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INFLUENCE OF SUDDEN COLUMN LOSS ON THE DYNAMIC RESPONSE OF A MULTISTOREY STEEL FRAME

Multistorey steel buildings are proved to be very susceptible to situations when one of their columns loses its capacity as a result of an accidental action. The above mentioned case concerning a steel framed building is the subject of investigation presented in the paper. Structural system of analyzed building was designed in accordance with ultimate and serviceability limit states in the persistent and transient design situations. Then its integrity in accidental design situation was assessed. According to EN 1991-1-7 [1], the strategy based on limiting the extent of localized failure was assumed. Firstly, the static analysis of the structure in Autodesk Robot Structural Analysis Professional software was performed. Then, the static and dynamic GMNA analyzes (materially and geometrically nonlinear) of the structure in Autodesk Simulation Mechanical were carried out. Calculations were made in reference to plane frame, which is the repeatable load bearing system of considered building. FEM models were made with the use of beam and shell elements. The results of performed analyzes were compared and discussed. Concluding remarks were drawn and directions of future research were outlined.

Keywords: robustness, dynamic analysis, accidental action, column loss, steel framed building, FEM

1. Introduction

In the case of identified accidental actions classical methods of structure analysis may be used, however, taking into account highly dynamic nature of the problem. In accordance with EN 1990 [2], these actions are considered only in accidental design situation.

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If actions impossible to determine are considered, e.g. terrorist attacks or random loss of bearing elements capacity, standard EN 1991-1-7 [1] recommends using strategies based on limiting the extent of localized failure. In practice, the structure subjected to accidental actions has to be calculated in two stages:

1) Structure dynamic response on an action with the impulse nature (e.g. explosion, impact) to be determined. In this case accidental loads are carried by the whole structural system including appropriate dynamic properties.
2) Structural robustness to be modelled in the case when one of load bearing elements could lose its capacity in stage 1.

This paper focused on the second stage of calculations. The case of sudden column loss on the lowest storey of steel framed building was considered. The strategy based on limiting the extent of localized failure was assumed [1]. Firstly, the static analysis of the structure in Autodesk Robot Structural Analysis Professional software was performed to achieve suitable cross-sections of frames members. Then, the static and dynamic GMNA analyzes (materially and geometrically nonlinear) of the structure in Autodesk Simulation Mechanical were carried out. Calculations were made in reference to plane frame, which is the repeatable load bearing system of a building analyzed in [3] and [4], according to the static approach.

2. Analysis methods

Safety assessment of steel skeletal structures with reference to codification of design rules was widely described in [5], [6]. Possible, but very simplified method to analyze the structural robustness of a building is the static analysis. According to [7], in this approach calculations could be divided into three study cases:

- analysis of degraded structure due to the loss of the internal column on the lowest storey,
- design of the key element regarding the recommended value of accidental load A_d,
- redundancy of ties to withstand the loss of any column on the lowest storey.

The static approach can be used in analysis of buildings in consequences class CC2 [5]. However it doesn't include the entire essential issue connected with the nature of analyzed phenomenon - the dynamic effect, which has to be taken into account during designing buildings in consequences class CC3. The loss of the capacity of element often happens in rapid way, which entails a sudden need to find the alternate paths of balance in structure. In this kind of situations the inertia of structure can have significant influence on forces distribution in load bearing components. Therefore the dynamic assessment of structural robustness on progressive collapse, widely applied and described, e.g. in [8] – [14] is more accurate approach for the mentioned problem.

3. Estimation of robustness on progressive collapse based on static analysis of frame

The single, repeatable steel frame with rigid joints (Fig. 1) representing the part of residential building (Fig. 2) was analyzed. According to Table A.1 in [1] this building is designated in the consequences class 2b (higher risk) and was widely analyzed in [4]: concerning design of the key element regarding the recommended value of accidental load A_d and analysis of degraded structure due to the loss of any column on the lowest storey. Naturally, including the connections between adjacent frames has substantial influence on obtained results – for example the transverse elements with length corresponding to spacing between each main frames and displacement boundary conditions could be added in model to take into account the catenary action associated with significant second order effects that plays an important role in resisting additional loads when structural column is destroyed under unexpected loads. Nevertheless, the simplification as an analysis of plane frame with connections between adjacent frames modeled as lateral restraints was assumed in this article.

Calculations of sway frame according to ultimate limit state in persistent design situation including appropriate imperfections resulted in members cross-sections presented in Fig. 1 (*Initial Frame*). Then the structure was analyzed including various scenarios of its degradation, which are showed in Figs 3 a–e [4]. Calculations were performed in Autodesk Robot Structural Analysis Professional software [15] with use of beam elements.

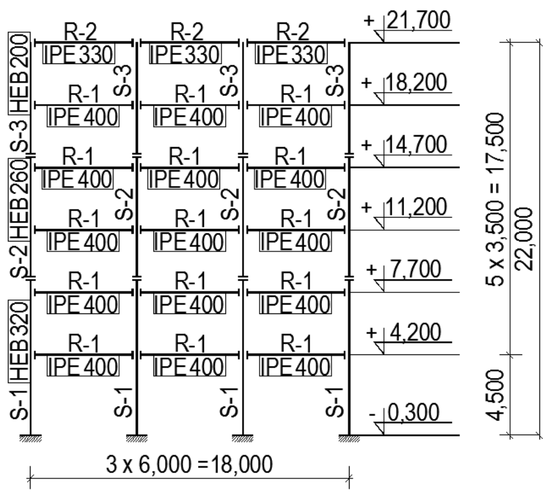

Fig. 1. Members of repeatable load bearing system resulting from ULS (*Initial Frame*)

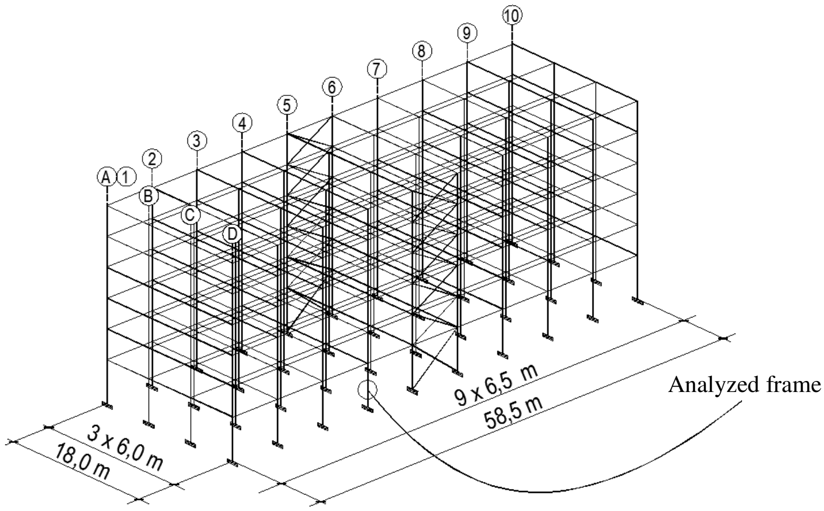

Fig. 2. Residential building taken into consideration in accidental design situation

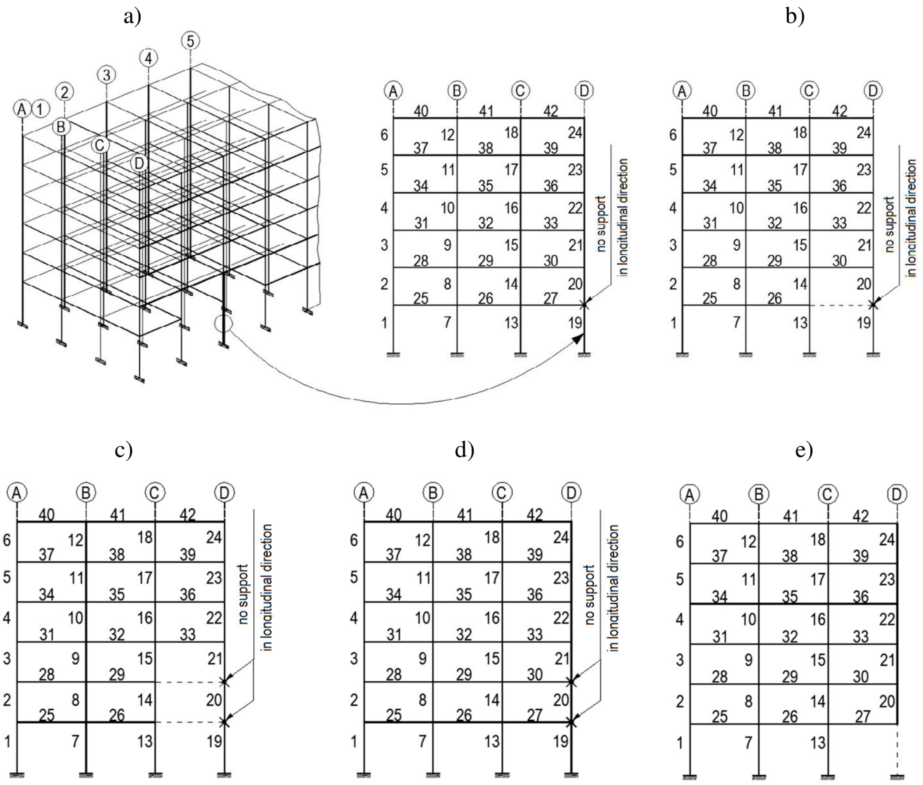

Fig. 3. Various scenarios of analyzed frame degradation due to external blast [4]

Static analyzes of degraded frame in accidental design situation resulted in significant increase of members cross–sections (*Upgraded frame*). Selected sections are shown in Table 1.

Table 1. Sections fulfilled Ultimate Limit States resulting from frame static analyzes

Frame	Main structure elements				
	column S1	column S2	column S3	girder R-1	girder R-2
Initial	HEB 320	HEB 260	HEB 200	IPE 400	IPE 330
Upgraded	HEB 500	HEB 280	HEB 200	IPE 750x173	IPE 330

Finally case e) (removing the side column on the lowest storey) proved to be most disadvantageous and was assumed to further analysis including dynamics effects.

4. Estimation of robustness on progressive collapse based on frame dynamic analysis

4.1. *Initial Frame* analysis

Structural system of analyzed building designed in accordance with ultimate and serviceability limit states in the persistent and transient design situations was taken under consideration. Geometrically and materially nonlinear analysis (GMNA) was conducted in Simulation Mechanical software, featuring integrated Autodesk Nastran FEA solver [16].

FE model of analyzed frame was made with the use of rectangular shell elements. Analysis of solution convergence including influence of discretization was carried out. The mesh size had been gradually decreased until it reached 40 mm, for which satisfactory solution convergence at acceptable calculation time was obtained. Finally the entire model consist of about 94 thousands of finite elements. However, in this case sufficient accuracy of dynamic analysis can be obtained using beam elements [8], [17], [18], shell elements were used in this study to more precise capture the potential plastic zones in most critical points of structure (Fig. 4).

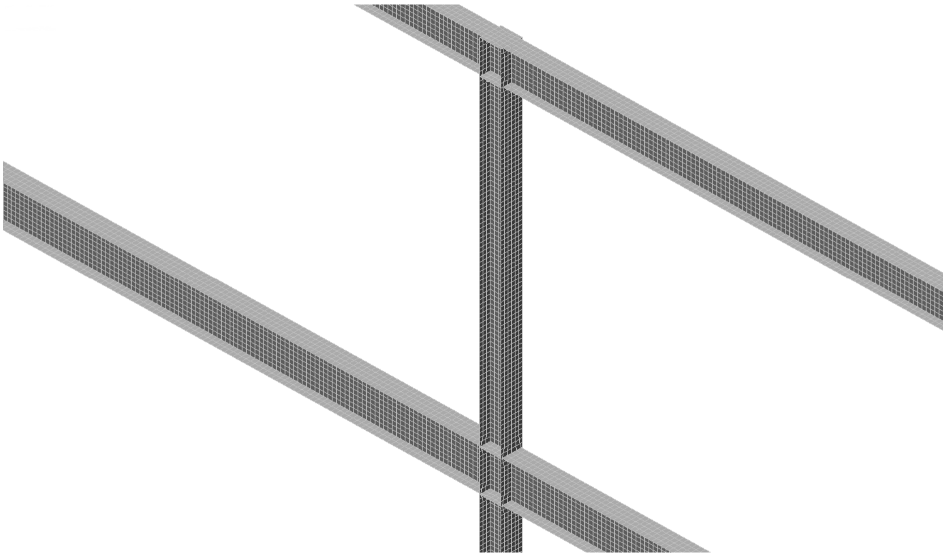

Fig. 4. View of the part of the FE model of *Initial Frame*

Dead load was modelled as lumped mass applied to top flanges of girders while live load as external load applied to top flanges of girders on side nave of frame (Fig. 5). Lateral restraints of girders' top flanges due to presence of floor slabs were assumed. To simplify numerical calculations, a side sway of the frame due to wind action was not taken into consideration. Furthermore, neither global nor local imperfections were included in analysis.

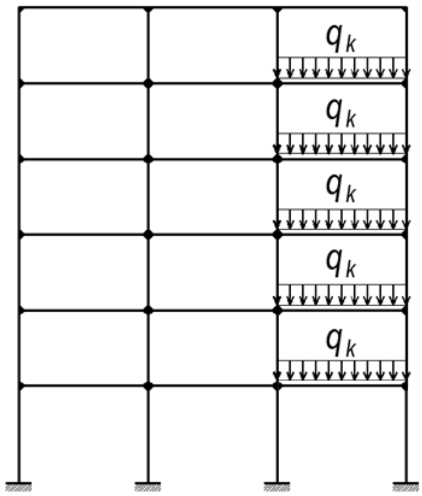

Fig. 5. Live load case included in frame dynamic analysis

Modal analyzes of complete frame and without a side column (degradated) were performed to determine natural frequencies and mode shapes. The analyzes were performed twice – including and not including mass of ceilings lying on girders. Obtained frequencies are shown in Table 2. Values of frequencies were used to determine suitable time step for dynamic analysis.

Table 2. Natural frequencies of the *Initial Frame*

Natural frequencies of analyzed structure [Hz]						
Mass of ceilings	included			not included		
Mode	1	2	3	1	2	3
Complete frame	0.51	1.45	2.53	3.09	8.43	14.31
Degraded frame	0.43	1.38	2.04	2.60	7.69	9.75

Based on results obtained from modal analyzes, the time step was set as 0,05 s [19]. Trial dynamic analysis revealed that further reducing of time steps had negligible influence on results. Implicit integration method available in Autodesk Simulation Mechanical was used in dynamic analysis.

The characteristic of the elastic-plastic material model with isotropic hardening [20] used in analysis is shown in Fig. 6 [21]. The following material parameters were assumed:

$\sigma_y = 235$ MPa,

$\sigma_u = 360$ MPa,

$\varepsilon_{st} = 0.02$,

$\varepsilon_b = 0.04$,

$\varepsilon_u = 0.30$.

Damping of the structure was defined with use of Rayleigh's method by setting the mass-proportional damping coefficient $\eta = 2.0$ and stiffness-proportional damping coefficient $\delta = 4.0$. This values were assumed based on probationary analysis, to achieve noticeable damping [22]. Correlation between this coefficients was obtained based on formula (1) for two first natural frequencies ω_1, ω_2.

$$\begin{bmatrix} \xi_i \\ \xi_j \end{bmatrix} = \frac{1}{2} \begin{bmatrix} \dfrac{1}{\omega_i} & \omega_i \\ \dfrac{1}{\omega_j} & \omega_j \end{bmatrix} \cdot \begin{bmatrix} \eta \\ \delta \end{bmatrix} \tag{1}$$

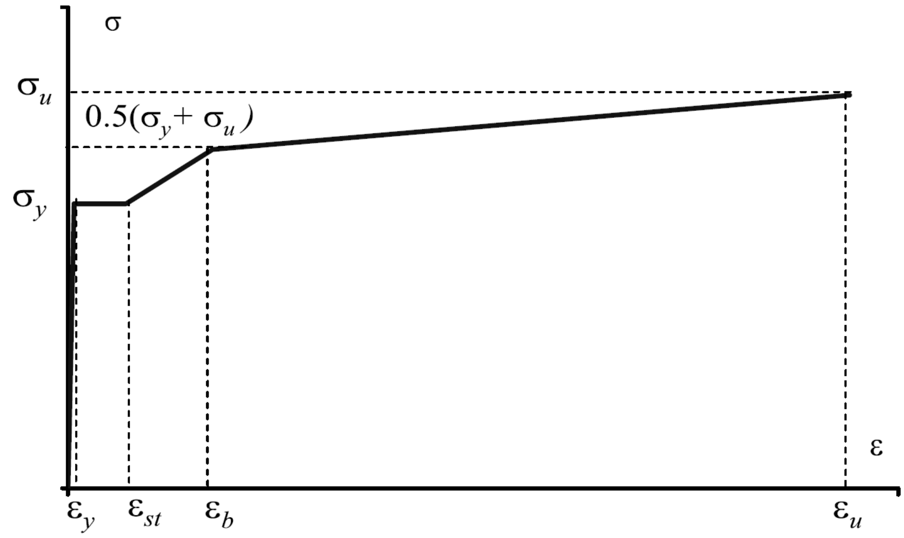

Fig. 6. Simplified stress-strain curve (in uniaxial tensile test) used in analysis, based on [21]

At the beginning of the analysis the frame was not degraded, so column was taking over all forces intended to it. Loads were applied in quasi-static way – dead load through first 10 000 seconds, live load through next 5 000 seconds and finally through another 5 000 seconds no additional load was applied, which was intended to stabilize the forces in structure (Fig. 7). In this part of analysis the time step was set equal 1 000 seconds.

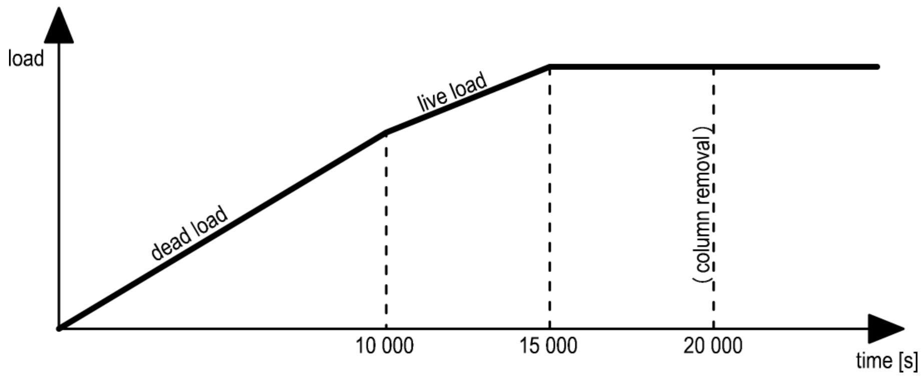

Fig. 7. Load multiplication factor curve in dynamic analysis

After described time the whole supporting constraints in side bottom column were removed rapidly during one time step and behavior of such degraded structure was analyzed. In this part of analysis time step was set as 0.05 s. Supporting zones of side girders started working above plastic limit after about 0.15 s (Fig. 8) as well as vertical displacement of side columns was growing very rapidly with average speed about 2 m/s. Analysis was stopped when the equivalent strain in mentioned zones exceeded 30% which corresponds to average relative elongation of steel samples during fracture (Fig. 9). At that moment of calculations, maximum vertical displacements amounted 1 105 mm and didn't stabilized, which means that the side nave of analyzed frame is going to collapse (Fig. 10–11).

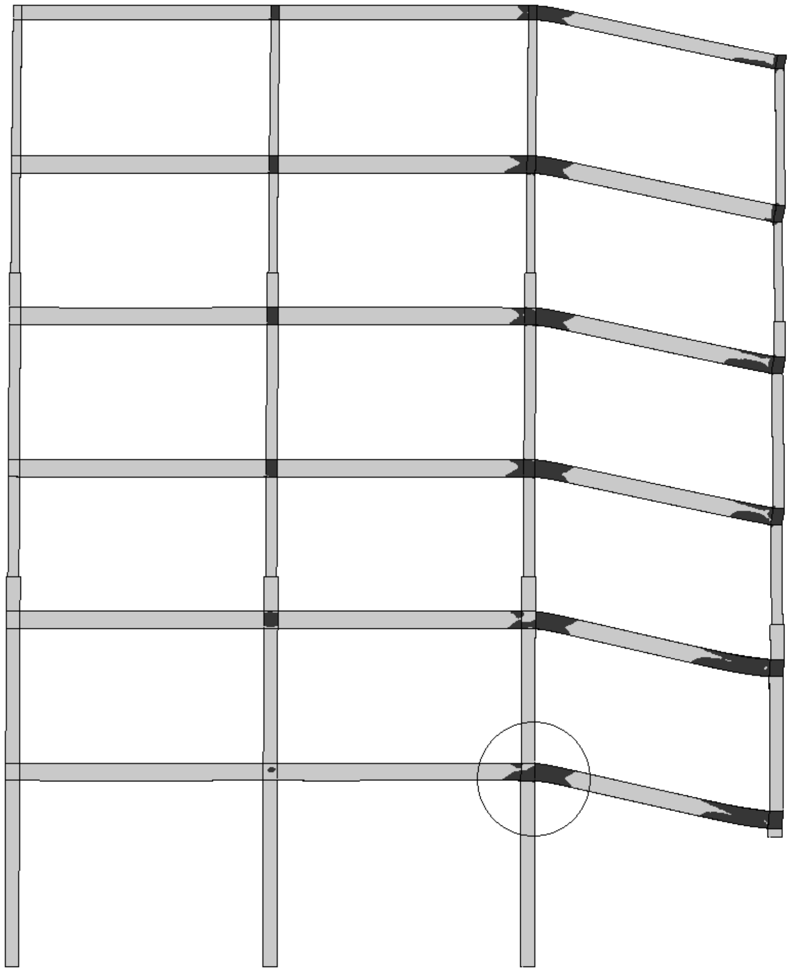

Fig. 8. Plastic zones (dark areas) in degraded *Initial Frame*

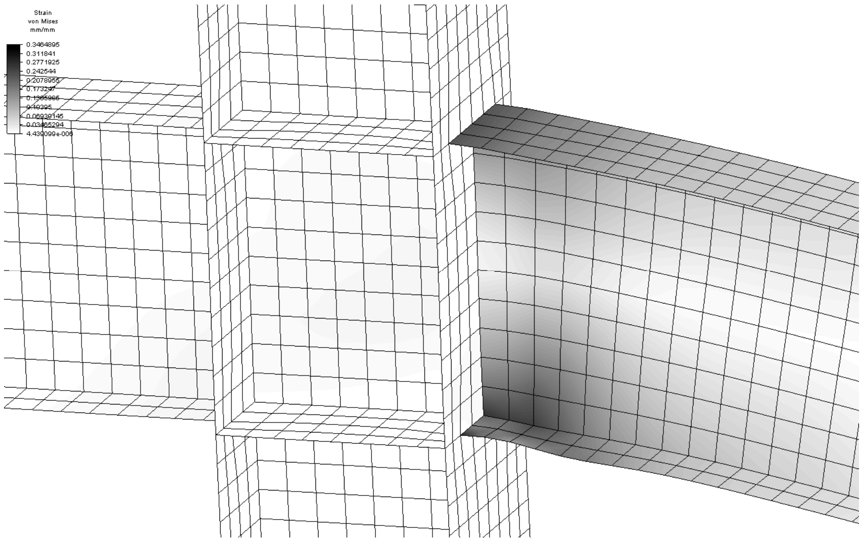

Fig. 9. Equivalent strain map around joint marked by a circle in Fig. 8

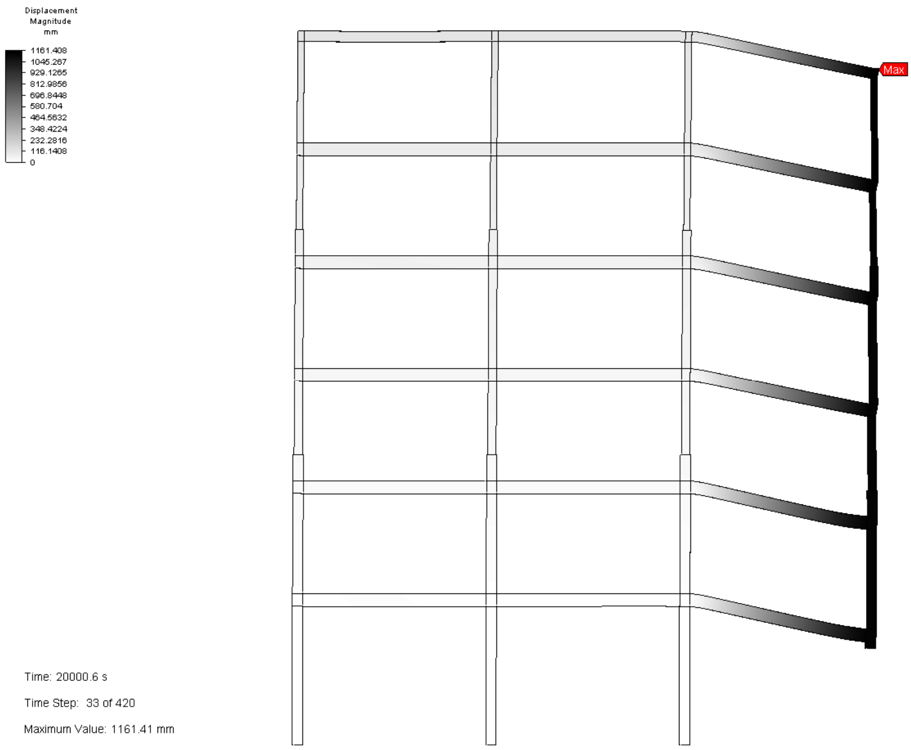

Fig. 10. Vertical displacement map in degraded *Initial Frame*

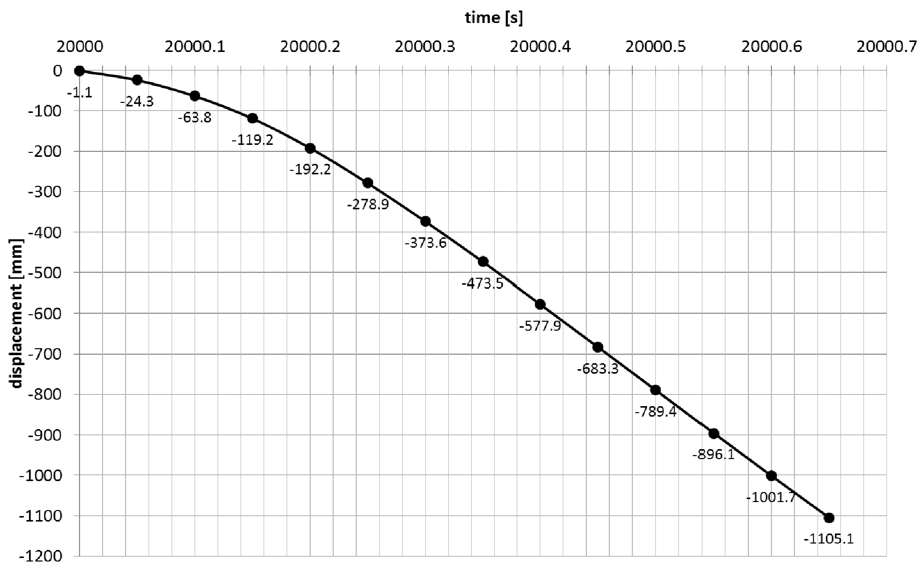

Fig. 11. Vertical displacement versus time of the point where the column was removed derived from dynamic analysis of *Initial Frame*

4.2. *Upgraded Frame* analysis

In this stage, structural system of analyzed building designed in accordance with ultimate limit state in accidental design situation was taken under consideration.

