steel structures may be authorized by:

- Using advanced reliability verification techniques – the main focus of this study;
- Optimizing target reliability;
- Obtaining data for a specific site or structure [1, 2].

At present, existing structures are mostly verified using simplified procedures based on the partial factor method commonly applied in the design of new structures according to actual codes. Such assessments are often conservative for existing structures and may lead to expensive upgrades [3, 4, 5]. A more realistic verification of the actual performance of existing structures can be achieved by using:

- Adjusted partial factors where the assessment values are obtained as fractiles of updated probabilistic distributions corresponding to probability defined based on sensitivity factor and a selected target reliability level. General guidelines for adjusting partial factors are provided by the basic Eurocode EN 1990 [6] and the international standard ISO 2394 [7].
- Probabilistic methods consider all basic variables describing loads and resistances as random variables using appropriate probabilistic models based on available experimental data. However, their applications require additional calculations and special experience. Further information about probabilistic analysis, reliability management and utilisiation of monitoring can be found in [8 – 11].

The submitted study is aimed at improvements methods of reliability assessment gained by applying advanced procedures in reliability assessment of an existing steel structure; a comparison with results obtained by application of the partial factor method recommended for structural design is provided. The analysis is carried out for the snow load as the leading variable action. Further, optimizing target reliability for existing buildings is briefly discussed briefly.

2. Adjusted partial factors

As the first advanced approach applied in the case study in Section 4, partial factors are adjusted considering structure-specific (information about materials, dimensions, permanent actions, system behaviour etc.) and site-specific (e.g. information about variable loads) conditions [9]. The assessment values are obtained as fractiles corresponding to probability from generalized values of sensitivity factors and a selected target reliability level. Adjusted partial factors are one of the basic approaches to assessment of existing structures introduced in the draft prEN 1990:2022, providing the basis for structural design and assessment of existing structures. This is why it is important to investigate and critically compare this approach with the partial factor method using the values of partial factors recommended for design (hereafter “fixed partial factors”).

Assuming a lognormal resistance given as the product of resistance model uncertainty θ_R, geometrical property a, and steel yield strength f_y, the partial factor for resistance R of generic steel members could be obtained as:

$$\gamma_M = R_k / R_d \approx [\exp(-1.645 V_{fy})] / [\mu_{\theta R}\, \mu_a\, \exp(-\alpha_R\, \beta \sqrt{(V_{\theta R}^2 + V_a^2 + V_{fy}^2)})] \tag{1}$$

where V is coefficient of variation; $\mu_{\theta R}$ is the mean value of resistance model uncertainty θ_R; μ_a is systematic deviation of random values of the geometric characteristics from its nominal value, expressed as the ratio of mean to nominal value; and α_R is the sensitivity factor of the FORM (First Order Reliability Method) for resistance; and β is the target reliability index. The subscripts "*k*" and "*d*" denote characteristic and design (assessment) values, respectively.

Assuming a normal distribution of the permanent load effect given as the product of load effect model uncertainty θ_E and permanent action g, the partial factor for the permanent load could be calculated as:

$$\gamma_G = G_d / G_k \approx 1 - \alpha_E \beta \sqrt{(V_{\theta E}^2 + V_g^2)} \qquad (2)$$

where G_d is the design value of permanent action effect; G_k is the characteristic value of permanent action effect; and α_E is the sensitivity factor of the FORM method for load effects.

Expression (2) assumes that tshe characteristic value of the permanent load effect corresponds to its mean value, the permanent load effect is normally distributed, unity characteristic value is considered for unbiased load effect model uncertainty and that a nominal value of a geometrical property corresponds to its mean.

In general, the partial factor for the variable load could be obtained as [5]:

$$\gamma_Q = Q_d / Q_k = F_{Q,tref}^{-1}[\Phi(-\alpha_E\beta), t_{ref}] / Q_k \qquad (3)$$

where Q_d is the design value of variable action effect; Q_k is the characteristic value of variable action effect; and $F_{Q,tref}^{-1}$ is the inverse cumulative distribution function of maxima of the variable load Q_{tref} during a reference period t_{ref}, for which a target reliability index β is specified.

A generic model for variable load effects may be written as follows:

$$Q_{tref} = \theta_E C_0 q_{tref} \qquad (4)$$

where θ_E is load effect model uncertainty; C_0 is the time-invariant component of variable load; q_{tref} is the time-variant component of variable load.

When a Gumbel distributed time-variant component is a dominating source of variability (commonly for climatic and imposed loads – see Section 3.2 for further details), the partial factor γ_Q can then be estimated as:

$$\gamma_Q = \mu_{Qtref} \times \{1 - V_{Q,tref}[0.45 + 0.78\ln(-\ln \Phi(-\alpha_E b_t))]\} \qquad (5)$$

where $\mu_{Q,tref}$ is mean of maxima of variable load effect (relatively to its characteristic value) and $V_{Q,tref}$ is its coefficient of variation related to t_{ref}:

$$\mu_{Q,tref} \approx \mu_{\theta E} \times \mu_{C0} \times \mu_{q,tref} \qquad (6)$$

$$V_{Q,tref} \approx \sqrt{(V_{\theta E}^2 + V_{C0}^2 + V_{q,tref}^2)} \qquad (7)$$

3. Probabilistic reliability analysis

The second advanced approach compared in Section 4 with the fixed partial factors is the probabilistic method. In contrast to adjusted partial factors, the probabilistic approach requires no assumptions on sensitivity factor values. In the general case limit state function for steel structural members may be written as follows:

$$g(\boldsymbol{x}) = \theta_R R - \theta_E [G + C_0 \mu_{tref}] \qquad (8)$$

where θ_R and θ_E are random variables characterizing the uncertainty in resistance and load effect models respectively, R is a random variable characterizing the resistance of the cross-section or of structural member, G is a random variable characterizing the permanent load, C_0 is the time-invariant component (e.g. shape factor), q_{tref} is the time-variant component of the variable load.

3.1. Resistance

For checks of the ultimate limit states of steel structures, resistance models are often based on yield strength. Reducing the uncertainty in yield strength by additional measurements on the existing structure thus often has a positive effect on the quality of the assessment. During design, the coefficient of variation of the yield strength is in range 5-8% [10], [11]. In the assessment of existing structures, it is possible to measure material and geometrical properties of steel members that may considerably vary for different steel grades, profiles and production processes adopted by various producers. For existing steel structures, the main source of uncertainty is a within-batch (within-rolling) variability. Based on these assumptions, $\mu_{fy} / f_{\mathrm{yk}} = 1.09$ and $V_{\mathrm{fy}} = 5\%$ are adopted in this study as representative values for the assessment. The variability of the geometric characteristics for steel structures is small compared to variability of members from other construction materials, the coefficient of variation is 2-5% [10], [11]. When dimensions are verified in-situ, unbiased values and a lower coefficient of variation can be considered [12], $V_{\mathrm{geo}} = 3\%$ is taken for further analysis. In this study, a resistance model of the cross section under bending (sufficiently braced to restrain instability; thus without buckling effects) is adopted with the following statistical parameters $\mu_{\theta R} = 1.1$ and $V_{\theta \mathrm{R}} = 5\%$ [13]. It is important to note that the resistance model is based on steel yield strength; resistance models based on steel ultimate strength would have different model uncertainty characteristics.

3.2. Action effects

Structures may be exposed to the effects of permanent loads, imposed loads, climatic (snow, wind, etc.) actions, differential settlements, water and earth pressures, earthquakes, accidental actions etc. The following analysis is focused on two key load types for structures – permanent loads and snow loads. Considering the snow load is the only variable action, the fixed and adjusted partial factors are applied according to the load combination rule 6.10(a,b) in EN 1990 [6]. According to this rule, either design value of the snow load effect, $\gamma_Q Q_k$, is combined with a reduced design value of permanent action effect, $\xi \gamma_G G_k$, or the design value $\gamma_G G_\mathrm{k}$ is combined with a combination value $\psi_0 \gamma_Q Q_k$; the maximum total load effect of the two is then considered. In the probabilistic approach, maxima of the snow load effect related to a reference period adopted for the reliability analysis are considered.

Permanent loads are caused by the self-weight of structural and non-structural members connected to the structure. The permanent loads may be commonly described by the normal distribution with the unbiased mean and coefficient of variation 3-10% [11]. To simplify the following analysis, the permanent load is assumed here to be a single-source, unbiased with respect to a nominal value and with the coefficient of variation of 5% (considering the possibility of measurements during the assessment).

Description of ground snow loads is typically based on sufficiently long records of annual maxima. The results of numerous studies indicate that a Gumbel distribution is often an appropriate model for annual maxima as also recommended in ISO 4355:2013 [14] and

EN 1991-1-3:2003 [15] for snow loads on structures. The background document [16] for EN 1991-1-3 suggested that a Weibull distribution provided the best fit to local measurements; Sadovsky [17] considered a flexible three-parameter GEV distribution for annual maxima of ground snow loads. It is noted that statistical uncertainty may be significant for the three-parameter distribution particularly when records span over short period only or when the records are affected by measurement uncertainty (e.g. for snow depth measurements) [18 – 20]. It may then be preferred to apply the two parameter distributions such as Gumbel, Weibull or lognormal accounting for generally good experience with these in particular climates.

The averaged values of the statistical parameters for describing the distribution of annual maxima of the snow load are adopted. Mean and coefficient of variation for annual maxima are taken equal to 0.4 Q_k and 50%, correspondently. These statistical characteristics can be objectively compared to similar data obtained in countries with a similar climate. The background report for Eurocodes [11] proposes the generalised values for annual maxima that are similar to those adopted here. Statistical parameters of Gumbel distribution for different reference periods (Table 1) are obtained as follows:

$$\mu_{Q,tref} = \mu_{Q,1}\,[1 + 0.78\,\ln(T)\,V_{Q,1}] \tag{9}$$

$$\sigma_{Q,tref} = \sigma_{Q,1} \tag{10}$$

where $\sigma_{Q,tref}$ and $\sigma_{Q,1}$ are standard deviation for snow load related to t_{ref}.

Snow load on the roof is obtained from the ground snow load by using shape, thermal and exposure factors. Uncertainties related to these coefficients are described here by the time-invariant coefficient C_0 according to [11].

In accordance with the generally accepted practice, load effect model uncertainty is described here by a unity mean and coefficient of variation of 7.5% [10].

The probabilistic models considered in the case study are presented in Table 1.

Table 1. Probabilistic models of basic variables considered in the case study

Basic variable	X	Dist.	μ_X / X_k	V_X
Yield strength	f_y	LN	1.09	5%
Geometry	a	N	1.0	3%
Resistance model uncertainty	θ_R	LN	1.15	6%
Permanent load	G	N	1.0	5%
Snow (1-year maxima)	q_1	Gum	0.4	50%
Snow (10-year maxima)	q_{10}	Gum	0.76	26%
Snow load – time-invariant component	C_0	LN	0.8	20%
Load effect model uncertainty	θ_E	LN	1.0	7.5%

μ_X – mean, V_X – coefficient of variation, N – normal distribution, LN – lognormal distribution with the lower bound at the origin, Gum – Gumbel distribution (max. values), X_k – characteristic value of basic variable.

4. Case study of steel beam

In this section, reliability requirements following from the fixed partial factors (*FPF*) provided in EN 1990 (values recommended for design), adjusted partial factors (*APF*)

(Section 2), and probabilistic method (*PM*) (Section 3) are critically compared. Reliability of the existing structure needs to be verified. Structural survey reveals no defects affecting structural reliability at the Ultimate Limit States. A particular focus of the case study is a steel beam – roof girder exposed to the dominant effect of snow load. Previous studies indicated low reliability of such structural members when they are verified considering the probabilistic models adopted in design [21-23]. The steel beam is fully laterally-restrained and stability issues do not affect structural reliability.

A target reliability index is recommended according to EN 1990 [6]: β_t = 3.8 for a reference period of 50 years. However, target reliabilities are intended to be used primarily for the design of members of new structures. In general, lower reliability levels can be accepted for existing structures in comparison to structural design as follows from the general principles of structural reliability provided in ISO 2394:2015 [7] and prEN 1990-2 [24]. The two standards suggest that besides failure consequences, the target levels should also be differentiated with respect to relative cost of safety measures that is often much higher for the existing structure than for a structure being designed. There are a number of studies in the field of optimization reliability levels [25] – [28]. Optimizing target reliability for existing structures by implementing cost optimization procedures and criteria for human safety is presented in *fib* Bulletin 80 [5]. According to [5] two reliability levels are recommended – the minimum level below which the structure is considered unreliable and should be upgraded - reliability index β_0; and the target level indicating an optimum upgrade strategy - β_{up}. The reliability indexes β_0 and β_{up} are presented as a function of the collapsed area due to the failure structural member and a reference period (it is assumed that a reference period is equal to the remaining service life). In considered case the collapsed area is smaller than 100 m^2. Reliability assessment should verify whether or not the structure can remain in service for next 10 years, it means reference period t_{ref} is equal 10 years. For middle Consequence Class (CC2), β_{up} = 3.3 and β_0 = 2.8 are obtained according to [5] for reference period t_{ref} equal 10 years.

EN 1990 [6] is the basic document that suggests the load combinations (such as those in equations (6.10) or (6.10a,b) therein) and relevant partial factors. The following partial factors are recommended for structural design for permanent loads: γ_G = 1.35 and ξ = 0.85 and for variable loads γ_Q = 1.5 and ψ_0 = 0.5 (snow). The load combination rule 6.10(a,b) according to [6].

Using the adjusted partial factors and the probabilistic method (FORM), partial factors are derived to provide for the adopted target reliability index. The values of the partial factors are presented in Table 2.

Table 2. Comparison of partial factors (load ratio χ = 0.8)

	APF ($\beta_{0,10}$ = 2.8)	*APF* ($\beta_{up,10}$ = 3.3)	*APF** ($\beta_{0,10}$ = 2.8)	*APF** ($\beta_{up,10}$ = 3.3)	*PM* ($\beta_{0,10}$ = 2.8)	*PM* ($\beta_{up,10}$ = 3.3)
γ_G	1.07	1.08	1.05	1.06	1.05	1.09
γ_Q	1.10	1.24	1.40	1.64	1.42	1.65
γ_{M0}	0.97	1.00	0.84	0.85	0.85	0.86

* Adjusted partial factors calculated with the actual values (according to Figure 2) of the sensitivity factors (α_R = 0.2, α_G = 0.2, and α_Q = 0.95).

The geometrical characteristic of a cross-section W_i, such as section module, required to satisfy the limit state in accordance with a particular approach to reliability verification (hereinafter referred to as a requirement) is calculated from the limit state function:

$$g(\boldsymbol{x}) = W f_{yk} / \gamma_{M0} - [\gamma_G G_k + \gamma_Q\ C_0\ Q_k] \tag{11}$$

To cover a wide range of load combinations, load ratio χ is introduced. The load ratio χ denotes the ratio of characteristic variable loads to the total characteristic load given as:

$$\chi = Q_k / (G_k + Q_k) \tag{12}$$

In most practical cases for steel beams the load ratio may vary within the interval from 0.3 (for example, a steel beam with a reinforced concrete deck) up to 0.8 (lightweight steel roofing for industrial halls) [29].

Figure 1 displays variation of $w_i = W_i / W_{EN}$ with χ where W_{EN} is the reference value based on the partial factors recommended in Eurocodes for structural design. When $w_i < 1$, the reliability requirements according to approach "*i*" are lower than those according to Eurocodes for structural design.

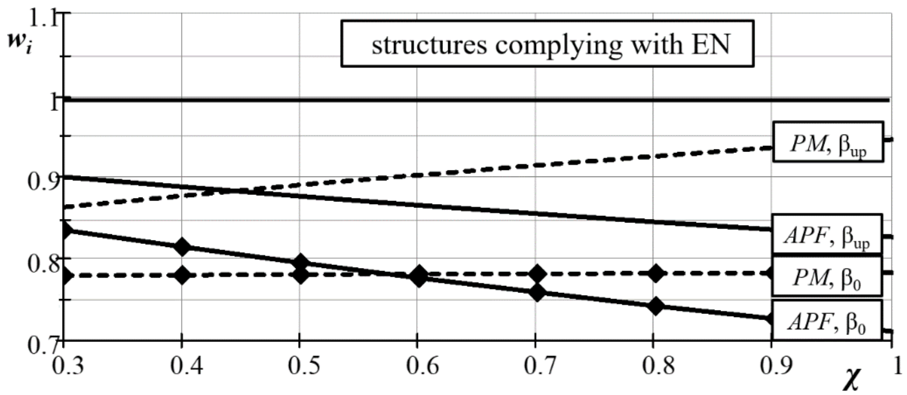

Fig. 1. Variation of w_i with χ (adjusted partial factors based on generalised values of sensitivity factors)

Figure 1 shows that an adopted target reliability level has a significant influence on the reliability requirement. For both adjusted partial factors (*APF*) and probabilistic method (*PM*) the lowest requirements are related to the β_0-level while the requirements based on β_{up} are between the β_0- and Eurocode requirements. Further, the *APF* and *PM* lead to different requirements. The main difference in these two methods is due to the assignment of generalised sensitivity factors α for the *APF*. It follows from Figure 1 that the generalised α-values may lead to unconservative requirements for $\chi > 0.6$ (β_0-requirement) and for $\chi > 0.45$ (β_{up}-requirement).

It is possible to determine the sensitivity factors using FORM to eliminate this deficiency of the *APF*. Figure 2 displays variation of the FORM sensitivity factors with the ratio χ. It appears that for steel structures, the dominant influence on reliability can be attributed to variability of the load effect.

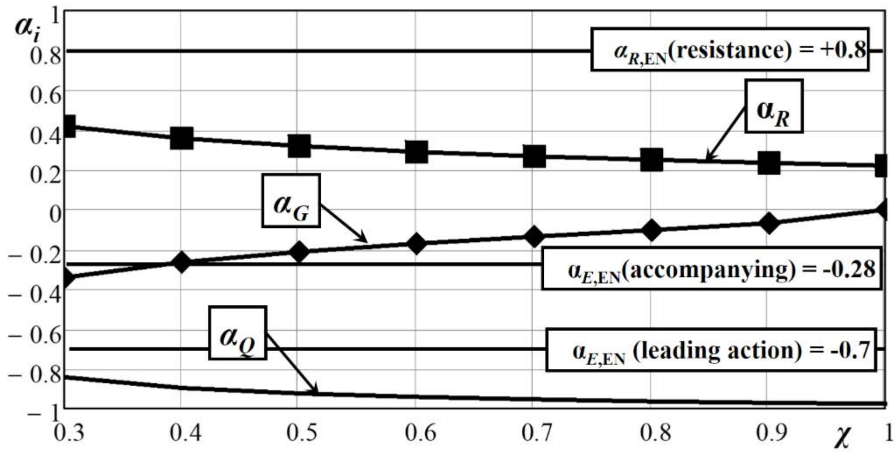

Fig. 2. Variation of sensitivity factors with load ratio χ

Figure 3 displays variation of w_i with χ for adjusted partial factors with the sensitivity factor α_E = -0.95 for the snow load and α_R = -0.2 for resistance. Using these α-factors, partial factor for snow load $\gamma_Q \approx 1.65$ and for resistance $\gamma_M \approx 0.85$ can be considered to comply with the β_0-requirement (Table 2). Similar reliability can be achieved by considering a commonly accepted $\gamma_M = 1.0$ along with reduced $\gamma_Q \approx 1.4$.

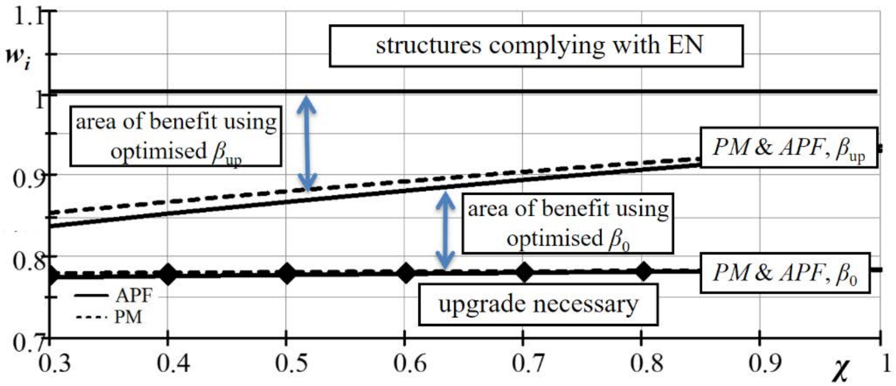

Fig. 3. Variation of w_i with load ratio χ (adjusted partial factors based on updated sensitivity factors)

The results of the *APF* and *PM* become close when using the actual values of the sensitivity factors. Figure 2 shows that the adjusted partial factors method (*APF*) and probabilistic method *PM* lead to the reliability requirements being lower than *EN.* The decrease in requirements is attributed to the use of the lower target reliability level for existing structure β_0 (lower than in *EN*) and case-specific probabilistic distributions for basic variables (measured statistical parameters of this distributions) that reduce the conservativeness of fixed partial factors. In contrast, the requirements for upgrades according to *APF* and *PM* (considering β_{up}) are close to those based on *EN*. The area between the curves for assessment (β_0) and upgrade (β_{up}) in Figure 2 is associated with the situations when the application of the advanced methods is expected to provide assessment benefit. In these situations, *EN* assessment requires an upgrade with economic and environmental impacts while the advanced methods authorise a continued use of the structure '*as it is'*.

For the structures designed according to the old standards valid before Eurocodes has been introduced and for which the snow load is the dominating variable load (e.g. wind load is comparably smaller), ratio w_i is expected to range approximately:

- From 0.8 when the snow load is the leading action in the load combination (χ close to 1.0)
- To 0.9 when the permanent load is the leading action (χ close to 0.3).

Such low w_i - values are attributed to increased design roof snow loads as introduced by Eurocodes. Similar observations were made for structures designed according to past Czech standards [30].

5. Discussion

The presented study provides a first insight into the performance of various approaches to reliability verifications of existing steel structures. Development and wider use of the adjusted partial factors seem to be reasonable considering the balance between demands on the input information, computational complexity, and achieved improvements in reliability assessments. Besides certain limitations of this method, it remains to define the target reliability levels for existing structures.

It seems that the most critical aspects in the application of the adjusted partial factors method is the setting of the sensitivity factor. It is generally accepted that a $\alpha_R = 0.8$ is suitable for the resistance model, and this value is recommended in EN 1990. This value seems reasonable for reinforced concrete, masonry and possibly for timber structures, for which the variability of the basic variables included in the resistance model is relatively large in comparison to uncertainties in the total load effect. For the steel structures, the variability of resistance is small compared to the variability of loads and it seems advisable to revise the recommended values of the sensitivity factors.

The probabilistic approach providing a reference level to simplified approaches requires further investigations as well. In particular, the review of available information shows incomplete empirical evidence to unambiguously justify the statistical parameters of variable load effects.

Modelling of degradation processes due to corrosion and fatigue seems to be another important challenge for further improvement of reliability studies. Special attention should be paid to reliability assessments of existing structures after fire exposure [31].

Cases with a single variable action are considered as a special issue of the reliability theory – combination of several variable actions is beyond the scope of this contribution. Previous studies revealed that the combination factors accepted in Eurocodes are often conservative and lower reliability levels were commonly obtained for the structures exposed to a single variable action compared to structures exposed to the effects of several variable actions [32].

6. Conclusions

Application of advanced probabilistic approaches allows reducing assessment requirements which the structure is considered unreliable and should be upgraded. This is attributed to the use of a particular target reliability level, case-specific statistical parameters and probabilistic distributions of the basic variables, and adjustment of partial factors. The application of fully probabilistic methods requires additional calculations and special experience while the adjusted partial factor method is easier to use. Nevertheless, the main conservatism of the latter method remains in the use of the generalized sensitivity factors.

The presented case study – detailed reliability analysis of an existing steel beam demonstrates that the dominant influence on reliability can be attributed to the variability of the variable load effect. The sensitivity factor α_E exceeding -0.9 for the dominating load and α_R smaller than -0.2 for resistance are obtained. It appears that reliability requirements for the minimum level below which the beam is considered unreliable and should be upgraded can be decreased by about 20%. Requirements for an optimum upgrade strategy of the beam might then be by about 10% in comparison to the design requirements.

For all the methods, the input data include the probabilistic models of basic variables. However, further studies and standardisation of such models is important and demanding task. As uncertainty in the load effects has the largest impact on reliability of steel structures, it is necessary to focus subsequent studies on a description of the models for loads and load effect model uncertainty. Within future research, the target reliability levels for existing structures should be specified.

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Orginal Article

Assessment of effectiveness of selected adaptation actions to climate change. The example of the New Centre of Lodz

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Abstract: The increase of the average annual temperature is observed in many cities around the world. Consequently, not only the Southern Europeans, but also the inhabitants of Central and Northern Europe are now exposed to thermal stress. Therefore, Nature Based Solutions (NBS) developed strategies for mitigation of this issue, i.e., the process of alleviating the negative effects of climate change in highly urbanized areas.

The main objective of this study is to answer the question whether the planned spatial activities involving the use of NBS solutions in the New Centre of Lodz can contribute to the improvement of the urban spaces' microclimate and thermal comfort of people in external environment. This research focusses on the microclimate of urban spaces, understood as climatic conditions set, and thermal conditions in particular, in a small area. The spatial scope of this research covers a 30-hectare area of the New Centre of Lodz bound within the following streets: Kilińskiego, Narutowicza, Piotrkowska, and Tuwima, which is currently undergoing a large-scale revitalization process. To determine the microclimate conditions and thermal comfort, numerical simulations conducted in the ENVI-met program were used.

Keywords: adaptation strategies in urban spaces, Nature-Based Solutions, microclimate, human thermal sensations, numerical simulations

1. Introduction

Forecasts indicate that the ongoing urbanization process[1] will affect the design of urban spaces. Transformations of settlement systems, including the structure, type of buildings functions, and natural components, will affect quality of life of residents; it will have a significant effect on health and well-being of people living in a given area. Researchers emphasize that not only the inhabitants of Southern Europe, but also Central and Northern European cities will be exposed to heat stress.

In recent years, numerous research projects were carried out, such as RUROS [2] and KLIMES [3], aimed at examining microclimatic conditions of urban spaces, such as streets, public squares, playgrounds, and parks. The research was conducted in various cities including Athens, Cambridge, Göteborg, Szeged, and Taichung City. The research emphasized the role and importance of planning activities in the study of thermal comfort [4]. When it comes to Polish cities, studies on the microclimate of urban spaces were carried out by Błażejczyk [5], Kłysik [6] (Lodz) and Lewińska [7]. Unfortunately, the obtained results were very rarely included in planning practice.

Local spatial development plans play important role in this aspect and should include matters related to urban spaces' microclimate shaping and human comfort. They are an integral part of the strategy for improving the quality of the living environment and the quality of life of those inhabiting urbanized areas. Very often, local spatial development plans do not fulfil their particularly important role. There is a pressing need to fill the gaps between urban climatology, spatial planning, urban design, and transferring microclimate knowledge to spatial planning.

This study has been limited to Lodz, the fourth largest and third most populous city in Poland. The research covered the area of the New Centre of Lodz, i.e., a 30-hectare area, which is considered to be of key importance due to the ongoing process of city revitalization. At the same time, it is the largest investment in Central Europe. For the selected area, research was carried out, which was divided into three main stages:

(1) Acquiring information on meteorological parameters in the New Centre of Lodz.

At this stage, a numerical base model was developed, covering the existing state of a 30-hectare part of the New Centre of Lodz bound within the following streets: Kilinskiego, Narutowicza, Piotrkowska, Tuwima. It was the starting point for further research.

(2) An analysis of the impact of the designed buildings on microclimatic conditions and thermal comfort of urban spaces.

An adaptive model (1) was created for a fragment of the New Centre of Lodz within which, in accordance with the city's design assumptions, new high-rise objects will be built. The construction of the Fabryczna Office complex has been considered.

(3) Assessment of the impact of the implemented passive adaptation strategies on the microclimate of urban spaces in the New Centre of Lodz. For this purpose, an adaptive model (2) was developed, considering blue-green solutions (NBS), in accordance with local spatial development plans for Lodz.

2. Urban adaptation strategies – Nature Based Solutions

The selection of optimal NBS is a key and extremely complex process. Currently, there is a very wide range of passive solutions. Every type of NBS has a significant impact – not only in terms of the environment, but also economic and social matters. This perception allows us

[1] By 2030, more than 60% of the world's population will live in cities, 70% of Europeans currently live in cities, and by 2050 it is expected to be 80% [1].

to state that the use of NBS solutions fits perfectly into the multifaceted idea of sustainable development.

It is worth emphasizing that shaping the microclimate of urban spaces should be an integral element of urban policy. NBS works well not only as an individual application. Thanks to city-wide programs integrated with spatial planning, cities can achieve even better results. It is possible to include NBS in a coherent network of a blue and green infrastructure. Such an approach naturally implements the last missing component of sustainable development in the form of institutional and political order. Examples of good practices in this area are the strategic instruments of German and Spanish cities such as Berlin, Hamburg, and Barcelona, i.e. [8]:

- Urban Landscape Strategy (Berlin) – a tool for integrated green management in the city. The main goal was to support the sustainable development of the city, with particular emphasis on increasing the area and improving the quality of green areas.
- Urban climate change adaptation plan (Berlin) – a plan to integrate adaptation to the effects of climate change as a permanent element of the city's development policy. The main goals were to maintain or improve the inhabitants' quality of life and to prepare solutions limiting the effects of climate change. The action plan includes 12 key projects using NBS to achieve specific goals.
- Project of adaptation of rainwater infrastructure (Hamburg) – integrated rainwater management strategy. The main goal was to develop a strategy for sustainable rainwater management in response to the progressive changes in the frequency and intensity of rainfall. The developed system aim is to restore the natural water circulation in the city and protect it against floods.
- The biotope area indicator – BAF (Berlin) – determines which part of the area, where new buildings are being introduced (construction projects and renovation) is to be biologically active areas. It is an obligatory or auxiliary planning instrument, depending on the area of the city. Its aim is to limit environmental degradation and to ensure the desired quantity and quality of blue-green areas in the city center. For different forms of land cover and different NBS, a different ecological weight was adopted – a different conversion factor for determining the ecologically effective area.
- Increasing the area of green areas by 2030 (Barcelona) – a standard of proximity and accessibility to green spaces established to support the implementation of nature-based solutions. The priority objective is to increase the green area per inhabitant by creating 160 ha of new natural zones.
- Estate management (Berlin) – the estate spatial development plan, supporting the sustainable development of housing estates. Directional programs on the scale of housing estates will enable them to be shaped directly by the inhabitants. Moreover, they are an instrument for the revitalization of estates in crisis. The main goal is to stabilize and strengthen social cohesion in three areas: housing, neighbourhood, and education.

3. The New Centre of Lodz

The New Center of Lodz is in the City's Metropolitan Zone. It covers an area bound within Narutowicza, Kopcinskiego, Tuwima, and Piotrkowska Streets. This investment was created to raise the rank of the city in the international arena, to promote and revitalize historical buildings as well as to create a new place, attractive for residents and tourists. Ultimately, it is to be a modern complex combining business, trade, culture, and transport. Currently, the New Center of Lodz is the largest investment in Central Europe.

The height of buildings in the New Center of Lodz is varied. The number of storeys in buildings ranges from 1 to 19. The dominant type are medium-hight buildings (95%). Most of the high-rise buildings are located in the Traugutta-Kilinskiego-Tuwima-Sinkiewicza quarter. The area is characterized by typical land cover of the downtown area. It is historically densely built up and artificial materials and impermeable surfaces (asphalt and concrete) dominate, while green areas are sparse. There are few lawns and trees (mostly deciduous), which make up a small percentage of the site total area, are located inside the courtyards of historical buildings and in the neighbourhood of the Fabryczna Office investment, which is currently at design stage. The areas intended for communication functions are mostly covered with impermeable surfaces. The only exception is the pedestrian and driving route – Traugutta Street (the so-called city woonerf), which was created in order to calm the traffic and increase the greenery.

The analysis of planning documents showed that there are three local spatial development plans for the area of the New Centre of Lodz. Regarding the protection and shaping of greenery, the following was established [9]:

- an order to preserve the existing green areas, shape new ones, and to preserve the existing rows of trees, with the possibility of their replacing and supplementing;
- an order to plant greenery with species resistant to urban conditions;
- a prohibition of logging trees, except for cuts necessary for safety reasons or colliding with buildings or infrastructure, and an order to compensate cut trees with new plantings.

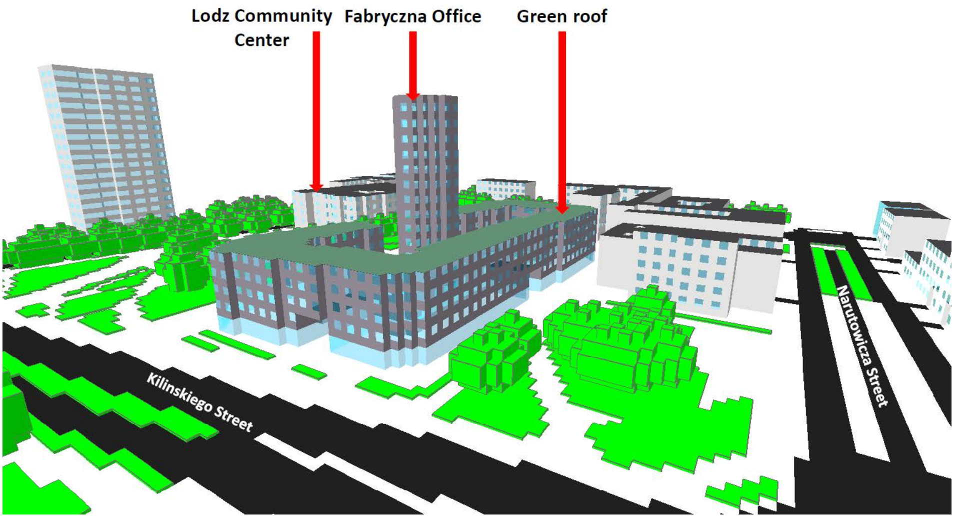

Fig. 1. The adaptive model (1). *Source:* own work

The area of the study was limited to a 4.3 ha zone bound within the following streets: Kilinskiego, Narutowicza, Sienkiewicza, Traugutta. The existing buildings constitute about 15% of the total area. The Fabryczna Office, an investment planned for implementation, served as the spatial dominant. According to the description, the building will have 13 overground storeys (55 m). The visualization of the Dutch MVRDV studio project was used as an inspiration.

The design assumptions were:

- adaptive model (1) – the design where the altitude dominant is the Fabryczna Office building (Fig. 1),
- adaptive model (2) – whose the core are the assumptions of the adaptive model (1) together with selected NBS. The introduced passive solutions include green roofs, green walls, rain gardens, blue infrastructure (fountains), new plantings, lawns, green lanes delimiting roadways, permeable surfaces, as well as a green car park (Fig. 2-3). The proposed solutions are in line with the local spatial development plans for the New Centre of Lodz.

As part of the adaptive model (2), a blue-green infrastructure was introduced in the neighbourhood of the Fabryczna Office (Fig. 2-3). The solutions were introduced in public spaces (in the courtyard of the Fabryczna Office, at the entrances to the complex from Kilinskiego and Narutowicza Street. Natural surfaces (lawns) have been added in the immediate vicinity of the Fabryczna Centre. The impermeable surfaces, which were a remnant of the demolished building, for example, the concrete parking located in the eastern part, were removed. In its place, a permeable surface (ground + grass) was implemented. Lines of trees were introduced at the entrance to the Fabryczna Office and also along Traugutta Street. Additionally, green belts with flower meadows were introduced, separating the main communication routes and fountains were implemented.

Buildings were covered with green roofs. In the case of the Fabryczna Office, both intensive and extensive forms of greenery were used. Extensive greenery was also introduced on the roof of the Lodz Community Centre (Fig. 2-3). Near the Fabryczna Office, the blind walls of the buildings were greened, and the potted plants were added. A green cubature car park was also designed (Fig. 2-3) with the use of vegetation on a special structure.

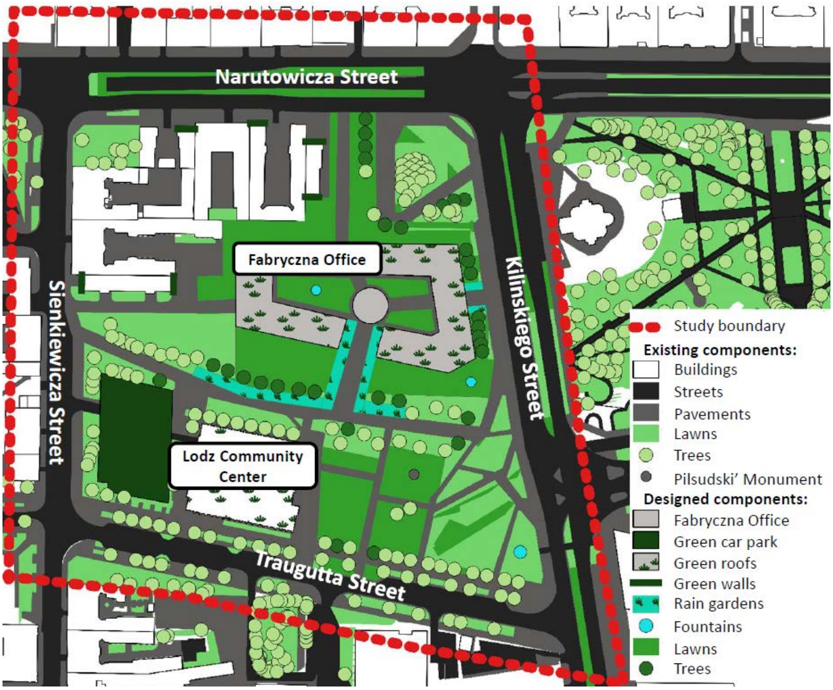

Fig. 2. The design concept for the adaptive model (2). *Source:* own work

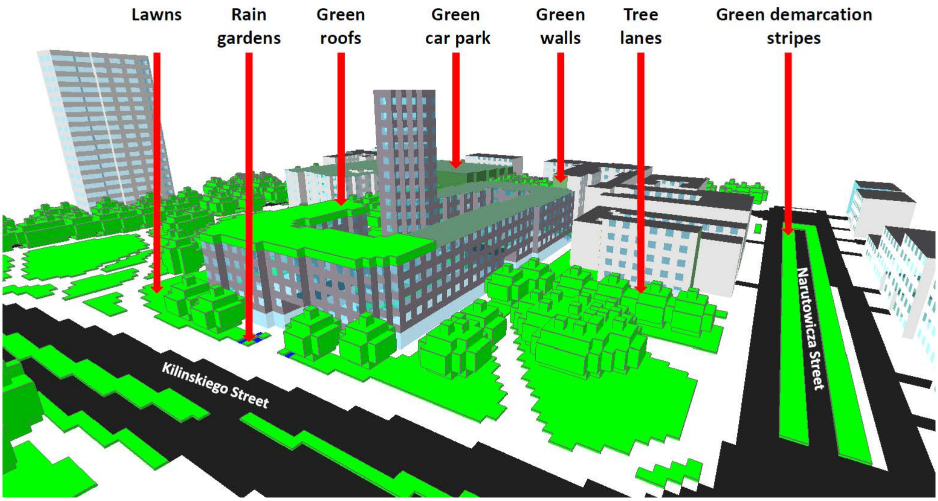

Fig. 3. The adaptive model (2). *Source:* own work

The existing pedestrian route running along the southern side of the complex was transformed into a green, pedestrian-friendly avenue. Tree plantings were introduced there. Numerous rain gardens in infiltration type containers appeared in the public space, i.e., along the mentioned green alley and at the entrances to the Fabryczna Office (Fig. 2-3).

4. Research methodology

One of the research methods used in the field of urban climatology is numerical simulation. Currently, one of the most widely used tools is ENVI-met, which is a CFD (Computational Fluid Dynamics) software. It was developed by a team of prof. M. Brus from the University of Mainz. It enables the simulation of the substrate – vegetation – air dependence in the urban environment in a daily cycle (from 24 to 48 hours). The program takes into account the flow of air between buildings, the processes of heat exchange of horizontal and vertical surfaces, turbulence, parameters of vegetation and dispersion of pollutants. S. Lenzholer includes ENVI-met among the tools that consider the parameters determining thermal comfort in the external environment, such as: air temperature, relative humidity, wind direction and speed, and average radiation temperature [10-11]. In this study, this tool was used to analyse the microclimatic conditions in the area of the New Centre of Lodz and to determine the people thermal comfort in the external environment.

4.1. Three-dimensional numerical model of the New Centre of Lodz

Carrying out the analyses required the creation of three-dimensional terrain models. The area bounded by Kilinskiego, Narutowicza, Piotrkowska and Tuwima Streets was treated as the base model. Adaptive models were created for the following quarters: Kilińskiego, Narutowicza, Sienkiewicza, Traugutta. The models are based on a rectangular grid of cells. An equidistant grid was used, the vertical structure of which was made of elements of a constant height. The grid parameters are presented in Tab. 1.

Then, the parameters of materials characteristic for construction objects located in the area of the New Centre of Lodz were defined. The thickness of the walls of the buildings as well as the physical parameters of building materials and roofing were determined (Tab. 2).

Table 1. Numerical model parameters. *Source*: own work

Model	Model resolution (x,y,z)	Grid size xyz [m]
Base	171×161×30	4×4×3
Adaptive 1/2	147×156×40	2×2×2
Common parameters for all models: Date: 5.07.2015 Location: the New Centre of Lodz Rotation: -9°		

The adaptive model (1) assumed the introduction of a newly designed facility – the Fabryczna Office. In this case, the impact of changing the structure of buildings on microclimatic conditions as well as thermal comfort was tested. The adaptive model (2) assumed the implementation of blue-green solutions in the form of tall greenery, green roofs, green walls, lawns, rain gardens and fountains. This activity was aimed at determining the impact of adaptation strategies on the conditions prevailing in the external environment. It should be mentioned that the adaptation scenarios considered the complex geometry of the buildings, gate passages, the diversity of building materials for the existing and designed structures (including glazing).

Table 2. Numerical model parameters. *Source*: own work

Parameter	Wall		Roof - roofing felt -
	Inner layer - brick -	Outer layer - plaster -	
Thickness [m]	0.64	0.02	0.01
Absorption [%]	60.00	50.00	94.00
Permeability [%]	0.00	0.00	0.00
Reflection [%]	40.00	50.00	6.00
Emissivity [%]	90.00	90.00	90.00
Specific heat [J/(kg×K)]	650.00	850.00	1460.00
Thermal conductivity [W/(m×K)]	0.44	0.60	0.18
Density [kg/m^3]	1500.00	1500.00	1000.00

4.2. Meteorological parameters

The microclimatic conditions for the warmest day of the Typical Meteorological Year (the 5th of July 2015) were used as input data for the simulation process of the base model[2]. Hourly values of air temperature, relative humidity, radiation, and the average daily value of air flow velocity were used (Tab. 3). The value of the subsoil roughness was chosen for the urbanized area.

2 This study uses the data of the warmest day of a Typical Meteorological Year, created based on information from 2004-2015, in order to show the maximum impact of the introduced adaptation strategies on microclimatic conditions, as well as the thermal comfort of a person in the external environment. More at: [12].

Table 3. Meteorological parameters of the base model. *Source*: own work

Hour	1:00	2:00	3:00	4:00	5:00	6:00	7:00	8:00	9:00	10:00	11:00	12:00
T_a*[°C]	15.50	14.80	14.60	16.70	21.10	23.40	26.80	29.60	31.00	31.70	32.70	33.30
RH*[%]	81.00	85.00	85.00	82.00	63.00	57.00	50.00	40.00	30.00	28.00	26.00	24.00
Hour	13:00	14:00	15:00	16:00	17:00	18:00	19:00	20:00	21:00	22:00	23:00	
T_a* [°C]	33.70	34.20	34.00	34.00	33.50	31.60	28.10	23.90	22.40	20.60	19.70	
RH*[%]	21.00	20.00	20.00	21.00	22.00	27.00	36.00	52.00	55.00	64.00	69.00	

Average daily wind speed: 0.80 [m/s]
Air inflow destination: west
The substrate roughness: 1.00 [m]
Adjustment factor for radiation: 0.80

*T_a – air temperature
*RH – relative humidity

The western direction of air inflow was assumed. Wind speed data was obtained from the Lublinek weather station, which is located in the suburban area of Lodz. Therefore, it was necessary to recalculate air flow within the strict city centre. For this purpose, the Simiu dependency was used, which made it possible to link the conditions of the suburban-downtown zone[3]. Consequently, the air flow velocity profile in the city centre was determined.

Then, the meteorological conditions were simulated for the base model, which made it possible to assess the influence of the building structure on the microclimate of urban spaces. The generated parameters served as input data for the simulation of adaptive models (1 and 2). The boundary conditions were the values calculated at the measurement point located in the vicinity of the Fabryczna Office complex (Tab. 4).