Analysis assumption remained the same as for *Initial Frame*. Natural frequencies calculated for *Upgraded Frame* are presented in Table 3.

Table 3. Natural frequencies of *Upgraded Frame*

Natural frequencies of analyzed structure [Hz]						
Mass of ceilings	included			not included		
Mode	1	2	3	1	2	3
Complete frame	0.88	2.18	3.45	4.24	10.59	13.55
Degraded frame	0.76	2.12	3.33	3.65	10.15	13.55

Based on results obtained from modal analyzes, the time step for dynamic analysis was set as 0.05 s, the same as for *Initial Frame*.

Analysis revealed that maximum vertical displacement occurred 0.5 second after column's removal. Inertia forces increased loads in comparison to static analysis, but induced only temporary plasticization actually only in connection between column S-1 (HEB 500) and column S-2 (HEB 200) in axis C (Fig. 12). Finally stresses stabilized on level about 205 MPa (Fig. 13).

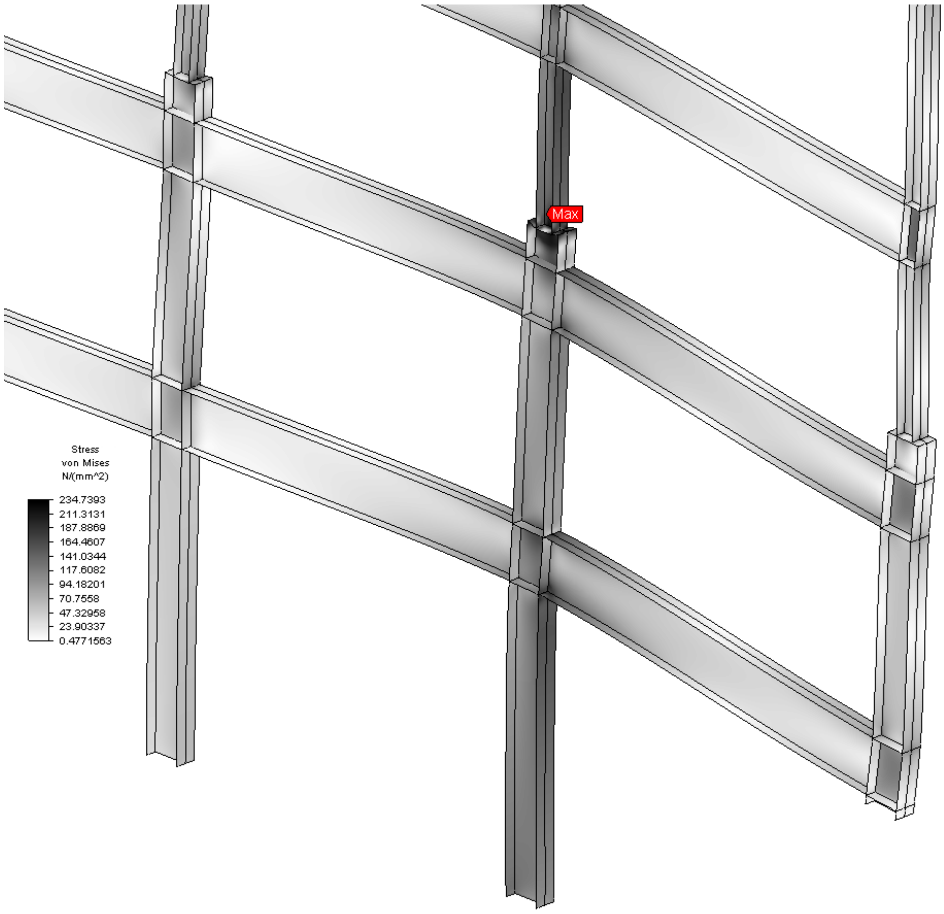

Fig. 12. H-M-H stress map in *Upgraded Frame* (0.5 s after column removal)

The maximum equivalent stress at the connection of bottom girder with side column was occurred a bit earlier – about 0.2 second after column's removal and reached value about 225 MPa and finally stabilized on about 160 MPa, which is significantly less value in comparison to 205.5 MPa derived from static analysis (Fig. 14). It proves that dynamic effects led to different way of forces distribution. Finally structure did not lose its stability, so it was resistant to assumed accidental action (side column removal).

Selected results of analysis are shown in Fig. 14–16.

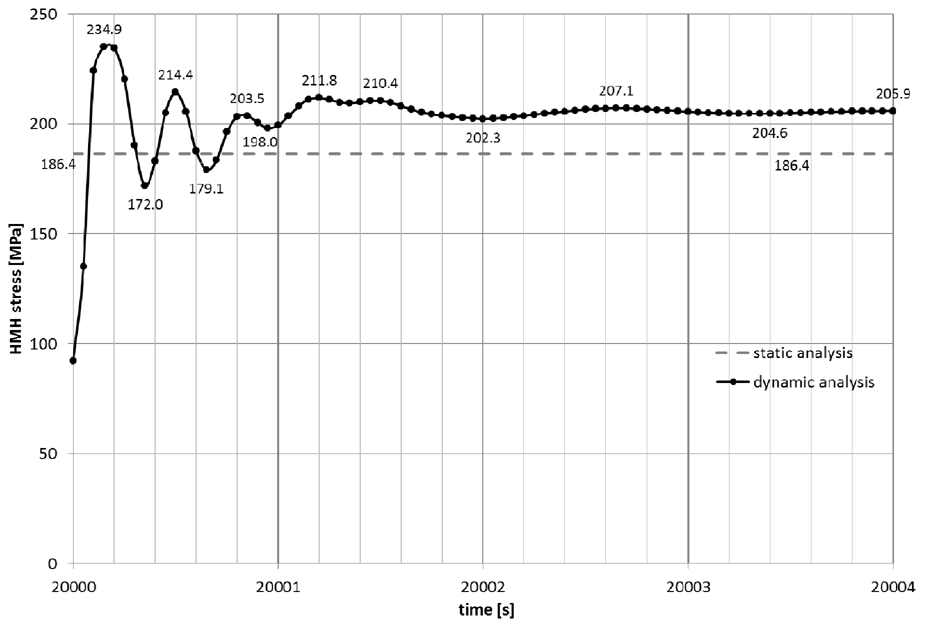

Fig. 13. H-M-H stress versus time of the point in connection between column S1 and column S2 in *Upgraded Frame*

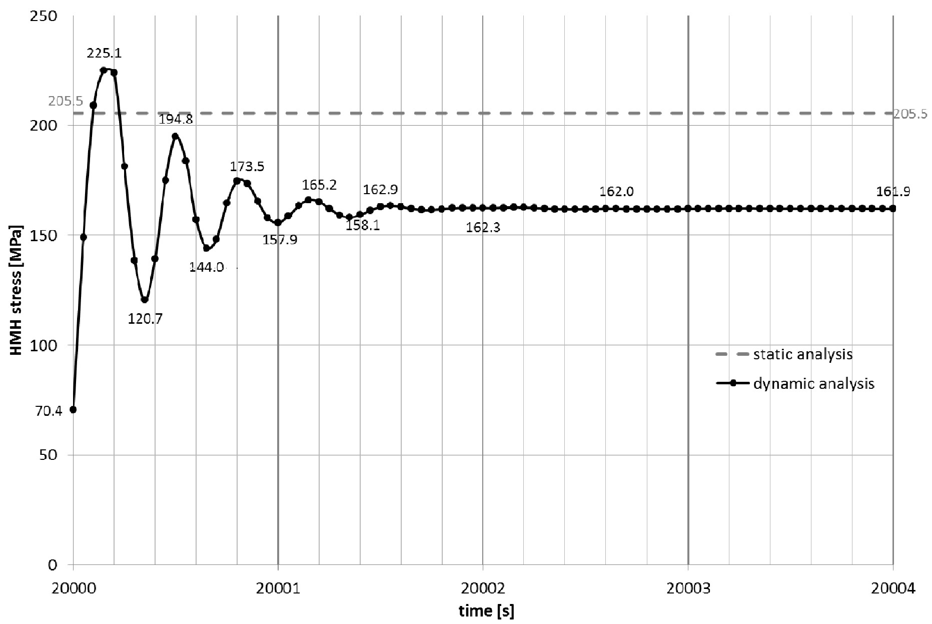

Fig. 14. H-M-H stress versus time of the point in girder-to-column joint in *Upgraded Frame*

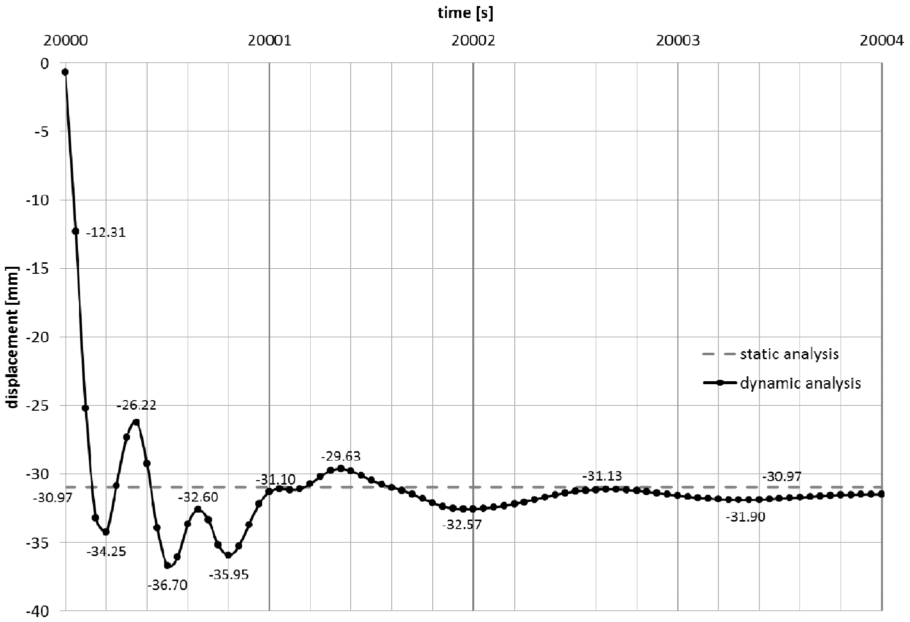

Fig. 15. Vertical displacement versus time of the point where the column was removed in *Upgraded Frame*

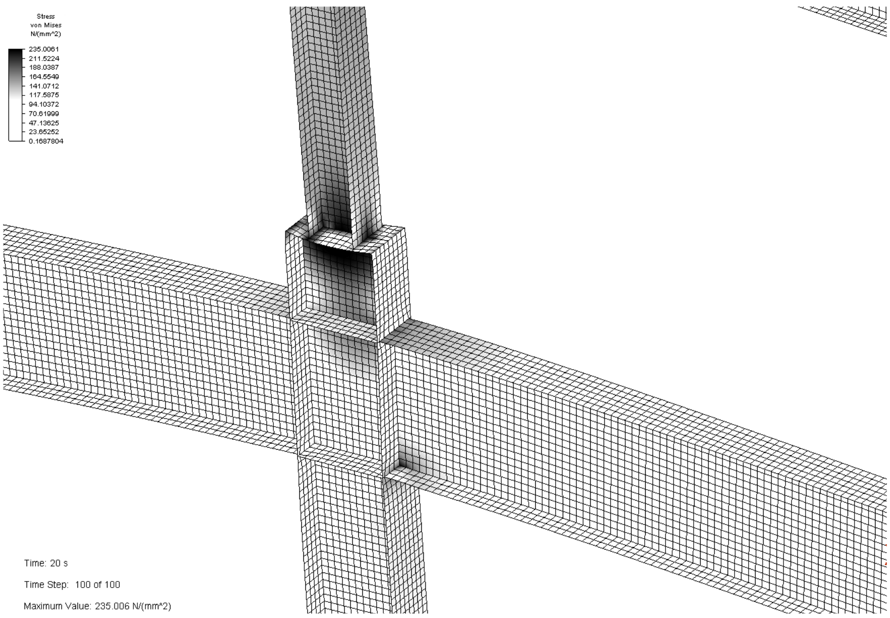

Fig. 16. H-M-H stress maps around column-to-column joint derived from static analysis of *Upgraded Frame*

5. Summary

Performed calculations presented in the paper revealed that sections of degraded structure's elements determined in static analysis (*Upgraded Frame*) using beam elements are sufficient to take over the designed load even with including dynamics effects of phenomenon. Simultaneously, frame with initial elements (*Initial Frame*) partially collapses in the case of side column loss. In comparison to static analysis a few percent increase of stresses and displacements was observed in dynamic analysis. In the case of plane frame with members of class 1 – 3 cross-section sufficient accuracy of calculations can be obtained using beam elements [8], [17], [18]. Despite of that, shell elements were used to more precise capture the potential plastic zones. Due to the necessity of the use of enormous sections, both for bottom columns as well as for girders, the possibility of the use of additional bracings to ensure integrity of structure subjected to accidental actions should be considered. Future research should take into consideration a spacious behavior of a structure instead of plane frame analysis. Furthermore, future investigations should also account for an influence of high temperature and distortion speed on material properties to improve accuracy of analysis results, as well as stiffness of steel joints (use of semi-rigid joints) and initial side sways due to imperfections and wind action.

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USING OF NEAR-CRITICAL FLOWS' THEORY IN PRACTICAL CALCULATIONS

The paper considers the problem of practical using of theory about near-critical flows. It describes the types of immovable and movable near-critical flow phenomena and cases of these phenomena formation during different hydrotechnical constructions operating. The paper gives generalized differential equation of free-surface profile of wavelike near-critical flows. The solution of mentioned generalized differential equation is given as well. The solution of generalized differential equation takes into account possible deviating from hydrostatic pressure in initial cross-section of considered flows. If the specifity of near-critical flows, especially wavelike free-surface profile and deviation of pressure distribution in initial section of considered flows, will not be taken into account, it can put to difference between designed and real hydraulic regimens. This factor can bring to miscalculation during designing, building and exploitation of hydrotechnical constructers. All that shows the issue urgency of near-critical flows characteristics determination and modelling for practical calculations. The equations for determination main depths (maximum and second conjugated) are given. Besides, the paper gives existence conditions of different types of near-critical flows. An objective of this work is to present the comparison between theoretical and experimental data of free-surface profile of cnoidal waves. The comparison shows good convergence of results.

Keywords: near-critical flows, non-hydrostatics, differential equations, laboratory researches

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1. Introduction

1.1. General comments

Near-critical flow is called free surface water flow that is steady with rapidly varied movement and depths, which are close to critical value, and also unsteady flow (translational wave) with rapidly varied movement and velocities, which are close to critical value [1]. Critical depth and critical velocity for two-dimensional problem can be found by such well known formulas:

$$h_K = \sqrt[3]{\frac{\alpha q^2}{g}} \tag{1}$$

$$c_K = \sqrt{g h_1} \tag{2}$$

where: α – Coriolis coefficient, q – specific water discharge, g – acceleration of gravity, h_1 – depth of undisturbed flow.

Given definition can be expressed by Froude criterion which are close to unit for near-critical flows. This criterion appertains to initial cross-section with minimum depth h_1. For steady flow, it equals

$$Fr_1 = \frac{v_1^2}{g h_1} = \frac{q^2}{g h_1^3} \tag{3}$$

$$Fr_1 = \frac{c}{g h_1} \tag{4}$$

where $v=q/h_1$ – flow velocity in initial cross-section of steady phenomena, c – movement velocity of translational wave front.

Near-critical flows have a number of characteristic properties which distinguish appreciably these flows from usual subcritical and supercritical flows with smooth or slowly varied movement. Such properties include wavelike or roller nature of free-surface curve, availability of inclination and curvature, and also non-hydrostatic pressure distribution in depth mainly in vertical section of these phenomena [1].

Nonsufficient investigation of near-critical flow, accuracy's low level of calculation data cause that near-critical regimes during hydrotechnical structures' operation are not recommended, or excluded at all by normative documents [2]. The methods to avoid the near-critical regimes during hydrotechnical structures' operation are not always apposite, these methods require additional costs, but sometimes the formation of these regimes is impossible to avoid. In these cases, it is needed to apply expensive hydraulic modelling of hydrotechnical structures' operation to provide a reliable solution of complex technical problems.

2. Types of near-critical flows Cases of near-critical flows' formation. Actuality of issue

Based on conducted experimental investigations [1] and analysis of many other scientists' publications, considering flows on horizontal (or slightly inclined) plane bottom without rapids and steps, it is possible to detach several types of immovable and movable (translational waves) near-critical hydraulic phenomena, which are shown in figures 1 and 2 respectively. It is necessary to note such things during consideration suggested classification of near-critical flows.

Roller conjugating of ponds without jump, classic hydraulic jump, and solitary translational wave with surface roller, which are shown in figures 1a, 1j, 2f, belong to domain of near-critical flow not at all interval of characteristics of their existing, but only when peculiar depths and velocities are close to critical values.

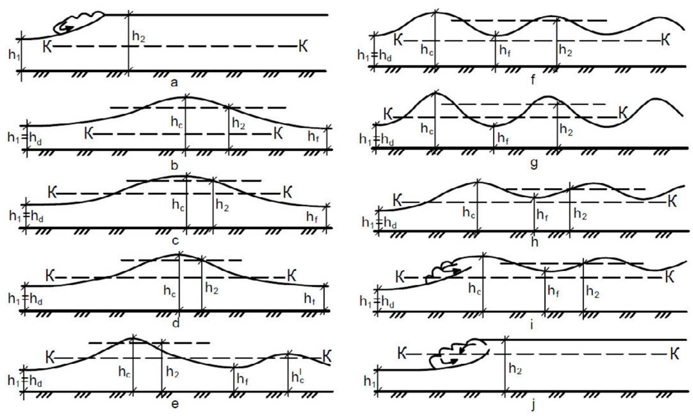

Fig. 1. Types of immovable near-critical phenomena: a – roller conjugating of ponds without jump, b – singular wave in subcritical, or critical flows, c – singular wave in supercritical flows, d – solitary wave, e – singular (solitary) wave with tail, f – cnoidal waves in subcritical, or critical flows, g – cnoidal waves in supercritical flows, h – undular jump with smooth surface, i – undular jump with surface roller on one or several wave crests, j – classic jump

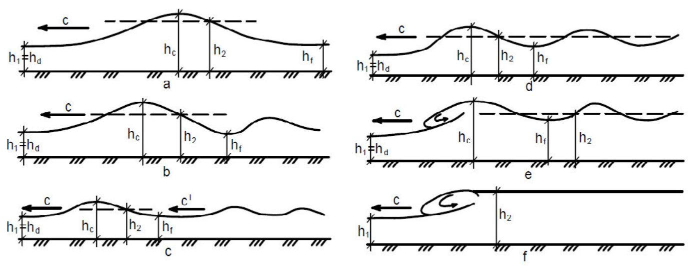

Fig. 2. Types of movable near-critical phenomena: a – solitary wave, b – solitary wave with tail, c – solitary wave with interrupted tail, d – grouped translational waves with smooth surface, e – grouped translational waves with surface roller on one or several wave crests, f – singular translational wave with surface roller (bore)

3. Cases of near-critical flows' formation. Actuality of issue

The near-critical flow may occur within different types of hydrotechnical structures: in tail water of water spillways, water outlets, hydropower plants, in channels, tunnels, passageways, pipes, during operating of geometrical shapes of flows, in the form of translational waves, etc. (fig. 3). Non-occurrence of general theory and reliable methods of near-critical flows' calculating, and indeterminateness of conditions of their existence are the reasons why near-critical flows sometimes aren't taken into account during designing. As a result many cases of damages and accidents of structures, which are operated in conditions of near-critical flows formation, happen. E.g. heavy damages of downstream floor were observed in water spillway Waco, shallow blankets were washed away of dams Krishna and Sardo, at a result, the dam Sardo was completely destroyed.

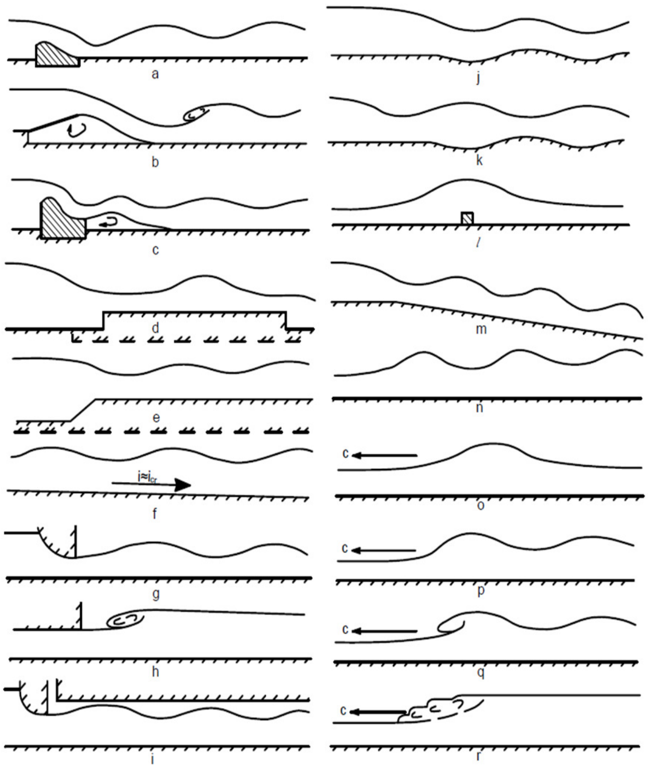

Fig. 3. Cases of near-critical flows' formation: a – after spillways, b – after regulator sluices and low pressure control structures, c – after spillways with drop wall and combine hydropower plants, d – on broad-crested weirs, e – at the inlet of channels or free-flow tunnels, f – in open channels, free-flow tunnels and pipes, g – during outflow from under gates with round or acicular configuration of bottom part, h – at the outlet of bottom discharges, i – in tunnels with boom, j, k – over undular bottom, l – during flow-around of bottom hindrance, m – at the inlet of inclined drops, n – during controlling of supercritical flows, o, p, q, r – in the form of translational waves

The damages of free-flow tunnels, that are Arpa–Sevan tunnel, Yalta tunnel, Spandaryan tunnel, Infiernillo tunnel, etc., were held fix.

For these cases of damages and accidents it is common that all of mentioned hydrotechnical structures were operated in domain of near-critical regimes' existence at certain stages. The analysis of operating conditions of free-flow diversion tunnels showed that Froude number in outlet section equalled Fr = 0.3 – 4.0. It shows that water flow was in domain of near-critical flows' existence. Obviously, noticed cases of damages and accidents of hydrotechnical structures happen as a result of large number of very different factors but these factors also include negative development of near-critical flow. Given information is declarative of actuality considered issue.

4. Generalized differential equation of the wavelike near-critical flow

Paper [1] shows generalized differential equation of the free-surface profile of near-critical flows with undular surface, which was developed as a result of investigation of near-critical flows' theory

$$h'^2 = \frac{3}{Fr_1}\left[-\eta^3 + (2\beta_1 + Fr_1)\eta^2 - (2\beta_1 - 1 + 2Fr_1)\eta + Fr_1\right] \tag{5}$$

where: h′ =dh/dx – first derivative of function h(x) at any point of free-surface curve, $\eta = h/h_1$ – dimensionless running ordinate of this curve, Fr_1 – Froude number in initial cross-section of considered phenomena which can be calculated by formula (3) or (4).

Integrating of this equation gives its general solution in the form of such system

$$\left.\begin{aligned} &\eta = \frac{h}{h_1} = 1 + (\eta_c - 1)\mathrm{cn}^2\left(\frac{x}{\Delta}, k\right),\\ &\Delta = 2h_1\sqrt{\frac{\eta_c Fr_1}{3(\eta_c^2 - Fr_1)}},\\ &k = \sqrt{\frac{\eta_c(\eta_c - 1)}{\eta_c^2 - Fr_1}},\\ &\eta_c = \frac{1}{2}\left[t_1 + Fr_1 + \sqrt{(t_1 + Fr_1)^2 - 4Fr_1}\right]. \end{aligned}\right\} \tag{6}$$

where: $\eta_c = h_c/h_1$ – dimensionless depth under the first wave crest, Δ and k – parameters of cnoidal waves.

It is necessary to emphasize that expressions (5) and (6) take into account possible deviating from hydrostatics in initial cross-section of near-critical flows. Such accounting is made by related coefficients of non-hydrostatics s_1, hydrodynamic pressure t_1, and potential energy β_1 in considered cross-section.

From the practical point of view, the most important matters among various manifestations of near-critical flows are determination of maximum depth, the existence conditions of different types of near-critical phenomena and the calculations of free-surface profile.

5. Determination of maximum depth

The maximum depth h_c of wavelike near-critical flows is one of the most important of their characteristics because this depth defines the upper level of side dikes of open channels, bottom level of bridge girders, the height of free-flow tunnels, pipes and galleries. The maximum height of such waves can outdo their average height over 60-80% [3]. This problem is more investigated for undular jump and solitary wave. In case of phenomena with surface roller the maximum depth is second conjugated depth h_2 which can be calculated by known Bélanger's equation.

The existed equations, which find the maximum depth h_c of undular jump and solitary wave, and second conjugated depth h_2 of hydraulic jump, have some imperfections:

- the formulas cannot be used for all domain of near-critical flows,
- the formulas cannot be used when Froude number is less than unit $Fr_1 \leq 1$,
- the formulas do not take into account the possible deviating from hydrostatics in initial cross-section of considered phenomena.

The developed theory of near-critical flow [1] gives equations, which determine depths h_1 and h_2, and avoids the above-stated imperfections:

$$\eta_c = \frac{h_c}{h_1} = \frac{1}{2}\left[t_1 + Fr_1 + \sqrt{(t_1 + Fr_1)^2 - 4Fr_1} \right] \tag{7}$$

$$\eta_2 = \frac{h_2}{h_1} = \frac{2}{\sqrt{3}} \sqrt{k_1 + 2\alpha_{01} Fr_1 - T} \cos\left\{ \frac{\pi}{3} - \frac{1}{3} \arccos\left[\frac{3\sqrt{3}\alpha_{02} Fr_1}{\sqrt{(k_1 + 2\alpha_{01} Fr_1 - T)^3}} \right] \right\} \tag{8}$$

where: α_{01} and α_{02} – coefficients of momentum in cross-sections with first and second conjugated depths respectively, T – dimensionless frictional force.

6. Existence conditions of different types of near-critical flows

From practical point of view the cognizance of reliable existence conditions of different types of near-critical flows is necessary to assign the favourable regimes of water movement through various constructions, to choose peculiar methodologies of calculations of considered phenomena's main characteristics, to determine the optimal size of structures, etc.

General imperfection of existed views on this matter is effort to find mentioned conditions only by one factor – Froude number Fr_1 in initial section of considered phenomena, and to except the influence of other factors. The disregard of possible deviating from hydrostatics in initial cross-section of considered flows can lead to absurdities and paradoxes [4].

The paper [1] showed that existence conditions of different types of near-critical flows should be characterized by not only one factor but two factors in their initial cross-section – by Froude number and one of the coefficients s_1, t_1, β_1, which take into account the possible deviating from hydrostatic pressure distribution in depth. This paper gave the existence conditions of different types of immovable near-critical phenomena by Froude number Fr_1 and coefficient of non-hydrostatics s_1.

7. Calculations of free-surface profiles

Practically the calculations of free-surface profile of wavelike near-critical flows are made for: determination of high-altitude size of constructions, determination of reach length of wave formation, operation of supercritical flows, checking of verity of developed theories of these flows.

Considered calculations are based on known differential equations of Korteweg-de Vries, Serre, Selezov, and others and use solutions of these equations in forms of solitary wave and cnoidal waves. In this connection undular jump and grouped translational waves are often considered as superimposition of mentioned waves' solutions. In this paper, the calculations were based on differential equation (5) and its general solution (6), which in explicit form takes into account possible deviating from hydrostatics in initial cross-section of considered flows.