Table 4. Numerical model parameters. *Source:* own work

Hour	1:00	2:00	3:00	4:00	5:00	6:00	7:00	8:00	9:00	10:00	11:00	12:00
T_a* [°C]	20.63	19.83	19.38	19.55	20.86	22.39	24.18	25.75	27.20	28.28	29.10	29.89
RH*[%]	57.63	60.51	62.09	62.89	59.95	55.96	51.61	46.11	40.95	37.06	34.59	32.55
Hour	13:00	14:00	15:00	16:00	17:00	18:00	19:00	20:00	21:00	22:00	23:00	
T_a* [°C]	30.40	30.82	31.01	31.07	30.87	30.32	29.47	28.02	26.75	25.71	24.79	
RH*[%]	30.63	29.12	28.49	28.34	28.63	29.88	32.35	36.50	40.07	43.35	46.49	

Average daily wind speed at a height of 10 m: 0.44 [m/s]
Air inflow destination: west
The substrate roughness: 1.00 [m]
Adjustment factor for radiation: 0.80

*T_a – air temperature at the height of 2 m
*RH – relative humidity at the height of 2 m

4.3. Thermal comfort

The ISO 7730 standard defines it as ‘a state of mind expressing satisfaction with the thermal environment’ [14]. In this study, thermal comfort was estimated using the PET (Physiological Equivalent Temperature) index. The following meteorological data were taken into account: air temperature, average radiation temperature, air humidity and wind speed. Information on human physical parameters, thermal insulation of clothing, as well

3 More at: [13].

as the type of physical activity performed was used. The assumptions adopted for this study are presented in Table 5.

Table 5. Parameters used to estimate the PET index value. *Source:* own work

Parameter	Value
Clothing insulation [clo]	0.50 (light summer clothes)
Movement speed [m/s]	1.21
Human metabolism [met]	1.43
Height [m]	1.75
Weight [kg]	75.00
Age [years]	35

5. Microclimatic conditions in the New Centre of Lodz urban spaces

The conducted research was limited to three parameters which, in the context of climate change and the increasing frequency of the urban heat island occurrence, have a significant impact on human thermal comfort. Air temperature, surface temperature, and wind speed were analysed. Measurements were made at the level of human movement in public spaces (at a height of 1.4–1.5 m, depending on the resolution of the model). The thermal comfort of people in an urban environment was determined using the PET index.

5.1. Air temperature

There was a clear differentiation of the parameter in the study area. During the day, the difference between the coldest and hottest points was 2.79°C (Fig. 4). The occurrence of local heat islands within communication routes and in areas covered with impermeable surfaces characterized by low albedo were detected.

In the case of the adaptive model (1), the maximum value of air temperature was lower by 1.05°C compared to the base model (Fig. 4). This was due to the different spatial management of both areas. Near the Fabryczna Office (adaptive models), a greater number of natural surfaces was observed.

The adaptive model (2) assumed the implementation of blue-green solutions (Nature Based Solutions). High greenery, green roofs, green walls, lawns, rain gardens and fountains were introduced in the area of the study. The introduced adaptation strategies contributed to the improvement of microclimatic conditions. The analyses showed that the air temperature was reduced. The value of the parameter during the day fluctuated between 28.82-31.65°C (Fig. 4). This meant a decrease in the minimum value by 1.12° C, and the maximum value by 1.08°C as compared to the base model.

Fig. 4. Air temperature at the pedestrian height, at 2 pm (the base model (a), the adaptive model 1 (b), the adaptive model 2 (c)). *Source:* own work

5.2. Surface temperature

The surface temperature oscillated between 13.74–48.66°C. The value of the parameter varied in study area. During the day the difference between the coldest and hottest points was 25.14°C, and at night it was 6.49°C (Fig. 5). During the day, the minimum values of the parameter were observable in places limiting direct solar radiation reaching the ground surface, shaded by trees and buildings. Additionally, the maximum values were detected in places with artificial surfaces.

In the case of the adaptive model (1), the surface temperature values fluctuated between 22.96-47.63°C during the day (Fig. 5). Critical places where the highest value of the parameter was observed were communication areas (communication routes, Lodz Community Centre parking), as well as the space in the vicinity of the City Gate. This is related to the type of surface used; those particular areas were impermeable surfaces covered with asphalt.

In the case of the adaptive model (2), the minimum value of the surface temperature decreased by 0.81°C, and the maximum value by 1.1°C during the day compared to the base model (Fig. 5). The removal of asphalt surface in the vicinity of the Fabryczna Office was of key importance in reducing the temperature.

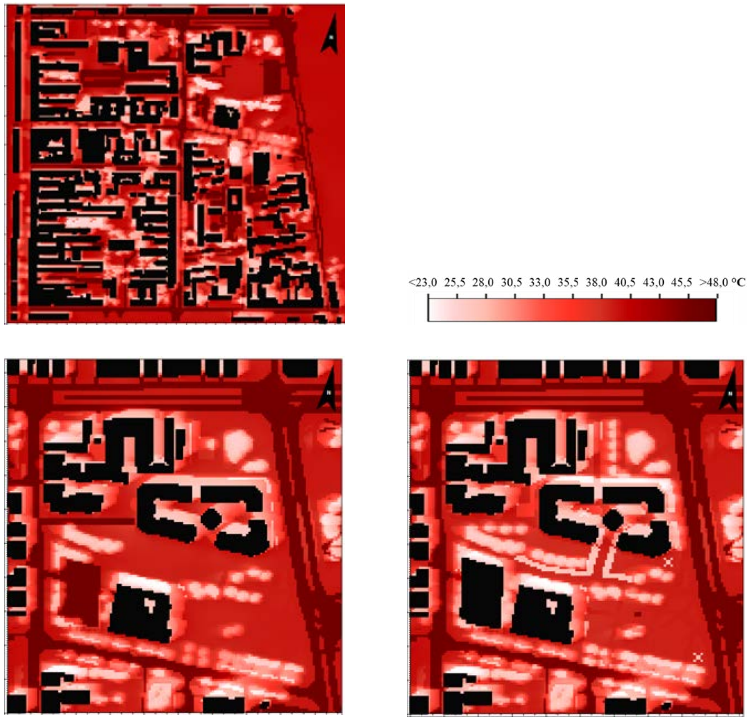

Fig. 5. Surface temperature at 2 pm (the base model (a), the adaptive model 1 (b), the adaptive model 2 (c)). *Source:* own work

5.3. Relative humidity

The distribution of relative humidity highlighted zones with the lowest density of buildings and the location of green areas. The maximum values of the parameter were observed particularly in the north-eastern part of the study area (Fig. 6).

In the case of the adaptive model (1), the areas characterized by the highest relative air humidity were located within the green belts along the streets, as well as on the east side of the Lodz Community Center. The minimum value of relative humidity was higher by 4.82% compared to the base model (Fig. 6). It was recorded in areas covered with impermeable surfaces (passageways, car parks).

In the adaptive model (2) was a visible increase in relative humidity in places where passive adaptation solutions were introduced. An increase in the minimum value by 4.9%, as compared to the base model was recorded (Fig. 6). The occurrence of 'cold areas' was observable in the eastern and central parts of the area.

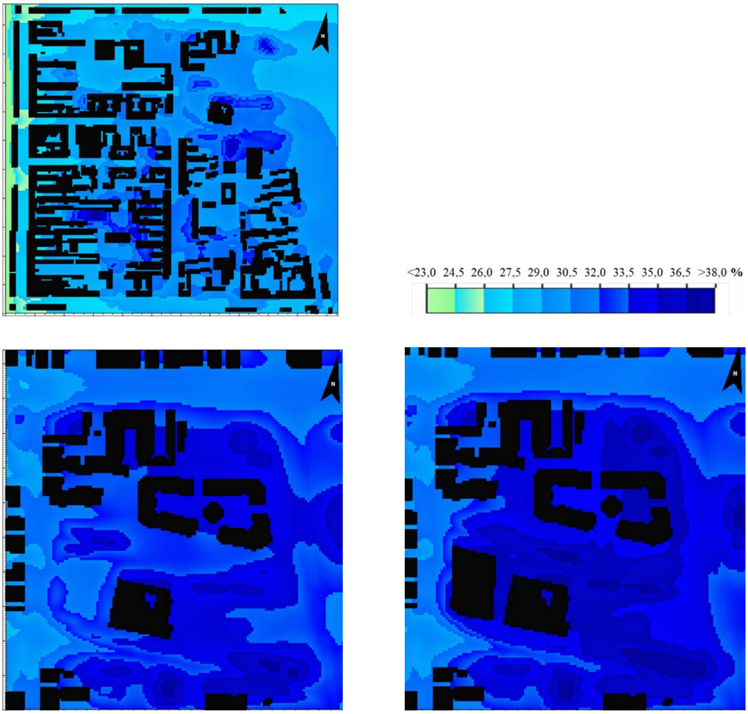

Fig. 6. Relative humidity at the pedestrian height, at 2 pm (the base model (a), the adaptive model 1 (b), the adaptive model 2 (c)). *Source:* own work

5.4. Air flow

For the considered cases, the western direction of air inflow to the city was assumed. The base model showed slow air flow in N – S street canyons ranging from 0.0 to 0.4 m/s (Fig. 7). This was due to the dense building structure located on both sides of the communication routes. Also, on the leeward side of the buildings, lower values of the parameter were observed. Minimal numbers were recorded in the area of closed courtyards of tenements.

In adaptive models, there was a noticeable slowdown in airflow in the vicinity of the Fabryczna Office. The lowest values were observed on the leeward sides of the buildings, inside the courtyards of historic buildings, as well as in the courtyard of the designed Fabryczna Office Centre. It should be noted that the Fabryczna Office traffic routes are located in the north, south and east. The shape of the building was closed from the west, which did not allow free air flow in the vicinity of the building. The described situation was the cause of low wind speed inside the courtyard, and thus air stagnation. As an effect, the air temperature increased, and consequently, thermal conditions experienced by humans deteriorated.

In the adaptive model (2), it should be additionally noted that wind speed slowed down at the Lodz Community Centre. The reason was the introduction of a green cubature car park, which acted as a barrier to inflow of air masses. The second place where a reduced value of wind speed was noticed was in the green alley. The plantings weakened air exchange in the vicinity of the Fabryczna Office (Fig. 7).

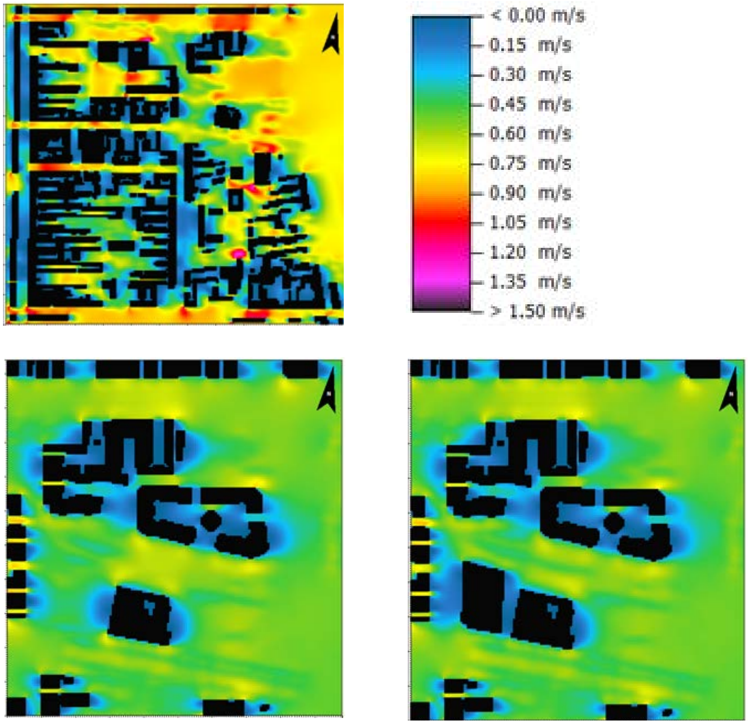

Fig. 7. Air flow at 2 pm (the base model (a), the adaptive model 1 (b), the adaptive model 2 (c)). *Source:* own work

5.5. Human thermal comfort – PET

Thermal comfort of a person in the external environment was estimated using the PET index. The value of the parameter was at the level of 11.55–57.80°C during the warmest day of the Typical Meteorological Year (Fig. 8). This meant the occurrence of 'hot' and 'very hot' conditions during the day. The highest values were recorded in the path of N–S orientation communication routes, as well as inside closed inner-city yards. Negative thermal impressions of people staying in urban spaces were largely the result of their spatial management. The surfaces were dominated by impermeable, artificial materials with a low albedo (asphalt, concrete) and there was a deficit of greenery. A characteristic closed building geometry of historic courtyards limited free flow of air, and air stagnation was observed within them. Due to the limitation of direct sunlight, the 'warm' conditions prevailed in the areas shaded by anthropogenic (buildings) as well as natural (green) elements (Fig. 8).

Fig. 8. Thermal comfort index – PET, 2 pm (the base model (a), the adaptive model 1 (b), the adaptive model 2 (c)). *Source:* own work

Supplementing the building structure with newly designed objects (the Fabryczna Office) influenced the modification of thermal conditions (adaptive model (1)). There was a decrease in the minimum value of PET by 1.73°C, and the maximum value by 3.20°C in relation to the base model. The indicator oscillated between 14.47–54.60°C. Daytime conditions were described as ‘warm’, ‘hot’ and ‘very hot’. Areas critical in terms of thermal comfort were communication routes, the Fabryczna Office courtyard and the parking area located at the corner of Sienkiewicza and Traugutta Streets. The most favourable (‘warm’) conditions were noticed in areas shaded by buildings and by trees.

The introduction of blue-green solutions (the adaptive model (2)) contributed to the modification of thermal conditions in the area. A reduction in the intensity of thermal stress was observed in the surroundings of the Fabryczna Office. There was a decrease in the maximum value of PET by 3.8°C, and the minimum value by 2.41°C during the day, compared to the parameters obtained in the base model. The improvement of thermal conditions was visible in the Fabryczna Office courtyard, in Narutowicza and Kilińskiego Streets, and in the vicinity of the green cubature car park at the corner of Sienkiewicza and Traugutta Streets. The lowest values were recorded in places shaded by trees.

6. Summary – assessment of the adopted planning solutions' impact on the microclimate of urban spaces and human thermal comfort

The aim of this study was to assess the impact of planning activities on the microclimate and thermal comfort of humans in the urban spaces of the New Centre of Lodz. The results of the carried-out research clearly indicate the importance of spatial development of the area. The adopted strategies using NBS solutions contributed to the improvement of microclimatic conditions and thermal comfort in the external environment. The analysed parameters improved, i.e., air temperature, surface temperature, relative humidity, and the PET thermal comfort index.

Table 6. Microclimatic parameters in the New Centre of Lodz. *Source:* own work

Parameter	Values 2:00 pm					
	Base model		Adaptive model			
			1		2	
	Min	Max	Min	Max	Min	Max
Air temperature [°C]	29.94	32.73	29.16	31.68	28.82	31.65
Surface temperature [°C]	23.52	48.66	22.96	47.63	22.71	47.56
Relative humidity [%]	23.84	37.82	28.66	37.14	28.74	37.61
Wind speed [m/s]	0.00	1.37	0.00	0.87	0.00	0.87
PET [°C]	34.38	57.80	32.65	54.60	31.97	54.00

The introduction of tall greenery and rain gardens was a significant development, thus increasing biologically active areas while eliminating building materials typical for the downtown zone – anthropogenic elements of the existing urban space development in the form of impermeable, artificial surfaces, such as asphalt or concrete. Also, particular attention should be paid to compact buildings, such as a city courtyard, where unfavourable microclimatic conditions prevailed including significant thermal stress. This place is a great challenge for the implementation of adaptation strategies – very often there is no possibility of significant interference. Limiting the proportion of hardened surfaces to the necessary minimum should be a priority within this type of structure. Passive solutions should be implemented there, such as elements of blue-green infrastructure that contribute to reducing thermal stress.

An example that proves the effectiveness of such solutions is the surroundings of the Fabryczna Office. The cooling effect of passive adaptive solutions was observed in the adaptive model (2), where the most favourable thermal conditions prevailed. The use of permeable surfaces, numerous lawns, as well as the introduction of fountains in the courtyard area allowed to reduce thermal stress. The results of the study of microclimatic conditions in the immediate vicinity of the building were also promising. The maximum reduction in air temperature was 0.37°C at the southern facade, and 0.21°C at the eastern façade (in comparison to similar studies, such a result is comparable and significant [13]). Additionally, the conducted analyses showed that the value of the daily air temperature amplitude in the green area was lower by 2.05°C in relation to the area of the asphalt street. Research conducted around the world confirms the effectiveness of the NBS in the mitigation process (Tab. 7). It should be noted that the extent of impact of the introduced solutions varies and depends on many factors, such as the selection of plant species, the size of trees, green area, water availability in cities and weather conditions. This is illustrated in the table below (Tab. 7). Even when the same NBS is used, the results depend on a variety of factors and the best effect was achieved in New York.

However, in this case, it should be emphasized that the research was conducted for the Central Park, which, due to the range of the green area or the size of the trees, most likely differs from the characteristics of other locations. The values obtained for the NCL are satisfactory, especially in relation to results obtained for Warsaw in the case of the park impact analysis. The implementation of additional solutions, primarily in the form of numerous high green areas, as well as increasing the percentage of blue infrastructure, could significantly improve the result obtained in the New Centre of Lodz.

Table 7. The influence of the applied NBS on the local temperature value reduction. *Source:* own study* based on [15-17]

Location	Mitigation temperature	Introduced NBS type
New York (Central Park)	12.2	Green areas / parks
Portugal	2.5-9.0	Green areas
Sweden	2.0-6.0	Parks
India	6.0	Trees
Israel	2.0-4.0	Lawns / trees
Netherlands	0.6-4.0	Green areas
Japan	2.0	Trees
Hong Kong (residential district)	0.5	Trees
Warsaw	1.0-3.0	Parks
New Centre of Lodz	0.3-2.5 *	Trees / green roofs / green walls / lawns / rain gardens

In the New Centre of Lodz, an unfavourable impact of changes in spatial development on anemometric conditions were observed. After introducing the new high-rise building (Fabryczna Office), the wind speed in its vicinity decreased. Due to its closed form from the west (from which air inflow was defined), air stagnation was observed inside its courtyard. The effect of the described phenomenon was an increase in air temperature and, as a result, deterioration of thermal conditions in the urban space. This case emphasizes the importance of the method of shaping the body of building objects that will consider the microclimatic requirements of a given area. Also, it shows how important it is to analyse the impact of the newly introduced cubature on the prevailing meteorological conditions at the design stage. Thanks to such modifications, such as a slight modification of the building's form (in the analysed case, introducing an opening allowing free air inflow from the west at the height of pedestrians' movement), ventilation of urban spaces could be improved. Moreover, the research showed the importance of method of shaping the structures of downtown buildings along communication routes. Slower air flow in N–S street canyons was evident (with the assumed flow of air from the west – the dominant direction in Poland), which resulted in intensification of thermal stress. In the case of canyons oriented in such a way, it may be good practice to introduce gaps between the buildings along the streets to allow free inflow of fresh air, as well as greenery – rows of trees and green belts separating the roadways.

It is worth emphasizing that shaping the microclimate of urban spaces should be an integral element of urban policy. NBS works well not only in individual applications. Thanks to city-wide programs integrated with spatial planning, cities can achieve even better results. They would make it possible to include NBS in a coherent network of blue and green infrastructure. Such approach naturally implements the last missing component of sustainable development in the form of institutional and political order.

The models used in the study can be treated as a guide for designers of urban spaces who want to create them in a way that ensures optimal, external thermal comfort. The usage of urban microclimatic models provides the ability to predict various thermal conditions.

The perception of the microclimate of a given urban space results in its different use by occupants in different climatic conditions. Therefore, an in-depth analysis of the microclimatic features of urban spaces, gives new opportunities for their growth, and increases the chances of optimizing their development in relation to the outside conditions throughout the year.

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Orginal Article

The sands of medium density and sandy loam density differences investigation while cement injection and pressuremetry crimping

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Abstract: The article focuses on the study of medium density sand and sandy loam density changes during water-cement solute injection and pressiometer expansion. Cement mortar injection is performed into boreholes under small pressure of up to 300 KPa resulting in radial well expansion. This expansion process also takes place during pressiometer studies. A cavity of compacted soil is formed around the expended boreholes in radial direction; the anchor and pile load determine field studies of the load, which they can perceive along lateral surface. It is worth noting that they frequently do not coincide with the calculated values. Following field anchor excavations, it was observed that root shearing can occur both directly at the contact between the cement root and the soil and at some distance from it. Subsequent laboratory studies revealed that when the soil is pressed, its density can change along a sinusoid, comprising pressed zones and decompaction zones. This gives rise to the displacement of the shear zone at the contact between the root and the soil.

Keywords: ground characteristic change, pressure, injection moulding, displacement, unit weight

1. Introduction

The soil injection method has become a widespread way of strengthening buildings and structures' foundations in Belarus. This method is used for making anchor and the pile for new buildings and during reconstruction and modernization of existing buildings and structures. This technology is currently used to increase the ground bearing capacity of plate foundations, anchors and piles, and also to prevent dangerous geological processes such as settlements, subsidence and suffusion.

The unit weight, angle of internal friction, and cohesion change take place when cement mortar is injected under small pressure of up to 300 kPa into boreholes; when anchors and piles are erected, and ground well expands. The same processes of well expansion and compaction of the surrounding soil also takes place during ground pressuremeter tests for determining the total deformation modulus. The value of the additional expansion of the well and dimensions of the compacted soil zone depend on the depth below the day surface, compressibility of the soil, and technological features of the injection. The well walls displacement determination can be attributed to the classical Lame solution of the thick-walled vessel walls stresses and displacements determination.

2. Justification of research objectives

Only a small number of works have been devoted to a purposeful study of the variability of soil properties around boreholes and wells expended by cement mortar injection. This is due to the technical impossibility of digging out natural anchors and piles in various soils in field, as opposed to the high accuracy experimental work performed in natural conditions, and the methodological functionality of modeling in laboratory conditions of the physical processes that occur during the introduction of cement mortar into soils.

The analysis of the studies of A. Cambefort [1], L. Hobst [2], V.A. Mishakov [3], M. Al Masri [4], A.A. Petukhov [5], M.I. Nikitenko [6], K.E. Povkolas [7], J. Warmer [11] and others helps us to determine:

- the ground (non-rocky soils) property change occurs to ground pressing and watering while cement mortar injection mainly due to ground pressing and watering (P) M.I. Nikitenko [6]). The compacted zone is equal to 3.1-3.2 of the initial diameter of the borehole ore well in the plane;
- the soil strength characteristics decrease with distance from the borehole Smorodinov [8];
- there are ground compaction and decompaction zone tracks (M. Hello Moussa [9]);
- borehole diameter increased due to the injection pressure (P) M.I. Nikitenko [6]);
- the waterproof soil quality does not depend on the pressure value, but on the time of injection (P) M.I. Nikitenko [6]);
- the injected can be characterize as an outflow along microfractures while injection with pressure of more than 0.4 MPa;
- penetration into the contact layer of up to 7-10 mm thickness and cementation of soil particles around the injection material are possible in loose, coarse, and gravel sands;
- excess moisture is drained from the water-cement mixture, which may be characterized by the W/C = 0.23-0.27 in the sands;
- they is no cement mortar impregnation in cohesive soils;
- high injection pressure adducts to hydraulic cracks around wells in clays;
- water is taken from the cement mortar into the surrounding soil, therefore, the clay soil plasticity increases and friction falls down in the first time after the mortar injection. At the same time, the resistance of the lateral surface increases with the soil strength increase and the plasticity decreases. [2, 6, 9] ;
- there is an increase in contact shear resistance in cylinders in 1.5-2.0 times compared to cement mortar injection and cement mortar filling holes [9]).

3. The description of the experiment and applied soils

3.1. The ground description

For laboratory experiments we used soils commonly found on the territory of Belarus – sands of medium density (γ = 16.5-16.7 kN / m^3, W = 0.7%, Pdmin = 1.5 MPa) and lean clay (γ = 19.1-19.2 kN/m^3, W = 12.7-13.0%, Pdmin = 3.5 MPa) (further in the article – clay).

3.2. The description of the experiment

Our research was carried out in the laboratories of the department of Geotechnics and Ecology in Construction of BNTU, using two methods:

– pressuremeter expansion,

– extension by injection of cement mortar.

3.3. The description of cement mortar injection expansion method

Cement mortar injection expansion experiment was carried out in cylindrical tanks with a diameter of 750 mm and a height of 0.9 meters (Fig. 1). The excess moisture from the injected cement slurry was released into the soil during experiment. The body of the well was formed by a pipe with a diameter of 110 mm, around which the soil was laid and compacted.

A B

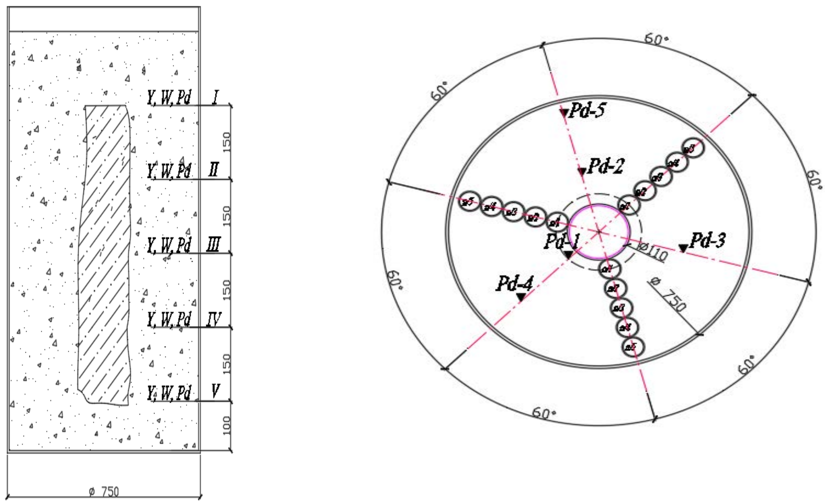

Fig. 1. The places of the selected soil samples (A – in height, B – in plan; I...V – sample place level; n/1...5 – sampling sites for determining soil moisture and density; Pd-1...5 – dynamic sounding locations). *Source:* own study

The well former was removed and injection of cement mortar was pumped under the pressure of 150 kPa, after placing the soil into the cylindrical tanks. M 400 cement grade was used in the experiments, the water-cement ratio of the poured solution equalled 0.5.

The initial diameter of the wells increased in sandy loam by 1.24-1.27 times, in sand by 1.19-1.23 times as the result of the injection.

After technological breaks lasting 7, 14, and 28 days, the excavation of injected bodies was carried out. A number of test samples were selected to determine the density and moisture content of the soil, and dynamic probing of the soil around the injection body and below it was also performed at 5 levels (with a step of 150 mm). In radial direction, soil density and moisture were determined with a step of 50 mm.

3.4. Pressiometer tests

The initial diameter of a borehole expanded by pressiometry methods was taken in the range from 130 to 140 mm based on the assumption that the soil properties around boreholes and cavities expanded by pressiometry method changes at a distance of up to 2.5-3 of borehole diameter.

The experiment was carried out in a metal tray of plan dimensions of 910 x 930 mm and the hight of 200 mm. The tray cover was made of transparent plexiglass. The rigidity of the tray cover and tray bottom was of a magnitude greater than that the soil, so the wells could expand only in radial direction with a corresponding change in soil properties.

The tray was filled with compacted soil by layers of 50 mm. Soil compaction was carried out by a dynamic density meter by dropping a weight of 30 kN from a height of 300 mm onto a stamp with a diameter of 100 mm. After that, a well was made in the centre of the tray and an expanding chamber was installed. Wire ground marks were dipped into the ground in radial directions in three quadrants of the tray at distances of 18-22 mm.

The wells formed in the soil mass expanded only in radial direction. The pressure created in the pneumatic chamber was up to 150 kPa. Prior to pressuremeter reaming of the well, measurements and binding of the locations of the marks were performed relative to the centre of the well, and the diameter between equidistant marks was also determined. After air was pumped into the chamber, the movement of the marks was determined by measuring their new location (Fig. 2). During the pressuremeter test, the initial radius of the well increased by 20-40%.

A *B*

Fig. 2. Carrying out laboratory pressuremeter tests, A) – location of the top cover and soil marks in sandy soil; B) – bursting cracks in the soil after an increase in the initial diameter of more than 40%. *Source:* own study

4. The results of laboratory tests

4.1. Pressiometer expansion

Based on the condition of constancy of the soil mass between two grades, the change in the density of the soil composition was determined.

Experimental processing of statistical data was performed in order for systematic errors and the measurement errors of the instruments and tools to be used as exception [3, 9]: the thesis was suggested that the relative deformation of each *i + 1st* layer should be less than that of the *i-th* layer; and also the total displacement of the borehole wall should be equal to the compression of all layers within the active deformable annular zone (Fig. 3).

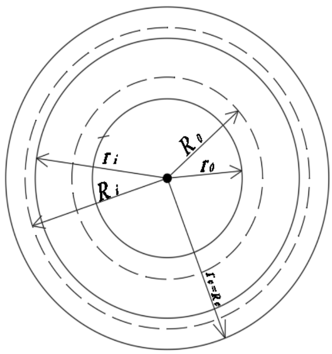

Fig. 3. Soil density change calculation scheme in the radial direction due to the well expansion (r_o is the initial radius of the well before expansion; R_0 is the radius of the well after its expansion; r_i is the radius up to the *i-th* mark before the well expansion; R_i – radius up to the *i-th* mark after the well expansion; r_e is the radius to the last mark before the well expansion; R_e is the radius to the last grade after the well expansion). *Source:* [4, 6]

Marks movement graphs were constructed relative to the zero point in sands and clays.

In this case, the condition must be observed that if

$$R_{i+1}-r_{i+1}<R_i-r_i \tag{1}$$

is taken into account, the amount of displacement should decrease with a growth of the distance from the borehole axis (Fig. 4).

The graph above shows that the boundaries were moving both in sandy and clay soils, which was accompanied by soil compaction in the volume of an elementary selected layer between the marks. The weight of the soil between adjacent marks *(M*) is a constant value, because within the corresponding i-th annular volumes of soil with the same heights *h*, only soil compression occurs. To compare the soils weight changes, the author proposed to use the soil compaction coefficient (K_{comp}). The soil compaction coefficient is the ratio of the weight of the soil after pressiometer expansion Y_i to the weight of the soil before expansion y_i and found the following expression:

$$K_{comp}=Y_i/y_i=\left(h\left(r_i^2-r_{i-1}^2\right)\cdot 3.14M\right)\Big/\left(h\left(R_i^2-R_{i-1}^2\right)\cdot 3.14M\right)=\left(r_i^2-r_{i-1}^2\right)\Big/\left(R_i^2-R_{i-1}^2\right) \tag{2}$$

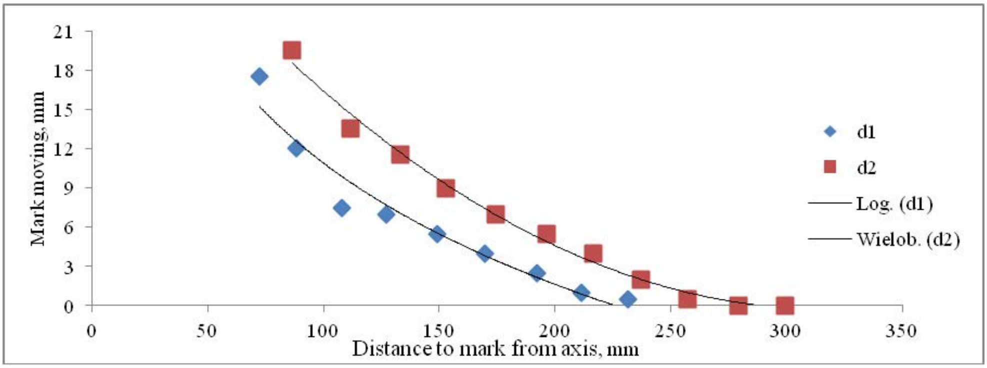

Fig. 4. Graphs of grades movement ΔR, mm with an initial radius of 80 mm. (d1 – sand; d2 – clay; Log(d1) – logarithmic trendline for sand data: wielob. (d2) – multinominal trendline for clay data). Source: own study

Based on the results of the experiments, it was revealed that the compaction coefficient K_{comp} takes values both greater than unity and less. When $K_{comp} > 1$, the soil is compacted; when $K_{comp} < 1$, the soil is decompacted (Fig. 5).

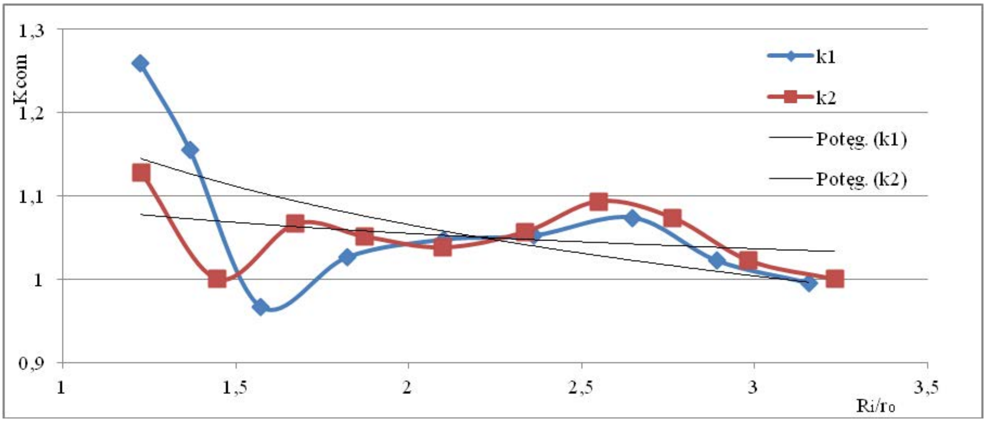

Fig. 5. The well pressiometer extension (k1 – sand compaction coefficient change; k2 – clay compaction coefficient change; Poteg (k1) – exponential trendline for sand compaction coefficient change; Poteg (k2) – exponential trendline for clay compaction coefficient change). *Source:* own study

The soil gravity changes graphs show the presence of zones of ground decompaction during pressiometer extension at a distance of 1.4-1,8 r_0 from the well wall. Such deconsolidation indicates the possibility of soil displacement at a similar distance from the side surfaces of the pile or anchor.

It should be noted that the increase in the initial diameter of the well r_0/R_0 more then 1.4 corresponds to the maximum density of soil composition and the minimum porosity coefficient. As a result of the performed laboratory experiments and statistical processing of the obtained data, the minimum possible soil porosity coefficients were determined, after which the processes of ruptures begin in the soils and contraction occurs. This coefficient is equal 0.33 for the medium sands and sandy loam.

4.2. Soil density change during cement mortar injection

The study of cement mortar injection to medium sand and sandy loam density changes was carried out during large-scale laboratory experiments. According to our research, the characteristics of the massif are constant and unchanged after 14 days from the injection of cement mortar into the sands, and after 28 days into clay.

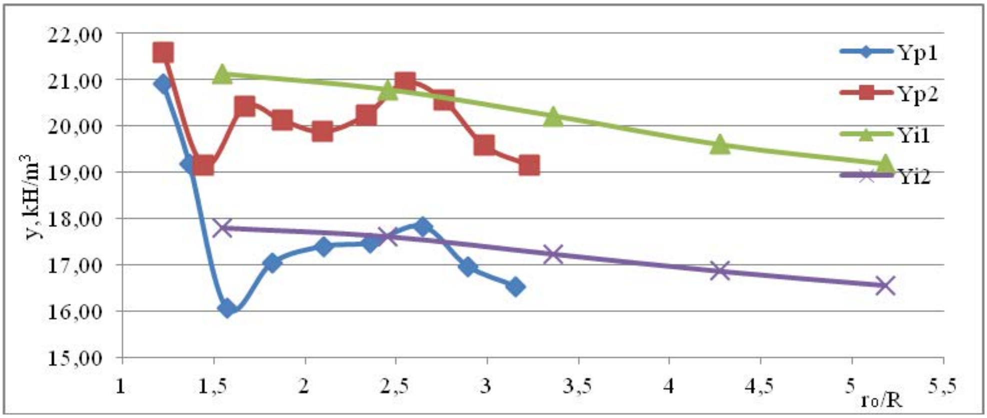

Fig. 6. Sand and sandy loam unit weight characteristics change in radial description (Yp1 – sand unit weight change after pressiometer extension; Yp2 – sandy loam unit weight change after pressiometer extension; Yi1 – sandy loam unit weight change on the 28th day after injection; Yi2 – sand unit weight change on the 14th day after injection). *Source:* own study

5. Conclusion

Based on the results of the laboratory experiment, it can be concluded that the soil area softening occurs during the pressiometer extension, and it also appears during cement mortar injection. The presence of softening zones can explain the occurrence of anchor and pile failure, also through the contact of the concrete body and the soil directly and through the softened soil. Furthermore, the pressiometer extension changes the unit weight of the sand in diapasons of 16.5-21.0 kH/m^3, and sandy loam 19.0-21.7 kH/m^3. The minimum porosity coefficients for these grounds equals 0.33.

According to our borehole injection expansion experiment, it was determined that the increase in the initial diameter of the well r_0/R_0 was more than 1.4, which corresponds to the maximum density of soil composition and the minimum porosity coefficient. After an increase in the initial diameter of more than 40%, discontinuous processes begin in soils.

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Local vertical compressive stress in the crane runway beam web

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Abstract: In this paper, the authors analysed several variants of connections between a block rail (60 mm × 60 mm) and a crane runway beam (IKS 800-6). They compared local vertical compressive stress in the crane runway beam web, calculated using an analytical approach and numerical simulations. In the case of the continuous block rail rigidly fixed to the beam flange, satisfactory convergence was obtained. For the remaining types of connections the results based on the analytical method were different from the results of the numerical simulations. The difference resulted from the fact that the analytical method did not take into account the crane rail joint. Furthermore, the impact of the elastomeric bearing pad on the local stress value was taken into account in a simplified manner in the analytical method by increasing the effective length by approximately 30%. The local vertical compressive stress in the crane runway beam web was significantly affected by the connection between the rail and the crane runway beam, the crane rail joint type, the use of the elastomeric bearing pad, the length of the elastomeric bearing pad, and the crane rail wear.

Keywords: crane runway beam, local stress, crane rail, steel structures

1. Introduction

Crane actions are separated into vertical and horizontal in the standard [1]. They induce dynamic and cyclic loadings [2]. In the designing process, the dynamic effects of the lifting of the weight, the crane movement and the trolley movement are taken into account using dynamic factors [3]. Vertical actions are caused by the self-weight of the crane and the hoist load. Horizontal crane actions are caused by acceleration or deceleration, skewing or other dynamic effects. The vertical loads of the crane runway beams result in global stress because of the bending around the y axis and torsion. The horizontal loads result in global stress because of the bending around the z axis and torsion. The vertical loads cause not only global stress but also local stress. The local vertical compressive stress is generated in the web of the crane runway beams. The global stress is superposed with the local stress in the beam web using Eq. (1) [4]–[6]:

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2+\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2-\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)+3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2\leq 1.0 \tag{1}$$

where: f_y – the yield strength, γ_{M0} – the partial factor, $\sigma_{x,Ed}$ – the design value of the longitudinal stress, $\sigma_{z,Ed}$ – the design value of the transverse stress, τ_{Ed} – the design value of the shear stress.

The local vertical compressive stress in the web $\sigma_{oz,Ed}$ is used in Eq. (1) as transverse stress. Due to the cyclic nature of the crane actions, it also is necessary to carry out fatigue assessments, taking into account the local vertical compressive stress in the case of crane runway beams [7]–[9].

The local vertical compressive stress in the web $\sigma_{oz,Ed}$ is calculated using the following equation from the EN 1993-6 standard [10]:

$$\sigma_{oz,Ed}=\frac{F_{z,Ed}}{l_{eff}t_w} \tag{2}$$

where: $F_{z,Ed}$ – the design value of the wheel load, t_w – the web thickness, l_{eff} – the effective loaded length.

The value of the local vertical compressive stress depends on the distance below the underside of the top flange:

$$\sigma_{oz,Ed}=\frac{F_{z,Ed}}{l_{eff}t_w}\left(1-\frac{2z}{h_w}\right) \tag{3}$$

where: z – the distance below the underside of the top flange, h_w – the overall depth of the web.

The real distribution of the local vertical compressive stress in the web is presented in Fig. 1a. In the standard [10] a constant stress distribution was assumed over the effective length (Fig. 1b).

The effective loaded length may be determined using the following equations from the EN 1993-6 standard [10]:

- for a crane rail rigidly fixed to the beam flange:

$$l_{eff}=3.25\left[I_{rf}/t_w\right]^{1/3} \tag{4}$$

- for a crane rail not rigidly fixed to the beam flange:

$$l_{eff} = 3.25\left[\left(I_r + I_{f,eff}\right)/t_w\right]^{1/3} \tag{5}$$

- for a crane rail mounted on a suitable resilient elastomeric bearing pad at least 6 mm thick:

$$l_{eff} = 4.25\left[\left(I_r + I_{f,eff}\right)/t_w\right]^{1/3} \tag{6}$$

where: I_{rf} – the second moment of area about its horizontal axis, of the combined cross-section comprising the rail and the beam flange with an effective width, I_r – the second moment of area about its horizontal axis of the rail, $I_{f,eff}$ – the second moment of area about its horizontal axis of a flange with an effective width.

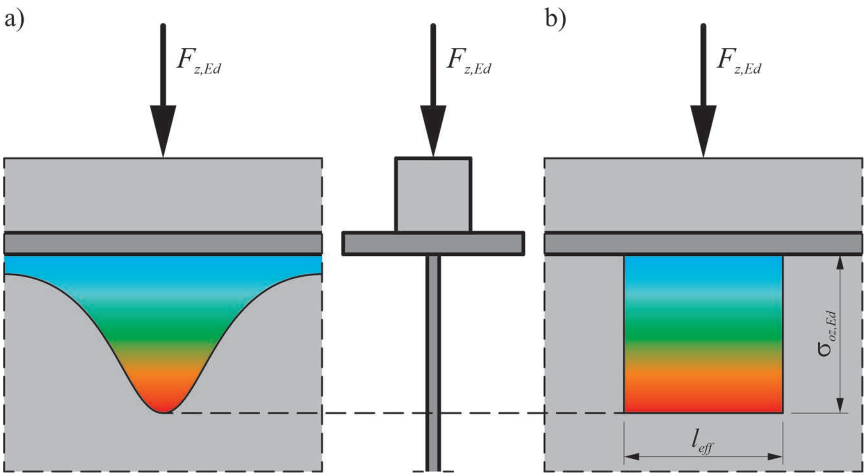


Fig. 1. Local vertical compressive stress in the web: a) real distribution, b) constant stress distribution assumed over the effective length

The effective width of a flange is calculated as [10]:

$$b_{eff} = b_{fr} + h_r + t_f \text{ but } b_{eff} \le b \tag{7}$$

where: b – the overall width of the top flange, b_{fr} – the width of the foot of the rail, h_r – the height of the rail, t_f – the flange thickness.

As outlined above, the effective loaded length depends on the type of the connection between a crane rail and a beam flange (rigid or flexible). Furthermore, the effective length may be increased when the elastomeric bearing pad is used under the rail. According to the

EN 1993-6 standard [10], the effective length for a crane rail mounted on a suitable resilient elastomeric bearing pad is 1.3 times higher than the one for a crane rail mounted without the elastic underlay. The elastomeric bearing pad provides for a smoother stress distribution in the beam web [11]–[13]. Mass [14] and Marcinczak [15] observed a significant decrease in the local stress value when the elastic underlay was applied. What is more, the value of the effective length and consequently the value of the local stress depends on the crane rail condition. Kurzawa et al. [16] observed the increase of the local vertical compressive stress in the crane runway beam web resulting from the crane rail wear. According to the EN 1993-6 standard [10] and papers [17]–[19], the wear of a rail should already be taken into account in the design process by reducing the head of the rail by 25%. Moreover, Chybiński et al. [20] demonstrated that the value of the local vertical compressive stress depends on the crane rail type. Rykaluk et al. [21] demonstrated that the value of the local vertical compressive stress depends on the shape of the crane rail splice. Last but not least, in his laboratory tests Kurzawa observed that the deformation of an elastomeric bearing pad subjected to vertical load depends on the friction force between the crane rail and the beam flange [11]. During the operation of the crane rails, oiling may occur on the contact surfaces. Kurzawa investigated the impact of oiling on the deformation of the elastomeric bearing pad. The behaviour of oiled elastomeric bearing pads was compared with that of dry bearing pads. Different elastomeric bearing pad thicknesses and types were taken into account, i.e., neoprene elastomeric bearing pads, polyurethane elastomeric bearing pads with and without steel plates, timber pads saturated under pressure with polyurethane. As a result of these tests Kurzawa obtained bearing stress–relative vertical deformation curves. The relative values of the vertical deformation were obtained by dividing the deformation by the thickness of the pad. Figure 2 presents the curve for the 15 mm neoprene elastomeric bearing pad. The deformation of the oiled elastomeric bearing pads was higher than the one of dry pads. The increase of the deformation due to oiling may have an impact on the stress in the crane rails and in the web of the crane runway beam. For this reason, it should be analysed in future tests.