8. Experimental validation of theoretical equations (5) and (6)

The complex laboratory investigations of near-critical flows were made in National University of Water and Environmental Engineering at four setups. During researches all types of immovable phenomena, which are shown in figure 1, were investigated. Considered phenomena occurred in two-dimensional conditions during outflow from under gates [1, 5], and also in three-dimensional conditions of single and double span regulator sluices with pivot-leaf gates [6, 7]. Each setup was equipped by system of bottom piezometers (all setups included about 346 piezometers). Existence of free-surface curve and piezometric line allowed to determinate the positions of sections with first and second conjugated depths, and also to calculate the coefficient non-hydrostatics in initial and other sections. Experimental setups and the investigated methodology were described in [1, 5-8].

Figure 4 shows collating of theoretical free-surface profiles of cnoidal waves, which were calculated by system (6), with experimental data. Adduced results and similar comparisons, which were made for different types of near-critical flows [1, 5-8], show their good convergence. It shows the principle verity idea about necessity of taking into account possible deviating from hydrostatics in initial cross-section of near-critical flows.

9. Conclusions

1. It is necessary to realize the calculation of free-surface profile, determination of the main characteristics (maximum h_c and second conjugated h_2 depths), and existence conditions of different types of near-critical flows with possible deviating from hydrostatics in initial cross-section of these flows.
2. It is recommended to take into account the results of developed theory of near-critical flow of this paper during making practical calculations of hydrotechnical structures.

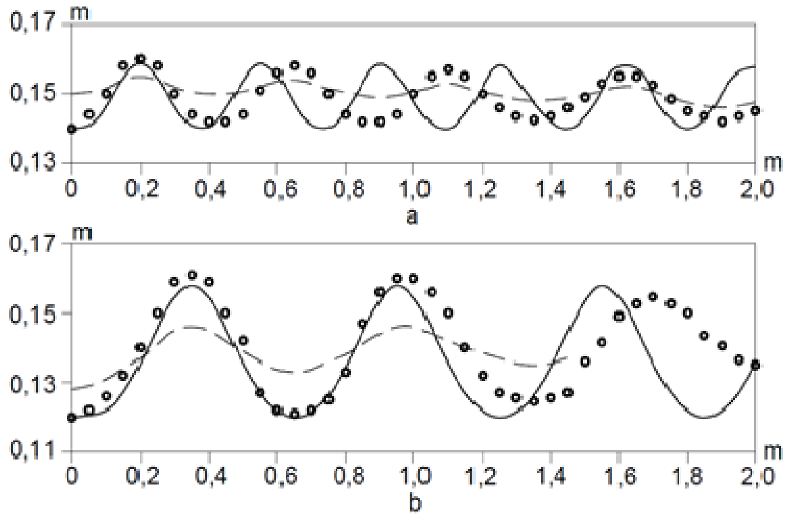

Fig. 4. The free-surface profile of cnoidal waves: a – q=0,094 m^3/s, h_1=0,14 m, Fr_1=0,322, s_1=1,05; b – q=0,111 m^3/s, h_1=0,12 m, Fr_1=0,722, s_1=1,06; 1 – piezometric line; 2 – fee-surface profile which was calculated by system (6); ○ – experimental data of free-surface

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PROCUREMENTS OF MODERN METHODS OF CONSTRUCTION BASED ON WOOD

In the last years we have witnessed increasingly frequent interest in an individual way of living in family houses. This provides a more natural way of living and living freely in contrast to the impersonal and restrictive living in panel housing estates. On this change largely responded companies offering a variety of system construction and technological solutions. With traditional and proven construction materials, the company also new, modern and fully-fledged alternative housing. One of them is the modern prefabricated structural systems based on wood. Even despite undeniable advantages that are associated with wooden buildings, preventing their more widespread low level of knowledge and awareness on the part of consumers and investors, as well as strong ties to traditional brick technology. An important factor in deciding the most building owners in choosing the construction of wooden houses is a measure of coping and recovery advantages of individual design systems that will mainly be reflected in the costs, quality and speed of construction. For this reason, we have decided to carry out a survey aimed at examining the impact of the procurement method on existing wood buildings in the context of construction time and acquisition costs.

Keywords: modern methods of construction, wood, construction costs, construction time

Generally, the modern methods of construction are technologies which make use of structures or their components manufactured in factory [1]. The production of more or less completed components of building structures in the plants has a high potential for increasing the construction efficiency at the production stage of building components as well as in the process of their integration in the site. The MMC [2] presents the technologies that provide effective procedures of construction preparation and execution, resulting in a larger volume of production with higher quality and reduced time of their

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procurement. The advantages of the MMC are shorter construction time, fewer errors in construction, and reduced demands on energy consumption or reducing of construction waste generation. Their ambition by [3] is to enhance the construction efficiency through reducing of construction time, improvement of quality, sustainability and impact of the building and of the building process on environment [4]. Authors [5] claim that the MMC in the construction industry have a higher productivity and better quality, as well as some benefits such as reduced construction time, lower overall construction costs, better quality, more durable and better architectural appearance, increased health protection at work and safety, reduce materials consumption, less construction waste, fewer emissions into the environment and reduce energy and water consumption.

A range of materials is used for MMC, the most common being wood, steel and concrete. The choice of basic building materials is a vital part of each project and is usually based on professional judgment taking into consideration the importance of such criteria as economic, environmental, functional, aesthetic and health-related [6]. Responses for efficient, economic and sustainable solutions are modern methods of based on wood. Regarding the modern methods of construction implementation in Slovakia, assembled buildings based on wood seem to be the most preferred construction system. This system is designed to build multi-storey buildings, apartment buildings, office buildings and houses [7]. By [8] they can be built as prefabricated panel constructions, framed constructions, timbered constructions, skeleton and half-timbered constructions. One of the advantages of wooden houses is the variability of structures and composition of the walls, which can be designed as a low cost, low energy and passive models. In addition, they are perceived as structures for the "healthy" housing, their main advantages are short construction time, lower the environmental impact of the construction and used materials, lower realization costs and costs of operation [9].

Despite the undeniable advantages associated with the use of modern wood-based construction systems, by Štefko [10,16] prevents a wider expansion of timber structures in the Slovak Republic from a low level of knowledge and information from customers and investors, as well as strong links to traditional brick technologies.

An important factor in deciding most builders when choosing a wooden construction system is the degree to master and reap the benefits of individual construction systems, which are reflected in the cost, quality and speed of construction. For this reason, we decided to carry out a survey aimed at examining the impact of the procurement method on the already wood constructions in the context of construction time and procurement costs.

This paper presents the partial results of the socio-economic exploration of modern wood-based construction methods. The results assess the impact of the procurement process on parameters construction time and the procurement costs of the wood buildings. The subject of the study was the real wood used already.

A total of 80 buildings were monitored on behalf of two of the most widespread wooden construction systems realized in Slovakia (Wooden frame system, Panel construction system). The comparison parameter was subjected to a correlation analysis to determine the dependence between the analyzed parameters.

2.1. Selected construction systems buildings based on wood

2.2.1. Wooden frame system

Wooden frame system originates from USA and Canada, where it is still the most widely used building system. The basic element of such a construction is supporting frame perimeter and partition walls of various timber profiles (Fig.1). Ceiling structure is composed of different profiles of timber and wood based materials. The stability is provided by the cladding of large agglomerated materials such as OSB board or gypsum board. Thermal requirements are secured by inserting thermal insulation (Fig.1). Standard construction of the walls is similar to panel construction system, but the individual elements and layers of walls are completed directly on site.

Fig. 1. Wooden frame system [10]

Construction and assembly of wooden frame system is less demanding on a large mechanization. All layers of the structure and operation of installations are carried out on site, resulting in higher labor demands a higher proportion of the on-site works. This causes a greater probability of low quality work, including the impact of climatic conditions [17].

2.2.2. Panel construction system

Panel construction system is a main off-site construction method based on wood. Structural elements - panels (wall, ceil, roof, gable, partition wall) are produced in different stages of completion in the production hall and subsequently transported to the construction site where they are assembled to the structure. Build-up process is characterized by speed and precision. The panel generally consists of a wooden frame of profiled timber, covered on both sides with large-scale plates, filled with thermal insulation material. Instalations are prearranged in the panels during the manufacturing.

Fig. 2. Construction of panel construction system [12]

Prefabricated construction panel system fully utilizes construction, manufacturing and assembly advantages of their production to the efficiency of the entire construction process. The key moment to increase the efficiency and degree of prefabrication is panel's finalization. Panel system has enormous potential for increasing efficiency in the design, production and construction phase [18]. Manufacturing can be automated, thus increasing the quality of production. Load bearing system of prefabricated wooden houses could be completed within a few days (Fig. 2). Other finishing and plumbing work follows the assembly of the individual elements.

On the basis of the correlation analysis, we found a statistically significant dependence between the method of procurement and construction time ($p = 0.5570$), the method of procurement and the procurement costs for procurement of wood building (EUR) ($p = -0.2776$), the method of procurement and the type of construction system ($p = 0.3553$). We also noticed the dependence between the type of construction system and the construction time ($p = 0.6903$).

A more detailed interpretation of the correlations between the construction system and the procurement of realization pointed out that the users of the panel construction systems prefer the realization of their construction mostly through the construction company and on the contrary, the users of the columnal wooden constructions used the way of realization self-help in combination with the realization of the construction through the construction company. A statistically significant impact has been observed between the type of timber construction system and the construction time, which suggests that panel timber constructions were realized in a shorter time horizon than a column construction system.

Table 1 presents a comparison of the average construction time of the individual construction systems broken down by the method of procurement, indicating the declared construction time from woodworking producers. The table shows the breakdown according to the method of procurement, due to the fact that correlation analysis revealed statistically significant differences in terms of type of construction system and method of procurement (P = 0.3553). Declared values of construction time parameters (Table 1) and procurement costs (Table 2) from producers are determined based on the findings made on promotional materials, websites and personal interviews with representatives of companies operating in the construction sector. From the findings from the mentioned sources, the most frequently mentioned declared parameters of timber constructions can be summarized as: construction time, investment acquisition costs and energy standard, which are subsequently determined by an individual arrangement, specified and anchored in the works contract. Manufacturers also state that the construction time of the assembled dwelling completely made depends on a number of factors such as the technology used, the size of the building, the number of floors, the severity of the foundation and the construction, and, last but not least, the annual construction period. Acquisition

Table 1. Analysis construction time of the comparative wood construction systems

Construction system	**Mode of procurement (number of buildings)**	**Average of construction time (months)**	**Construction time declared by suppliers (months)*** (complete building)
Panel construction system	Through the supply company (40)	**4.26**	3 – 6*
	Realization by self-help (3)	**7**	-
	Combination (2)	**10**	-
Wooden frame system	Through the supply company (20)	**10.47**	3 - 6*
	Realization by self-help (13)	**17.91**	-
	Combination (2)	**16**	-

*depending on the complexity of the project

costs as well as construction time depends on the particular technical and design. The material composition also has a significant impact on the price, the other cost is if you use a diffusion-sealed polystyrene-insulating construction and the cheapest rendering system, or if a wood-based thermal insulation with a vented wood facade is used in the diffusion-open structure. Of course, such qualitative variants apply to all construction parts of the building.

From the data in Table 1 it can be stated that the shortest construction time was recorded in the panel construction system in all three ways of realization compared to the comparative construction system. The representative of the on-site construction system (wooden frame system) is largely implemented on a building site with a higher workflow and a higher demand for craftsmanship of workers, not excluding the weathering effects of the environment. By comparing the average construction time of the construction systems and the declared construction times by the manufacturers it can be stated that only the panel construction system has actually fulfilled the predefined parameter.

Table 2 presents a comparison of the average procurement costs of individual construction systems in terms of conversion per m^2 of useful area. On the basis of considerable data dissemination, there was no statistically significant effect between the procurement cost and the building energy standard, therefore we did not calculate the recalculated cost per m^2 of useful space in terms of the energy standards of the monitored buildings in Table 2.

Table 2. Analysis procurement costs of the comparative wood construction systems

Construction system	**Mode of procurement (number of buildings)**	**Average of procurement costs** (EUR) per m^2 of floor space	**Procurement costs declared by suppliers** (EUR)* per m^2 of floor space plochy (complete building) with DPH	
		Overall without a difference in the energy standard	Low energy standard	Passive Energy Standard
Panel construction system	Through the supply company (40)	**933**	900 – 1200*	1400 – 1600*
	Realization by self-help (3)	**647**	-	-
	Combination (2)	**1046**	-	-
Wooden frame system	Through the supply company (20)	**925**	900 – 1400*	1400 – 1600*
	Realization by self-help (13)	**635**	-	-
	Combination (2)	**694**	-	-

*depending on the complexity of the project

By correlation analysis we recorded a statistically significant negative dependence between the method of realization and investment costs for the procurement of constructions (p=-0.2776), which means that if the construction was carried out by the supply company, the acquisition costs increased, whereas the decrease was made when the construction was realized either alone or in combination. These findings have also been guessed as they are a standard in practice. On the basis of the average values calculated per m^2 in Table 2 it can be stated that in almost all methods of realization of panel and column woodwork comparable cost of acquisition per m^2 was recorded, except for the combined realization of the construction.

In the present article, we analyzed the impact of the procurement method on the building time parameters and the procurement cost of real woodworks. They were analyzed two of the most widespread wooden construction systems realized in Slovakia (Wooden frame system, Panel construction system). On the basis of the correlation analysis, we found a statistically significant dependence between the method of realization and the time of construction (p=0.5570), the realization method and the investment costs for procurement of wood buildings (EUR) (p=-0.2776), the realization method and the timber construction system (p=0.3553). We also noticed the dependence between the type of construction system and the construction time (p=0.6903). The conclusions of the analysis of the assessed wood construction parameters point to the fact that the timber construction based panels are the most effective in terms of construction time and are realized through a supply company. The least efficient in terms of the construction period is the construction carried out by a combined construction method (a combination of the way of realization through a supplier company and self-realization). From the point of view of procurement costs, panel and column construction system were comparable in almost all ways of realization.

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TECHNICAL AND ARCHITECTURAL PROBLEMS IN REVITALISATION OF PREFABRICATED RESIDENTIAL COMPLEXES

The subject of this paper are the issues connected with revitalisation of prefabricated residential complexes built in industrial technology. The article focuses on modernisation of prefabricated residential complexes in terms of architectural and technical issues of the buildings. The reason for this is the fact that there is a systematic improvement in the quality of housing environment and attractiveness of housing areas and also the issue of revitalisation should be popularized. Popularizing this subject in Poland serves to refute false information that the exploitation of these residential estates is coming to an end. Current research confirmed with the technical expertise on possibilities of further use and an unflagging interest in old 'high-rise housing developments' may be an encouragement for revitalisation works. In this paper the attention is paid to technical issues of the housing developments connected with building defects, exploitation damage, diagnosis and modernisation of the buildings and also the architectural possibilities of new solutions within revitalization. Additionally, the paper presents the results of the survey carried out among the residents concerning the architectural aspects of Sadyba residential estate in Warsaw and diagnosis of some problematic issues.

Keywords: revitalization, modernization, thermomodernisation, prefabricated housing estate, technical issues, image

1. Introduction

In Poland, contemporary programs of revitalisation of the housing estates concentrate on the invested areas but also on the ones coming from the period of prefabrication. The main aim of this activity is the current maintenance of the building objects, including carrying out thermal modernisation as well as transformations in the housing environments leading to an increase in their attractiveness. Housing standards represented by newly built housing developments of various aesthetics and spatial options force to think about more thorough revitalisation activities in comparison with an older commonly used tissue so that the disproportions in the quality of housing developments would not

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show such striking differences. According to statistics, half of the housing stock in Poland are the buildings erected in industrialised technology and so far half of the population in the cities lives in this kind of housing estates. Striving for an improvement in the standards of housing estates is also a sign of the growing interest in high-rise residential estates, especially among young and elderly people due to their financial capability of purchasing such a flat. For these social groups the most popular are the flats of approximately 40 m^2, which are situated in a good localization with a suitable communication access and functional facilities offered by the city or near the city centre. Another important factor are the hygienic conditions of the blocks of flats, the size of green areas and their development, solar exposure, which makes up for the poor aesthetics of slightly varied housing developments. The issue of revitalisation of the housing areas concerns mostly the communication and parking problems, the access to services within the housing estates and attractiveness of green and recreational areas. However, it does not refer to the issue of the housing development itself or any spatial changes in the structure of objects and the aspects of architecture. Meanwhile, in other European countries the industrialised housing developments are the main object of basic transformations, connected with partial rebuilding, reducing the number of floors, partial or full demolitions. In Polish conditions such drastic activities are not possible because there are not any uninhabited buildings in the housing stocks. What is more, the government does not finance any modernisation activities. The subject of revitalisation understood as a transformation of the buildings is looking forward to being popularised and undertaken in the near future also due to a lack of technically documented constraints for the future exploitation of the objects. Taking into account the current situation and real possibilities, transformations may include new architectural solutions of the common areas, stairways, entryways, improvements in a functional quality of the entrance space and aesthetics of the facades but also improvements in the technical standards of the prefabricated buildings. In this paper some of the architectural and technical issues concerning revitalisation of the prefabricated housing estates are presented, which are necessary for their further exploitation and improving the conditions and comfort of living in the buildings.

2. Basic concepts in revitalisation of the housing spaces

Revitalisation is commonly defined as ‘bringing back to life’ or a renewal of the neglected parts of the cities with the help of many coordinated undertakings in social, economic, spatial and ecological spheres. Revitalisation of housing developments built in the industrial technology is directly connected with the degraded housing development, neglected surrounding areas, a program of services functioning in the housing development and a social program concerning inhabitants of the housing estates. The activities should be carried out in many precisely defined areas such as technical, architectural, urban and social ones. They also should be done in accordance with the scale of the destruction of the tissue or depreciation of the space and the inhabitants' needs. Modernisation of the buildings itself and their infrastructure is a separate issue connected with a technical condition of the buildings. In Polish conditions, it is difficult to carry out modernisation due to a lack of possibility to do any internal works directly in the structure of the flats and buildings.

The concept of 'revitalisation', according to 'The Revitalisation Act', is understood as 'a process of moving the degraded areas out of a state of the crisis, conducted extensively through integrated activities dedicated to a local community, space and economy, focused territorially and carried out by the interested parties of the revitalisation.' [1] Although the law does not directly refer to prefabricated housing estates, it clearly defines the degraded and revitalisation areas. The degraded area, according to the law, is the one with the concentration of the negative social phenomenon, but also spatial, functional and technical ones: 'degradation of the technical conditions of the building objects, including the ones serving the housing purpose, and non-functioning of the technical solutions enabling an effective use of the building objects, particularly in the field of energy-saving and environmental protection.' [1]

Modernisation is the concept commonly applied and functioning interchangeably with revitalisation. However, it has a completely narrowed meaning, referring to the technical side of the objects and the elements of infrastructure. Modernisation is part of the revitalisation activities of residential estates and might be understood in many ways, for example, as beneficial changes, modernising a building object, enhancing its value of exploitation, functioning and aesthetic values in accordance with current trends or fashion. However, this concept does not exist currently in the Building Law and various terminology is used interchangeably, such as extension, transformation, or renovation (as restoring the original conditions, not being the current maintenance). It is not correct to use the concept of 'modernisation' in terminology connected with building activities and administrative decisions. [2] A similar case concerns the word 'restoring', which is replaced with 'renovation' in the nomenclature.

Revitalisation is an element of the whole socio-economic strategy of urban development and one of its aims is the improvement of living conditions and housing environment. The chances for success have only the activities, which include cooperation between many partners such as authorities, social organisations, local councils, housing co-operatives, local communities and other participants, for example, designers, contractors, and coordinators. The key to success is the ability to cooperate together with particular local partners and creative involvement in the transformation process.

3. Review of the systems and technical issues in conducted modernisation of the buildings

Using multi-dimensional prefabricated elements in housing developments has been observed in Poland since the second half of the 50s. The first ones were prefabricated hollow core slabs, so called, ‘Żerańska brick’[3]. In 1957 the first building of large prefabricated concrete panels was erected in the PBU-Jelonki system and in 1961 in the WUF-T system. We can talk about some regional systems, which spread in the 60s such as Winogrady, Domino, ‘J’, and, which operated with sets of finished buildings. Central systems of large prefab concrete panels, such as, OWT-67 and WUF-T were also produced, which offered sets of components, prefabricated elements within typical units and buildings. In 1967 so called ‘open’ systems appeared, for example, ‘Szczeciński system’ and W-70.

Their characteristic feature was the base (type series) of the prefabricated components and typical connectors, which gave the possibility of constructing flats and buildings with different functional layouts. [4] It might be mentioned that in Poland from 1970 till 1989, 24 systems of large prefab concrete panels of housing development, in their basic and modified forms were used [5]. After the period of fascination connected with planning huge residential estates and numerous realizations of spatial development strategies, the time of ‘doubts came, which brought the nostalgic longing for lost values of the urban space’, and it was followed by a wake-up call in the 70s [6].

‘Production’ of the space full of monotony created by the system, turned out to be faulty. Large-panel building systems contained many mistakes in their construction and utility. Because the anonymity of the tower blocks and flats was criticized, more distinct character of the buildings was introduced by individualisation of the details of entrances or balconies. It was difficult to eliminate the anonymity of the flats due to a large number of similar features in construction methods and repetitive layouts of the flats. [6] In the 90s, when the production of the prefab systems was finished, time for conclusions came.

It was then that technological and construction defects appearing in the buildings were emphasized together with bad functionality of flats. The assumptions of modernisation defined at that time described works, which should be carried out to maintain the buildings in such conditions so that it would still be possible to use them and to ensure the inhabitants about the elimination of any ‘defects’. Transforming, modernising and upgrading the buildings was the main subject of many thematic conferences,[5] reconstructive pilot plans and activities within the international cooperation with Germany, which after 1989 introduced a widespread revitalization program of housing estates.

Thermomodernisation of buildings was a very common activity, which in the 80s became more complex and was conducted in order to lower the costs of exploitation. Building envelopes, external walls, constructions of the flat roofs walls and the ceilings of the basements were insulated and installations were modernised. It turned out that the main defect of building the walls were thermal bridges and their influence on yet very low thermal insulation of the building envelopes in buildings constructed of large prefab concrete panels. The situation was regulated by the law on supporting thermal modernisation works in December 1998. It enabled cooperative societies to apply for publicly available investment loans to carry out modernisation works connected with enhancing the insulation of the building envelopes and modernisation of the internal central heating installations. The research conducted in the late 90s by the Building Research Institute on buildings of large prefab concrete panels proved that except of insulations of the building envelopes it is essential to limit 'the exchange of the air in the flats and stairways' and replace the glazing with better parameters of the U-value. [3]

In March 2009, the Act on supporting thermomodernisation and renovations came into force [7]. The law in its new version, together with suitable regulations, defined the activities and regulated procedures of conducting energy and renovation audits.[8] The law also defined a 'thermal modernisation project' and it was understood as an improvement to 'lower the energy demand supplied for the needs of heating and warming drinking water (…) , an improvement, as a result of which, there is a reduction in the loss of primary energy in the heating networks' and centralisation of the heat sources for residential buildings and so called collective dwelling. Under the Act 'a renovation project' is about renovations connected with thermomodernisation of multi-family residential buildings, including balconies, glazing replacements, supplying installations and necessary equipment but also 'transformations of multi-family residential buildings, as a result of which, their improvement takes place.' [7]

In 2015, the Revitalization Act, which was mentioned before, came into force. It presents a set of general rules of conducting this process putting the emphasis on ensuring social participation and preparing detailed analyses as the base for corrective actions. The act does not regulate, however, any particular rules of carrying out revitalisation. It does not refer to any financial issues and does not mention appointing a person, so called, 'revitalisation operator', to carry out any activities in the best public interest. For the first time the law mentions the issue of revitalisation, referring to the degraded areas, so it gives a chance for a reversal from the negative trends, for instance, by restoring the greatness of residential areas and also improving housing standards while taking into account the needs of local communities. [9]

Summing up all the activities, which have been done so far in the field of revitalisation, it should be emphasised that in Poland, except of thermomodernisation, any other works, neither complex nor partial, connected with the planned revitalisation of prefab housing estates were carried out. However, even the issue of thermomodernisation raises many doubts. Firstly, such activities require continuation and improvement and, secondly, they do not have any particular influence on the changes when the character of housing developments and their aesthetics are considered, let alone the improvement in living standards. They just serve to keep the buildings in a condition that is suitable for use.

Thirdly, the support of housing co-operatives in the field of thermomodernisation have not been sufficient enough so far and these works were just odd activities, often carried out thanks to financial resources obtained by the heads of the co-operatives boards. What is more, the country policy in terms of financing, turned out to be insufficient and made it impossible to carry out thermomodernisation works at the expected level. [3]

Referring to the work entitled ‘Precision of Completion and the Necessity of Modernisation of Large-Panel Buildings’, in which the authors made an analysis of sample thermomodernisation works in the prefab buildings from 70s, 80s and 90s in the residential estates in the Lubelskie region, some conclusions might be quoted. Thermal insulations of the facades were carried out in the ‘light-dry’ method from mineral wool and was coated with concrete slabs on the steel or wooden reinforcements or in the ‘light-wet’ one, which was insulated with Styrofoam and covered with thin-coat plaster on fibreglass mesh. It turned out that the ‘light-dry’ method was better because it provided better ventilation and after thermal insulation there are not any problems with mould and the rooms’ humidity. After conducting the analysis of modernisation works in the prefab buildings, it might be concluded that the applied method of thermal insulation turned out to be merely satisfactory. However, heat losses were observed in a vertical and horizontal directions in particular zones, for example, through joints with the basement ceilings, in the areas of plinths and walls adjacent to the elevators, near cantilevered balconies and window frames, jambs, cornices, concrete lintels and at joints with insulation panels. This means that thermomodernisation solutions are not complete and require further improvement, possibly another thermal insulations and eliminations of new point and linear thermal bridges, and particularly ‘bonding texture layers with the construction layer of the slab and modernisation of the building by using mechanical heat recovery ventilation’ of the prefab buildings. [3] All activities should serve to lower the maintenance costs of the prefab housing stocks and increase the standards of their further exploitation, especially in technical and energy terms.

4. Diagnosis of destructions and current trends in revitalisation of the residential estates in Poland

Current trends in revitalisation of the prefab housing estates are focused on the activities, whose aim is to carry out a credible diagnosis of functional and technical issues. The research serves to refute the opinion about the forthcoming end of their performance abilities and is also the basis for elimination of any defects and modernisation of the buildings. According to a lot of studies conducted by the Building Research Institute, there are not any indications that the end of exploitation of large-panel constructions and tower blocks is coming in the nearest future. Durability of the buildings, defined as the ability to fulfil certain required functions, may be estimated to be at least 100 years, however, the exception are the prefab components, as defects in their production and assembly are found and also some negligence in their maintenance and repairs. [10]

Nowadays, the buildings should undergo technical inspections and maintenance controls, especially when it comes to components exposed to the negative effects of atmospheric conditions and factors appearing during their exploitation. Current requirements in terms of energy-saving and thermal insulation parameters are more restrictive than they were when the buildings were erected. That is why, another current thermo-insulation works are carried out in the building objects, which previously underwent thermodernisation. However, such activities may lead to an excessive load on the hangers of concrete texture layer and, as a result, it might be necessary to reinforce their connectors. [10]

According to available technical publications, large-panel buildings are stable objects in terms of their durability and do not pose any direct threat to residents. The damage caused by improper exploitation may be divided into two groups, the traditional one related to partition walls, coatings, ceilings and installations but there are also destructions caused by technological errors relating to internal and external walls, ceilings and balconies.[11] The assessment of technical conditions and safety of construction of prefab buildings (conducted by BRI) proves that the biggest problem are the triple-layer prefab components of external walls. The most common destructions include falling off the texture layers, decrease in insulation of the warming layer, defects and wearing of the joints, faulty binding of the prefab components by connectors and their corrosion, destructions of ceilings, external wall elements, joinery of installation, and the areas of lifts and rubbish chutes. [12]

One of the basic ways of assessing the buildings' safety is the diagnosis of the connectors of the prefab components and hangers in triple-layer external walls. The inspections of constructions, analysis of destructions and cracks are done by experts through cut-and-cover and visual methods. They mainly rely on taking samples for tests, analysing the results and making statistic calculations. Current non-destructive diagnostic tests involve scanning walls and ceilings' surface, defining position and diameter of reinforcement, the condition of anchor bars and their placement and they also predict where the corrosion of reinforcement may appear. These are the methods, which have the future in the diagnosis of destructions due to the fact that residents often do not approve of doing cuts in the walls of inhabited flats. The conducted diagnosis of different types of prefab buildings shows that construction safety after the years of use is not threatened. However, experts do not rule out possible dangers in case of malfunctions, which took place at the stage of production, assembly, during improper exploitation and maintenance of the buildings. [13] The trends in revitalisation of building objects move towards implementing a program of complex renovation works of construction systems preceded by diagnostic tests. In Poland diagnostic and material tests of destructions and technical conditions of housing developments should be carried out in accordance with current standards of building inspections. Renovation and modernisation works should include such elements as an improvement in the safety of using prefabricated slabs, fixing balconies, loggias, and also complete thermal insulation of objects. There should be an improvement in ventilation systems and airtightness, a replacement of installations and heating equipment, an improvement in energy balance of objects and microclimatic comfort of the flats but also in the image of architectural objects. [14]

5. Problematic architectural aspects

Architectural transformations within revitalization are an indispensable element of improving living conditions, the comfort of using the objects and their aesthetics. In Polish conditions, any changes interfering with individual flats are not possible, only transformations in the areas of common space are allowed. It is possible to implement functional changes within flats, their connections, extensions of stairwells, lifts, transformations of the ground floor entrance areas, building additional rooms for bikes and prams and improving the access for the disabled. Additionally, changes in buildings' facades and possible ceiling transformations, for example, in the form of extensions or new roof constructions, are necessary.

While executing functional changes, there are new holes made in internal load-bearing walls of the buildings and that is allowed by the construction of a building. It is followed by stiffening the walls with new holes only through internal load-bearing walls. What is more, a load-bearing wall may be treated as stiffening one by increasing the capacity of a load-bearing internal wall. Inner holes may have bigger width while external strip should be kept intact by the edge of the construction. [15] Changes in the facades and flat roofs should be carried out within the load-bearing capacity and should require in-depth constructional analysis.

6. The example of Sadyba residential estate in Warsaw

In practise, revitalisation of the prefab residential estates is not very common in Poland but it is very up-to-date and needed. To popularize the possibilities of architectural transformations and to familiarize the residents of the estates with the problems of their living place, a survey was conducted within the seminar "Revitalisation of the Housing Development" by the students of Architecture at Warsaw University of Technology. It concerned the quality of housing environment, conditions of building development and their aesthetics. The residents answered the questions on such issues as: the conditions of building development and their aesthetic preferences. On this basis, the key problems were defined. Additionally, there was an inspection of eastern part of the housing development and some 'sensitive' areas. It resulted in creating a base of solutions of architectural concepts, which were put forward by the students.

Sadyba is the residential estate, which was built at the end of 60s, in the typical housing development in the large-panel construction system 'Ż' - Żerańska brick. This system was based mainly on wall slabs made of aerated concrete and 1.5 meters wide ceiling channel slabs. The internal walls were made of ceramic brick, which at the same time allowed quite big freedom in creating solutions for flats and buildings. The use of a typical construction of the blocks of flats in Sadyba was balanced with developed arrangement of greenery and recreational areas. Some of the buildings had flats for disabled people, which was an innovative approach towards design issues. Currently this housing development is in a quite good technical condition. It represents typical and not varied facades and also monotonous architecture of the blocks of flats.

The conducted survey was multi-range but in this paper only the results concerning the housing development itself will be included. 90 people took part in the survey. The residents expressed their opinions on such topics as the aesthetics of the buildings, their technical conditions and preferred modernisation changes. The results revealed that according to the residents the aesthetics is rather good (58.2%) and almost 10% described it as very good. The visual image was evaluated as rather bad by only 13.2% respondents and bad by almost an equal number of 15.4%. The residents also assessed the technical condition of the buildings and for majority of respondents it is very good (21.1%) and rather good for 63.3%. When the questions about the buildings and flats are considered, the residents would be interested in enlarging the size of their flats (26.7%), functional rearrangements (15.4%) and equally important is improving ventilation systems (17.4%) and acoustics of the buildings (17.4%). The residents expressed their willingness to insulate facades (14%), change the size of the balconies (12.8%) and showed a significantly smaller interest in enlarging the windows (7%). The biggest group of respondents did not have the opinion on the subject of transformations inside the buildings (40.7%), which is quite a sad result. When the question about the expected changes in the staircases and entrance areas is considered, the residents were mostly interested in building more lifts (42.2%) and 28.9% expressed their willingness to make rooms for pushchairs. It means that the residents realize that it is necessary to transform entrance halls of the buildings in order to improve the comfort of living. However, only 6.7% respondents wanted to enlarge the entrance enclosure zone of staircases and 14.4% people were interested in building driveways for pushchairs. A big number of respondents did not have any expectations (25.6%) regarding the changes of entrances in the common space.

The survey revealed some discrepancies between the residents' opinions and real assessment of the needs in the diagnosis made by independent observers. It shows that a flat is still a deficit commodity in Poland and the residents feel some concern about losing it or the necessity of changes in their housing environment. The residents do not hide their concern over an increase in rents and any other costs connected with maintaining the objects and residential infrastructure, which may appear after revitalisation. Lack of consistency in opinions is also connected with a low level of awareness of current standards functioning around the world and expectations, which should be fulfilled by housing developments in cities.

7. Conclusions

Revitalization of the prefab residential estates requires the attention of the government, coordinated programs and financing the activities connected with the improvement of the whole housing environment. Complex modernization works preceded by diagnosis of technical conditions require bigger participation of the owners and administrators. What is more, they should be carried out on higher constructional and more functional levels concerning at least common space in the buildings. During technical inspections it is necessary to check critical elements obligatorily, which are connected with the prefab system, construction and exploitation level of substances. A system method for diagnosis within created standards should concern the technical and energetic issues but also living comfort and even the image of architectural objects. Architectural changes are an indispensable element, which is integrated with modernisation of the building structure. No matter what the residents' opinion is, communities of experts should popularize this issue and revitalization activities in order to adjust the prefab housing estates to modern standards.

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SYSTEM OF HEAT SUPPLY *'ad hoc'* WITH SOLAR WALL

For the implementation of solar thermal installations it is necessary to optimize energy-efficient external enclosures due to the correct installation and design of these units at different orientations of external walls. Investigated the effectiveness of using solar energy in the conditions of shortage of energy in Ukraine. *Ad hoc* solar wall look at two modes of its operation. Given a variety of graphical, analytical expressions for understanding the operation of the proposed design. Given these changes of the heat carrier temperature solar wall income the amount of the specific instantaneous heat output through time. The article considers the possibility of using experimental models, solar wall under the influence of her unfavourable *ad hoc* factors. It is established that the proposed model is solar wall is quite effective and can be used in solar heating systems.

Keywords: solar wall, temperature, specific heat capacity, a coefficient of performance system

1. Introduction

One of the important issues of energy policy is the expediency and economic efficiency of energy use for technological processes in different

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branches of industrial and agricultural production. In addition to this, an important issue is the reduction of anthropogenic impact on the environment and as a consequence of this improving the environment.

In the 60s of the XXth century the population consumed approximately 50 % of that consumed by the population in the XXI century. This is primarily due to the fact that the use of energy has become the Foundation for a variety of processes as well as energy has become more available for use. This increase to using of energy is the root cause of modern environmental problems such as energy shortages and the problem of global warming.

On the average on one inhabitant of the planet Earth there are 2.5 [tfe] (tons of fuel equivalent) of energy resources. In the future, to 2100 the population on the planet Earth will grow to 10 bn, while the average specific energy – up to 10 [tfe], that is in overall energy production will reach 100 bn [tfe]. The level of air pollution also continue to grow, which will lead to the slow destruction of the biosphere. According to the group of American engineers, in 1800 for 1 million molecules of air there were 280 molecules of carbon dioxide, in 1960 there were 315, and at the beginning of the XXIth century – 370. By the end of that century, this number could rise to 550, which will lead to average temperature increases from 3 °C to 6 °C [1].

According to Carrington College, in the world may develop the Energy Revolution. Therefore, when comparing the baseline scenario of primary energy consumption with the implementation of renewable sources by 2050, the world could reduce the using of traditional fuel practically in 1.5 times (Fig. 1) [2].

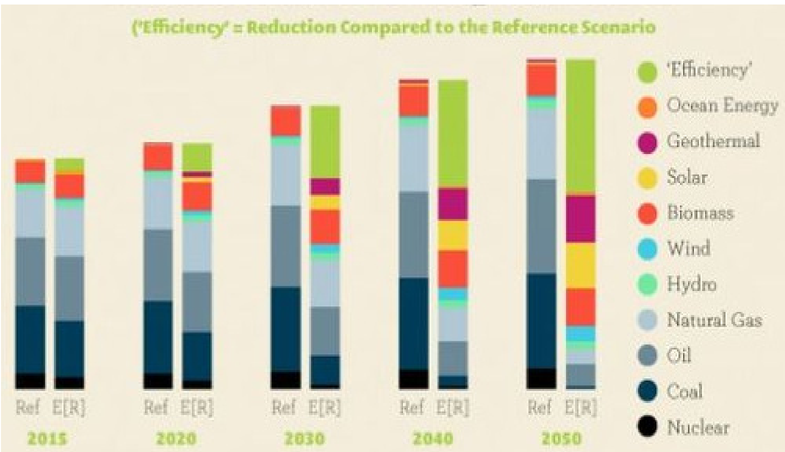

Fig.1. The forecast baseline scenario primary energy consumption and Energy Revolution

Interesting that according to Greenpeace, the five key principles for an energy revolution are:
1. creating greater equity in the use of resources;

2. respecting the environment's natural limitations;
3. phasing out unsustainable energy source;
4. implementing renewable solutions i.e. decentralizes energy systems and grid expansions;
5. decoupling economic growth from the consumptions of fossil fuels.

American scientist John Ricardo Cole asserts that the world stands on the threshold of the "Age of solar energy" [3].

Various designs of solar systems that contain a protective coating, insulating layer and a heat-conducting layer [4, 5].

A flat-plate collector – the most common type of solar collectors. Flat-plate collectors used in low temperature processes up to 80 [°C]. For higher temperatures necessary system with concentrators [6].

For the implementation of solar thermal installations it is necessary to optimize energy-efficient external enclosures due to the correct installation and design of these units at different orientations of external walls [7].

The aim of this work was to offer cost-effective design of a solar system, without losing the efficiency of the system by reducing its cost. On the other hand, to investigate the proposed construction of the solar heat supply system and to establish the thermal characteristics of the influence on it of the proposed factors.

2. The Main Material

The research was used an experimental setup design of solar walls as solar heat supply systems, with subsequent analysis on the coefficient of performance (COP). The design of the solar wall are a universal solution to conserve traditional energy resources, because it is a combination of the outer structure of the building and the solar collector. Scheme of the experimental installation of solar walls in a gravity mode depicted in Fig. 2.

This solar heating system was studied in the regime of 'gravity' and 'flow'. Water from the storage tank 6 is supplied into the tubes of the circuit 4, which is heated on the principle of natural convection through the radiator 12 and is returned into the storage tank 6 and possible for the selection of the heat carrier through the pipe 7.

Mathematical processing of the results obtained in the measurement of the physical properties performed on the developed special programs.

The intensity of the flow of energy that radiates the source must be measured by the actinometer.

The temperature of the heat carrier at the inlet and outlet in the solar collector and in the storage tank it is necessary to measure thermal converters of resistance 50M, working with a meter controller type PT-0102. The ambient air temperature and its speed was measured by thermo-electro-anemometer TESTO 405 – V1.

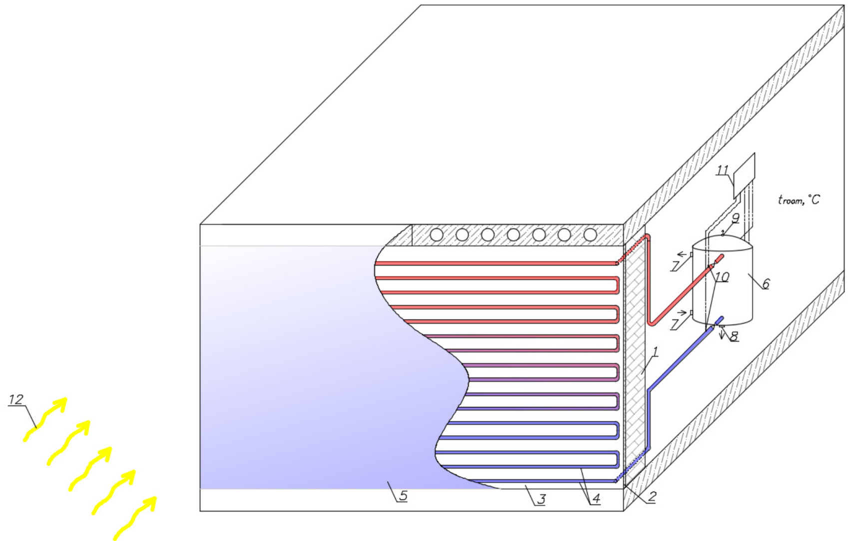

Fig.2. The experimental setup, where 1 – solar wall; 2 – thermal insulation layer; 3 – heat reflecting layer; 4 – tube circuit (serpentine heat absorber); 5 – plaster, 6 – storage tank; 7 – pipes for selection and flow of heat carrier; 8 – the discharge pipe of the heat carrier; 9 – air outlet valve; 10 – thermometer; 11 – display; 12 – radiation source

The basis of determining the coefficient of performance system of solar heat supply η_{SSHS} in general, the amount of energy received by the storage tank Q_{rec} use formula 1:

$$\eta_{SSHS} = \frac{Q_{rec}}{Q_{rad}} \cdot 100, (\%) \tag{1}$$

where: Q_{rec} – the amount of heat received by the storage tank for the time ΔT - was determined experimentally;
Q_{rad} – the amount of radiant heat received on the surface of the solar wall during the same time period ΔT.

Specific heat capacity of solar collector (SC) Q_{SC}, (W/m^2) determined by the formula (2):

$$Q_{SC} = \frac{m \cdot c \cdot (T_{outlet} - T_{inlet})}{F_{SC}}, (W/m^2) \tag{2}$$

where: m – mass of the heat carrier in the tank-battery, (kg);
c – the average specific heat capacity (at constant pressure) at the arithmetic average of the heat carrier, (J/(kg·K));
T_{inlet}, T_{outlet} – the heat carriers temperature at the inlet and outlet SC

respectively, (K),
F_{SC} – the square SC, (m^2).

At the heart of the experimental research methodology of the proposed construction of the wall was carried out the analysis of its heat characteristics. The methodology was taken from collection of published data, analysis of existing designs of solar installations, mathematical and experimental studies of the proposed design of the solar wall on the thermal characteristics.
Along with experiments was carried out monitoring of factors that could cause errors in the measurements.

Conducted the following measurements: water temperature at the inlet and outlet of the solar wall, temperature of water in the storage tank. Alternatively, the experimental part of the solar wall studied in two regimes, and processing of research results was carried out in different ways.

In research was used the intensity of the flux of solar energy I = 600 W/m^2 at the plane of the collector. Since the intensity of solar radiation is the radiant energy flux received per unit surface area that passes through any point perpendicular to the direction of radiation. Therefore, for the optimization experiments is permissible to use at the laboratory solar simulator.

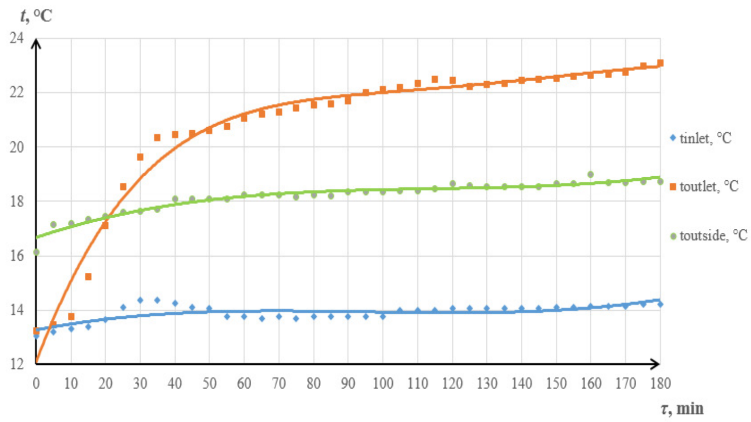

Fig.3. The temperature of the heat carrier due to the research of solar walls with the serpentine heat absorber in the regime of gravity.

The temperature of the heat carrier with the same design feature sof the solar wall, namely d=10 mm, l=20 mm, δ = 20 mm and the volume of the storage tank 0.015 m^3 has varied over the experiments and at the output from the solar wall was reached 15 °C in flow mode and 23 °C in the gravity mode, for a comparison of these temperatures on the charts, it was proposed to bring the ambient temperature and the inlet temperature in the solar wall (Fig. 2 and 3).

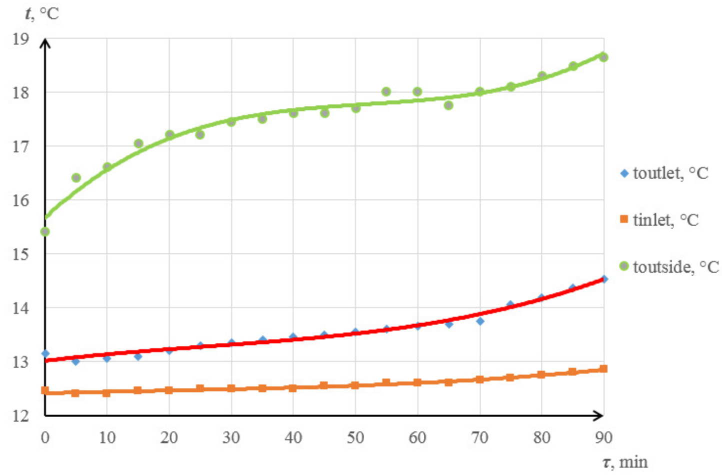

Fig.4. The temperature of the heat carrier due to the research of solar walls with the serpentine heat absorber in the flow mode

Interesting question in these structural parameters in the regime of gravity was more to consider changing the heating temperature in the storage tank and lead it to the average (Fig. 5) for such parameters, namely d =10 mm, l=20 mm, δ = 20 mm and the volume of the storage tank 0.015 m^3.

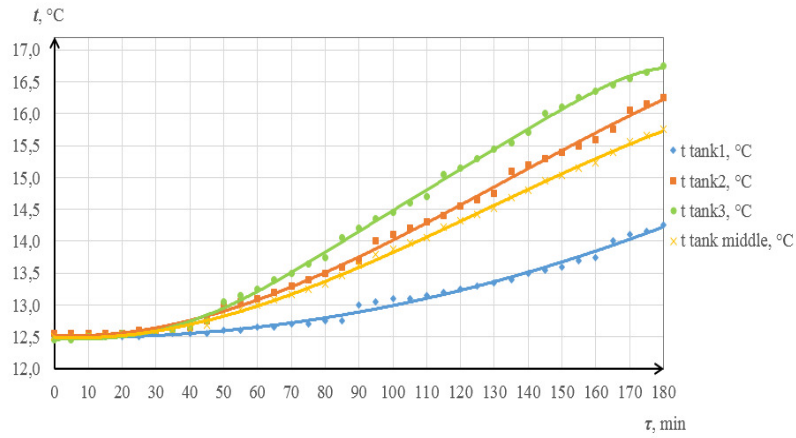

Fig.5. The temperature change of the heat carrier in the storage tank

An important issue in the regime of the flow was to investigate the instantaneous values of specific heat capacity for the solar heat supply system, which is shown in Fig. 6. Instantaneous specific heat capacity at d =10 mm, l=20 mm, δ = 20 mm, and the volume of the storage tank 0.015 m^3 and at a constant volumetric flow of 0.25 l/min is acquiring growing importance, which may be associated with the heating system, the instantaneous change in specific heat capacity for these parameters occurred from 61-147 W/m^2 for a constant solar radiation of 600 W/m^2.

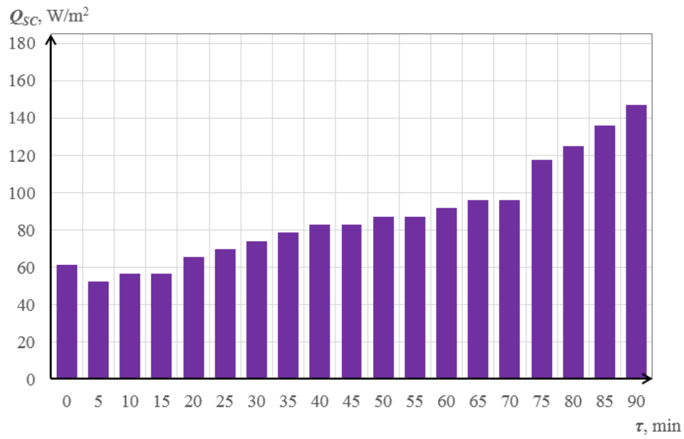

Fig.6. Instantaneous specific heat capacity of the system in the flow mode

The important point of the study of the solar wall was to examine the change in the coefficient of performance of the heat supply system as a whole a solar wall in gravity mode depending on the tube diameter of the circulation loop d and the distance l between them at constant v = 0.25 m/s and δ = 20 mm (Fig. 7).

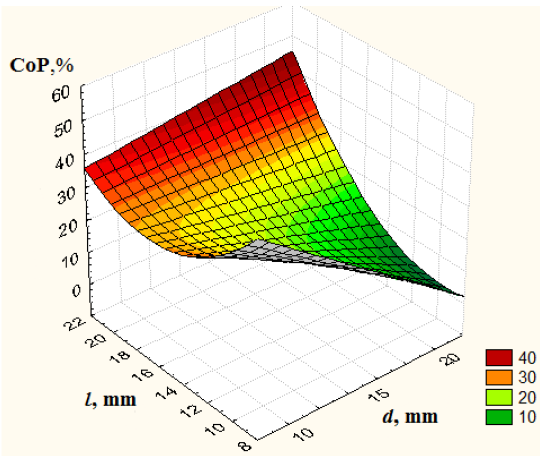

Fig.7. Change of the COP as a whole with serpentine heat absorber in the regime of gravity depending on the diameter of the tubes of the circuit d and the distance between them l

Such a solution would enable the engineers to calculate such systems for practical use with the assumption of the practical efficiency of such a system, depending on the diameter of the tubes of the circuit d and the distance l between them. The maximum achieved efficiency of the system of heat supply with solar wall in gravity mode was approximately 60 %.

3. Conclusion

The main thesis of this work is the installation of accessibility to the consumer, because the design does not require a separate installation and can be installed in an existing wall. In addition, the solar wall in both modes, has a sufficient efficiency for combined domestic hot water or pre-heating the heat carrier heating system. For example, the radiation intensity of 600 W/m^2 in the mode of flow the coefficient of performance was 49 %, while in gravity mode was 60 %.

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SUSTAINABILITY ASSESSMENT OF DESIGNED FAMILY HOUSE ALTERNATIVES WITH APLICATION OF GREEN TECHNOLOGIES

The paper aimed at an assessment of environmental and energy impacts of designed family house using green technologies. Investigated buildings are located in Kosice region. The analysis investigates the role of applied green technologies in proposed variants of family house from embodied energy and equivalent emissions of CO_2 and SO_2 by using LCA assessment method within "cradle to gate" boundaries. The main contribution of the study is underlining that green technologies have significant part in the reduction of the environmental and energy impacts.

Keywords: green technology, green building, sustainability principles to building design, sustainability assessment of buildings

1. Introduction

To achieve the goal of reducing CO_2 emissions, a life cycle approach is required. A life cycle assessment (LCA) quantifies the potential environmental impact of a product or a service according to the ISO 14040:2006 and ISO 14044:2006 standards. The application of LCA in architecture is constantly growing, and it has been used repeatedly to evaluate new buildings [1]. Study [2] denotes that a lot of LCA studies differ in approach, system boundaries, database and scope, and therefore cannot be compared. They reveal that calculations over the whole life cycle show a slightly different picture. Here, the advantages are generated mainly through benefits outside system boundaries (module D). In Europe the EN 15978:2012 is a widely agreed framework to clearly specify LCAs. Through division of the life cycle in modules, there is an agreement to

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calculate the benefits regarding energy recovery separately and to further show all potential benefits like recycling outside the life cycle of the building. This understanding is also applied on product level (EN 15804:2014). Product category rules for various material fractions were introduced. This means based on this framework comparative LCAs can be conducted on comprehensible manner. Another study [3] is focused on an addressing the gaps between the requirement of dynamic consideration and static implementation of LCA methodology. Therefore, the main goal of this study is developing a dynamic LCA (DLCA) withtime-varying factors and occupancy behaviors into consideration. Four dynamic properties (i.e., technological progress, occupancy behaviors variance, dynamic characteristic factors, and dynamic weighting factors) are brought into a static LCA model to develop a dynamic one that could be used to quantify building environmental impacts overtime. In addition, residential occupancy profiles were described at personal level, family level, and social level; and three potential quantification methods were introduced to explore the relationship between occupancy profiles and household energy consumption. The new DLCA framework could help improve the building LCA theoretical base, extend the connotation of LCA system from a new perspective, assess the influences of occupancy behaviors, and promote sustainable buildings. A novel approach which allows evaluating the relative importance of climate change and energy transition on environmental impacts of buildings is presented in study [4]. The methodology is illustrated using a simple case study: a low-energy single family house located in France. Two design options were evaluated: the choice of a heating system and the integration of photovoltaic (PV) modules on the roof. Using an attributional approach and compared to a static LCA considering no prospective parameters, the carbon footprint of the house (total life cycle) varies from +21% to +43% for the electric heating alternative, −7% to +4% for the gas boiler alternative, −6% to +15% for the PV alternative depending on climate change intensity and evolution of the energy mix. By using the consequential approach have a larger magnitude of variation from −36% to −13% for the electric heating alternative, 0 to +16% for the gas boiler alternative and −14% to +1% for the PV alternative compared to a static LCA. Accounting for climate change and the evolution of the energy system has a large influence on LCA results.