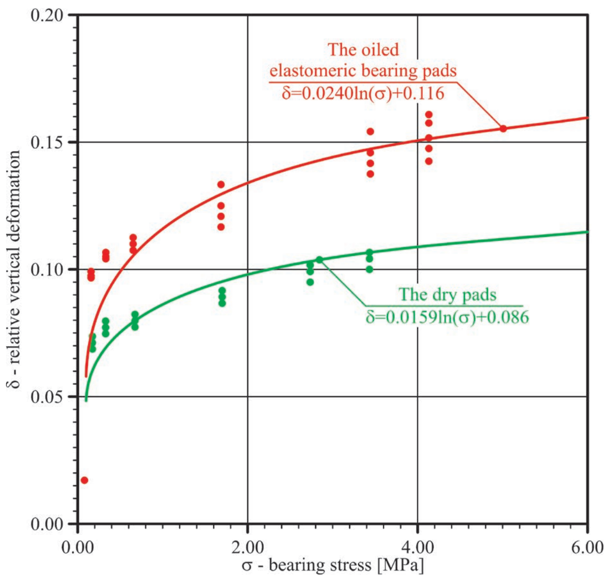


Fig. 2. Bearing stress–relative vertical deformation curves for oiled and dry 15 mm neoprene elastomeric bearing pads [11]

2. The aim of the current study

In this study a block rail (60 mm × 60 mm) connected with a crane runway beam (IKS 800-6) was analysed in several variants (Figs 3, 4).

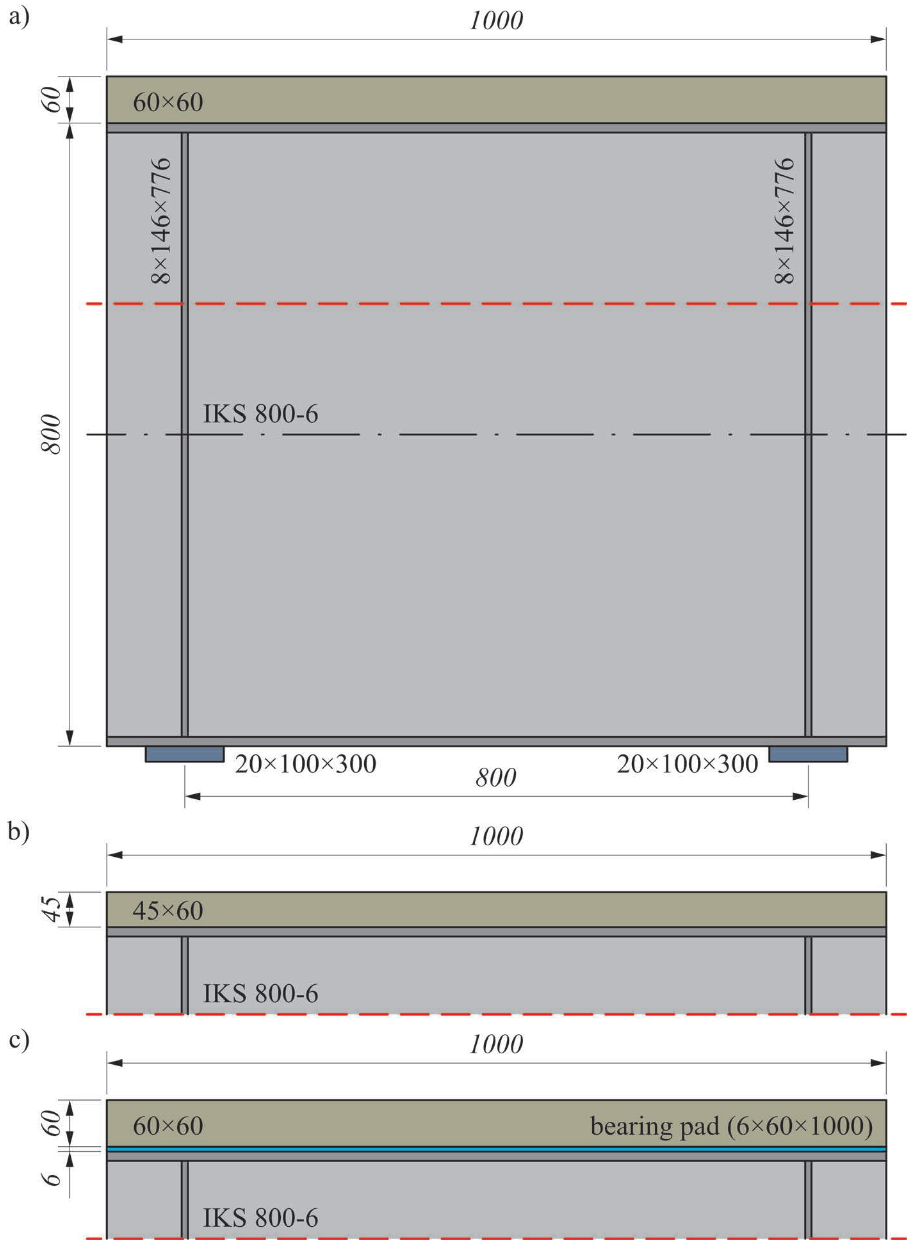


Fig. 3. Variants analysed in this study: a) the block rail rigidly (analysis 1) or flexibly fixed (analysis 3) to the beam flange; b) the reduced block rail rigidly (analysis 2) or flexibly fixed (analysis 4) to the beam flange; c) the block rail mounted on a 6 mm × 60 mm × 1000 mm elastomeric bearing pad (analysis 5)

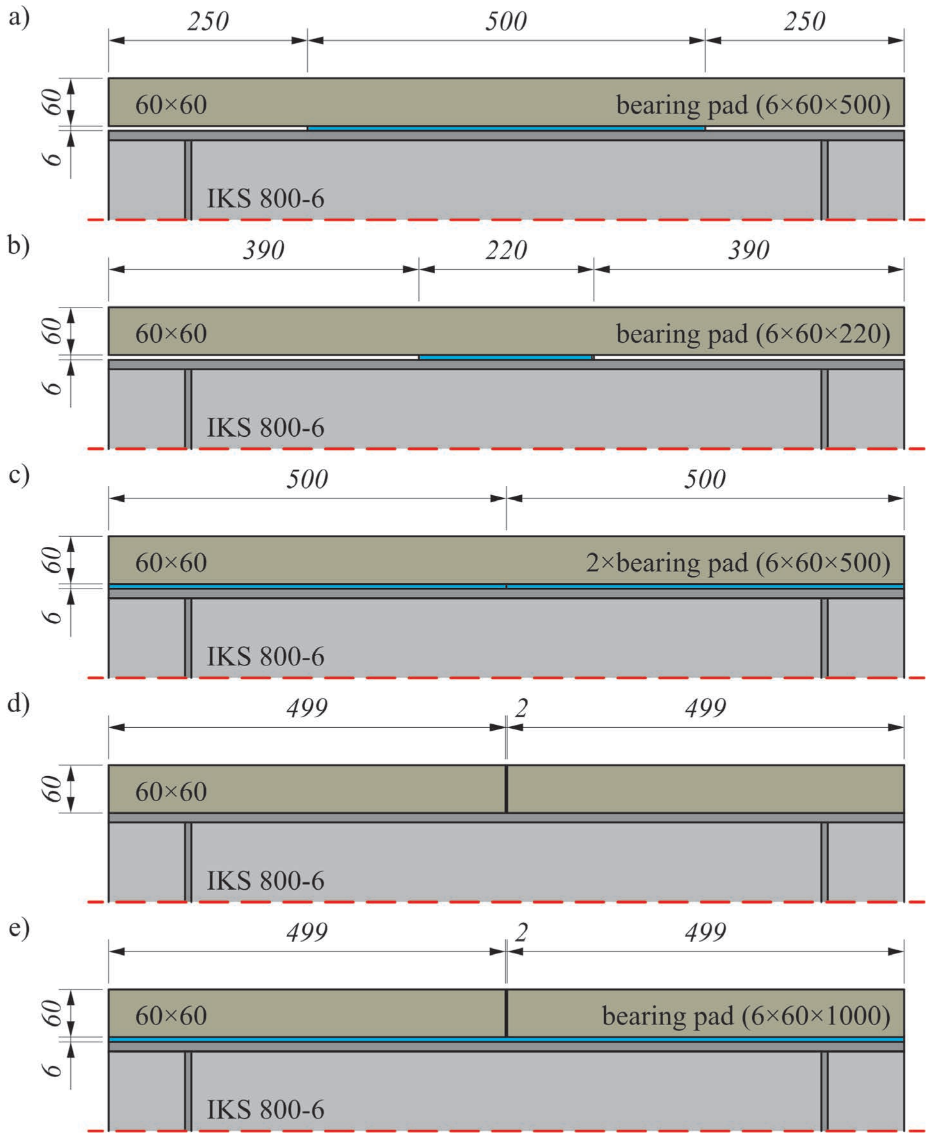


Fig. 4. Variants analysed in this study: a) the block rail mounted on a 6 mm × 60 mm × 500 mm elastomeric bearing pad (analysis 6); b) the block rail mounted on a 6 mm × 60 mm × 220 mm elastomeric bearing pad (analysis 7); c) the block rail mounted on two elastomeric bearing pads (analysis 8); d) the block rail with a 2 mm gap rigidly (analysis 9) or flexibly fixed (analysis 10) to the beam flange; e) the block rail with a 2 mm gap mounted on an elastomeric bearing pad (analysis 11)

In the case of the crane rail joint, two types were investigated, i.e., a continuous crane rail (Figs 3a–c and 4a–c) and a crane rail with a 2 mm gap (Figs 4d and 4e), to evaluate the impact of the crane rail joint type. The gap should not be larger than 2 mm [22], [23]. The crane rail joint may have different designs. Rykaluk et al. analysed three different types of crane rail joints, i.e., orthogonal contact, bevel contact, and stepped bevel contact [21]. The lowest value of the local vertical compressive stress in the web was observed for the stepped bevel contact, while the highest value of the vertical compressive stress in the web was observed for the orthogonal contact. For this reason, the orthogonal contact, which provided for the highest stress value, was analysed in this paper.

In the case of the connection between the crane rail and the beam flange, three variants were studied. In the first variant, the crane rail was rigidly fixed to the beam flange (Figs 3a, 3b and 4d). In the second variant, the crane rail was flexibly fixed to the beam flange (Figs 3a, 3b and 4d), and in the third variant, the crane rail was mounted on a 6 mm elastomeric bearing pad (Figs 3c, 4a–c, 4e).

In the case of the elastomeric bearing pad, four variants were investigated. In the first one, a continuous 1-m-long elastomeric bearing pad was used (Figs 3c and 4e). In the second variant, a continuous 0.5-m-long elastomeric bearing pad was used (Fig. 4a). In the third variant, a continuous 0.22-m-long elastomeric bearing pad was used (Fig. 4b). In the fourth variant, two 0.5-m-long continuous elastomeric bearing pads were used with the joint located in the middle of the beam (Fig. 4c).

In the case of the rail wear, two variants were analysed. In the first variant the crane rail was not worn (Figs 3a, c and 4a–e). In the second variant the crane rail wear was taken into account by reducing the head of the rail by 25% (Fig. 3b).

3. Analytical approach

The local vertical compressive stress in the web $\sigma_{oz,Ed}$ was calculated using Eq. (2). The results of these calculations are presented in Table 1.

Table 1. The local vertical compressive stress in the web $\sigma_{oz,Ed}$ based on the standard [10]

No.	Crane rail	Connection	Elastomeric bearing pad	η [%]	t_r [mm]	h_r [mm]	b_{eff} [mm]	I [cm^4]	l_{eff} [cm]	$\sigma_{oz,Ed}$ [MPa]
1	*c*	*r*	–	0.0	60.0	60.0	132.0	252.5	22.1	56.5
2	*c*	*r*	–	25.0	45.0	45.0	117.0	122.3	17.4	71.9
3	*c*	*f*	–	0.0	60.0	60.0	132.0	108.0	16.8	74.5
4	*c*	*f*	–	25.0	45.0	45.0	117.0	45.6	12.7	98.8
5	*c*	*f*	6 × 60 × 1000	0.0	60.0	60.0	132.0	108.0	21.9	57.0
6	*c*	*f*	6 × 60 × 500	0.0	60.0	60.0	132.0	108.0	21.9	57.0
7	*c*	*f*	6 × 60 × 220	0.0	60.0	60.0	132.0	108.0	21.9	57.0
8	*c*	*f*	6 × 60 × 500 (× 2)	0.0	60.0	60.0	132.0	108.0	21.9	57.0
9	*g*	*r*	–	0.0	60.0	60.0	132.0	252.5	22.1	56.5
10	*g*	*f*	–	0.0	60.0	60.0	132.0	108.0	16.8	74.5
11	*g*	*f*	6 × 60 × 1000	0.0	60.0	60.0	132.0	108.0	21.9	57.0

c – continuous; *g* – with a 2 mm gap; *r* – rigid; *f* – flexible; η – rail head reduction; t_r – head height after reduction; h_r – rail height after reduction; b_{eff} – effective width of the top flange; I – second moment of area; l_{eff} – effective loaded length; $\sigma_{oz,Ed}$ – local vertical compressive stress in the web

4. Numerical analyses

Non-linear finite element models of the crane runway beam were developed in the Abaqus program [24]. The calculations were performed using the Newton-Raphson method. Each numerical model consisted of a crane runway beam, a crane rail and two support plates. In some models a suitable resilient elastomeric bearing pad was applied under the crane rail. Each model was 1.0 m long. The material behaviour of steel was modelled as bi-linear elastic-plastic with strain-hardening. The crane runway beam was made of S235 steel, and the crane rail and the support plates were made of S355 steel (Table 2). The material behaviour of the elastic underlay was modelled as elastic (Young's modulus E = 7.2 MPa, Poisson's ratio v = 0.45).

Table 2. Steel parameters [4]

Steel grade	Yield strength f_y [MPa]	Ultimate tensile strength f_u [MPa]	Young's modulus E [GPa]	Poisson's ratio v [–]
S235	235	360	210	0.3
S355	355	490	210	0.3

The wheel loading of 100 kN was introduced fully centrically. The vertical load was spread out over a 50-mm-wide area based on two European standards [25], [26]. In the case of the support plates and the runway beam contact, surface-to-surface contact was defined between the support plates and the beam flange. "Hard" contact was modelled in the normal direction and friction was modelled in the tangential direction. The friction coefficient between steel–steel elements was equal to 0.3. Depending on the model, three types of rail connection were modelled. In the case of the crane rail rigidly fixed to the beam flange, the rigid connection was modelled using the tie function. A surface-based tie constraint tied the crane rail and the beam flange together for the duration of the numerical simulations [24], [27]. In the case of the crane rail not rigidly fixed to the beam flange, surface-to-surface contact was defined between the crane rail and the beam flange. "Hard" contact was modelled in the normal direction and friction was modelled in the tangential direction. The friction coefficient between steel–steel elements was equal to 0.3. In the case of the crane rail mounted on a suitable resilient elastomeric bearing pad of 6 mm, surface-to-surface contact was defined between the crane rail and the bearing pad as well as between the bearing pad and the beam flange. "Hard" contact was modelled in the normal direction and friction was modelled in the tangential direction. The friction coefficient between steel–elastomeric bearing elements was equal to 0.3. The crane rail, the beam flanges and the elastomeric bearing pad were divided into eight-node cuboidal finite solid elements (C3D8R) and the crane runway beam stiffeners were divided into four-node shell elements (S4R). The maximum mesh size for these elements was 10 mm. The crane runway beam web was divided into four-node shell elements (S4R) of the maximum mesh size of 5 mm.

The chosen mesh size based on the analyses of the sensitivity of the numerical model to the mesh size presented in the literature [20, 21]. Chybiński et al. analysed two mesh sizes (5 mm and 10 mm) [20]. The local vertical compressive stress value in the web for the mesh size of 5 mm was 2.06% higher than for the mesh size of 10 mm. Rykaluk et al. presented that the difference in the local vertical compressive stress value in the web between the mesh size of 2 mm and 5 mm was only 0.53% [21]. They selected the mesh size of 5 mm for the numerical analyses. For this reason, in this paper, the mesh size of 5 mm was chosen for the web in the numerical simulations.

The shell web of the crane runway beam was connected with the solid flanges and the shell stiffeners using the tie function. Figure 5 presents the mesh and the boundary conditions used in the model (fixed displacements and rotations). The numerical analyses conducted in this study are presented in Table 3.

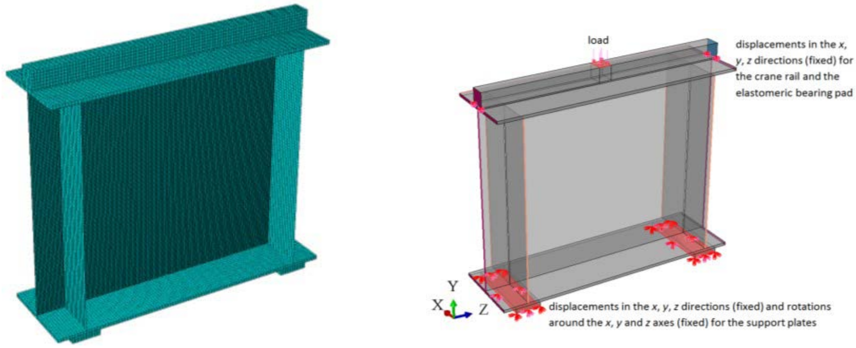


Fig. 5. The mesh and the boundary conditions used in the numerical model

Table 3. Numerical analyses

No.	Crane rail	Connection	Elastomeric bearing pad	η [%]	t_r [mm]	h_r [mm]
1	*c*	*r*	–	0.0	60.0	60.0
2	*c*	*r*	–	25.0	45.0	45.0
3	*c*	*f*	–	0.0	60.0	60.0
4	*c*	*f*	–	25.0	45.0	45.0
5	*c*	*f*	6 × 60 × 1000	0.0	60.0	60.0
6	*c*	*f*	6 × 60 × 500	0.0	60.0	60.0
7	*c*	*f*	6 × 60 × 220	0.0	60.0	60.0
8	*c*	*f*	6 × 60 × 500 (× 2)	0.0	60.0	60.0
9	*g*	*r*	–	0.0	60.0	60.0
10	*g*	*f*	–	0.0	60.0	60.0
11	*g*	*f*	6 × 60 × 1000	0.0	60.0	60.0

c – continuous; *g* – with a 2 mm gap; *r* – rigid; *f* – flexible; η – rail head reduction; t_r – head height after reduction; h_r – rail height after reduction; b_{eff} – effective width of the top flange; I – second moment of area; l_{eff} – effective loaded length; $\sigma_{oz,Ed}$ – local vertical compressive stress in the web

5. The results

The results of the analytical approach were compared with the results of the computer simulations in Table 4.

In the case of the continuous block rail rigidly fixed to the beam flange (analyses 1 and 2), satisfactory convergence of the local vertical compressive stress in the crane runway beam web was obtained. The stress values calculated using the analytical method in accordance with the standard [10] were only 3.7% (analysis 1) or (8.2%) (analysis 2) higher than the ones obtained from the numerical simulation. For the remaining analyses the stress obtained from the computer simulation was different from the one based on the analytical approach.

In the case of the continuous block rail flexibly fixed to the beam flange (analyses 3 and 4), the values of the local vertical compressive stress in the crane runway beam web calculated from the standard [10] were 21.9% (analysis 3) or 26.7% (analysis 4) higher than the ones obtained from the computer simulation. The numerical calculations underestimated the maximum stress. This fact calls for further investigation and it should be verified in laboratory tests. It is worth emphasizing that the numerical calculations were only conducted for centric loading. What is more, the maximum stress in the numerical simulation increased significantly when the non-continuous block rail was used (analysis 10).

Table 4. The results of the analytical approach and computer simulations

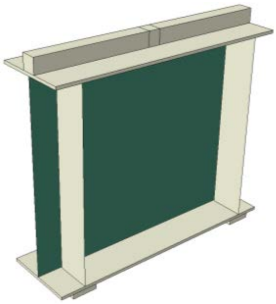
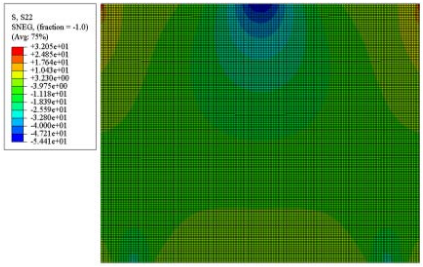
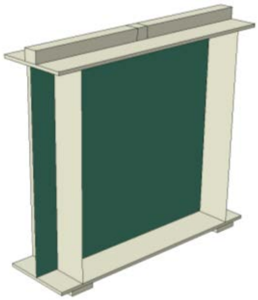
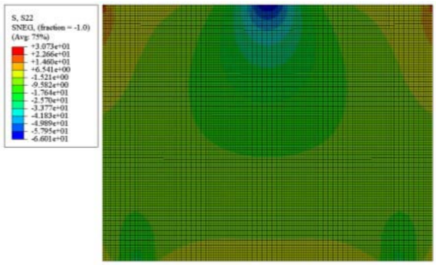
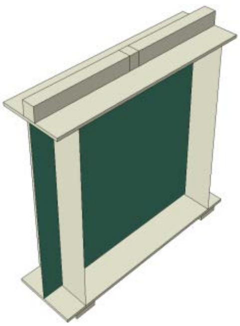
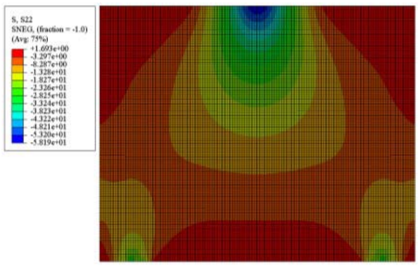
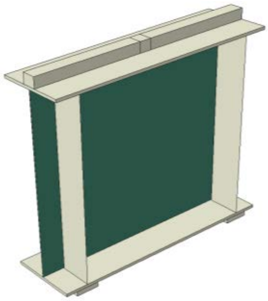
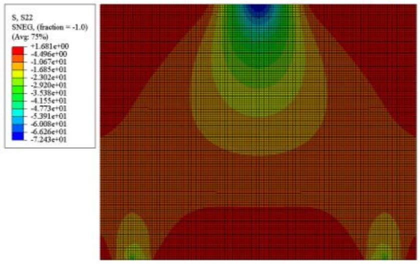
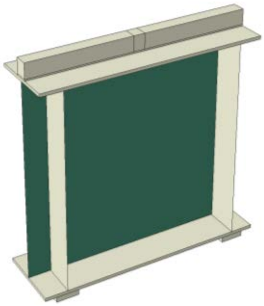
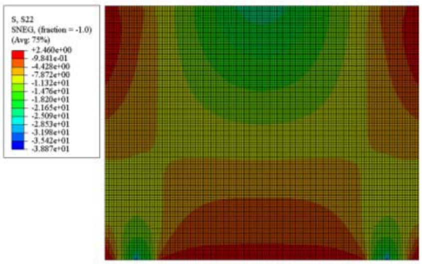
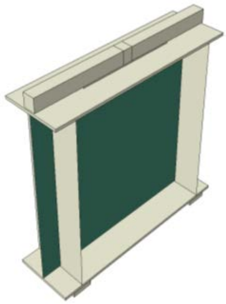
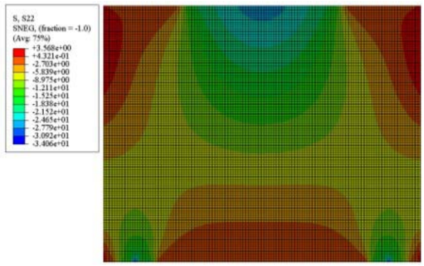
No.	Model	Stress distribution	$\sigma_{oz,Ed,na}$ [MPa]	$\sigma_{oz,Ed,s}$ [MPa]	μ [%]
1			54.4	56.5	–3.7
2			66.0	71.9	–8.2
3			58.2	74.5	–21.9

Table 4. The results of the analytical approach and computer simulations, continued

No.	Model	Stress distribution		
4				
		$\sigma_{oz,Ed,na}$ [MPa]	$\sigma_{oz,Ed,s}$ [MPa]	μ [%]
		72.4	98.8	–26.7
5				
		$\sigma_{oz,Ed,na}$ [MPa]	$\sigma_{oz,Ed,s}$ [MPa]	μ [%]
		23.4	57.0	–58.9
6				
		$\sigma_{oz,Ed,na}$ [MPa]	$\sigma_{oz,Ed,s}$ [MPa]	μ [%]
		26.2	57.0	–54.0

In the case of the continuous block rail mounted on a 6 mm × 60 mm × 1000 mm elastomeric bearing pad (analysis 5), the local vertical compressive stress in the crane runway beam web calculated from the standard [10] was 58.9% higher than the one obtained from the computer simulation. A similar difference (43.3%) between the results of the analytical approach and the numerical simulation was obtained by Marcinczak [15]. In accordance with the standard [10], the effective length based on Eqs. (5) and (6) can be increased 1.3 times if an elastomeric bearing pad with a minimum thickness of 6 mm is used [28]. The maximum stress in the numerical simulation decreased 2.5 times when an underlay was used (compare analyses 3 and 5). This observation is optimistic and it calls for further investigation in laboratory tests. Nevertheless, it should be noted that the numerical calculations were only conducted for centric loading. Furthermore, the maximum stress increased significantly in the numerical simulation when the non-continuous block rail was used (analysis 11) or when a shorter underlay was used (analysis 7). The non-continuous underlay investigated in analysis 8 did not have any impact on the web stress.

Table 4. The results of the analytical approach and computer simulations, continued

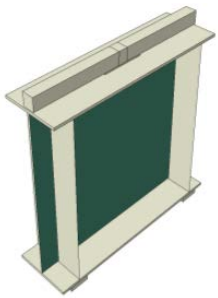
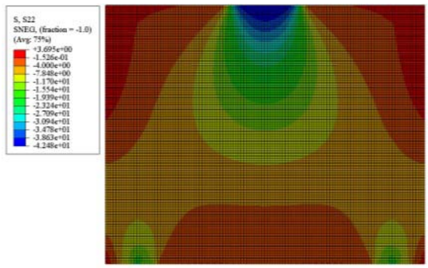
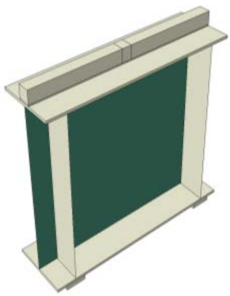
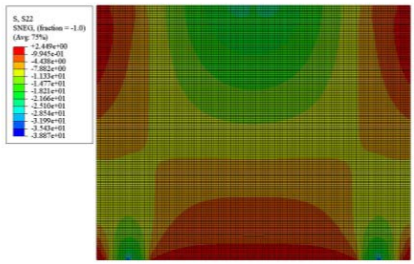
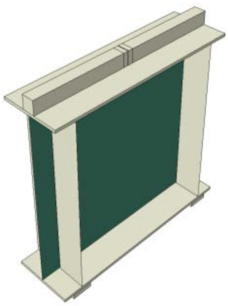
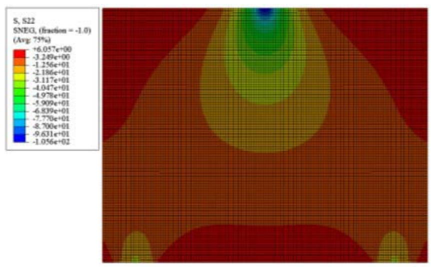
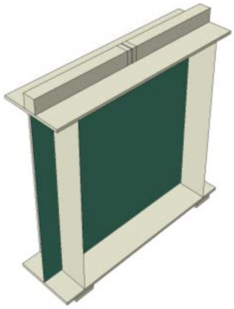
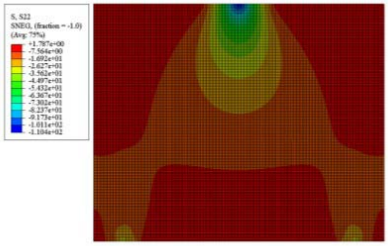
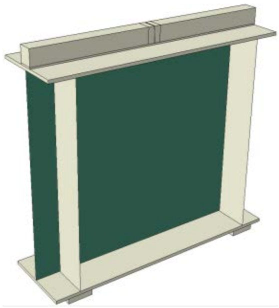
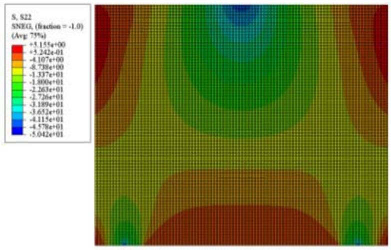
<table>
<tr><th>No.</th><th>Model</th><th colspan="3">Stress distribution</th></tr>
<tr><td rowspan="3">7</td><td rowspan="3"></td><td colspan="3">
</td></tr>
<tr><td>$\sigma_{oz,Ed,na}$ [MPa]</td><td>$\sigma_{oz,Ed,s}$ [MPa]</td><td>μ [%]</td></tr>
<tr><td>42.5</td><td>57.0</td><td>–25.4</td></tr>
<tr><td rowspan="3">8</td><td rowspan="3"></td><td colspan="3">
</td></tr>
<tr><td>$\sigma_{oz,Ed,na}$ [MPa]</td><td>$\sigma_{oz,Ed,s}$ [MPa]</td><td>μ [%]</td></tr>
<tr><td>23.0</td><td>57.0</td><td>–59.6</td></tr>
<tr><td rowspan="3">9</td><td rowspan="3"></td><td colspan="3">
</td></tr>
<tr><td>$\sigma_{oz,Ed,na}$ [MPa]</td><td>$\sigma_{oz,Ed,s}$ [MPa]</td><td>μ [%]</td></tr>
<tr><td>105.4</td><td>56.5</td><td>86.5</td></tr>
</table>

Table 4. The results of the analytical approach and computer simulations, continued

No.	Model	Stress distribution		
10				
		$\sigma_{\text{oz,Ed,na}}$ [MPa]	$\sigma_{\text{oz,Ed,s}}$ [MPa]	μ [%]
		110.4	74.5	48.2
11				
		$\sigma_{\text{oz,Ed,na}}$ [MPa]	$\sigma_{\text{oz,Ed,s}}$ [MPa]	μ [%]
		42.7	57.0	–25.1

$\sigma_{\text{oz,Ed,na}}$ – local vertical compressive stress in the crane runway beam web from numerical the analysis;
$\sigma_{\text{oz,Ed,s}}$ – local vertical compressive stress in the crane runway beam web based on the standard [10],

$$\mu = \frac{\sigma_{oz,Ed,na} - \sigma_{oz,Ed,s}}{\sigma_{oz,Ed,s}} \cdot 10^2$$

6. Conclusions

In this paper, the authors focused on the local vertical compressive stress in the crane runway beam web. The stress was calculated using an analytical approach and numerical simulations. The connection between the block rail (60 mm × 60 mm) and the crane runway beam (IKS 800-6) was analysed in several variants. The study presented in this paper has certain limitations as within it, only the centric load position was investigated. What is more, no laboratory tests were performed on the crane runway beams. The results obtained in the numerical analyses were only compared with the results calculated using the analytical approach. It would be advisable to perform complementary laboratory tests to validate numerical models in the future. For this reason, the authors plan to conduct laboratory tests of crane runway beams.

The main conclusions, based on the calculations and the numerical analyses, are as follows:

- In the case of the continuous block rail rigidly fixed to the beam flange, the local vertical compressive stress in the crane runway beam web obtained from numerical simulations was in agreement with the results based on the standard [10].

- In the case of the continuous block rail flexibly fixed to the beam flange, the continuous block rail mounted on a 6 mm elastomeric bearing pad, and the non-continuous block rail, satisfactory convergence was not obtained. This fact calls for further investigation and it should be verified in laboratory tests.
- The crane rail joint had a significant impact on the web stress. For example, when the continuous crane rail was used (analysis 1) the stress was 1.94 times lower than when the crane rail with a 2 mm gap was used (analysis 9). In this paper the authors analysed the non-continuous crane rail with the worse orthogonal contact and the maximum possible gap (2 mm). The rules presented in the standard [10] do not take into account the crane rail joint. The standard underestimates the local value of the web stress. This fact is very important for designing crane runway beams, where crane rail joints are often used. The local web stress is added to the global stress due to bi-axial bending. A realistic evaluation of this local stress in the web is very important for verifying if the crane runway beams meet the ultimate limit state requirements of Eurocode 3 and for making accurate predictions of the fatigue life of the crane runway beams. For this reason, the standard [10] should be supplemented with the formulas for calculating the web stress also in the case of the non-continuous crane rails. What is more, a transverse stiffener should be used in the beam web under the rail joint, and stepped bevel contact should be used instead of orthogonal contact [21].
- The use of the a 6 mm elastomeric bearing pad provided for a significant reduction of the local stress value. For example, when comparing analyses 3 and 5 it can be seen that the maximum stress in the numerical simulation decreased 2.5 times when the underlay was used. However, the use of elastomeric bearing pads also has negative effects, as it provides for the increase of the deflection and tensile stress in crane rails [12], [14], [15].
- The length of the elastomeric bearing pad had an impact on the local stress value. This fact is of limited importance for designing crane runway beams, because in general continuous elastomeric bearing pads mounted under the rails are used.
- Last but not the least, the crane rail wear had a significant impact on the web stress. For this reason, a reduced rail head (by 25%) should be taken into account during the design process.

The results of this study indicate that further investigations, including laboratory tests, are necessary. Therefore, the authors plan to test real crane runway beams and study the effects of elastomeric bearing pads on the local vertical compressive stress in the crane runway beam web.

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Orginal Article

Methods of protection and the state of preservation of the wall topping of Gothic brick castles

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Abstract: This article is an analysis of methods of securing historic walls in objects in the form of so-called permanent ruins. The research group consists of the brick Gothic castles of Mazovia in Ciechanów, Czersk, Liw and Sochaczew. Based on the analysis of documentation and our research, the effectiveness of the solutions applied in 4 different technological variants was evaluated. The state of preservation of the introduced solutions after several years of work was also assessed. The entire study was summarised in a table and conclusions were drawn.

Keywords: historical ruin, castle, castle ruins, wall topping degradation, protection of wall topping

1. Introduction

One of the most characteristic features of the structures and complexes left as so-called permanent ruins is the relatively large number of free-standing walls. These structures consist, in addition to historically free-standing walls, of walls from volumetric objects. Free-standing walls are most often devoid of weather protective elements. The lack of protection thus exposes them to accelerated degradation. These processes are particularly intense in originally volume masonry, which has a smaller thickness and was often erected with lower-quality materials. This is due to the fact that buildings of this type did not always have a defensive function.

The greatest damage is observed on the crown of the masonry, i.e. the part that is most highly exposed and vulnerable to damage. The main factors responsible for the degradation of masonry crowns, include the following:

– Climatic, caused by environmental factors related to the climate – precipitation, frost, insolation, the intensity of temperature change, wind.

– Chemical, related to the action of chemical compounds existing in the masonry and supplied from outside – leaching of substances from mortars and masonry material, decomposition of plants growing on the masonry, salt crystallization.
– Biotic, caused by the action of microorganisms and living organisms – algae, bryophytes, fungi moulds, lichens, grasses, perennials and succulents and trees and shrubs.
– Mechanical, caused by external and fatigue stresses, abrasion, and mechanical impacts.

The destructive factors listed above and the degradation processes which imply them practically never occur individually. The condition of masonry crowns in ruins is most often responsible for all or almost all of them.

2. Methodology and aim of the work

This article aims to analyse the protection of the wall topping at selected Gothic castles found in the Mazovian region. For this purpose, the following was performed:
– Study visits were carried out,
– Necessary photographic documentation was made,
– 3D scanning was carried out,
– An analysis of the applied safeguards was carried out,
– Assessments of the technical condition of the over-built walls topping and historic wall fragments were carried out.

All work of an inventory and research nature was carried out between 2019 and 2020.

3. Methods of protection and state of preservation of the wall toppings of Mazovian Gothic brick castles

Works to protect the wall topping are usually combined with the partial or complete reconstruction of the wall. The method of protecting the wall topping is chosen depending on the objectives of the conservation program. They differ in terms of durability, legibility and reversibility. The solution applied depends primarily on the type of masonry (its current form and state of preservation, the material from which it was built, the type of construction and the concept of conservation and architectural work proposed for the entire building).

There are two groups of methods for protecting masonry crowns. The first is to make a new layer on the historic wall. This layer is assumed to be a lost layer, i.e. it can deteriorate and should be cyclically restored. This group includes: rebuilding part of the masonry, overbuilding, protecting the crown with mortars or concrete and the technical-green method.

The other group of protective actions is characterised by covering, shielding the historic fabric from the effects of rainwater. This group can include various types of canopies, protection of the wall toppings with sheet metal and chemical coatings.

In all cases, more or less prior repair of the degraded historic wall is required.

The Table 1 shows only the solutions applied at selected Mazovian castles:

Table 1. Selected ways of protecting the wall topping of brickwork castles. *Source:* author

Protection of the wall topping with mortar or concrete

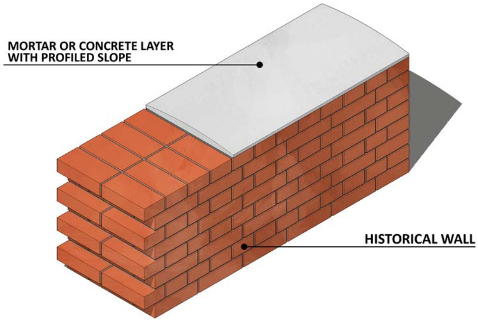

Fig. 1. Scheme for the protection of the cope of a brick wall with mortar or concrete

Construction of an airtight layer of mortar or concrete on the degraded wall topping. This method consists of making the protection directly on the historic masonry or on a layer of insulating material as a separating layer and allowing the reversibility of the solution applied. The historic wall to be protected must first be properly prepared for the application of the finishing layer.

Protection with a layer of mortar or concrete allows any slope to be shaped to allow rainwater to drain into or out of the building. The plasticity of the material makes it possible to form the crown according to the line formed by natural factors. The plasters used for protection should be as tight as possible to prevent them from penetrating the protection layer and, in the absence of insulation, from penetrating the historical layers of the masonry.

This method of protection for lower walls may be considered not entirely aesthetically pleasing.

Insertion of an insulation layer with addition of a new layer of native material

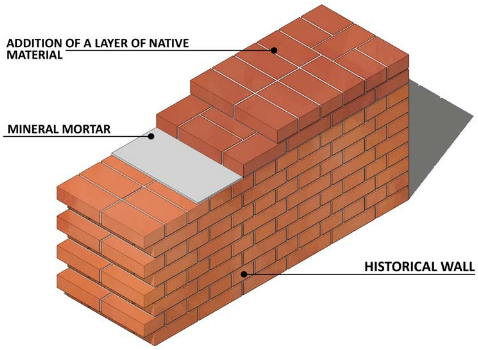

Fig. 2. Scheme for the protection of the cope of a brick wall by over-bricking with native (historicising) material.

Overbricking of the existing masonry with additional layers of material, either native or foreign, depending on the preservation program. In the case of the Mazovian castles discussed here, this was a lining with native material. As the basic and simplest way of securing the wall topping, this method is most frequently used. The upper part of the wall is filled in and appropriately shaped. Overbricking transfers the destructive action to the new material, but does not stop the wall topping deterioration process itself. Periodic inspection and replacement of the material is necessary.

For this solution, it is extremely important to select the strength of the masonry material and the strength of the joint. When there is too strong a mortar, the brick or stone will quickly crumble and deteriorate. On the other hand, a mortar which is too weak will lead to sections of the brickwork falling off and the wall topping deteriorating again.

Insertion of an insulation layer with addition of a new layer of native material

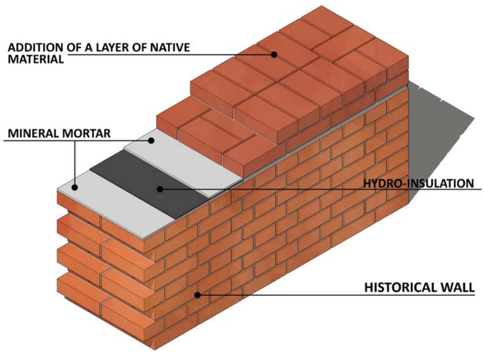

Fig. 3. Scheme to protect the wall topping of a brick wall by applying a layer of insulation and over-bricking with native (historicising) material

The insertion of an insulation layer in the form of roofing felt or metal sheeting is a cheap and quite effective way of protecting the wall topping. It is also a much better solution than to merely over-brick the wall topping. Insulation gives additional protection to the old masonry, provides a separating layer and makes the solution reversible. In contrast, brickwork provides a pressure layer for the insulation and is considered a lost layer that needs to be restored periodically.

Addition of a new layer to the wall topping with insulation and capping

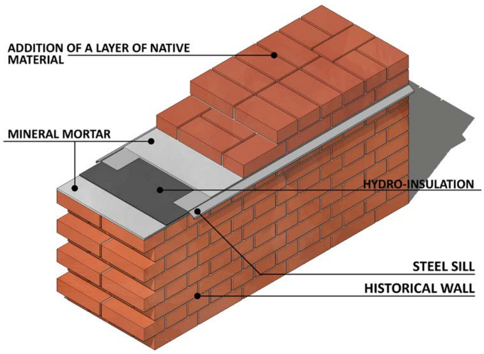

Fig. 4. Scheme for the protection of the wall topping of a brick wall by over-bricking with native material (historicising) with insulation and capping

A major problem in the case of masonry crowns protected by lintels in the form of so-called permanent ruins is the seepage of rainwater onto the faces of the masonry. The core of the masonry remains protected by the insulation layer, but the seeping water starts a series of processes which degrade the near-surface layers. The solution is to fix slightly visible drips, so that the water is drained away and does not cause the masonry to become damp. Steel or non-ferrous metal drip caps are used. Due to the separation of the lintel by a layer of insulation, the solution is reversible. The lintel should be treated as a lost layer.

4. Protection of wall toppings of Mazovian castles – analysis of selected objects

4.1. The castle of the Dukes of Mazovia in Ciechanów

The Castle of the Dukes of Mazovia in Ciechanów dates back to late 14th century. Built on a rectangular plan, the walls were made of ceramic brick on a stone foundation. The fortress was located on a flat area, which was a floodplain of the Łydynia river and surrounded by a wide and shallow moat. The defensive layout consisted of two round towers and curtain walls with blanketing. The castle was rebuilt several times and expanded with buildings inside its perimeter. Never conquered by enemies, the structure began to deteriorate as early as the end of the 15th century due to its loss of importance. The first restoration work took place in the second half of the 16th century. At the beginning of the 19th century, the demolition of the building was started by the Prussians. Since the 20th century, restoration and conservation work has been carried out on the ruins [1].

Today it is one of the best-preserved Polish Gothic brick castles. It is maintained as a so-called permanent ruin. In 2013, work to revitalise the castle was completed. Inside the castle perimeter, a new museum pavilion was erected in place of the so-called Little House, and a complex of sanitary facilities and utility rooms was located under the courtyard.

Ciechanów Castle is in overall good to very good technical condition. There are two types of protection of the wall toppings on the site. One curtain wall and two towers have wall toppings over-built with native material with a crenelation, without an insulating layer and a capinos. The other wall toppings of the perimeter walls are over-bricked with a slope towards the courtyard and a metal capstone installed under the top layer of bricks.

The wall toppings of the external walls protected by over-bricking with a crenelation without an insulating layer are in good technical condition. Damage to the crenelation, i.e. spalling of contemporary bricks, carbonate efflorescence on the surface, cavities in the joints and biological corrosion are present in places. At the Gothic basements, the crenelations are in varied technical condition. The wall toppings are in good condition where the walls have been restored. There is point damage to the bricks and loss of pointing. However, the wall toppings of the Gothic cellar walls are in a poor condition. There is complete degradation of the surface layers, numerous defects in whole sections of the topping, cracks and separations. This causes rainwater to penetrate deep into the masonry. There is algal, lichen and vegetation growth at the preserved surfaces.

Where the masonry has been protected by over-bricking with native material and the addition of a metal capping, the technical condition of the protection is good to very good. There is some sparse damage to the vertical and horizontal surfaces of the lintel with minor damage to the brickwork and joints.

Fig. 5. The wall topping of tower is in good technical condition. Locally visible damage to the top layer of bricks. Defects in pointing, damage to bricks. *Source:* author, 2019

Fig. 6. Tower wall topping. Severe salt corrosion of the brick face of the tower terrace exit. Salt corrosion damaged brick faces. Carbonate streaks on the surface of the masonry. *Source*: author*, 2019*

Fig. 7. The wall topping. Localised damage to the surface of the upper layer of bricks. The wall topping contaminated with bird droppings, spotty lichen growth. *Source*: author, 2019

Fig. 8. The wall topping. The wall topping of the wall on the courtyard side. The junction of the historic wall and the over-brickwork. Brick used and pointing to ensure the layers are distinguishable. Sheet metal dripline draining rainwater. *Source*: author, 2019

4.2. The castle of the Mazovian Dukes in Czersk

The Castle of the Mazovian Dukes in Czersk was built at the turn of the 14th and 15th centuries [2], on a Vistula escarpment, on an irregular polygonal plan. The fortress consisted of a square gate tower (eastern), two round towers (southern and western) and a defensive wall surrounding a large courtyard. The building material of the castle is brick on fieldstone foundations. The building was rebuilt several times. During the Swedish Deluge, the castle was conquered and ruined. In the second half of the 18th century, attempts were made to restore it, but at the turn of the 18th and 19th centuries, the Prussian government ordered the demolition of the walls.