2. Green technologies and green buildings

The term "green technology" generally refers to the application of advanced systems and services to a wide variety of industry sectors in order to improve sustainability and efficiency. That means that goals could include: reduction of waste, spoilage and shrinkage; improvement of energy efficiency and energy conservation; creation of systems that are energy self-sustaining; the reduction of carbon emissions; a reduction in toxic waste and emission of toxic gasses such as volatile organic compounds; creation of products that are biodegradable; enhancement of water conservation and water quality; and promotion of the reuse and recycling of materials of all types [5]. Green technology development is accelerating in some areas. The number of patented inventions in renewable energy (+24 %), electric and hybrid vehicles (+20 %), and energy efficiency in building and lighting (+11 %) increased more rapidly than total patents (+6 %) between 1999 and 2008. Most of the green technology development is concentrated in a relatively small number of countries and there is a considerable specialisation across countries. For selected climate mitigation technologies, Japan's patent applications in 2008, for example, were relatively more concentrated in innovation related to energy-efficient buildings and lighting, as well as electric and hybrid vehicles, while the United States was particularly prominent in the area of renewable energy. While some data are available on green technologies, much less information is available on the related non-technological changes and innovation, such as in the introduction of new business models, work patterns, city planning or transportation arrangements, that will also be instrumental in driving green growth. There is some evidence that the scope of green innovation is broadening, however. For example, manufacturing firms have moved from end-of-pipe solutions to approaches that minimise material and energy flows by changing products and production methods and reusing waste as a new resource for production. Advances are also being made through better management practices and integrated strategies that are contributing to a range of new business models [6]. Fig. 1 depicts some examples of green technologies.

a) b) c)

Fig. 1. Green technologies: a) Cool roof, based on [7]; b) Electro kinetic road ramp, based on [8]; d) Crosslam timber/CLT, based on [9]

3. Design of family house

Up-to-date design of buildings requires a multidisciplinary approach. However, even today, there are often proposed drawings which do not respect the requirements for sustainable building construction. Total investment costs are preferred to sustainability aspects like environmental impacts, health and well-being of building users. It leads to the need to present sustainable or green designs of buildings with application of green technologies. For this reason, this article is aimed at design and evaluation of two alternatives of family house. Family house is designed in flat terrain in village of Kokšov Bakša, district of Košice in the Košice Region. The alternative 1 of family house denotes design from conventional materials and building services. In contrast, the alternative 2 represents one of the possible solutions concerning the using of resources, the comfort of building users and the protection of natural environment. It is a single storey family house without basement with numbers of occupants of four. On the ground floor there is a vestibule, entrance hall, kitchen, living room, dining room, two bedrooms, and bathroom with toilet, pantry, and boiler room. Table 1 presents basic data about designed alternatives of family house.

Table 1. Information about designed alternatives of family house

	Alternative 1	**Alternative 2**
Built up area	250 m^2	224 m^2
Living area	98.06 m^2	117.11 m^2
Floor area	183.52 m^2	165.61 m^2
Built up volume	1350 m^3	986 m^3

3.1. Alternative 1

Family house has two entrances, the main oriented to the northeast side and entrance from the terrace into the living room from the northwest side. It is designed as a single-storey brick house basement with garage, covered with a flat roof. Foundation strips are designed as monolithic with wide of 600 mm, high of 650 mm from concrete C16/20, on which foundation structures are designed in two rows shuttering formwork blocks. External and internal bearing walls are designed from concrete blocks with thickness of 300 mm, respectively 240 mm; windows and doors as plastic. Floors are designed as ceramic, laminate and screeding. The horizontal structures consist of reinforced concrete ceiling with EPS liners above the ground floor and reinforced concrete ring beam wreaths and lintels from concrete of C20/25. The roof structure is designed as a flat sheathed with modified asphalt strips.

Family house is designed to be connected to a public network of wiring and water. Sewage water will be drained to a septic tank. Family house will be connected to the public water supply. Floor heating is designed in all rooms of house. Source of the heating is gas boiler.

3.2. Alternative 2

Alternative 2 of family house has two entrances, the main oriented to the northeast side and entrance from the terrace into the living room from the northwest side. It is designed as a single-storey house with wooden structure and without basement as well as garage, covered with a flat green roof. Foundation strips are designed as monolithic with wide of 600 mm, high of 650 mm from concrete C16/20, on which foundation structures are designed in two rows shuttering formwork blocks. External and internal bearing walls are designed from CLT panels with thickness of 170 mm. Windows and doors as well as floors are designed as wooden structures. Horizontal structures consist of CLT panels with thickness of 170 mm above the ground floor. The roof structure is designed as a flat green roof using the extensive vegetation to a height of 100 mm; substrate for green roof has a thickness of 100 mm. Water drainage is provided by drainage pipes and a built-in gravel drainage layer. Drainage layers are placed around the perimeter of a width of 500 mm. Pipes are fitted into a gravel layer at the lowest roof height. Family house is designed to be connected to a public network of wiring, water and sewage. Family house is connected to the public water supply. Sewage is designed as pressure sewage system. Floor heating is designed in all rooms of house. Source of the heating is heat pump. Hot water is stocked in tank of 300 litters.

Table 2 presents designed material compositions of building envelope for two mentioned alternatives.

Table 2. Constructions of building envelope

Assembly	**Alternative 1**	**Alternative 2**
External wall	Concrete blocks EPS 100 PENOGREY	Crosslm /CLT panel Fleece Diffusive open wall
Floor	Laminate floor Ceramic pavement Thermal insulation EPS	Wood floor Ceramic pavement Mineral wool
Roof and ceiling	Flat roof Modified asphalt strips Reinforced concrete ceiling	Green roof CLT panels
Opening construction	Plastic windows Plastic door	Wooden windows Triple insulating glass 44 mm with argon

On the figures 2 and 3 we can see the ground floor dispositions and views for designed alternatives of family house.

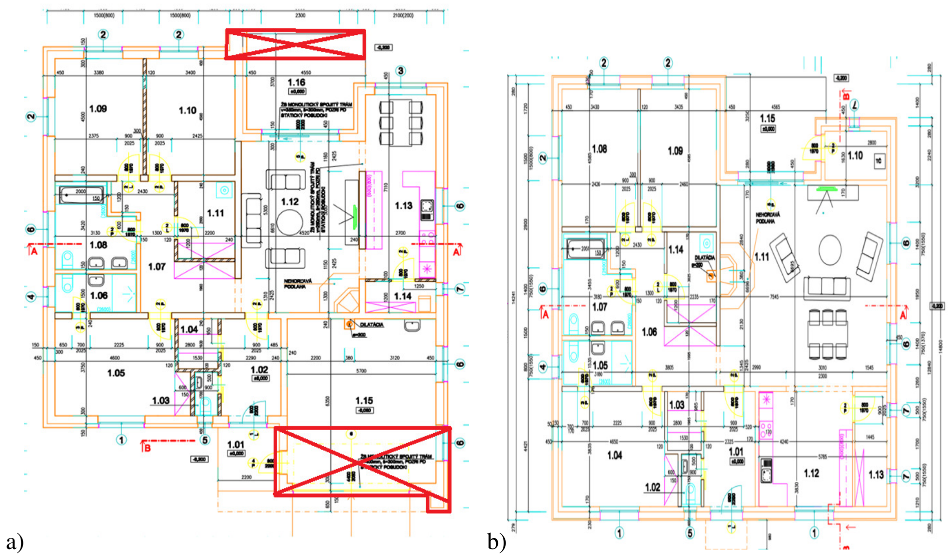

Fig. 2. Ground floor of family house: a) Alternative 1; b) Alternative 2

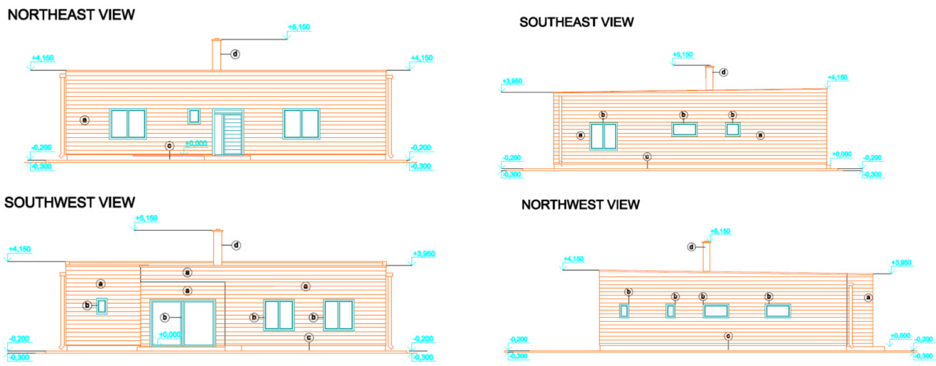

Fig. 3. Views of alternative 1 of designed family house

4. Sustainability assessment of designed alternatives of family house

Building environmental assessment system (BEAS) is used for evaluation of selected family house. Evaluative program BEAS contains six evaluation areas and 52 indicators. The main fields are site selection and project planning, building construction, indoor environment, energy performance, water management and waste management. Each indicator is assessed according to scale: negative practice (-1 point), acceptable practice (0 point), good practice (3 point) and best practice (5 point). The results of each indicator assessment are obtained so that the point from the scale is multiplied by weight of the indicator.

After the assessment of the whole building and its site, building is certified according to scale introduced in Table 3 [10].

Table 3. Key for total assessment of building and certification by BEAS

	Key for assessment	Certification scale
-1	negative practice	Environmentally unacceptable building
0	acceptable practice	Environmentally acceptable building
3	good practice	Environmentally friendly building
5	best practice	Sustainable building

4.1. Site selection and project planning

Environmental regional classification of Slovakia represents a cross-sectional source of information on the state of the environment and reflects its differentiated situation in different parts of the Slovak territory. Slovak regions show diverse load situation for individual components of the environment and the risk factors show various degree of representation in them. A map assessing the Slovak territory by 5 degrees of quality of environment developed by the Slovak Environment Agency represents one of the outputs. This map helped identify the most loaded areas - their core typically comprises territories within the 5th degree with the most damaged environment. Family house is designed to be situated in the southern part of eastern Slovakia. Kokšov Bakša is located in the south-eastern part of the city of Košice on the side of the river Hornád. It is 191 m above sea level. Construction site is located in an area with environmental class level that falls within a category 5, i.e. environment heavily deteriorated/disturbed environment. The most important pollutants are suspended particulate matters, SO_2, NO_x, CO. Based on the assessment in the field "A - Site selection and project planning" the designed house reached points of 2.49 from 5 possible points. According to the tool of BEAS the significance weight is 14.71%.

4.2. Building constructions

Currently we are highly strived to incorporate into construction work materials with the lowest negative effects on humans and the environment. Efforts of building materials producers are to be designated as environmental labelling of products. In the EU we can found on the market the official European label for products and services known as Ecolabel. Products with the Ecolabel meet strict environmental and quality requirements, plus it is also assessed their life cycle from the inception to the destruction. According to LCA analysis the embodied energy for alternative 1 is 4221.14 MJ/m^2, total CO_2 emissions causing global warming are 245.79 kg/m^2 and total SO_2 emissions are 1.28 kg/m^2. Alternative 2 achieved values for embodied energy 1072.66 MJ/m^2,

CO_2 emissions -56.5 kg/m^2 and SO_2 emissions 0.355 kg/m^2. In the figure 5 we can see the comparison of values for environmental indicators. Based on the assessment in the field "B - Building constructions" the alternative 1 and 2 reached points -0.82, respectively 2.07 from possible 5 points. According to the tool of BEAS the significance weight is 20.59%.

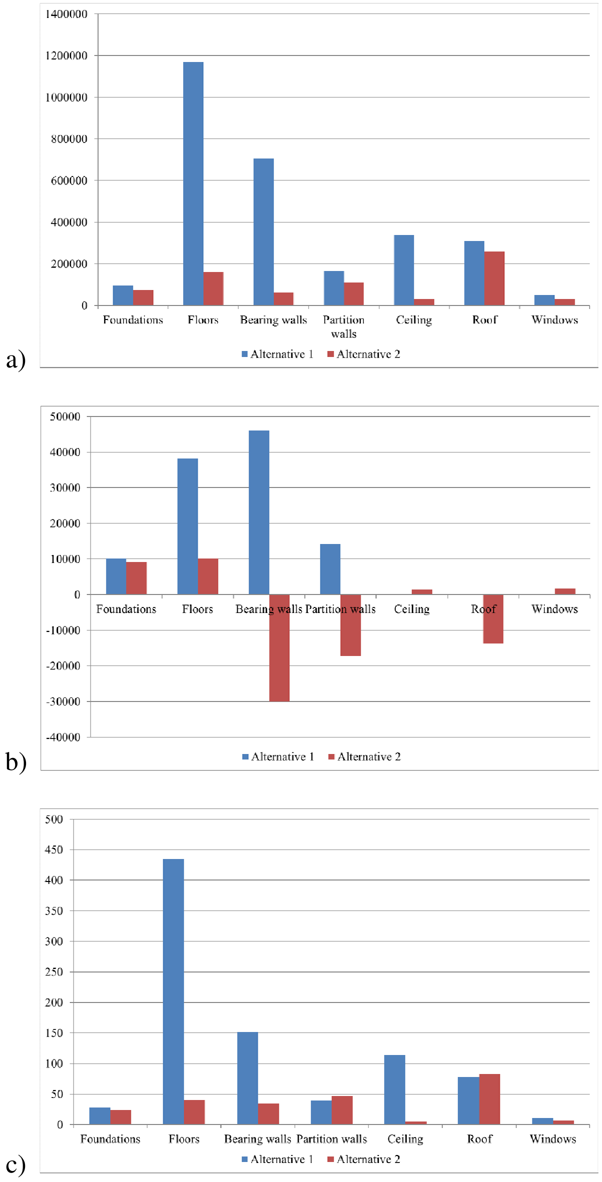

Fig. 4. Comparison of environmental indicators: a) Embodied energy; b) CO2 equivalent emissions; c) SO2 equivalent emissions

4.3. Indoor environment

The total area of openings in the enclosure is at least 5% of the total floor area, and over 50% of the cross-ventilation. Mechanical ventilation in some areas meets the minimum requirements of the standard. External walls are proposed in terms of sound insulation in accordance with requirements of legislation and ensure the required degree of protection of the internal spaces. The windows that are most exposed to the source of the noise from the outside according to drawings have good quality class of sound insulation. According to the drawings, airborne sound insulation between some space meets the minimum requirements of the standard. Ensure sufficient daylight in some areas reaches a minimum value for the scheduled tasks. To ensure minimum glare in the main occupancy areas in the period with a maximum brightness from outdoor is proposed appropriate measures by shading elements. All interior materials, including paints, sealants, adhesives, carpets and composite wood products are selected as materials with low-level of VOC emissions and are not used in composite wood products containing urea formaldehyde resin. Based on the assessment in the field "C - Indoor environment" alternatives 1 and 2 reached points of 1.20 and 3.30 from possible 5 points. Significance weight of is 23.56%.

4.4. Energy performance

The field D–Energy Performance, was evaluated in subfields: D1 - Operation energy, D1.1 - Energy for heating, D1.2 - Energy for domestic hot water, D1.3 - Energy for mechanical ventilation and cooling, D1.4 - Energy for lighting, D1.5 - Energy for appliances, D2 - Active systems used renewable energy sources, D2.1 - Solar system/heat pump, D2.2 - Photovoltaic technology, D2.3 - Heat recuperation and D3.1 - System of energy management. Based on the assessment the alternatives 1 and 2 reached values of 0.74, respectively 2.51 from possible 5 points. Significance weight is 26.47%.

4.5. Water management

The field "E - Water Management" was assessed in subfields: E1 - Reduction and regulation of water flow in water systems, E2 - The water management of surface runoff, E3 - Drinking water supply and E4 - System of grey water. Based on the assessment in the "E - Water management" the alternatives 1 and 2 reached 1.65 and 2.40 points from possible 5 points. Significance weight is 8.88%.

4.6. Waste management

The field F-Waste Management, was assessed in subfields: F1 - Plan of waste disposal originated in construction process, F2 - Measures to minimize waste resulting from building operation and F3 - Measures to minimize emission resulting from building construction and demolition. Based on the assessment the alternatives 1 and 2 reached 0.58 respectively 1.62 points from 5 possible points. Significance weight is 5.80%.

5. Summary

Evaluation of designed alternatives of family house reveals that these alternatives meet the requirement for energy demand. Significant differences are noted in the comprehensive assessment. Values of heat conductivity for building envelope are almost the same, and both alternatives can meet the desired aims, whether being a classic house or a house designed from environmentally friendly materials and using green technologies. In terms of energy demand both alternatives meet requirements for energy performance of buildings, but alternative 1 complies with the requirements determined for years to 2016 and alternative 2 meets the requirements designated for buildings built up since 2016. The advantage of the use of environmentally friendly materials in alternative 2 is in increasing the useful area by reducing the thickness of external walls from 450 to 400 mm, and also reducing the thickness of internal bearing walls. The most significant differences were observed in the assessment of the two alternatives by BEAS where it was clearly showed that alternative 2 is more appropriate and acceptable with respect to the environment and to the comfort of the user. In Table 4 it can be seen the results of assessment of two alternatives of designed family house. Table 4 shows the whole assessment of designed alternatives of family house with presenting the main fields and their percentage weights of significance.

Table 4. Comparison of designed alternatives of family house by BEAS

Fields		**Percentage weight**	**Alternative 1**	**Alternative 2**
A	Site selection and project planning	14.71 %	2,49	2.49
B	Building constructions	20.59 %	-0.18	2.07
C	Indoor environment	23.56 %	1.20	3.30
D	Energy performance	26.47 %	0.74	2.51
E	Water management	8.88 %	1.65	2.40
F	Waste management	5.80 %	0.58	1.62
Total assessment		100 %	0,99 Environmentally acceptable building	2.54 Environmentally friendly building

6. Conclusions

Results of sustainability assessment of two alternatives of family house show that the house with environmentally friendly building materials and green technologies is preferable. Thermo-physical parameters of both alternatives meet up requirements for energy performance, but alternative of green design met the target recommended requirements for all constructions of building envelope. It can be said that the alternative 1 can also meet the advanced requirements by minor adjustments. But the benefits of green alternative are also in the reduced thickness of external walls, which ultimately means the increase of living space inside the house. Energy demand is comparable in both alternatives of the house. Benefits of alternative 2 are the better shape factor of the house. The difference between alternatives occurred in the comprehensive assessment by BEAS. Here, the alternative 1 reached the score near to 1, therefore this design of family house is certified as Environmentally acceptable building. The alternative 2, which can be consider as green alternative obtained higher level and is certified as Environmentally friendly building. This result is achieved by changing the building materials, replacement of gas boilers for heat pump, as well as by modifications the roof for the green roof. In conclusion it can be stated that the alternative 2 of family house is more appropriate and more acceptable to the environment.

In the end it is possible to conclude that the green technologies are on the rise. There are a lot of materials and technologies that can function effectively or have suitable properties and at the same time be acceptable to the environment. In Slovakia there are a number of buildings classified as green or high performance green buildings that are documented and have the required certifications from sustainability aspects. Design of high performance green buildings for the future of a sustainable life on Earth is indisputable. Certification of buildings from three dimensions of sustainability (environmental, social and economic) gives some assurance that the buildings do not burden the environment. Building users will create an environment where they can carry out their daily activities with the objective and subjective positive feelings.

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RISK ANALYSIS WITH AN APPLICATION TO LOCAL ROAD INFRASTRUCTURE

The paper presents the concept of the risk evaluation for road infrastructure exposed to natural hazards – floods and landslides. Floods and surface mass movements impose a serious threat to the contemporary activities and people's lives in modern economy. The natural meteorological and hydrological phenomena are main causes of a landslide activation. Typically, heavy or prolonged rain is combined with the progressive flooding. In river valleys, an increased lateral erosion of rivers and rapid snow melting in early spring would also lead to flood events. In Poland, the Carpathian regions are mostly predisposed to the formation of landslides. This may be favoured by the nature of shapes associated with high and steeply sloping slopes of the valleys and flysch geological structure. The paper presents the general characteristics of precipitation in Poland and the concept of a risk assessment with risk matrix. The issue is illustrated by an exemplary detailed risk matrix for a selected section of the road infrastructure in Subcarpathian province.

Keywords: natural hazard, road, flood, landslide, risk matrix

1. Introduction

Over the last decades, extreme natural phenomena, such as heavy or prolonged precipitation and the associated increase in landslide risk, cause significant economic and financial losses in the areas affected by these phenomena**.** The current situation should force to constant improvement of the principles and research methods used in hydrology, including monitoring of precipitation, both design and dimensioning of construction and communication infrastructure, drainage and sewage systems, and drainage of endangered areas. The issues of modeling and forecasting climatic impacts caused by atmospheric precipitation on building constructions are the matter of great importance, especially in the case of strongly urbanized areas, where the financial, economic and social consequences can be serious. In the literature and such standards as EN 1990-1999 [1] and ISO 2394 [2], the strategies and rules for ensuring an adequate level of safety of buildings and other structures for exceptional

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loading, including climatic influences, have been defined. The hitherto models of climatic impacts are adapted to semi-probabilistic methods of structural design, i.e. they allow to estimate characteristic values (quantiles of the order 0.05–0.02 interactions treated as random variables) multiplied by partial safety coefficients γ_F, in order to get their design values [1]. Semi-probabilistic methods of calculations of engineering structures are based on values of actions similar to mean values. However, in the probabilistic methods of 2nd and 3rd order the design values should be estimated directly [3]. With this approach, the extreme values of actions and effects of actions are decisive for the safe and reliable structural work, in comparison to those for which there is a high probability of occurrence. The extreme values of the interactions with a very low probability of occurrence decide about exceeding the ultimate limit states and destruction of the structure. The doubts about classical principles and research methods regarding climatic actions, including recommended standard methods, are caused by the disproportion of the time scale (the length of the time interval) from which the measured data originates (most often several dozen years), which indicates the contractual character of the recommended exceptional actions [4,5]. The repeatability obtained for values assessed as extreme may reach even several thousand years. Therefore, determining the values of exceptional loading, their forecast and risk assessment in relation to the planned and existing buildings and communication infrastructure is a very important task due to the potential social, economic and financial consequences.

2. Spatial development and climate monitoring

The area of Poland is located in the moderate climate zone [6,7] and is exposed to the occurrence of various extreme phenomena of natural threats directly affecting building structures. These include landslides. In the Carpathians, which occupy about 6% of Poland, the number of 23000 landslides have been identified and documented, which is about 95% of all landslides registered in Poland. In the most susceptible regions of these mountains, about 40% of the area is covered by landslides or other forms of mass movements. The area of Polish Carpathians is about 19500 km^2, therefore, it can be estimated that in this region of Poland for every square kilometer of area and for each 5 km of road infrastructure network, there is one landslide on average [8].

2.1. Ground based weather stations in Poland

In Poland, the tasks in the field of hydrological and meteorological protection of people and the economy are performed by the State Hydrological and Meteorological Service (PSHM), and their fulfillment under the Act of 18 July 2001 Water Law [9] was entrusted to the Institute of Meteorology and Water Management – National Research Institute (IMGW). The main tasks of PSHM are as follows: measurement and observation works, data transmission, processing and distribution. In the case of climatic actions on historical constructions, historical data is the matter of highest importance.

Both short-term intense rainstorms with a local territorial range, as well as long rainfall of lower intensity, but with a broad range of terrain, may cause damage to the environment and urban and industrial risk. Such phenomena occur now and will happen in the nearest future. Therefore, it is necessary to aim at limiting the detrimental effects of such random events by determining both the probability and performing risk quantification.

The PN-EN 752: 2008 [10] standard limits the frequency of flooding from sewage systems or the lack of possibility to a rainwater collecting, to the rare socially acceptable repetition of their occurrence once every 10 years for rural areas and 20, 30, or 50 years, respectively, for urban areas – according to their spatial zoning. That is the reason why a systematic research on precipitation and determination of the statistical frequency of their maximum amount, intensity and unit exertion are so important, even for rare rainfall repetitions. As far as statistical analysis is concerned, sufficiently long and continuously updated archival material from rainfall observations is necessary. A verification of statistical material regarding precipitation is a task justified not only economically, but also technologically. The data acquires particular significance in relation to quantitative differences, such as calculated yield rainfall for reliable channel dimensioning [11,12,13], or retention reservoirs [7,14], as for models and standards used in Poland [7,8]. Precipitation in moderate climate zone occurs both in liquid drops (rain, drizzle) and in solid state (snow, hail).

Neither the rain intensity is constant in time nor in the area covered by precipitation. Momentary unit intensity of precipitation may be several times higher than the average. High intensity may occur once or more times during the rainfall, appearing in random sequence. These phenomena, variable in nature, are difficult to describe in individual terms in time and space, e.g. in the urban catchment scale, but necessary for generalizations or simplifications for design purposes. At present, it is recommended to take into account various scenarios of atmospheric precipitation (variable in time and space) taking into consideration either actual series of local rainfalls measured for at least 30 years, or a model rainfall, based on e.g. Euler 2^{nd} order or Weibull distributions created from IDF or DDF curves the so-called synthetic hietograms [4].

As for a design of safe building facilities and infrastructure, both short-term (fleeting) rainfall with high unit intensity (q) and long-term rainfall (widespread) with significant territorial range (A) and high efficiency are decisive ($Q=q(A)$).

Heavy or volatile rains come from stormy clouds (cumulonimbus), usually lasting several dozen minutes (sometimes few hours) and are characterized by high intensity and varied local range, covering the area from several to even several hundred square kilometers. They occur in Poland in the summer time, from May to September, and usually in July. Rain phenomena lasting longer (up to several days) usually consist of several precipitation, occurring immediately after each other, separated by periods without precipitation. As for the design of slopes and the occurrence of landslide phenomena, long-term (widespread) rainfall affecting the state of the soil is the matter of the greatest importance. They cause the largest water infiltration of the subsoil [7].

In order to increase the reliability of the road infrastructure and construction objects designed on the slopes (in accordance with EN 1990 [1] and ISO 2394 [2]), the need of refining the principles of dimensioning based on continuous precipitation measurements over several decades becomes crucial. This is a need to capture the possible trend of climate change, especially in recent decades and to develop methods for estimating both the threat and risk taking into account economic consequences. The efficiency and credibility of each calculation method, as evidenced by the principles of reliability, determines the fulfillment of the requirements for safe and durable use in the designed lifespan.

2.2. General characteristics of precipitation in Poland

Precipitation being a discontinuous meteorological phenomenon is characterized by high temporal and spatial variability of occurrence, as well as a significant variation in the sum of heights. A number of environmental determinants determine the precipitation phenomenon of a specific area. Among them, the most important are the geographical location, the distance to water reservoirs (including seas and oceans), the shape of the surface and the elevation of the area above sea level, vegetation cover and specifics of terrain, and others. The rainfall measurement takes place pointwise, in a given network of measuring stations, what in relation to larger areas requires the use of appropriate interpretations of the measurement results obtained. The relationships of the intensity (or height) of the rainfall with the duration and frequency of occurrence, developed for many geographical regions of the world (America, Asia or Europe) are qualitatively similar to each other. [7,15,16]. However, it does not mean that they are quantitatively identical, especially at the microscale of local precipitation.

Referring to the extreme rainfall of long duration occurring in Poland, it should be pointed out that the greatest value of precipitation for a 24-hour period is the rainfall recorded on June 30, 1973, the sum of which was equal to 300 mm (Hala Gąsienicowa). The sum of precipitation for the 48-hour period (29 to 30 June 1973) was equal to 372.8 mm, and for the interval of 72 hours (June 29 - July 1, 1973) was measured at 384.5mm. The amount of rainfall over 24 hours indicates that the area of Poland is characterized by the highest values among neighboring countries in region. In Poland, the highest sums of precipitation lasting 48- and 72-hours occurred in the area of the Śnieżnik Kłodzki massif on July 5-7, 1997. The highest 48- and 72-hour precipitation in total was then equal to 422.0 and 557.0 mm, respectively [17]. During the flooding period of July 1997 on the IMGW network, the highest daily sum was measured at the station in Międzygórze and amounted to 200.1 mm (July 6, 1997), the 48-hour total was equal to 364.6 mm (July 4, 07), and the 72-hour in total reached 431.2 mm (July 05-07). The monthly rainfall amounting to 702.0 mm was also record-breaking (measured at Kamienica) [7,17,18].

On the basis of the analysis of maximum precipitation levels with duration of 5 minutes up-to 72 hours (Table 1) for the climatic conditions of Poland and selected European countries, a comparable amount of precipitation may be observed. For most European countries, 24-hour volumes have a rainfall level close to Poland - 300 mm. Also extreme amounts of short-term precipitation, most often convective rains, in Polish climatic conditions are comparable with the recorded rainfall levels in neighboring countries. The reader may find many details of torrential and heavy rainfall patterns occurring in Poland [17,18].

Table 1. The maximum precipitation for selected European countries [mm] according to [7]

Country	**Duration of precipitation**					
	Min.			Hours		
	5	10	15	24	48	72
Poland	25.3	80.0	79.8	300.0	428.0	557.0
Germany		126		312.0	379.9	458
Czech Rep	29.8	39.8	50.2	345.1	380.0	536.7
Hungary		64.2		260	288	
Norway	17.9	31.5		229.6	378.9	402.4

3. Natural hazards and spatial management in Poland

In the case of Poland, the landslide threat applies to all counties and communities located within the Carpathians, in the following provinces: Silesia, Małopolska and Subcarpathian, where after rainfall the state of the ground changes. Landslide processes generally trigger with a certain delay in relation to rainfall, that is the reason why their intensity may take place in the following weeks after the occurrence of heavy and continuous precipitation. Usually in the Carpathians, after the rainfall, the sum of which exceeds the range of 70-100 mm, generally shallow landslides become active whereas, after over 400 mm rainfall - large and deep landslides may be formed.

The main reason for the high financial losses occurring in landslide areas in Poland is not well thought-out activity of investors, manifested in the location of the construction and communication infrastructure on the threatened or active slopes. Financial expenditures for reconstruction and/or repair of damaged facilities and roads are significant. The losses caused by landslides are counted in millions. In the Małopolska province alone, they amounted to approximately PLN 170 million in just two years (2000-2001). It should be emphasized that often, however, the effects are removed, not the cause of such mass movements. Often, incorrect stabilization of landslides is the result of insufficient geological recognition, based only on theoretical premises, not confirmed by appropriate geological-engineering studies. [19]. Financial and economic losses caused by landslides may be estimated, while social and moral losses are almost impossible to be valued. The only effective solution to the landslide problem is the exclusion from the new development of the areas of currently and periodically active landslides and the limitation of buildings predisposed to their occurrence, and, above all, appropriate monitoring and development of landslide hazard maps. The System Guards Against Landslides (SOPO) project, implemented in recent years in Poland, offers the chance to avoid major problems related to mass movements in the future by introducing landslide cards, that is, indication, description and regular observation and monitoring in the most-threatened areas, to limit private, public construction works in areas at risk. The SOPO project is an opportunity to limit the negative effects of mass movements throughout Poland.

The difficulty in forecasting landslides is closely related to rainfall and results from irregularities in the occurrence of various weather phenomena. Catastrophic rains may occur once every several, a dozen or even several hundred years. Their occurrence is in practice unpredictable, only the statistical probability of their occurrence may be determined.

The end of the last decade of the 20^{th} century and the beginning of the 21^{st} century were marked by the exceptional severity of catastrophic events in the Polish Carpathians. After a long break between 1980-1990, when there was no high precipitation in this area, since July 1997 there has been a radical change in the amount of precipitation and spatial distribution. After the rainfall in July 1997, the great amount of water contributed to the launch of the Carpathian slopes. New landslides were created and old renewed in the western and central part of the mountains, in areas with new housing and communication infrastructure, which often led to its deterioration. The winter weather conditions of 1999/2000 contributed to the renewal of great many landslides mainly in the area of the Carpathian Foothills (over 2500 reported cases).

L. Starkel [20] writes about a specific "cluster of extreme phenomena", which began with catastrophic precipitation in the summer of 1996, marking the activation of an enormous number of landslide processes in the Carpathians. The cluster lasted for several years until 2010. A strict determination of the end of the landslide process is very difficult, because once the stability of the slope is disturbed, even with less efficient driving force, it is in a state of unstable equilibrium.

Landslides alongside with floods contribute more to the development of landforms. In contrast, however, from the flooding, which is a catastrophic but episodic phenomenon, landslides are a continuous phenomenon, when the mass movements occur on the slopes constantly, even without the action of a stronger triggering force. And often landslide suddenly becomes a catastrophic phenomenon of unpredictable strength and intensity.

Landslides formed as a result of recent catastrophic precipitation and floods associated with them together with losses counted every year in hundreds of millions caused that the mass movements entered the catalog of natural disasters and were included into the Polish law.

4. Risk analysis and expected losses for the selected section of road infrastructure

4.1. Methodology

In the risk assessment is to determine the risk acceptance criterion. There are numerous proposals for quality criteria and the criteria mixed, usually not very precise and led to substantially different results [21,22,23]. Most often these are different variants of the principle of ALARP, i.e. the risk as low as reasonably practicable. The PN-EN 1991-1-7 [24] presented a mixed criterion in tabular form as the mathematical expectation of the consequences of an undesired event (Table 2).

Table 2. Matrix of acceptable quantitative risk levels according to [24]

Probability of Failure / Consequences	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}
Very large	X				
Large	X				
Medium		X			
Low			X		
Very Low				X	
X – represents the largest acceptable level of risk					

The social acceptability of risk is determined by comparison with other types of individual risk and lack of social acceptance for disasters, in which great many people may lost their lives. In EU countries acceptable level of probability of loss of life or *health of a person p_{fd} is estimated depending on the natur*e of the risk [12,25] as:

* The voluntary risk (e.g. professional work, sport) $p_{fd} = 10^{-3}$/year;
* Natural risk (e.g. floods, hurricanes, landslides) $p_{fd} = 10^{-4}$/year;
* The imposed risk (such as construction failure, acts of terrorism) $p_{fd} = 10^{-5}$/year.

The criteria proposed in the form of matrices of mixed quality and acceptable levels of risk are presented in many publications, including [26,27,28]. Depending on the consequences and costs of damage, the level of risk associated with damage of the road infrastructure located in the area of floods may be considered as acceptable, when the risks of loss of life of people as a result of the damage of the road is small, and its economic, environmental and social consequences are small, or inadmissible (Table 2).

The common form of presentation of risk dependent on the probability of occurrence of hazards and their consequences is a risk matrix. If the available knowledge on the consequences of hazard is imprecise, incomplete, subjective and/or is qualitative, the risk R may be considered as a fuzzy parameter and estimated with linguistic variable or fuzzy sets using the following formula (1):

$$R = H \, x \, C \tag{1}$$

where: H - is imprecise occurrence associated with the occurrence of hazards, and
C - is imprecisely, fuzzy defined loss caused by a hazard, consequence.

The qualitative form of both the hazard and the financial loss caused by damage of transport infrastructure, may be expressed in the form of linguistic variables, which are described by fuzzy sets, or fuzzy numbers.

In a situation, where the risk and financial consequences of damage of the road infrastructure may be estimated as linguistic variables, subjectively, and/or imprecisely, a relevant assessment is presented in the form of fuzzy matrix or as linguistic variable (Fig. 1) [29].

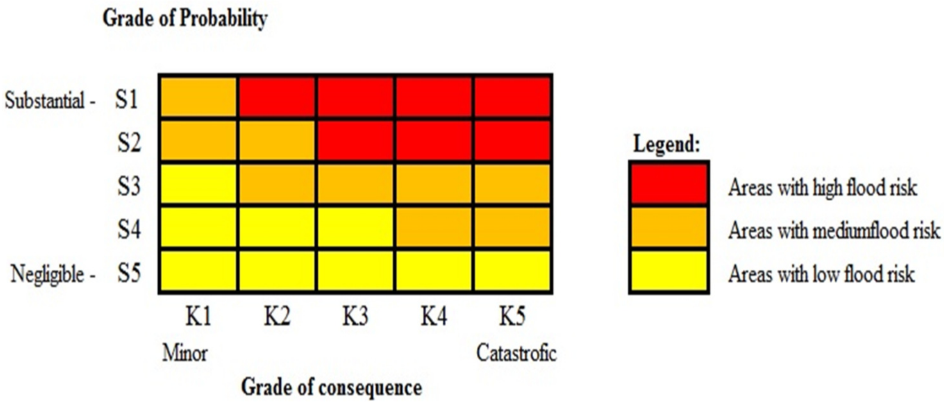

Fig. 1. Example of risk matrix based on flood probability and consequence according to [29]

4.2. Case study

This study therefore sought to develop a landslide risk map showing the risk of floods and landslide occurrences in selected area of Niebylec community. The identification of areas prone to landslide is a fundamental component of disaster management and an important basis for promoting safe human occupation and infrastructure development in areas of Subcarpathian region, considering the devastating impacts of land movements. The study were assisted with investigating the characteristics and factors which determine the value of a risk map for road infrastructure. There were several reasons responsible for a the formation of landslides in the considered area. Undoubtedly, the direct cause is a torrential rainfall. The association of the formation or activation of landslides in the Outer Carpathians (named Flysch Carpathians) with the height and intensity of precipitation is indisputable and has already been described many times [30,31,32]. In addition, the flysch geological structure, which is characterized by alternating layers of water-permeable sandstones and poorly permeable shales, clays and marls becomes another serious reason. Also, the presence of quaternary weathered cover more susceptible to landslide processes and tectonic structure, i.e. arrangement of rocks, cracks, faults become a source of threat. The landslide in analyzed does not significantly differ, when considering its parameters and local precipitation (Fig. 2).

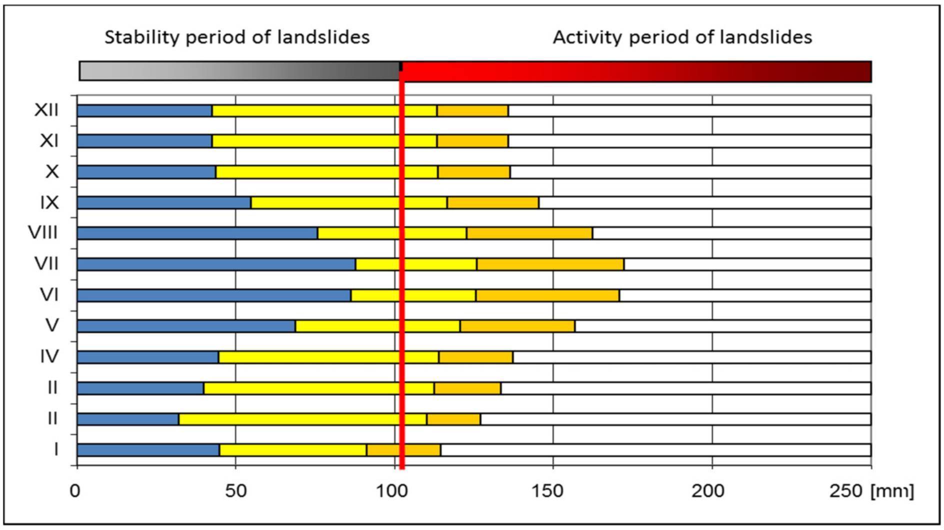

Fig. 2. Potential stability and activity period of analysed landslides in relation to the year's average of maximum rainfall amount of rainfall for Niebylec community

Risk assessment of flood for the sketch of road infrastructure was performed according with the algorithm for the selected area (Table 3):
1) the class of hazard of flooding of the land surface was determined,
2) the clas of consequences of the road section was determined.
3) risk matrix for sketch of the road infrastructure was determined (Table 4):

Table 3. Grade of hazard and consequence

Hazard /Consequences	Grade of hazard/ consequences
Very small	1
Small	2
Medium	3
High	4
Very High	5

Table 4. Risk matrix

Hazard xConsequences	Very small	Small	Medium	High	Very High
Very small	1	2	3	4	5
Small	2	4	6	8	10
Medium	3	6	9	12	15
High	4	8	12	16	20
Very High	5	10	15	20	25

Risk	Grade of risk
Very small	1,2,3,4
Small	5,6,8,9
Medium	10,12
High	15,16
Very High	20,25

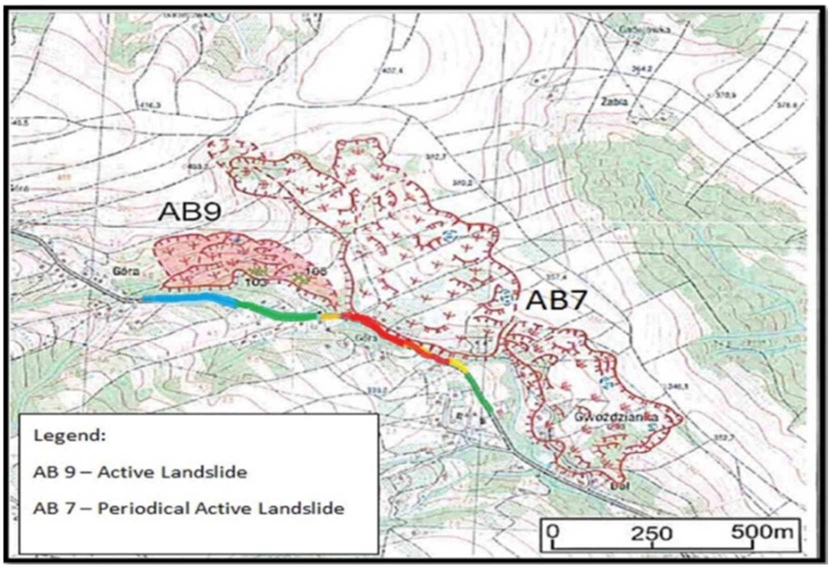

Fig. 3. Final risk map of the road section for Niebylec community [own elaboration based on 33,34]

The last stage of analyzes was the elaboration of the final risk map of the road infrastructure (fig. 3).

Analyzed local road has pavement of flexible structure. The construction cost of 1 km road distance is 720 950 PLN for this type of structure and road category. The financial consequences associated with the destruction of the road are high. The risk of road damage is very high for 0,3 km section, because the road is located near a watercourse and landslide, so the hazard risk of landslides and flooding for this road is significant.

5. Conclusions

Floods and landslides belong to natural threats and cause losses both in the natural environment, as well as in infrastructure and construction. In the communes of south-eastern Poland, there is both a flood risk and a landslide risk. These two threats very often go hand in hand as long-term precipitation causes flooding of buildings located near watercourses and, at the same time, starts landslide processes on slopes above river valleys. The knowledge of hydrogeological conditions and monitoring of geotechnical and hydrological parameters of a given terrain is the basis for forecasting the occurrence of these threats and the development of a landslide risk map. Therefore, the article underlines the issue of monitoring of these two threats in parallel.

The implementation of an investment on a landslide or in the vicinity of landslide area or flood risk area is possible, if it is combined with technical activities enabling both hazard identification and estimation of the risk of both threats. After recognizing and assessing the risk, the economic calculation of the investment will indicate whether it is economically justified to invest in such areas. On the other hand, due to the cost of the necessary safeguards it may suggest the withdrawal of investment in such risky area. This problem concerns not only private, but also public investments. It also involves the design and construction of linear infrastructure facilities used for roads and railways, which should be carried out taking into account the risk assessment. Furthermore, it is also important to ensure the safety of people and the existing building infrastructure. Increasing the level of security each time increases the economic costs of projects, especially in relation to the societies of developing countries. However, such investments in the nearest future may provide better development conditions and oppose migrations of people from the endangered areas.

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ANALYSIS OF THE INFLUENCE OF SLIP IN BOLTED JOINTS ON GLOBAL STRUCTURAL DEFORMATIONS

Shear connections of bearing type are one of the most commonly used types of bolted connections. This type of connection effectively transfers the load, when there is no gap between the bolt and the connected elements, the elimination of slack is often associated with a slipping. Neglecting the slip in an non-preloaded connection is a design error, which in some cases might lead to structural damage. The study presents the ways of taking into account the slip in connection in the analysis of the structure. The authors discuss the analytical approach based on the Fontviolant formula and the numerical approach in which software supporting the design of building structures is used. The aim of the study is to determine the influence of slip in non-preloaded bolts on the deflection of truss structures. The basis for the conducted analyses was the damage to a conveyor that was subject to major deformations as a result of excessive slip in incorrectly designed bolted connections. Theoretical analyses were carried out for flat trusses and they were expanded to cover the most commonly used types of trusses. The paper discusses the difficulties, pointing, among others, to random parameters that affect the capacity of the connection. The obtained results confirm that it is necessary to take into account the slip and its significant influence on the value of structural deflection. They also provide a set of results that can be used as initial reference for the slip calculation. Especially, the influence of slip in non-preloaded overlap bolted connections on the global deformations of the structure.

Keywords: slip of bolted connections, deflections of the structure, bar model, numerical analysis

1. Introduction

The design of building structures usually does not take into account the slippage in bolted connections. The usage of bearing type bolts in deflection-sensitive structures may result in failure to meet capacity condition in SLS, and in the case of statically indeterminable structures, may result in the redistribution of internal forces and overloading of specific structural elements.

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Depending on the distance between the bolt shank and the opening, and on the force of tightening of the bolt, connections with high or low slip capacity are obtained [1].The extreme case is the preloaded connection, which is considered as destroyed if any slips occur [6], [9]. However, using preloaded connections in all cases seems economically unreasonable. Hence, the study presents an attempt to determine the effects of use non-preloaded shear bolted connections in truss structures and in particular the influence of the slip of connecting elements on the global deflection of the truss. The subsequent tasks included determining the relation between deflection, caused by taking up of the hole clearance in the connection, and the localisation of such connection in the truss structure. The effects of the accumulation of holes clearance, from many connections used in the whole structure, on the global deformation of the analysed element were also investigated. The analysis of this phenomenon was inspired by a failure of a structure of gypsum conveyor. The structure showed excessive deflection under the self-weight, which occurred after assembly and prior to the start of operations. Due to its large dimensions, the conveyor was designed with use of the member-to-member assembly technique. The division into numerous elements to facilitate transport to the construction site and a large number of incorrectly designed connections led to damages of the structure. The analyses indicated the main reason of excessive deflection, which was the slip in shear bolted connections. Figure 1 shows a part of the structure after disassembly.

Fig. 1. Disassembled span of the conveyor structure [photo: A. Biegus]

1.1. Slip of bearing type bolts

Force transfer in the non-preloaded shear bolted connection (category A according to EN 1993-1-8 [9]) can be divided into two phases, separated by the bolt slip phenomenon. In the first phase, the main force transfer mechanism is the friction between the connected elements. It depends on the force of tightening the connectors, which is small and uncontrolled in non-preloaded bolts, and on the faying surface condition. The increase in the applied load, overcoming the friction forces, causes the bolt slip by reducing the slack between bolt shank and the opening. The bolt slip ends when the connectors start pressing on the connected elements.

Relative displacement between the connected elements, called major slip, theoretically can equal to two hole clearances (fig. 2) [10].

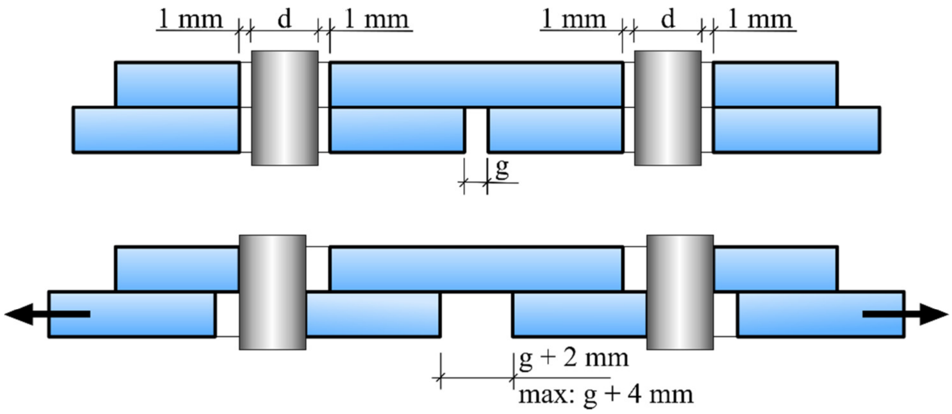

Fig. 2. The slip effect in the joint due to load, based on [2]

According to the provisions of the PN-EN 1993-1-1 standard [8] in the global analysis of the structure, the slip in non-preloaded connection should be taken into account whenever justified. This refers in particular to structures in which the influence of deformations on the global static is significant. However, it is noted that no guidelines concerning the manner of determining such slip were provided.

Although newly designed structures use butted connections, which completely prevent the occurrence of this type of slips, structures with overlap connections are also used. This results, among others, from the tradition of forming the structure and the connections between its elements. Structures with overlap connections are commonly used in the USA, where such connections are considered as the basic solution. Additionally, as it is much easier to calculate the bearing capacity of shear type connections in comparison to the complex procedure of dimensioning butted connections, many designers prefer to use the first ones.

1.2. Scope of the study

The point of reference for the conducted analyses is the problem of excessive deflection of conveyor ramp, whose parts were connected with use of non-preloaded shear connections.

The damage of the structure, which subjected to self-load, in reality deflected far beyond the results of static calculations, gives a clear signal that the applied calculation model is not proper for the construction system with shear overlap connections. Thus, a structure using such connections requires an analysis that would take into account the slip between connected elements [8] (such as in [3]). However, this is a very complex issue, and the parameter defining the final deformation of the structure is a random variable dependent on slips in specific connections.

The importance of the discussed subject is also confirmed by the analyses of connection slip on structural elements of the Louvre Abu Dhabi Dome [5] and PhD thesis concerning verification of bolted connection for large span roof [4].

In order to determine the influence of slip in non-preloaded connections on the deflection and redistribution of internal forces, the authors analysed selected flat truss systems. The parameter in numerical analyses was the type of truss. Trusses with “N”, “K”, “V” and “X” bracing type were analysed (fig. 6). The aim of using various types of trusses was to determine, which of them is the most prone to taking up hole clearance in shear connections.

The study presents two approaches to estimating the influence of slip in shear connections on the vertical deflection of trusses. The first one consists in numerical analyses considering the member models of flat trusses, taking into account the non-linear nature of the deformation of specific members in the lower and upper chords and the diagonals. The second approach is based on analytical calculations with use of the Bertrand Fontviolant equation to determine the effects of slack in connections.

2. Problem of excessive deflection of the conveyor belt ramp.

The bearing structure of the conveyor ramp consists of spatial, four-chord trusses. The side wall trusses (“wall trusses”) of the conveyor have parallel chords, with N-type bracing. The “wall” trusses are connected with each other in the planes of their top and bottom chords by “floor” and “ceiling” trusses. All the members of the truss spans of the ramp were designed as hot-rolled I-beams. The member-to-member assembly technique was adopted, and the spatial truss structure was assembled at the construction site from single members connected in nods, with use of shear bolted connections. High strength bolts M16, M20 and M24 in grade 8.8 were used. The bolts in connections were classified as non-preloaded.

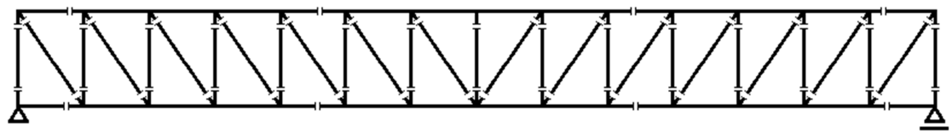

Fig. 3. Schema of the side wall truss of the selected span of the conveyor

The structure designed in such way was supposed to meet the requirements concerning the bearing capacity and required functionality, including the values of acceptable deflection [7]. As arranged with the investor, the threshold deflection of conveyor span was $w_{max,k} = \frac{L}{350}$, i. e. 33750/350 = 96 mm.

Geodetic measurements performed after the ramp had been assembled confirmed excessive deflection which occurred under self-weight of the structure (straight parts without precambering were used).

For the analysed span, the measured deflection in the middle of the span length is $w_{(1,rz)}$ = 81 mm, while according to the design calculations (no slip considered), the same deflection is $w_{(1,st)}$ =11 mm.

3. Methods of analysing slip in nodes of truss structures

Detailed analysis of slip in non-preloaded shear connections requires experimental research or extensive numerical analyses. This is a complex process, and even if it is conducted with all due diligence, it refers only to the analysed connection and cannot be generalised for all analogical instances. The obtained results are caused by the random nature of the connection assembly. The final slip value is affected by such factors as: hole diameter, hole pattern or the initial position of the bolt in the hole.

In order to determine the influence of slip in non-preloaded shear connections on the deflection and redistribution of internal forces, authors analysed various truss structure diagrams. Trusses with “N”, “K”, “V” and “X” bracings were analysed. Figure 4 shows half of the selected truss scheme subjected to static analysis. The span of the analysed elements was 33.6 m. The dimensions of each “panel” of the truss are 2.4 x 3.26 m. In order to broaden the scope of tests each of the N and K type trusses were analysed in two variants, i.e. when the diagonals are extended or compressed (under self-load).

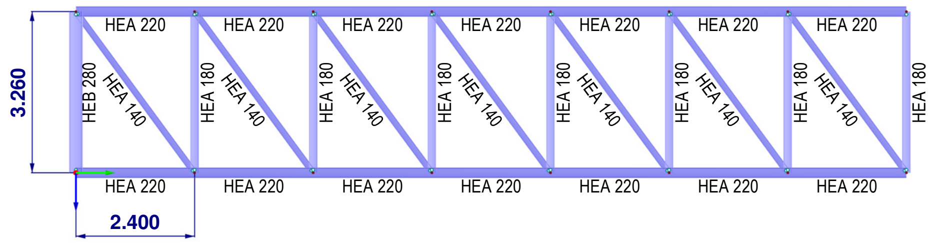

Fig. 4. Dimensions of the truss and cross-sections of members in the analysed truss element

3.1. Numerical modelling of slip in connections

In order for a connection with clearance to transmit to the node the load required by the attached member, the bolt must come into contact with one or other of the connected parts: This is often referred to as ‘taking up slack’.

The phenomenon of taking up slack can be considered in static calculations, among others, by assigning a non-linear characteristics of the node (Fig. 5a) or "changing" the length of the member (Fig. 5b, c). For a connected tension member, this slack can be assimilated as an additional extension that is added to the elastic elongation of the member in tension (Fig. 5b). Likewise, for a connected compression member, the slack is consider as a reduction in length that is added to the elastic shortening of the compressed member (Fig. 5c). However, it should be noted that assimilating the slack by changing the member length may be used to calculate the deflections of the system, but special care is recommended when estimating the internal forces in members (change in member length affects the internal forces in elements).

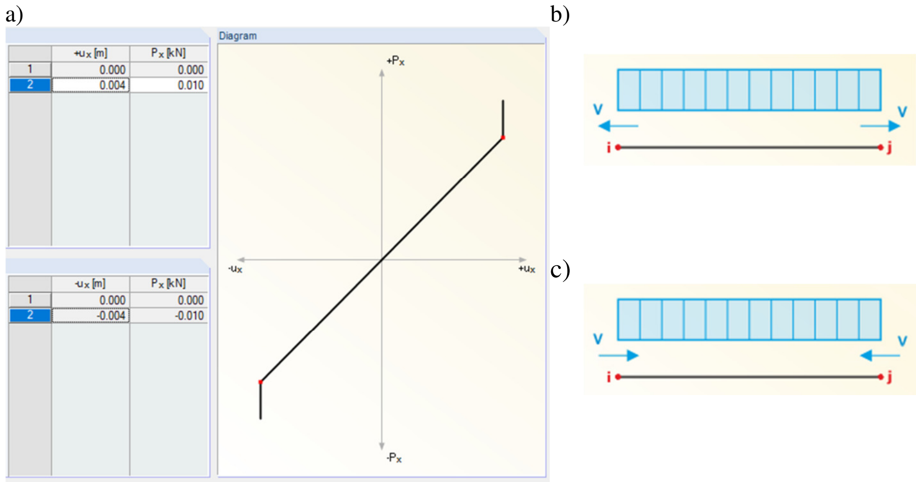

Fig. 5. Different numerical approaches to the determination of slip influence on connections: a) non-linear node model, b) elastic elongation of the member in tension, c) elastic shortening of the compressed member

For the purposes of estimating the influence of slip in connections on structural deflection, an e^1, p^2 class model was defined and geometrically non-linear elastic analysis (GMA analysis) was conducted. Trusses were analysed as simply supported elements, loaded only with self-weight. The geometry and cross-section characteristics correspond to the used in the conveyor discussed in Section 2, and the analyses were expanded to cover various types of trusses. Displacements caused by taking up of the holes clearance in connections were simulated by introducing joints of a potential set displacement in the node of the member (cf. Fig. 5a). The geometry of the analysed structures is presented in Fig. 6a-f.