The castle in Czersk was preserved in the form of a so-called permanent ruin. Its present form is the result of conservation work carried out on the site since the end of the 19th century. The castle walls were rebuilt with contemporary bricks similar to the Gothic ones, differing in colour and texture from the historical material.

There are two types of protection for the wall topping on the site. All towers have been protected by a layer of cement mortar with a profiled slope, while the brick perimeter walls have been protected with solid ceramic brick over the entire surface. The wall topping of the south and west towers are additionally reinforced with steel bars.

Technical condition of the cement mortar protection on the outer wall topping of the main tower and the west tower is without significant damage to them. Only in the areas of direct contact between the cement layer and the brick masonry is there biological corrosion (lichen). The toppings of the south tower masonry are in good technical condition. There are localised defects in the joint. In poorer technical condition, there are the walls topping of the south and west tower shelves, where numerous spalling and damage to the concrete surface and the presence of vegetation were found.

The protection of wall topping of the perimeter walls created by re-bricking with contemporary bricks is in sufficient condition. There is a lack of horizontal insulation at the junction between the historic masonry and the over-brickwork. There is substantial damage at the junction between the face and the wall topping. In parts, the wall topping is detached from the masonry proper. In places, the cope brickwork is not connected to each other due to defects in the mortar. As a result of inadequate protection (no bond between the last layer of the face and the crown lintel), the surface of the bricks has become detached and a deep crack has formed, allowing precipitation to penetrate into the masonry. Perennial vegetation has sprung up in these leaks, with its root system causing further damage at the interface of the layers. There is also a localised infestation of algae (mainly on the final of the face), which is evidence of the markedly higher and persistent moisture content of the bricks.

Fig. 9. East wall, view from the tower. Detaching of the face layers of the wall. In areas of damage, water enters the wall. *Source*: author, 2019

Fig. 10. North wall on the courtyard side. Damage to contemporary brick topping. Damage to pointing, salt deposits. *Source*: author, 2019

Fig. 11. The north wall from the exterior. Degradation of the surface layers of the modern superstructure. Intense carbonate efflorescence on the face of masonry. *Source*: author, 2019

Fig. 12. South wall. Localised damage to wall topping repointed with fieldstone. *Source*: author, 2019

4.3. The castle of the Mazovian Dukes in Liw

The castle of the Mazovian Dukes in Liw was built at the turn of the 14th and 15th centuries. It was built on a square plan with a gate tower located outside the outline of the walls. The foundations were made of pebbles, the walls were built of Gothic brick. The castle precinct also included a Large House and a Minor House (on the site of the present manor house). The fortress was first rebuilt in the 16th century and had to be reinforced due to the many dangers. The castle was destroyed twice. The first time was during the Swedish Deluge; the building was rebuilt quickly enough. The castle was destroyed a second time after the Northern War and no more attempts were made to rebuild it. At the end of the 18th century, the Minor House was replaced by the starost's office. From the beginning of the 20th century, work was carried out to secure the castle. [3] The last comprehensive preservation works took place in 2017-2019. The overall technical condition of the fortress is good to very good. The only damage is mainly to the surface layers of the elements.

There are two types of protection for the wall toppings at Liw Castle. The first type is re-bricking of the wall topping with contemporary bricks with the top layer of bricks laid flat. The second is the construction of a mortar coating. The repointing occurs on brick walls, which are in good to very good condition. In places there are carbonate deposits associated with carbonates leaching from the mortar. However, mortar protection is found on the lower parts (at the stone wall) of the gate tower buttresses. The technical condition of this type of protection is poor. The mortar layer is cracked and delaminated, and rainwater penetrating into the wall has damaged the joints directly under the protective coating.

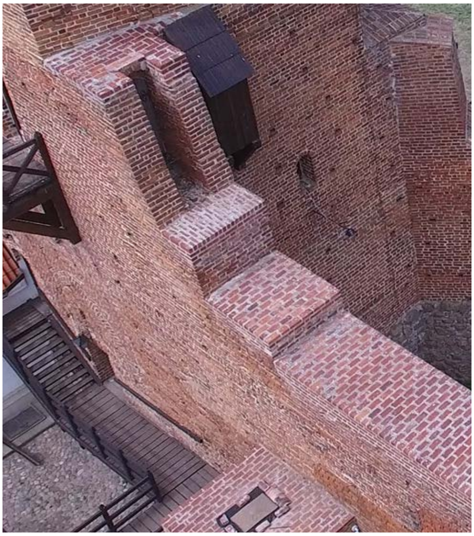

Fig. 13. The wall topping of north-west wall. Localised mortar staining of face, careless pointing of bricks. *Source*: author, 2019

Fig. 14. Tower, view of south-west elevation. The repointed wall topping of the tower buttresses. The brickwork and pointing used ensures that the layers are distinguishable. *Source*: author, 2019

Fig. 15. The wall topping of north-east wall. Unaesthetic pointing, localised carbonate deposits on surface of bricks. *Source*: author, 2019

Fig. 16. South-east wall. Damage to joints, detachment of individual tones of face and top layers. *Source*: author, 2019

4.4. The castle of the Mazovian Dukes in Sochaczew

The castle of the Mazovian Dukes in Sochaczew dates back to the 14th century. It was situated on a hill, separated from the town by a natural ravine and a river flowing through it. The walls of the fortress were made of finger-brick on fieldstone foundations. The structure was built on a trapezoidal plan. Originally, the defensive system consisted of two towers and a defensive wall. During the Swedish invasion, the castle was burnt down and functioned in a state of disrepair for more than a century. Subsequently, due to the unstable hill, it was decided to demolish the Gothic castle and a new 18th century foundation was built on its previous shape. [4] Only the north tower remained of the former fortress, and the form of the modern castle consisted of three wings, an octagonal tower and a defensive wall.

Currently, the castle in Sochaczew is kept as a permanent ruin. The overall technical condition of the castle ruin in Sochaczew is very good. In the years 2011-2013, conservation work was carried out to comprehensively protect the remains of the walls, make new floors in the courtyard as well as in the former rooms of the fortress and drainage of rainwater. These protections were carried out according to the standards accepted for so-called permanent ruins.

The toppings of the walls were over-bricked with contemporary material and made as lost layers, clearly recognisable from the historic substance. Above the brickwork a mortar layer with a slope. Protection on the toppings was carried out piecemeal. They are currently in varying degrees of repair. Some of the mortar caps have cracks, which cause rainwater to penetrate deep into the historic masonry. In places there is carbonate crystallisation on the face of the modern bricks, mortar contamination, and vegetation growth has been found at the interface of the layers. In places above the historic wall there is no contemporary re-bricking, sections of the wall have only been secured with contemporary mortar. The lack of regularity of the protections and their inadequate execution results in rainwater ingress and destruction of the masonry.

Fig. 17. The wall topping of tower in good technical condition. Locally visible damage to the top layer of bricks. Defects in pointing, damage to bricks. Source: author*, 2019*

Fig. 18. Tower wall topping. Strong salt corrosion of the face of the brickwork of the tower terrace exit. Salt corrosion damaged brick faces. Carbonate streaks on the surface of the masonry. Source: author*, 2019*

Fig. 19. The wall topping. Localised damage to the surface of the upper layer of bricks. The wall topping contaminated with bird droppings, spotty lichen growth. *Source*: author, 2019

Fig. 20. The wall topping. Topping of the wall on the courtyard side. The junction of the historic wall and the over-brickwork. Brick used and pointing to ensure the layers are distinguishable. Sheet metal dripline draining rainwater. *Source*: author, 2019

4.4. Summary

On the basis of the analyses and studies carried out, Table 2 has been developed taking into account the character of the work carried out. The following columns refer to: target or temporary character of the protections, material compatibility, reversibility, scope of work (overall, piecemeal, local) and distinguishability of the materials and applied solutions.

Table 2. Selected ways of protecting wall topping of brickwork castles. *Source:* author

	Ciechanów	Czersk	Liw	Sochaczew
Protection of the wall topping YES/NO – character	YES Addition of a new masonry layer Addition of a new masonry layer with execution of a capping	YES Addition of a new masonry layer	YES Addition of a new masonry layer	YES Addition of a new masonry layer Addition of a new masonry layer with mineral mortar sloping layer
Target/Temporary	Target	Target	Target	Target
Materials compatibility	YES	YES	YES	YES
Reversibility of the solution YES/NO	NO	NO	NO	NO
Scope of work Overall/Partial	Overall	Overall	Overall	Partial Fragments of wall faults unprotected
Distinguishability of the solution YES/NO	YES	YES	YES	YES
Slope profiling YES/NO	YES	YES	NO	YES

5. Conclusions

On the basis of the above analysis of the protections made on the toppings of the Gothic brick walls of the Mazovian castles and the summary in Table 2, the following was concluded:

– The least durable solutions are those in which the masonry has been protected by lintels without the introduction of an insulating layer.
– Significantly better durability and effectiveness were achieved when using lintels materially compatible with the insulation layer at the interface between the historic and modern layers.
– The use of additional capping between the historic layers and the brickwork or within the layers of the brickwork (usually 1-3 brick layers from the top of the brickwork) protects the face of the wall.
– The protections should be applied comprehensively without leaving sections of masonry unprotected.
– The protections are targeted and, in most sites, their reversibility is linked to interference with the historic structure.
– Due to the use of materials with similar colours and measurements, there may be a problem with the distinguishability of the protection after several decades.
– The protections made were designed as lost elements that should be continuously monitored and restored.

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Orginal Article

Planning protection of Ciechanow castle versus contemporary exposition in the landscape

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Abstract: The castle in Ciechanow is classified as a Gothic castle of the lowland type. The building, which is an example of defensive architecture, is located to the north of the historic urban layout of Ciechanow city centre. At present, the castle is covered by the following forms of protection: protection for the Natural and Landscape Complex "Łydynia River Valley", provisions of the Local Spatial Development Plan and an entry in the Register of Historic Monuments. The authors of the paper analyse the legal regulations concerning the castle and their impact on the contemporary castle's exposition in the landscape.

Keywords: cultural landscape, formal and legal protection, exposition, castle

1. Introduction

Poland is a unique European area in terms of its diversity and is a distinctive "open-air museum" of defensive architecture incorporated into the open landscape. The landscape in Poland is almost entirely a cultural landscape, except for small enclaves of primary landscape and natural landscape [1]. Based on the 2003 law, cultural landscape is defined as: "the space perceived by people containing natural elements and products of civilisation historically shaped as a result of natural factors and human activity" [2]. Defensive architecture (the so-called "Ars militaris") has always been linked to the surrounding landscape, terrain and land cover, thus benefiting from natural elements such as rivers, hills or moors, which have been used as factors to enhance the fortification of the site. "Depending on the complex military function of the object and the possibilities of the builder, as well as the material and technical conditions of the time, the castles took on different shapes and dimensions and underwent successive extensions both in terms of quantity and in terms of function, technology and space. When interpreted subjectively, the castle itself contained something in its external function, i.e.

important communication routes, crossings and strategic points, while when interpreted objectively it contained something inward, i.e. it enclosed, fenced off, guarded and secured the castle users and their external function, i.e. the function of the enclosure" [3]. Castle ruins are therefore one of the groups of defensive structures which are an indispensable element of Polish cultural landscape. Historic defensive works such as castles form a dominant landscape feature and this is the extent to which they are exposed [4]. We can speak of a historic ruin when it is a complete object with a defined technical condition, functionality, legibility and communicability [5]. Most historic ruins are isolated from other buildings and are located on elevated ground in a favourable viewing position [6]. Like any defensive work, they are a kind of document of the era "as a monument of architecture, engineering and techniques".

The cult of the ruins originated in the second half of the 18th century from romantic motivations, developed through interest in history and stemmed mainly from patriotic motivation of the 19th century [7].

The first attempts to get to know the deteriorating defensive buildings were most intense at the end of the 18th century and the beginning of the 19th century, at a time when they were abandoned and gradually degraded [8]. Identification work shows that the collection of castles in Poland is estimated at 402 objects entered in the register of immovable monuments, which are in a varying degree of preservation [9].

2. Methodology and aim of the work

The aim of this paper is to analyse the formal and legal provisions concerning the protection of the landscape values of the castle in Ciechanow and to try to verify them in situ. For this purpose, the following were performed:

1. an analysis of the legal records concerning the protection of the castle in Ciechanow,
2. study visits were carried out,
3. necessary photographic documentation was made,
4. on the basis of the collected materials, analyses of the exposition of the castle in the panorama were prepared.

3. Castle in Ciechanow

3.1. The history and architectural form

The Castle of the Dukes of Mazovia in Ciechanow is an example of a Gothic, lowland castle. The castle was probably erected in the second half of the 16th century. The founder of the castle is considered to be Janusz I of Warsaw [10] . The idea to build the castle in the floodplains of the Łydynia iver was at the time the result of a threat from the Teutonic State, which, after accepting the Zakrzyń Land as a fief, was increasing its territorial scope towards northern Mazovia [11].

Over the years, the castle was repeatedly modernised and rebuilt. Thus, it changed its purely defensive function into one of both residence and defence. The last period of splendour of the whole complex dates back to the times of Queen Bona, then, as a result of the turmoil of wars – specifically the Swedish wars – the castle underwent significant destruction and partial demolition. Repair and partial reconstruction work was undertaken throughout the 20th century. An extensive renovation was carried out in 2013. Currently, the castle in Ciechanow is managed by the Regional Museum, while the castle itself is owned by the Marshal of the

Mazovian Voivodeship [11]. At present, the object is maintained in the form of a so-called “historical ruin”. The form of maintenance and scope of development of the castle is subordinated to the preservation of its historic values.. The current castle development mainly serves the needs of tourist traffic.

The building was founded on a rectangular plan measuring 48x57 metres with a courtyard with a side length of 44 metres. The castle is surrounded by high walls with a crenelation and two cylindrical towers, which frame the southern curtain with a gate in the middle [11]. Visible relics of the gate house (south curtain) and the Big House (north curtain) have survived to the present day in the courtyard space. The main entrance to the castle is located in the western curtain – the western gate, with visible relics of the former drawbridge. The aforementioned elements constitute the landscape value of the castle and its exposure from the town [11]. The castle was built on a stone foundation in a marshy area. Due to the unstable ground, the ground was reinforced by gravel, bricks and oak stumps before construction began. The complex was surrounded by an extensive 18-metre-wide moat, which was quite shallow, as it was up to 1.4 metres deep.

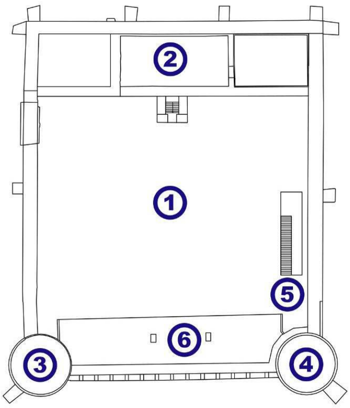


Fig. 1. Plan of the castle in Ciechanów. Legend: 1. the courtyard, 2. the dwelling house, so called. “3. west (arsenal) tower, 4. eastern (prisoner) tower 5. Entry to sanitary supply 6. Small house (new volume), *Source:* K. Drobek

Fig. 2. Plan of Ciechanow from 1816

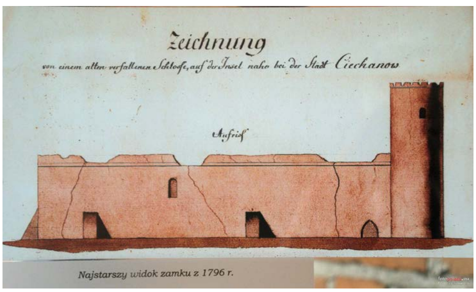


Fig. 3. The oldest view of the castle from 1796

Fig. 4. Photo of Ciechanow castle. Photo: A. Broimska, 1905, postcard

3.2. The viewing relationship of the castle and the city – determinants

The primary aim was therefore to protect the landscape zone, i.e. the scenic links between the castle and the city, in addition to protecting the historic structure of the castle. This is because the castle in Ciechanow is integrally connected with the surrounding riverside landscape. Thus, the "Conditions for the landscape zone of viewing relations of the castle and the city" emphasise the need to create a tourist route highlighting the cultural values of the castle and its surroundings, as well as to build a connection – via a pedestrian route – with the urban system (by combining the functional programme of the market square) with highlighting the historic gate, while preserving the currently existing pedestrian traffic system (western curtain). The elements selected for landscape protection of the entire complex are:

1. the floodplains of the Łydynia river (eastern side of the city of Ciechanow),
2. panorama of the city from the view of the tower, through appropriate composition of buildings in the first line of plots in terms of architectural form, material solutions and scale,
3. protection of the panorama from the side of the city with a line of trees,
4. protection of the Łydynia river landscape,
5. protection of the city panorama from viewpoints and platforms located within the castle.

The recommendations also pointed out the already existing problems in the form of villa developments on the northern side of the castle, which got too close to the viewing foreground of the building, and recommended designing isolating greenery in sensitive points and detailed elaboration of the rules for introducing the developments.

3.3. Formal and legal situation of the castle in Ciechanow

For a long time Ciechanow played a key role in the economic system of Mazovian cities, as well as in the system of defence of Mazovia. The so-called "new town" was founded near the castle itself and right by the river, on a high ground, in the northern – less developed – part of the town. The northern frontage of the market square of the "new town", due to its looseness in terms of development, allowed the market square to be linked to both the river and the castle located behind it. Without a doubt, according to a 2005 study by Danuta Kłosek-Kozłowska, the scenic compositional link between the market and the castle was provided by the still existing narrow street dividing the northern frontage in half. In addition, along Nadrzeczna Street there were numerous view openings to the castle, which are currently not legible due to the addition of development to the plots and the tall greenery present in them [11].

The castle is located to the north of the historic urban layout of the city centre of Ciechanow. The building is listed in the register of monuments, together with its surroundings within a radius of 200 m. It is located in an open landscape, among meadows, in the floodplain of the Łydynia river.

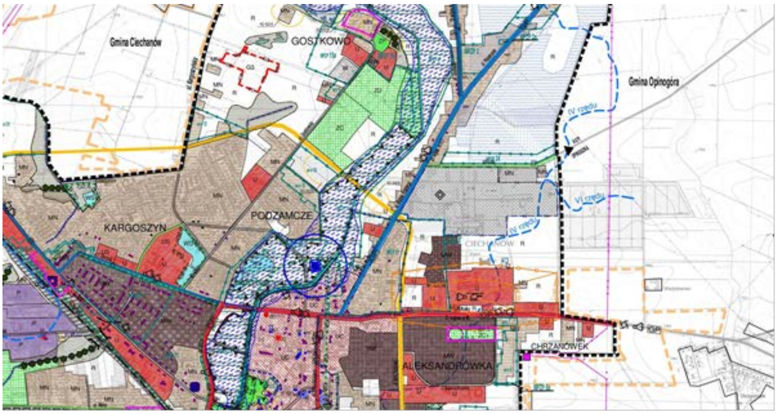
The great scenic value of the Castle of the Dukes of Mazovia in Ciechanów is influenced by the fact that it is located within the natural and landscape complex "Łydynia River Valley". Together with its surroundings, it is protected for the Natural and Landscape Complex "Łydynia River Valley". It is a unique area located in the centre of Ciechanów, characterised by a natural and unaltered landscape of the river valley, with fauna typical of forested areas located away from urban settlements.

In addition, the castle is also the starting point of the route of the Medieval Route of Ciechanów, which covers the area of the historical part of the city together with the oldest sacral buildings located in this area and buildings from the 19th and 20th centuries listed in the register of monuments as a historical urban establishment. The route runs through Wodna Street, Jana Pawła II Square, Warszawska Street, Kościuszki Square, Zielona Street, a path to Farska Góra and through Dąbrowskiego Park and Ściegiennego Street and part of 11 Pułku Ułanów Legionowych Street.

Fig. 5. Location of the castle of the Mazovian princes in Ciechanów, spatial relations, *Source*: K. Drobek

Table 1. The forms of protection of Ciechanow Castle

Form of protection	Content of the record/ Landscape context
Protection for the Nature and Land-scape Complex "Łydynia River Valley".	„The specific aim of the protection of the Complex is to preserve fragments of the natural and cultural landscape of the Łydynia river valley, in particular: 1) an area overgrown with a wide range of plant communities constituting a review of plant succession from hay meadows, through tallgrass meadows and herb forests to woody willow-poplar riparian forests; 2) a habitat of several dozen species of breeding birds; 3) an area of great health, climate-forming and recreational importance; 4) the valley of the Łydynia river together with estuarial areas of the watercourses; 5) areas under conservation protection: the bailey of the Castle of the Mazovian Dukes, the Farny Church and the Farska Gora.” [12] “The natural and landscape complex "The Łydynia river Valley" – is an area of 57 hectares, located on the territory of the city of Ciechanów, is a legally protected part of the valley. The protection covers the natural and landscape values of the area and the plant and animal species occurring here, as well as material culture resources – registered monuments: Castle of the Mazovian Dukes, Farny Church, Farska Góra.” [12]

Form of protection	Content of the record/ Landscape context
Local Spatial Plan	Service area U1, recommendation to preserve naturally low greenery, designation of exposure protection zones in which no development is allowed and land use that could disturb the exposure of the castle and the historic centre [13]. 
Study of conditions and directions of spatial development of the city of Ciechanow	The need to control changes in land use and development, not allowing elements that disharmonise or overshadow protected sites [14]. 
Entry in the register of Historic Monuments	

3.4. The castle in Ciechanow – landscape analysis

The Castle of the Mazovian Dukes in Ciechanów is a dominant landscape feature. The foreground of the exposition is the castle's meadows. The Łydynia river is a mirror reflecting in its surface the picturesque surroundings and, above all, the castle itself. Along the axis of the river, as well as along Castle Street, there is a wide view opening onto the castle. Overall, the castle is very well exposed in the landscape, as exemplified by the following photographs. In several places, the view of the stronghold is somewhat shielded by the tall greenery growing along Zamkowa Street at the junction with Kmicica Street and from the north-west from the field areas located behind the castle.