Each of the arrangements was analysed for five slip values, i.e. for the slip, respectively, of 0 to 4 mm, where 0 is to be understood as reference value – a non-slip connection. Slips were set on truss chords by simulating the contact of transmission elements, in panels 1, 2, …, 6 from the support to the center of the truss span respectively (fig. 3).

The results were provided as relative ones, in reference to the slip value of 1 to 4 mm in relation to the basic (reference) value, and presented in Fig. 7–10.

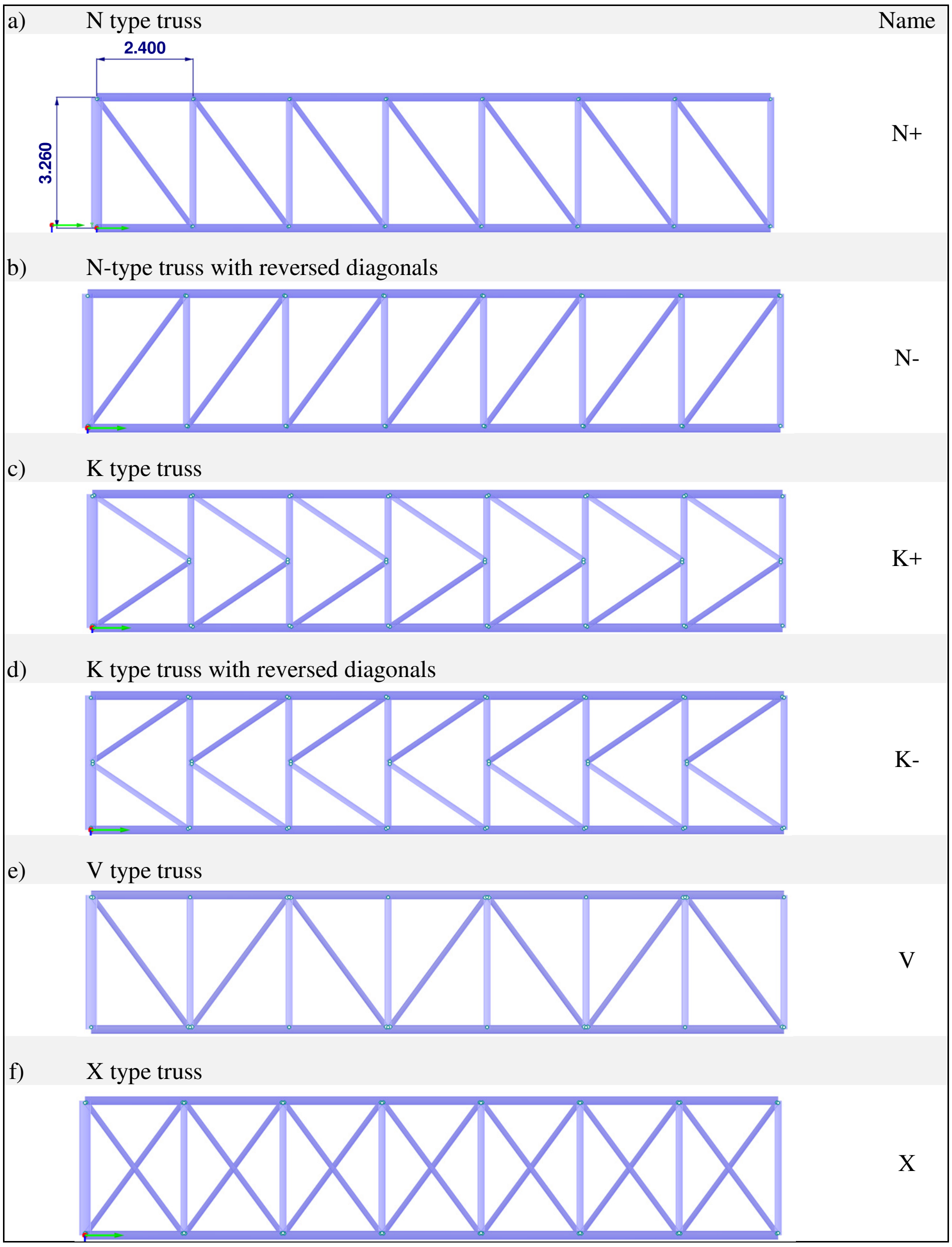

Fig. 6. Types of trusses whose proneness to connection slip was analysed. Trusses with bracing: a) type N, b) type N with reversed diagonals, c) type K, d) type K – reversed diagonals, e) type V, f) type X. For simplification purposes, only half of the system was presented

3.2 Analytical approach to estimation of the slip in connections

The literature [2] provides the method of analytical consideration of the slip phenomenon in truss shear connections. The formula is a function of the axial force in a member in which connection was designed, and the single force applied in the middle of the element span. The Bertrand Fontviolant formula has the following form:

$$v = \sum_{i=1}^{i=b} N_{1,i} \frac{F_i l_i}{ES_i} \tag{1}$$

where: v – displacement at a given point from unit force,
$N_{1,I}$ – is the axial force in the i-th element, caused by the unit force applied at the point where the deflection is investigated,
l_i – is the length of member i,
S_i – is the section area of member i,
b – is the number of elements with bolted connections,
$\frac{F_i l_i}{ES_i}$ – is the variation in length of member i due to slack recovery ±4mm according to whether the chord is in compression or tension.

This equation was used to estimate the deflection of the trusses presented in Fig. 6a–f. The results for trusses where shear connections are on the top and bottom chords in the panel No. 6, symmetrically to the centre of the system, are shown in the Table 1.

Table 1. Deflection results considering slip according to the Bertrand Fontviolant formula

Bracing type	N+	N-	K+	K-	V	X
Force in member from unit load -*N_i*	2.2	1.83	2.21	1.83	1.83	2.01
$\frac{F_i l_i}{ES_i}$	4	4	4	4	4	4
Deflection considering slip $v = \sum_{i=1}^{i=b} N_{1,i} \frac{F_i l_i}{ES_i}$, **[mm]**	35.2	29.28	35.36	29.28	29.28	32.16

4. Results of numerical analyses

Diagrams of the relationship between displacement and the value of potential slack in overlap shear connections were created for the conducted numerical analyses. These relations are presented in Figures 7–10. Results are presented as relative values, for each truss scheme the displacement values 'u' refer to the case of zero slip, to enable comparing the diagrams, in other words the influence of the bracing type on deflection was eliminated.

The diagrams in Figures 7 and 8 present the relationship between relative displacement of the structure and the value of slack that exist in the connection. The connections in the top and bottom chords, located in the extreme positions were shown as a representative case, i.e. the first panels of the truss near supports and sixth panels of the truss near the midspan (cf. Figure 6). The shear connections were symmetrically distributed on the truss.

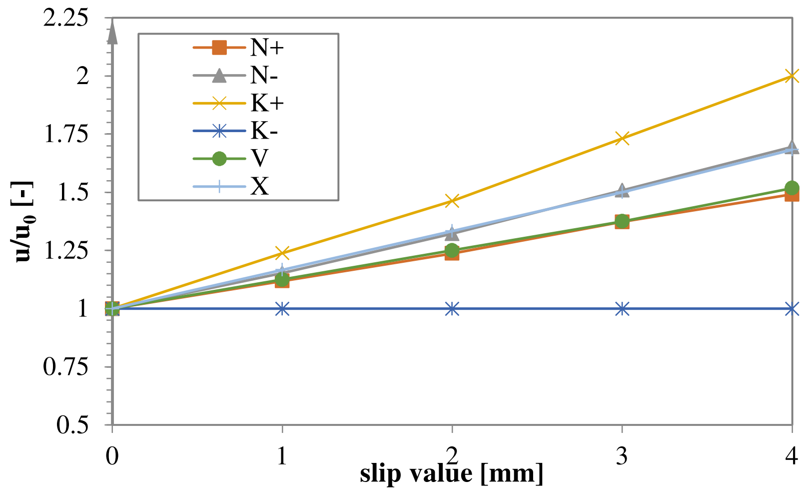

Fig. 7. The relationship between the relative displacement of the truss and the slip value, derived connection localised in the first panels from the supports

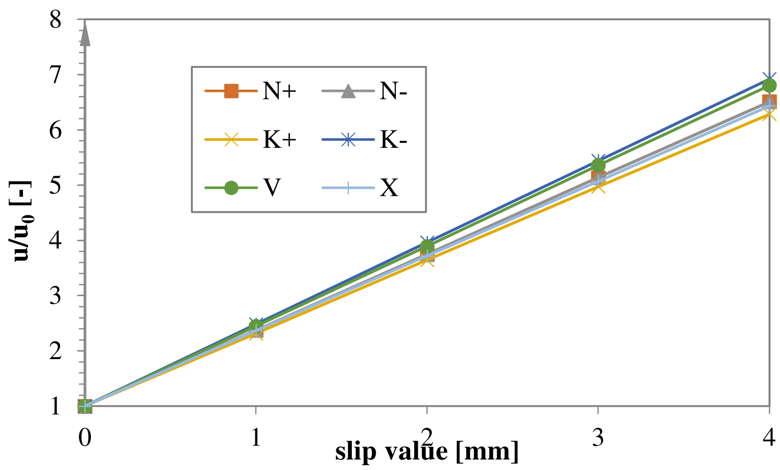

Fig. 8. The relationship between the relative displacement of the truss and the slip value, derived connection localised in the sixth panel from the support

Increasing the value of potential slip in connections in the truss structure causes a significant increase in displacements.

Only for truss K- (Fig. 6) and the connection located in the extreme panel a constant deflection value was obtained, i.e. i.e. the slips in chord connections do not affect the displacement value, which results from the location of connections on the so-called zero force members.

The numerical analyses for trusses, where the connections were modelled directly at the support resulted in a twofold increase of displacement as a result of maximum slack equal to 4mm. When the connections were modelled in panels close to the mid-span of the truss, the deflection increased near seven-fold in comparison to the model without slack in connections. Additionally, the analysis of diagrams N+ and N- (Fig. 7), where in the first panel the top and bottom members are zero force members, reveals that the influence of slip in the bottom chord on the value of these deformations is higher than the slip in the upper chord.

The analyses are summarized in the graph of the impact of the connection location on the length of the structure on the relative deflection of the truss (Fig. 9). The connections in the model were always in pairs (on the top and bottom chord) and they were placed symmetrically in relation to the centre of the truss. Six situations were analysed, starting from the position of connection in the panel No. 1 to its position in the panel No. 6, i.e. located 2.4 m from the centre of the truss. The slip of 2 mm in the connection was assumed as the meaningful slip level.

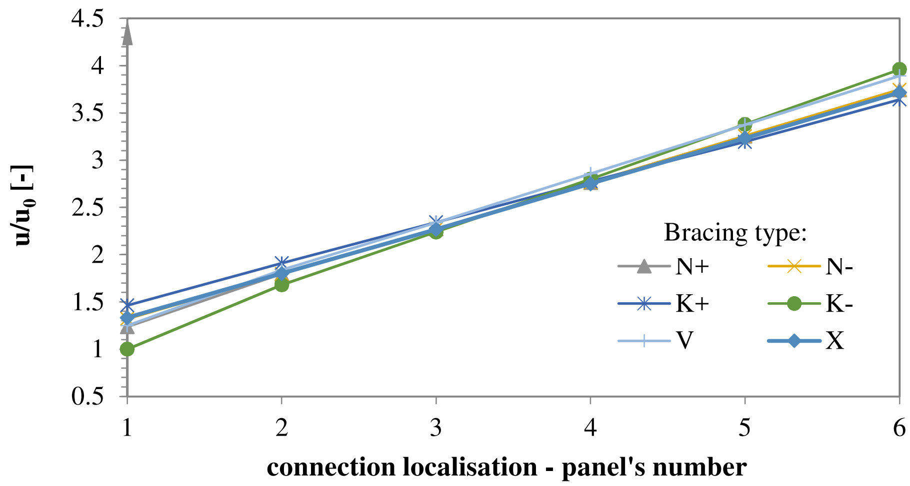

Fig. 9. The relationship between the relative displacement of truss structures and the position of overlap shear connections (panels No. 1 to 6)

When designing two bolted splice connections on the length of the truss and placing them in the fifth panel of the truss, one should expect about four-fold increase in the actual deflection, regardless of the type of truss.

In reference to the geometry of the truss of the damaged conveyor discussed in Section 2 and, at the same time, an arrangement that is one of the most prone to slip in connections (according to the conducted analyses, Fig. 9), detailed results were provided for V type truss for different slip values (Fig. 10).

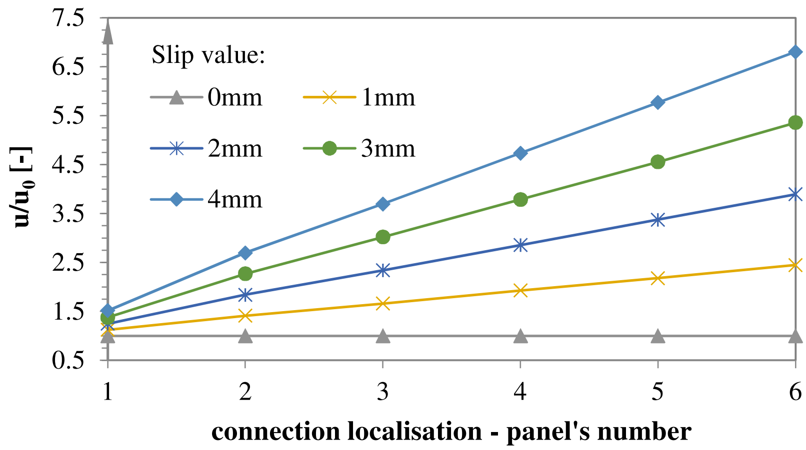

Fig. 10. The relationship between the relative displacement of truss structures and the position of overlap shear connections on the top and bottom chords of trusses

The diagram shows the results for truss pattern V, for which the highest increase in deflection was noted. In this case, the ratio of deflection increase with the assembly connection approaching the mid-span of the truss is 6.8, and the maximum deformation at 4 mm slack is 38.1 mm, while for analytical analysis (according to the Bertrand Fontviolant formula) it is 29.28 mm, which means a difference of approx. 30%.

In the analysed conveyor, the actual deflections were about 8 times greater than those calculated in the design process. This results from the fact of the influence of accumulated slips from several connections on the total deflection of the structure.

5. Conclusions

In accordance with conducted analyses the following conclusions were drawn:

1. The value of the obtained vertical deflections considering the potential slip in non-preloaded shear connections corresponds to the measured deflections $w_{rz.}$ They were obtained for a model with fixed slip of a constant value not exceeding 4 mm. However, uneven slips on specific connections were not taken into account, as they are of a random nature.

2. Structures, where the deflection criterion is essential, or those with a large number of assembly connections due to their size should be designed with use of preloaded connections. If non-preloaded shear connections are used in such structure, the slip impact on the value of deflection and internal forces should be taken into account.
3. Structures which are loaded with dynamic loads or in case when there is no possibility to use fitted bolts in non-preloaded bolted connection, than connections need to be designed as preloaded slip-resistant connection.
4. Neglecting the connection slip results in incorrect deflection results.
5. It is possible to estimate deflection considering the slip in overlap shear connections with a certain accuracy with use of the Bertrand Fontviolant equation. Still, it does not provide an accurate result of structural deformation.

In conclusion, control of slack in non-preloaded connections is important for trusses. In order to reduce or eliminate the problem of slack caused by deflection, it is necessary to: reduce slack in A category connections, e.g. by drilling with an excess of +1 mm or using fitted bolts, preloaded connections or welded connections instead of bolted ones.

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CRISIS PHENOMENA IN "CITY-DISTRICT"

Crisis phenomena occur in the majority of settlement units regardless of location, size, social structure and other significant factors influencing the quality of life and correctness of urban processes. This paper is an attempt to present relations of social problems and urban layout and genesis of the spatial structure of the selected small town. Unit selected for analyses – Poniatowa town, characterizes with specific layout and time of founding – this is a town founded "in cruda radice" during the construction of structures of the Central Industrial Region (Polish: Centralny Okręg Przemysłowy, abbreviated COP) built on the basis of residential development for nearby industrial plants associated with military production.

Keywords: city-district, multi-family development, Poniatowa

1. Introduction

In general belief small towns are settlement units with specific morphological structure - centrally located main (market) square surrounded by a more or less compacted frontages, then dense small town tissue gradually transforming into dispersed single-family and farmstead developments [1], [8], [12]. Sometimes this structure includes multi-family buildings occurring as a single or in groups identified by researchers as a pathology of urban layout of the small town [2].

However, there are cities with a residential development at its core, which were constructed as a result of location "in cruda radice" of polycentric settlement unit, e.g. industrial plant and ancillary residential development intended for engineering, civil service and working staff brought for the operation of the plant [11]. These developments, at the time of construction, characterized by a specific social structure - people involved in production in local plants and their families were almost exclusive inhabitants. City-forming process followed over time. New services were created and city tissue was filled with new residential buildings. Developing residential estate has acquired features of urban environments and received town privileges.

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A town located in eastern Poland - Poniatowa (Lublin Province) was selected to present the problem. Settlement system of Poniatowa was formed during construction of the Central Industrial Region, which was intended to be a source of development of the re-emerging Poland at the beginning of the 20th century. Location of the Central Industrial Region, besides the territorial and geographical dimensions, had also very high social value. First of all, it is worth pointing out that Poland at that time characterized with very inconsistent economic development - two areas could be distinguished. Area A, that is western part of the country concentrating 73% of the industry and area B with marginal industrial resources what has been influencing living conditions of the inhabitants, their education and cross-section of the society [13]. Due to investments in the development of the Region over the period 1936-1939 a major change in social-economic functioning in this part of the country was observed: increase of work productivity, an increase of professional qualifications of the employees and in consequence improvement of their standard of living. Requalification from agricultural activities to a group of citizens living from gainful employment resulted in a considerable reduction of unemployment [13]. At the same time, dynamic changes in the intensity of industrialization of the Region resulted in an influx of migrant population of different social statuses what caused series of negative effects: reduction of public safety, thefts and burglaries, robberies and excessive and uncontrolled price increase [4].

2. Functional and spatial transformations of the selected urban centre

Poniatowa is a town classified as small (up to 20 thousand inhabitants) inhabited by about 9 thousand people. First investments have been made over the period 1937–1939 and a status of the town was granted in 1962.

Construction of urban-type residential development in Poniatowa village began from three factory buildings of the Państwowe Zakłady Tele- i Radiotechniczne [National Tele and Radio Rearch Plant], worker's hostel and residential building for technical staff. Further on the implementation of the urban project for large residential development, which was the biggest investment planned for the Central Industrial Region, was started. Twenty two multi-family residential houses in comb layout were successfully completed until September 1939 [5]. During the war German troops were stationed here and then buildings of the production plant and residential facilities were used as a camp for Russian prisoners of war and then for a labor camp for Jews [7]. The Red Army stationed in the camp after the war and following the departure of the troops Poniatowa was an abandoned and deteriorating town.

The nationalised company was transformed into Zakłady Wytwórcze Sprzętu Instalacyjnego (a branch of the company from Bydgoszcz) in 1944 and new inhabitants started to appear in the town. The 1950s and 1960s is the period of a heyday for the town. It was at that time when Nowe Osiedle residential development was constructed - which is currently a core of the urban unit and important city-forming services: “Czyn” movie theatre, Anti-tuberculosis Sanatorium, Factory’s Cultural Center, summer camp centre and stadium. Social activity was concentrated around the factories where most inhabitants and visitors were employed. Similarly to many towns the turn or 1980s and 1990s, that is system transformation period, was a time of slump in the functioning of the industry in Poland. The factories were transformed into EDA joint stock company, dismissals of the employees began and a number of unemployed people started to increase. EDA company announced its bankruptcy in 1998 [3]. Currently several companies are operating at the premises of the factory: Zakład Produkcji Granulatu Gumowego Orzeł S.A., a manufacturer of films for thermoforming Polifolia sp. z o.o., Metalton specializing in the production of precision tools and Browar Zakładowy.

The urban tissue of Poniatowa preserved a character of residential development although the multi-family buildings constructed in the last forty years do not have so compact and strictly defined structure. Single-family houses were constructed mainly on the outskirts and service areas and potential public spaces are located at the meeting point of old tissue of Nowe Miasto and new, more dispersed residential structure.

3. Crisis phenomena in the spatial structure of the town

In the context of history and functional transformations of the town, the following question seems to be very interesting: what phenomena and problems relate to the city area with such specific genesis and characteristic urban structure? A map extract (Fig. 1) was prepared using the Local Revitalisation Program for the Poniatowa Borough for 2017–2023 (hereinafter referred to as LPR) by overlaying the boundaries of auxiliary units on the topographic map obtained from the BDOT10k database [9]. Referring the identified crisis phenomena to indicated part of the structure it is possible to indicate social problems accompanying the areas of specified genesis, history, urban layout and architecture.

The Local Revitalisation Program identifies the major social problems in the town:

- the steady decline in the number of inhabitants,
- negative natural growth rate,
- negative net migration rate,
- the decrease in the number of people in pre-working age and an increase in the number of people in post-working age,
- a large number of inhabitants being dependent on unemployment benefits,
- increase in the number of dysfunctional families,
- accumulation of dysfunctions in the town centre: alcoholism, drug addiction, hooliganism, high level of crime,

- insufficient infrastructure for care and mobilization of older people,
- relatively high level of unemployment resulting from stagnation and discouragement of people.

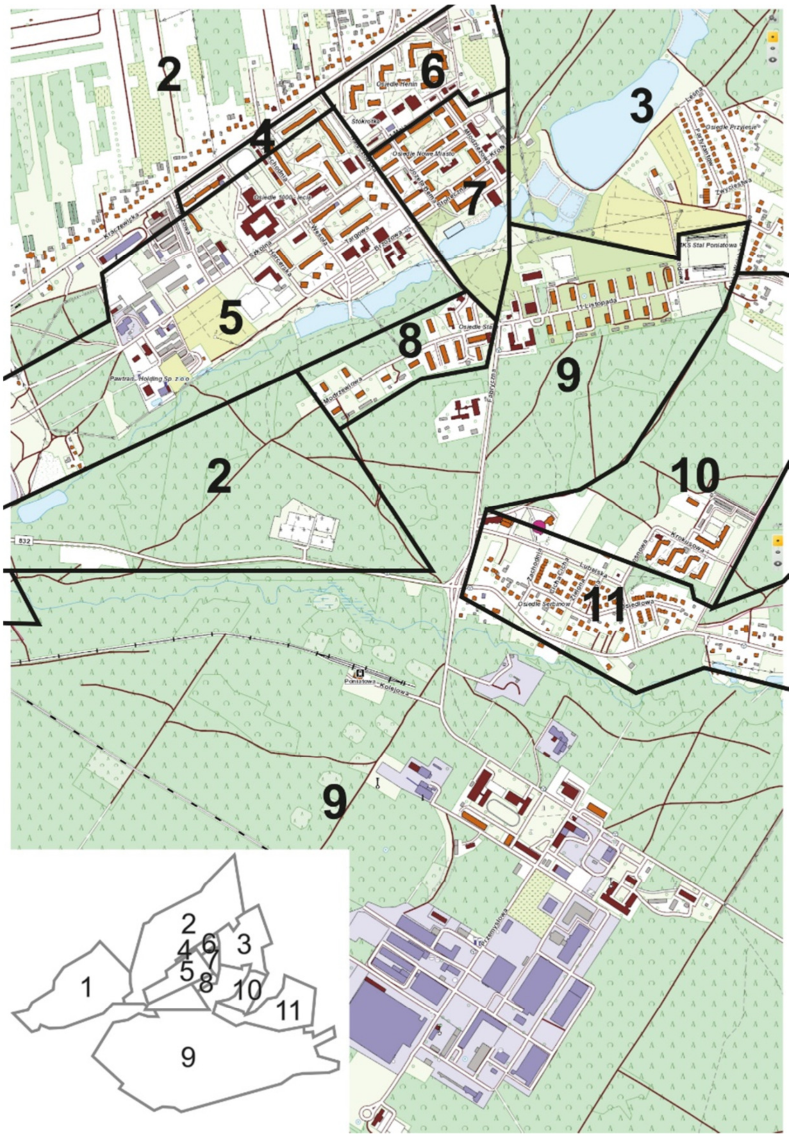

Fig. 1. Map extract from the BDOT10k database with marked boundaries of auxiliary units indicated in the Local Revitalisation Program for the Poniatowa Borough for 2017–2023 for identification of crisis phenomena in the area of the town. Boundaries of the borough with division to auxiliary units at the lower left: 1 – Leśniczówka, 2 – Henin, 3 – Przylesie, 4 – Tysiąclecia – Kraczewicka, 5 – Tysiąclecia, 6 – Nowe Miasto – Północ, 7 – Nowe Miasto – Południe, 8 – Stare Miasto – Modrzewiowa, 9 – Stare Miasto, 10 – Przedwiośnie, 11 – Młynki (own work based on the LPR)

Concentration of occurrence of the above-mentioned phenomena differs depending on the auxiliary unit: Leśniczówka '1', Henin '2', Przylesie '3', Tysiąclecia – Kraczewicka '4', Tysiąclecia '5', Nowe Miasto – Północ '6', Nowe Miasto – Południe '7', Stare Miasto – Modrzewiowa '8', Stare Miasto '9', Przedwiośnie '10', Młynki '11'. Structural units were separated based on the culturally established names of residential developments, districts, areas, specific urban development, architectural character and period of construction of housing stock.

Analysing the population figures [6] (Fig. 2) within the 5-year period (2010–2015) it can be noticed that the highest decrease in the number of inhabitants was noted for Tysiąclecia '5' (-8.3%) and Nowe Miasto-Północ '6' (-10.7%) units, that is the areas of relatively young urban structure, where development was constructed in the 1970s, 1980s and 1990s. In urbanised units, Stare Miasto – Modrzewiowa '8', Stare Miasto '9' depopulation is smaller but it is still high, '8' (-6.8%), '9' (-6.6%) respectively. Analysing housing structure in terms of time of construction and the structure these are the oldest parts of the town of interwar genesis and with prevailing comb layout. Against this background, an interesting issue is a change of population on the area of Nowe Miasto-Południe '7' that is in city core region with quarter-meander development formed in the 1950s. Depopulation in this part is relatively small and equals to -2.2%. The largest population growth of 13.7% was noted for Leśniczówka '1' unit where single-family housebuilding industry is developing and it is in line with the trends where city inhabitants move to suburban or peripheral areas [10].