Fig. 6. The View of the Castle of the Dukes of Mazovia in Ciechanów and the Łydynia river from the west, Exposure of the castle in the panorama. View from the intersection of Parkowa and Zielona streets, 2019. *Source:* K. Drobek

Fig. 7. Exposure of the castle in the panorama. View from the junction of Parkowa and Zielona Streets, 2019, *Source*: K. Drobek

Fig. 8. Exposure of the castle in the panorama. View from the intersection of Zamkowa Street, 2019, *Source:* K. Drobek

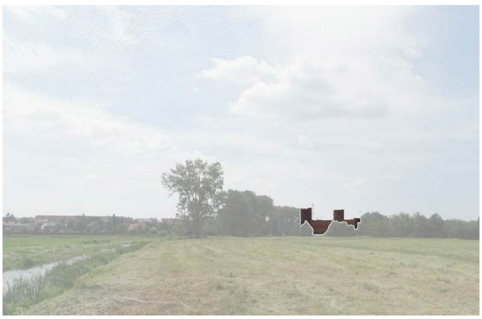


Fig. 9. Exposure of the castle in the panorama. View of the castle from the north-east from the Łydynia river, *Source:* K. Drobek

Fig. 10. Exposure of the castle in the panorama. View of the castle from the north-east, 2019, *Source:* K. Drobek

Fig. 11. Exposure of the castle in the panorama. View of the castle from the north, 2019, *Source:* K. Drobek

Fig. 12. Exposure of the castle in the panorama. View of the castle from Zamkowa Street from the bridge over the Łydynia river, 2019, *Source:* K. Drobek

Fig. 13. Exposure of the castle in the panorama. View from Jana Pawła II Square, *Source:* K. Drobek

Fig. 14. Exposure of the castle in the panorama. View from Warszawska Street, 2019, *Source:* K. Drobek

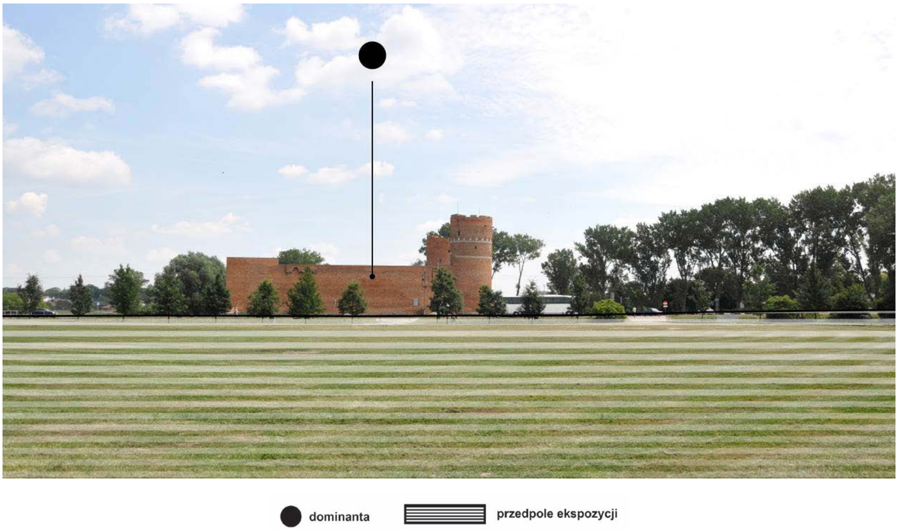


Fig. 15. Exposure of the castle in the panorama. The view from the footbridge on the meadows, 2019, *Source:* K. Drobek

The Castle of the Dukes of Mazovia in Ciechanow offers a view of the extremely green surroundings. The two towers and a shooting porch are good observation points. Looking to the north, one can see a fragment of a single-family housing estate and cultivated fields. To the north-east and east, there is a view of fields and the floodplain of the Łydynia river. South of the castle, beyond the river, there are buildings in Nadrzeczna Street and the market square. From the western tower. Looking south-east one can see Wodna Street and the Ciechan Brewery. To the west of the castle are located the meadows with a footbridge.

Fig. 16. Photo View of the surrounding area and the Lydynia river from the shooting porch, 2019, *Source:* K. Drobek

Fig. 17. Photo View from the prison tower to the northeast, 2019, *Source:* K. Drobek

Fig. 18. View from the prison tower to the southeast., 2019, *Source:* K. Drobek

4. Summary

The exposition of the castle in Ciechanow and the legibility of its composition have been preserved to the present day. The work of Ars militaris is visible in the landscape of Ciechanow both from the side of the riverside boulevards and the city space with the fact that some of the original views and links with the urban system have been lost through high plantings of greenery (Zamkowa Street) isolating the north-western side. The legibility of the panoramas has also been lost along Nadrzeczna Street as a result of newly developed riverside areas and the tall greenery growing there. It therefore seems crucial that the buildings in the first line of plots are properly composed in terms of architectural form, material solutions and scale, as well as appropriately firm provisions in local plans. Formal and legal protection in the form of an entry in the Register of Historic Monuments, provisions in the Local Spatial Development Plan and protection for the Natural and Landscape Complex of the 'Łydynia River Valley' ensures a high level of protection for the castle's exposure in the landscape. The very fact that the provisions of the Local Spatial Development Plan note and indicate the zone of protection of the exposure of the Masovian Dukes' Castle both from the side of the city of Ciechanow and from the Podzamcze area is noteworthy. Thus, a conclusion is drawn that further model protection of this type of monuments should be detailed in the local plans as it is an act of law compared to the provisions of the study of spatial conditions of the city of Ciechanow.

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Improving functional standards of apartments in buildings from large-panels, on the example of solutions applied in Lublin in the 1970s and 1980s of the 20th century

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Abstract: Housing blocks constructed from prefabricated elements have been the backbone of housing resources in many Polish cities in the last fifty years. Over the time, the residents' expectations regarding the residential amenity standards, as well as demographic structure of the estates built in the communist era have changed. Therefore, the current residents' needs significantly differ from the needs and expectations of those who took over the newly built flats in the last quarter of the 20th century. Fundamentally, the functional and spatial standards of flats built before 1989 are already out of date. The proper functioning of the usable dwelling space of these apartments has gained additional importance due to the inhabitants getting progressively older and, especially recently, the prolonged SARS CoV-2 pandemic. The introduction of the lockdown, which involved strict restrictions on movement outside the place of residence during the pandemic, made the apartments the centre of life and a place of work for many families. Suddenly, for psychophysical reasons, the importance of having larger floor space, than the functional solutions typically used in the 1970s and 1980s has increased. It appears that the residents begun to take interest in all possible solutions to improve the functionality of their homes. The aim of the article is to present proposals for contemporary architectural solutions that can improve the functionality of these apartments that would increase residents' standard of living and everyday comfort.

Keywords: large panel, Polish People`s Republic, modernization, living environment, functionality

1. Introduction

In Poland, there are currently over 60,000 residential buildings dating from last century that were built in large-panel technology. These residential blocks of flats are inhabited by nearly 12 million people, which constitutes almost 1/3 of Polish population [28]. The implementation of housing development on such a large scale was the result of the state's housing policy between 1952 and 1989, aimed at solving the serious housing crisis. As a result of the government commissioned research, a multi-storey residential structures built from large precast concrete panels were introduced and mass production was implemented. 1970s and 1980s were the decades of the most intensive development of this type of construction in Poland. The facilities were built in a hurry and their spatial parameters (both interior and exterior) were rigorously governed by strict architectural standards [13] and the specific structural arrangement was required to be adhered to. The vast majority of the facilities erected at that time, have been in use continuously to this day.

For at least a dozen of years, both politicians and construction experts have been urging for need of assessing the current condition and subsequent modernisation of large-panel residential buildings[1]. Furthermore, in the light of the ever-growing demand for new houses, architects, economists, and sociologists draw attention to the issue of the shortage of housing resources. Increasingly, the issues of interior functionality, appropriate technical equipment, aesthetic quality, and energy efficiency are coming into light. Another significant problem of the communist era residential architecture *moral deterioration*[2], a consequence of the buildings no longer meeting the amenity and comfort expectations of the inhabitants. Furthermore, this issue is very apparent in the residents' feedback, a confirmation that the problems of large-panel constructions have intensified and become a real obstacle to efficient functioning. Strictly defined by law and rigorously controlled spatial standards of flats inside buildings made with industrialized technologies in the 1970`s and 1980`s does not meet the modern needs of residents, and the functional layout clearly deviates from the solutions proposed today.

This article presents selected technical possibilities and proposals of architectural solutions aimed at improving the functional quality and amenity of flats in prefabricated constructions using south-eastern Poland as an example. The identified problems, analysed examples, and propositioned solutions attempt to indicate the possible functional changes to stop, or even reverse, the process of sub-standardization of the housing estates of prefabricated buildings. Studied issue is one of many factors affecting the standard of housing in large-panel buildings and is intended only as an introduction to a broader study of restoring full functionality to this type of buildings.

2. The research status

The historical background, taking into account the broad political legal and economic context of the construction of the analysed types of buildings, is widely described by Basisa [3] and Korzeniewski [13].

Many scientists emphasise the necessity of modernisation and revitalisation of buildings constructed in prefabricated modular technology. The urgent need to outline these innovations is indicated in publications of authors including, but not limited to: Ostańska [17],[18], Cibis and Olejko [4], Gronostajska [7],[8], or Kłopotowski [11]. The analysis of housing standards

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1 In 2013, the Deputy Minister of Construction in the Sejm of the 7th term, Janusz Żbik, announced a tender for examining the technical condition of large-panel buildings, while the Minister of Investment and Development in the 8th term of the Sejm, Jerzy Kwieciński, assured in 2019 that funds for modernization of 60,000 would be secured in the state budget. this type of facility..

2 Defined as the persistent dissatisfaction of residents with the surrounding architectural space, resulting from its insufficient spatial, functional and aesthetic standard. Following [12]

and their changes over the years is carried out in their research by Płachcinska and Grudziński [9] and Płachcinska and Zaniewska [22].

The progressive depopulation processes in housing estates of a similar type in countries other than Poland, as well as actions undertaken by individual countries are covered in publications by Balache and Salagnac [2], Łodziński [16], Czado [5], and Dmitruk [6]. The current epidemiological situation confirming the intensification of problems is covered in Ostańska's [18] social researches and calls for an urgent need to take remedial action. As a direct result of many months in isolation, elderly residents were forced to remain in their apartments (SARS CoV-2), which also affected their physical fitness. These adverse effects of the lockdown, as well as restrictions of the movement of the residents within the building and on the premises, contributed to the development of the proposals presented in the article.

Issues concerning pro-environmental solutions applied to buildings of a similar type to those described in this article are studied by Życzynska and Cholewa [23], and Ostańska and Medvedeva [20] in particular in terms of thermal insulation technologies. In a broader context, pro-environmental and pro-social issues that can be applied to the process of modernization of buildings built with the analysed constructional systems are studied by Krężlik [14] and again by Ostańska [21]

Interior arrangements of residential buildings from the 1950` and 1960` are studied and analysed by Kruk [15], from the point of view of users' needs, as well the political and social realities of the communist era Poland. The possibilities of urban revitalization of a housing estate built in the OWT-67 technology, aiming to improve the living conditions of the residents is studied in the works of Janus and Janusz [10].

3. Scope of the study and research methodology

Exemplary flat layouts of apartments within the buildings made with WBLŻ and OWT-67 structural systems[3] were subjected to analysis. These systems were chosen because of the substantial amount of multifamily buildings constructed in the Lublin region in the above-mentioned technology. For this reason, conducted issue addresses the regional, rather than individual significance. The research was preceded by inventory work, archival searches, photographic inventory and an extensive study of the original design documentation, made possible by the courtesy of the administration of the St. Moniuszko housing district in Lublin. Research and analysis of existing solutions were conducted on the basis of a literature study, helpful in determining the contemporary needs of residents, a site inspection, as well as many interviews with residents of Lublin's communist-era housing estates. The proposed solutions are based on the author`s design experience, as well as on the evaluation of the implemented solutions by both the author and users after several years of functioning.

4. Functional layouts of apartments

Over the four decades since the construction of the most of the multi-family buildings in prefabricated technology, significant changes have occurred in the expectations regarding spatial standard of flats, room layout, and mutual relationship of dwelling spaces (zoning). The limited functional areas of flats were the result of the construction laws, applied construction systems, and socio-economic conditions of the communist era. According to architect Rita

[3] Technical abbreviations of the names of Large-panel building technologies from the communist era Poland.

Nowakowska[4], between 1952 and 1989, there was creative freedom in urban design, while residential architecture was completely subordinated to the limits of the construction system, limited by tight budgeting, giving the architect no possibility of creative shaping of building and its interior.

The architectural norms in the communist Poland [24][25][26], which regulated the size of residential premises, had to be strictly followed, and any deviation, such as increasing the usable area of the object, often resulted in the rejection of the project by the authorities [3]. The spatial standard of flats was determined by the authorities based on the expected number of tenants, naming them successively from M-1 to M-7[5]. It can be observed that despite the gradual increase in the normative values of flat square meterage in subsequent resolutions (Table 1), these parameters differ from the modern expectations of apartment owners. According to the information from the Central Statistical Office for 2017 [1], the average living space per 1 tenant is 27.8 m^2, which is already in the upper limit of the area allowed for the M-1 category from the 1974 norm. Flats intended for two people, have a floor area of 40-50 m^2, which corresponds to the normative value, expected for 3, or until 1974, even for 4 people. Thus, it can be observed that the expected spatial standard of a contemporary apartment it is noticeably higher than that offered in large-panel construction[6].

Table 1. List of spatial standards for residential buildings from 1959 and 1974

Appartment category	Useable floor area 1959 (m^2)	Useable floor area 1974 (m^2)	Permitted increase of upper limit for technical reasons (m^2)	The maximum upper limit of the size of a flat in a given category (m^2)	Number of residents
M-1	17-20	25-28		28	1
M-2	24-30	30-35	1	36	2
M-3	33-38	44-48	4	52	3
M-4	42-48	56-61	2	63	4
M-5	51-57	65-70	3	73	5
M-6	59-65	75-85		85	6 or 7
M-7	67-71	No norm			

Source: [13]

A significant obstacle in the unrestricted arrangement of residential interiors in large-size prefabricated panel buildings is a system of structural walls. They are often located inside the premises[7], preventing or considerably hindering any adjustments to the layout of the rooms that the residents may wish to introduce.

4 The interview was conducted on September 6, 2016 in Warsaw. Rita Nowakowska is the author of many architectural and urban projects in Lublin, incl. the building of the municipal office in Lublin, at ul. Wieniawska 14, housing estate Tatary, housing estate Kalinowszczyzna, housing estate them. Stanisław Moniuszko, the estate of Henryk Wieniawski and the campus of the Lublin University of Technology.

5 The numer indicating the numer of residents.

6 The actual phenomenon of selling micro-apartments, smaller than 25 m^2, was deliberately omitted due to a clear violation of building regulations relating to the minimum floor space of a flat and mainly due to the temporary nature of living in this type of premises.

7 And not only constituting the external walls of residential premises, as is often solved nowadays.

Making openings in structural walls to improve the functionality of the apartment's interior is technically possible, although rarely implemented, due to difficulties in obtaining the consent of the building community or the property manager. This is probably due to the lack of technical knowledge and willingness to understand the current needs of the residents.

As part of the research, functional and spatial solutions of buildings constructed in accordance with OWT-67 and WBLŻ systems, which are two highly popular Polish building technologies from the 1970s and 1980s, were analysed. 20th century design solutions significantly differ from the concepts used today. Table 2 compares the solutions adopted primarily in the analysed buildings with the contemporary interior layout design trends. Modern design trends were determined on the basis of an analysis of numerous examples of contemporary realizations of multifamily housing facilities in the country, as well as the author's practical experience in the field of designing such facilities and the knowledge of customer requirements.

Table 2. Comparison of interior layout solutions adopted in prefabricated systems from the 20th century, with contemporary design trends

No.	Systemic solutions in 1970s and 1980s	Contemporary solutions
1	No clear zoning of the interior of the apartment	Rooms divided into a day zone (living room, kitchen, dining room) and a private zone (bedroom, bathroom, wardrobe, children's bedrooms)
2	Kitchen as a separate room	Possibility to connect the kitchenette with the dining room or living room area
3	Bathroom and toilet as separate rooms	Bathroom and a toilet as one room + additional toilet in the living area
4	The front door opens into the private zone (WBLŻ) or opposite the toilet and bathroom (OWT-67).	The entrance door opens into the living area
5	Extensive hallway space inside the apartment.	Minimal space dedicated to hallways, increased floor area of functional rooms
6	A living room is also the parents' bedroom, it is not possible to fit in a double bed	A separate bedroom with a double bed as a separate room, in addition to the living room
7	Small, non-functional balconies, no communal gardens	Large balconies and/or communal gardens constituting additional functional space
8	No foreseen service premisses/utilities on the ground floors of the building	Service premises are frequently included on the ground floor of the building
9	No passenger lifts in under five-story buildings (OWT-67)	Residential buildings over 4 storeys must be equipped with a passenger lift. Often this type of amenities is available in lower buildings.
10	The ground floor, raised by half a storey, requires climbing 7-8 steps	The entrance to the building is designed to be accessible from the ground-floor level

As part of research, reworked at Lublin University of Technology practical classes, as well as dissertation theses of Architecture faculty students and numerous alike designs implemented by author, a number of design solutions were provided to help adjust the interiors of the communist era buildings to modern standards, while maintaining the existing structural layout, the size and the location of windows, and the number of rooms. The large-block system (WBLŻ) turned out to be more susceptible to changes allowing for fuller interior optimisation. WBLŻ has two basic types of flats, 47.14 m^2 and 57.15 m^2, which are mirrored in the layout of the entire building. For the purposes of the article, a study of the rearrangement of the residential interior of the premises with an area of 57.15 m^2 was made, and the effects of the work are presented in Fig. 1.

a) premises before modernization b) premises after modernization

Fig. 1. Comparison of layouts of living quarters in a WBLŻ type building. Original state and designed state. Compiled by M. Dmitruk

The above example shows modern zoning of the apartment into a living zone and private zone. The direction of the front door opening, leading to the living area, was maintained, and the the kitchen and the living room were merged. There is a spacious sleeping area with room for a double bed. In addition, a study/office area was included. This was possible due to the access to the window in the gable wall. The correct technological line of kitchen appliances was restored. The bathroom and toilet were merged, increasing its area by 0.78 m^2 and provided the required manoeuvring space in front of the bathroom fittings. The hallway was reduced by 0.81 m^2, but the storage space was almost doubled. The hallway was significantly shortened as a result of a merged day zone, originally situated on two opposite ends of the apartment. The total usable area was increased by 0.11 m^2. The ventilation of the rooms, sewage system and gas installation were unchanged, while water supply system was changed only slightly. If necessary, a variant of this design allows to change the location of the cooker and to replace of the gas cooker with an electric one. It is important to note that in order to ensure optimal spatial comfort, the layout of the apartment after the changes will be designed for a family of the 2+1 model, rather than 2+2 as originally envisaged. However, it should be emphasized that the apartment has never actually met the spatial needs of a family with two children, and the number of optional sleeping places in the proposed variant has de facto increased.

The design analyses (Fig. 1) provide for thermal insulation of the facade and, due to the change in the location of the living room, an additional large balcony, which will undoubtedly meet the expectations regarding the residents' leisure needs.

Unquestionably, there are more difficulties with optimizing the interior layout of OWT-67 system buildings. This is due to the cross-structure layout, which makes the layout of passages in individual premises challenging to adjust. In addition, there are as many as five types of apartments in this system, which requires a more individualized approach to design analyses. Although the suggested modernization of the interior layout (similar to the case of the WBLŻ system) can be solved within the existing arrangement of openings in the façade and structural walls, it requires some major changes to the kitchen and bathroom utilities[8].

In the sample apartment analysed (Fig. 2), the kitchen and the living room were combined, moving the day zone's location to the south side, and increasing its area by 1.44 m^2. A proper technological line of kitchen appliances and a place for a full-size refrigerator was provided. Furthermore, the bathroom was relocated, with the use of an existing opening in the structural wall. For structural reasons, the toilet remained separate from the bathroom, but its floor area was extended for comfortable use. There is space for a double bed and a large wardrobe in one of the bedrooms. The direction of the entrance door opening has also been corrected, making it easier to access the wardrobe. These changes will not require additional permits as the design analysis assumes the replacement of the gas cooker with an electric one. As in the case of the WBLŻ system, the thermal insulation of the building was considered. It was also proposed to add a large balcony to the exterior of the day zone.

a) premises before modernization b) premises after modernization

Fig. 2. Comparison of the layouts of living quarters in a building in the OWT-67 technology. Original state and designed state. Compiled by M. Dmitruk

The possible changes to the layouts of the rooms analysed in the above examples are legally insignificant, which means that they can be without the need to report work or obtain a building permit. However, such adjustments can only be applied within residential premises.

[8] Ventilation channels will have to be led into the kitchen and bathroom above the false ceiling in sanitary rooms.

This evidences that despite the prevailing opinion about rigidly defined interior layouts in prefabricated constructions, numerous functional improvements are still possible. A proposal to relocate and enlarge balconies may be more difficult, due to the need to obtain the consent of residents, the property manager and the relevant building supervisory authorities. In addition, it would be necessary for all residents to participate in the cost of such a project. However, practice shows that this type of solution is used in similar facilities across the country[9].

5. Summary

The main advantage of the presented proposals for reorganizing the space inside apartments in large-panel buildings is the possibility of universal implementation of solutions and making upgrades, according to a single design study. Development of a comprehensive building modernization plan for these constructions, supported by the real needs of residents and considering technical possibilities could be used throughout the country. Similar solutions are already used abroad; the Czech website panelplus.cz [27] offers designs for both interior and facade solutions, tailored to specific building systems, and offers case studies of successfully implemented modernisations. Moreover, the proposed changes in interior layout do not require additional permits or interference with the structure of the building, which greatly simplifies their implementation and gives the possibility to introduce them on an individual basis, according to the needs and financial capabilities of users.

It is possible to improve the quality of life in a prefabricated building. However, it is important to note that improving accessibility for people with disabilities by increasing the functional and spatial standard of flats requires determination, specialist technical knowledge, general consent of residents and a willingness to share in the costs. Also, removal of architectural barriers, e.g., replacement of balconies, requires a team of qualified professionals. Yet, examples of proprietary solutions confirm it is possible to counteract the growing problems of prefabricated buildings' residents, especially people with disabilities and the elderly, but also entire families.

Improving housing standards in communist era developments is a broad and multifaceted issue. It concerns both technical, architectural, urban planning, social and economic issues. Only through comprehensive and coordinated modernization measures, supported by active participation of residents, can a lasting improvement in housing standards be achieved. The study cited in this paper is intended as a starting point for further research and as a contribution to further discussion of the functioning of large-plate buildings in the city space in the coming decades.

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