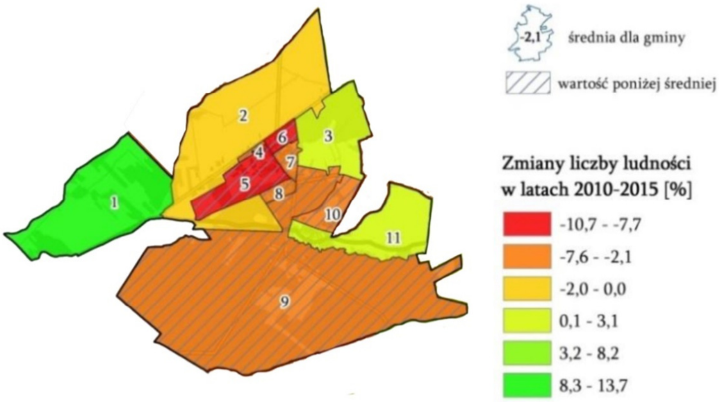

Fig. 1. Population changes for 2010-2015 [%] (own work based on the LPR)

Another social problem found in the area of Poniatowa is a high percentage of post-working age population (Fig. 3). The highest share of older people is found in Stare Miasto – Modrzewiowa '8' (31.9%), Nowe Miasto – Południe '7' (29.2%) and Tysiąclecia '5' (26.7%) units. These are adjacent areas but varying in terms of time of formation of spatial structure (from the 1930s to 1980s).

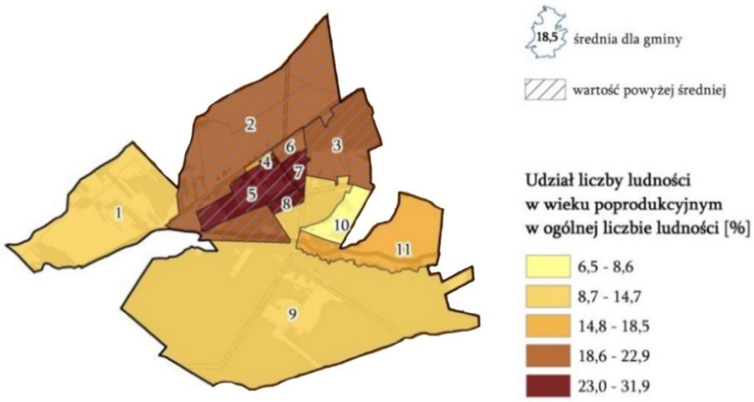

Fig. 2. The share of post-working age population in the general population [%] (share of older people) (own work based on LPR)

Areas with the smallest share of the population in post-working age are units of peripheral character with single-family development Leśniczówka ‘1’ (11.5%) and Przedwiośnie ‘10’ (6,5%) and, interestingly, Stare Miasto ‘9’ (12.0%) [6]

Map of offenses and crimes (Fig. 4) shows that the highest intensity of events is located in the units forming a city core, constructed between the 1950s and 1980s: Nowe Miasto – Południe ‘7’ (54.8 per 1 thousand inhabitants), Tysiąclecia ‘5’ (52.7 per 1 thousand inhabitants) and for Stare Miasto ‘9’ (48.4 per 1 thousand inhabitants). No events of infringement were noted for Leśniczówka ‘1’ area.

Totally different distribution of intensity of occurrence of the studied phenomenon is presented on the extract from percentage share of long-term unemployed persons in the general population (Fig. 5). The largest number of people live in Stare Miasto ‘9’ (9.6%) oraz Leśniczówka ‘1’ (8.6%) units that is

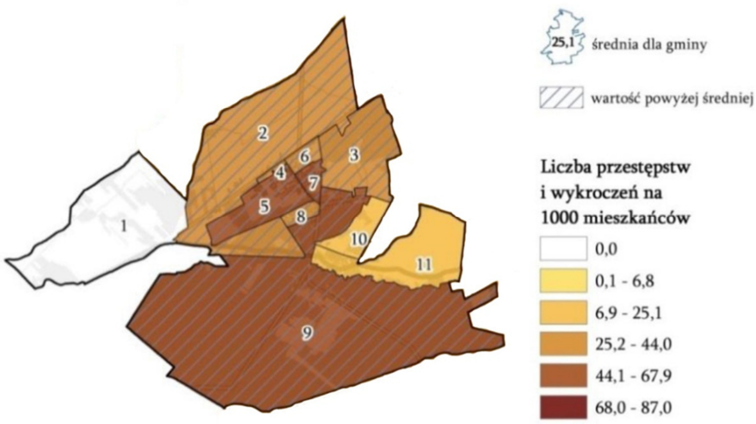

Fig. 3. Number of crimes and minor offences per 1000 inhabitants (own work based on LPR)

areas of various time of origin (interwar period and on the turn of 20th and 21st century) and various prevailing form of use (multi-family residential buildings and single-family buildings). Central areas Nowe Miasto – Południe '7', Nowe Miasto – Północ '6', Tysiąclecia '5', Stare Miasto – Modrzewiowa '8' and Tysiąclecia – Kraczewicka '4' indicates at least half of number of long-term unemployed persons, respectively: 4.4%; 3.9%; 3.3%; 3.2%; 3.1%; The lowest unemployment rate was noted in the peripheral Młynki '11' unit (2.2%) [6].

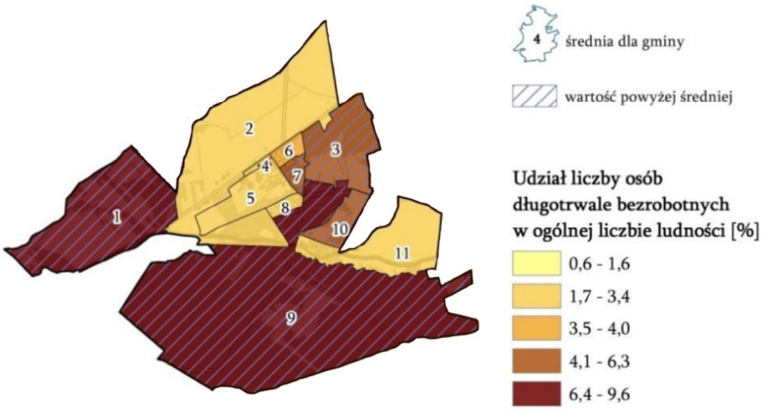

Fig. 4. The share of the number of long-term unemployed persons in the general population (own work based on LPR)

4. Summary

Small towns, where groups of multi-family buildings are town-forming centres occur very rarely and are the result of planned spatial actions of designers and location of structural units "in cruda radice". In general, their functional structure is polycentric: residential development or developments in relation to industrial plant, for purposes of which the residential developments were constructed. A segregation of urban and architectural form can be noticed while analysing the local scale.

Looking at existing layout, from a sociological point of view, it can be assumed that because the tissue grown in roughly controlled manner in terms of time, function and form (new developments were added to these already existing) it can be assumed that units (developments, districts, etc.) formed in this way have a diversified social structure and thus social phenomena, including crisis phenomena, occurring in the units differs.

In the case in question, of Poniatowa town, information related to the functional-spatial structure of the town and basic data diagnosing condition of social infrastructure obtained from an analytical part of the Local Revitalisation Program for the Poniatowa Borough for 2017-2023 were compared. This allowed to draw conclusions (concerning given example) related to the correlation between the morphology of the studied area and crisis phenomena occurring in separate auxiliary units. This analysis led to conclusions that negative social phenomena (decline in population, demographic ageing and crime rate) concentrate in the western part of Poniatowa, where multifamily blocks from the 1970s and 1980s, Tysiąclecia residential development, dominate. The oldest part of the town (Stare Miasto, Stare Miasto – Modrzewiowa and Nowe Miasto – Południe) the discussed crisis phenomena are also visible, but to a substantially lesser degree. The interesting conclusion is a fact that area, where a share of long-term unemployed persons in general population is the highest does not match with the area delimited by analysis of the other crisis factors and covers Stare Miasto unit, that is the oldest part of the Poniatowa, with prevailing development from the 1930s.

Considerations carried out cover only an outline of the identification of social problems in towns-districts, however it seems that they can be useful for the shaping of sustainable development policy for small towns of specific layout of the spatial structure.

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APPLICATION OF AN INTERFACE DAMAGE MODEL TO STEEL REINFORCED CONCRETE: A STUDY OF THE SIZE EFFECT

Influence of the size of steel reinforcement of a concrete structure on crack initiation at the interface between the steel fibre and the concrete body of the structure is under consideration. Numerical analysis is provided using a quasi-static delamination model for interface rupture based on an energetic approach using a cohesive zone model for providing the interface stress-strain relation. The obtained results confirm expected dependence of the critical load which causes triggering of the interface crack on a structure dimension parameter.

Keywords: interface crack, damage evolution, quasi-static delamination, critical load, crack mode

1. Introduction

Several experiments confirmed that the size of a fibre placed in a fixed volumetric unit of a composite material influences various macroscopic properties of such composites. For instance, the tensile strength was found to depend on the size of an inclusion in works of Fisher et al. [3] or Cho et al.[2]. Such a kind of the size effect was explained by using different cohesive zone models (CZM) for describing the interface stress-strain relations by Carpinteri et al. [1], or Tavara et al. [9]. Based on such results, the aim of the present work is to apply the model developed in [5][10] to a problem which leads to the

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aforementioned size effect. Additionally, numerical tests also show the dependence of the size effect on the transverse loading, as shown by Carpinteri et al. [1], or Mantič [6][7], therefore the proposed tests are to demonstrate in some sense such influences by introducing various boundary conditions in the solved problem.

The predictions of interface failure are calculated by implementing a quasi-static rate-independent model of interface rupture which uses a bilinear CZM as in [11]. The evolution in time is governed by the total potential energy functional E and the functional of energy dissipation R [8][12].

The evolution model is described in Section **Błąd! Nie można odnaleźć źródła odwołania.**, very briefly with underlying energy functionals. The main part of the analysis is summarizes in Section **Błąd! Nie można odnaleźć źródła odwołania.**, where the size effect is assessed in a particular example of steel fibre reinforcement placed in concrete matrix.

2. A brief description of the interface damage model

The section briefly reviews the mathematical formulation of the used model which is based on energy balance. The total potential energy E is given by the mechanical energy stored in the bulk and at the interface as a function of displacements u and the internal variable for interface damage ζ, ($0 \leq \zeta \leq 1$), decreasing from 1 (undamaged state) to 0 (crack initiation and propagation), i.e. $\dot{\zeta} \leq 0$. The damage evolution is considered rate independent so that the dissipation potential R, being a degree one positively homogeneous function of the damage rate $\dot{\zeta}$, is also the dissipation rate. The model consists of linear elastic solids in contact along adhesive interfaces whose constitutive law includes both a linear elastic and a softening branch. The numerical approach is formulated as a recursive time-stepping procedure for finding local minima of the sum of the changes of total potential and dissipated energies, one with respect to u and the other with respect to ζ. By a special choice of the interface energy functional, the interface constitutive relation of the bilinear CZM is obtained in the following form, cf. [11]:

$$p_{\mathrm{n}} = k_{\mathrm{n}}\phi(\zeta)\delta_{\mathrm{n}} + k_{\mathrm{g}}\langle\delta_{\mathrm{n}}\rangle_{-}, \quad p_{\mathrm{t}} = k_{\mathrm{t}}\phi(\zeta)\delta_{\mathrm{t}}, \quad \phi(\zeta) = \tfrac{\beta\zeta}{1+\beta-\zeta}, \beta > 0 \text{ on } \Gamma_{\mathrm{c}} \quad (1)$$

where $p_{\mathrm{n}}(x)$ and $p_{\mathrm{t}}(x)$, respectively, are the normal and tangential components of the interface tractions, $\delta_{\mathrm{n}}(x)$ and $\delta_{\mathrm{t}}(x)$, respectively, are the normal and tangential relative displacements between opposite interface points, i.e. $\delta = u^{\mathrm{A}} - u^{\mathrm{B}}$ (superscripts A and B refer to the bodies adjacent to the interface from both its sides), k_{n} and k_{t}, respectively, denote the normal and tangential stiffnesses of the adhesive in the interface. The additional term with $\langle\delta_{\mathrm{n}}\rangle_{-}$, denoting the negative part of the relative normal displacement, provides the normal compliance contact model with finite interpenetration ($k_g >> k_n$) at the interface Γ_c.

The interface damage (and subsequent crack) evolution is controlled by the energy release rate G whose critical value $G_d(\delta)$ is the fracture energy being defined as fracture-mode sensitive, e.g. according to the Hutchinson-Suo law [4]

$$G_d(\delta) = G_{Id}\left(1+\tan^2\left(\left(\frac{2}{\pi}\arctan\sqrt{\frac{G_{IId}-G_{Id}}{G_{Id}}}\right)\arctan\left|\sqrt{\frac{k_t}{k_n}}\frac{\delta_t}{\langle\delta_n\rangle_+}\right|\right)\right), \tag{2}$$

where the parameters G_{Id} and G_{IId} express the fracture energies in the pure Mode I and in the pure Mode II, respectively, and $\langle\delta_n\rangle_+$ denotes the positive part of the relative normal displacement to exclude a state of compression from damage propagation.

2.2. Energetic formulation of the model

Let us consider the energies which are taken into account in the model for two domains Ω^A and Ω^B with boundaries Γ^A and Γ^B, respectively. The stored energy functional is defined as

$$\begin{aligned} E(\tau;u,\zeta) = {} & \int_{\Gamma^A}\frac{1}{2}u^A.p^A(u)\mathrm{d}\Gamma + \int_{\Gamma^B}\frac{1}{2}u^B.p^B(u)\mathrm{d}\Gamma + \\ & \int_{\Gamma_c}\frac{1}{2}\left[k_n\phi(\zeta)\delta_n^2 + k_t\phi(\zeta)\delta_t^2 + k_g\langle\delta_n\rangle_-^2\right]\mathrm{d}\Gamma, \end{aligned} \tag{3}$$

for the admissible (time τ dependent) displacement $u^\eta = w^\eta(\tau)$ on a part of boundary Γ_u^η with η=A, or B. The first two integrals, representing the elastic strain energy in the subdomains Ω^η, are expressed in their boundary form. The last integral is the interface term which corresponds to the expected interface stress-strain relation (1).

The potential energy of external forces (acting only along a part of the boundary denoted Γ_p^η) is given by the relation

$$F(\tau;u) = -\int_{\Gamma_p^A} f^A.u^A\mathrm{d}\Gamma - \int_{\Gamma_p^B} f^B.u^B\mathrm{d}\Gamma, \tag{4}$$

where $f^\eta(\tau)$ are the prescribed forces on the part of boundary Γ_p^η . The dissipated energy is introduced by the (pseudo)potential R which reflects the rate-independence of the debonding process

$$R\left(u;\dot{\zeta}\right)=\int_{\Gamma_{\mathrm{c}}} G_{\mathrm{d}}(\delta)\left|\dot{\zeta}\right| \mathrm{d}\Gamma, \tag{5}$$

with fracture energy defined by (2). The relations which govern the evolution of damage can be written in form of nonlinear variational inclusions with initial conditions

$$\begin{aligned} &\partial_u E(\tau;\, u, \zeta) + \partial_u F(\tau; u) \ni 0, \qquad u(\tau=0) = u_0, \\ &\partial_{\dot{\zeta}} R\left(u;\dot{\zeta}\right) + \partial_{\zeta} E(\tau;\, u, \zeta) \ni 0, \quad \zeta(\tau=0) = \zeta_0, \end{aligned} \tag{6}$$

where ∂ denotes (partial) subdifferential of a convex non-smooth function, see [8], which can be replaced by Gateaux differential if the functional is sufficiently smooth, as e.g. F. The initial condition for damage is usually $\zeta_0=1$, pertaining to the undamaged state. It should also be noted that the energy release rate G is hidden in the second inclusion of (6) as it expresses a change of energy E due to a change of damage ζ.

3. Prediction of the size effect for a steel inclusion

The size effect for a steel-concrete structure may be caused by changing size of the steel inclusion. It can be modelled as a variable-size inclusion in a fixed matrix size representing a volumetric unit of the whole structure. Of course, also other dimension changes can be applied. The size effect also depends on the type and the form of the loading. Therefore, several types of boundary constraints will be taken into account.

In the described model, the damage initiation pertains to the instant when $\zeta<1$ and crack initiation to the instant when $\zeta=0$. The difference between these two instants determines the cohesive zone in the interface. In order to eliminate influence of the cohesive zone in the numerical analysis, the two instants are set actually close to each other by appropriate adjusting of the model parameter.

3.1. Problem definition

The geometry of the solved problem is shown in Fig. 1, where three choices of the applied boundary conditions are depicted, all of them using the dimensions a and r. In all cases, compression is considered. It can be guessed that no significant difference appears between the cases (A) and (B). The case (C), however, corresponds to a biaxial load, in fact compression, therefore the form of the size effect may be distinct. The changes of critical loads depending on the actual scale of the domain will be done in three modes:

I. Inclusion changing size.
II. Matrix changing size.
III. Inclusion and matrix changing sizes proportionally.

The material parameters of the domains are E=210 GPa, ν=0.3 for the steel inclusion, and E=20 GPa, ν=0.2 for the concrete matrix. The interface characteristics

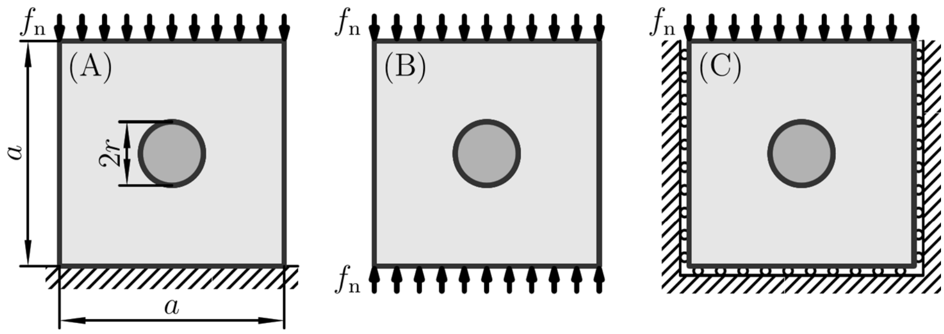

Fig. 1. Geometry of the solved problem and three options of boundary conditions

for the physical model are based on the assumptions that maximal normal tensile stress is p_n^c=25 MPa, and fracture energy for an opening crack is G_Id=0.101 kJm^{-2}, the bilinear CZM (1) is considered such that it provides an almost brittle interface, which occurs, if the parameter β is large, namely β=100. The model is crack mode sensitive if G_IId is different from G_Id, namely G_IId=0.690 kJm^{-2}, so that for the choice of the interface stiffnesses in (1) or (2) k_n=3.125 TPam^{-1}, and k_t = 0.25k_n the maximal shearing stress is p_t^c=32.66 MPa.

The loading f in (4) prescribed by the normal pressure f_n is applied in increments until total rupture occurs at a part of the interface. The main aim is to find the critical value of the load f_n^c at which the interface crack is initiated. The discretisation has the time step τ=0.01 s which causes also the load step 0.01 MPa. The interface is discretised by 128 boundary elements.

3.2. Results

We intend to compare the critical magnitudes of the applied load f_n^c as functions of some characteristic dimension of the structure for the three introduced in Fig. 1 configurations. Simultaneously we try to explain changes in graph curves based on some observations for stresses. Generally, the boundary conditions (C) cause biaxial compression so that normal stresses cannot initialize damage and the crack appears in Mode II. An effect of the boundary conditions (A) and (B) is similar, therefore also the results in these two cases are similar for each of the scaling options I, II, and III. Additionally, the crack mode can be separated due to the fact that at the places with the maximal normal tensile stress the tangential stress vanish or it is close to zero, and at the places with the maximal absolute value of the tangential stress the normal stress is compressive not affecting the damage evolution and a crack propagation. Therefore, if the ratio $\left|p_{\mathrm{t\,max}}\right| / \left(\left\langle p_{\mathrm{n}}\right\rangle_{+}\right)_{\max}$ is greater than $p_{\mathrm{t}}^{\mathrm{c}} / p_{\mathrm{n}}^{\mathrm{c}}=1.31$, a crack appears in Mode II, otherwise it is in Mode I.

The first size changing option shows the dependence of the critical load on the radius r of the steel inclusion as shown in Fig. 2, while the concrete matrix size is kept fixed at a=150 mm. The paragraph above outlined that in the cases (A), (B), which behave similarly, there may appear a kink in the dependence of the critical load for some inclusion radius due to a change of stress magnitudes (of p_{n} and p_{t}) at the interface, while in the compression state of the case (C) no such a kink has appeared. Simultaneously, closeness of inclusion and outer matrix boundary also affect the results. Therefore, the stresses for the cases (A) and (C) are shown in Fig. 3 for a small inclusion and a large inclusion relatively to the matrix.

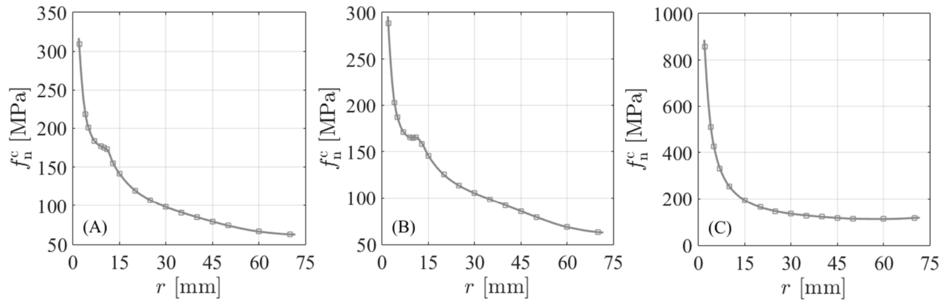

Fig. 2. Critical load for three types of boundary conditions: (A), (B), (C) according to Fig. 1, scaling of the inclusion with a=150 mm

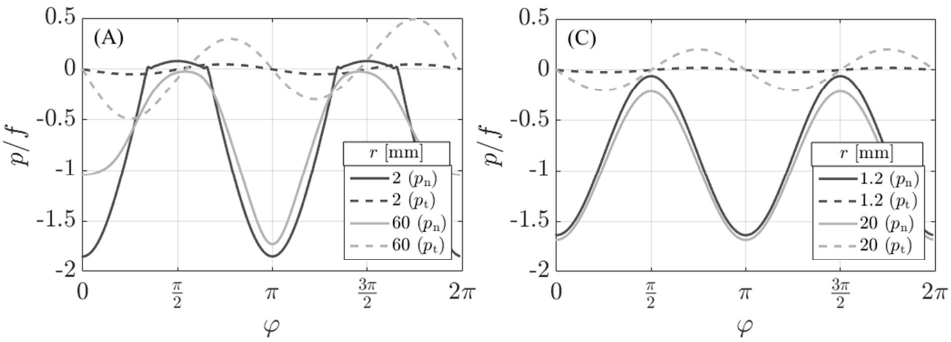

Fig. 3. Interface stresses for the constraints (A) and (C), a difference between small and big inclusions, scaled inclusion, a=150 mm

The graphs in the case (A) show different ratios between the stresses p_{n} and p_{t} so that a change in the character of the curve in Fig. 2 is easily explained: for small r maximal normal tensile stress is greater than the tangential one, therefore the crack develops in the opening mode, for big r the situation is opposite and the appearing crack is rather in the shearing mode. This is also documented by Fig. 4 which shows the arising Mode I crack (clear opening) for a small fibre of radius r=2 mm and the form of Mode II crack for r=60 mm as it is also used in Fig. 3 for the case (A).

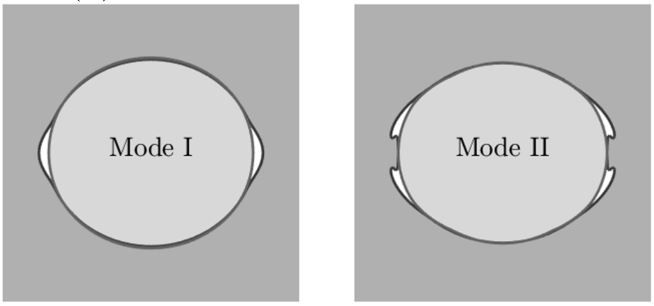

Fig. 4. Cracks at Mode I and Mode II

The effect of close contours is the most evident for the case (C) where the critical load $f_{\mathrm{n}}^{\mathrm{c}}$ has even increased a bit.

The second option for size changing mode shows the dependence of the critical load on the size a of the matrix as shown in Fig. 5. Here, the radius of the fibre is kept fixed r=10 mm. As before, in the cases (A), (B), there is a kink in the graph of the critical load for some matrix dimension caused by the observation demonstrated by Fig. 6 that, depending on the size of the matrix, either tensile normal or tangential stress is dominant at the instant of damage triggering. The graphs demonstrate various ratios between stresses p_{n} and p_{t} so that for the constraints (A) it causes different modes of the interface crack for various matrix sizes. This is the reason for the change of character of the curve in Fig. 5: for small a maximal tangential stress is greater than the tensile normal, therefore the tangential stress prevails to control the damage initiation, for big a the situation is opposite and the appearing crack is more in the opening mode. For the case (C), there is only compression.

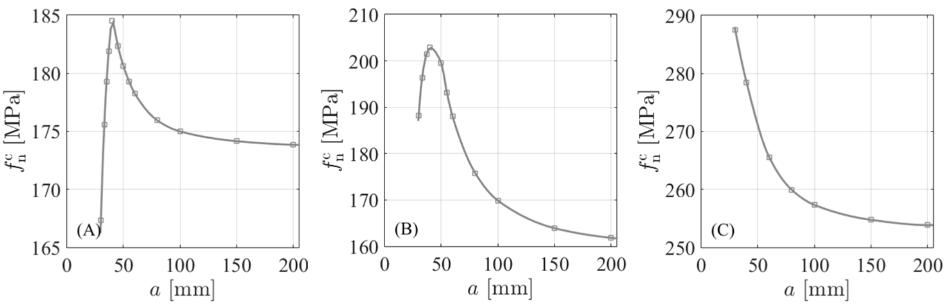

Fig. 5. Critical load for three types of boundary conditions: (A), (B), (C) according to Fig. 1, scaling of the matrix with r=10 mm

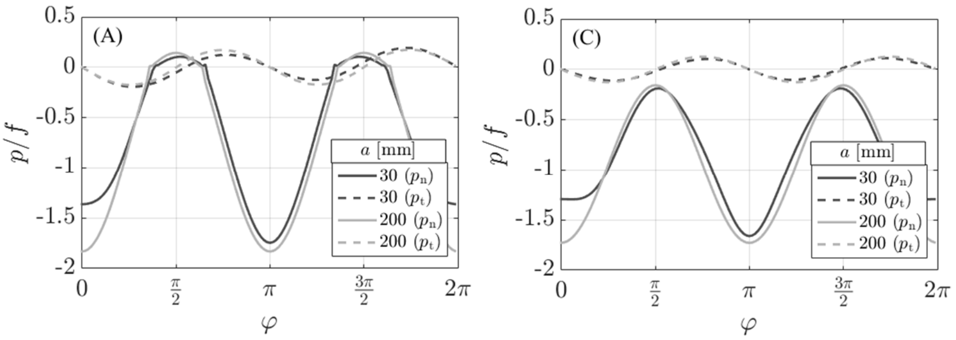

Fig. 6. Interface stresses for the constraints (A) and (C), a difference between small and large matrices, scaled matrix, r=10 mm

The last size changing mode provided the dependence of the critical load on the radius r of the inclusion as shown in Fig. 7. The ratio between structure's dimensions used in the test is $a:r$=75:4. Again, in the cases (A), (B), there is a kink of the graph in the middle of the shown span as, depending on the scale of the problem, either tensile normal or tangential stress is dominant at the instant of damage triggering, see also in Fig. 8. The figure shows the values of interface stresses relatively to the applied load f_n at the moment of damage initiation. The compressive forces, which are the greatest in fact, do not influence the damage process. The graphs reveal various ratios between maximal tensile normal and tangential stresses p_n and p_t, respectively. For the constraints (A) shown in Fig. 8, it causes different modes of the interface crack and it is the reason for the change of the slope of the curves in Fig. 7(A,B): for small r maximal normal tensile stress is greater than tangential, therefore the normal stress prevails to control the damage initiation, for big r the situation is opposite and the appearing crack is more in the shearing mode. For the constraints (C), again only tangential components affect damage triggering.

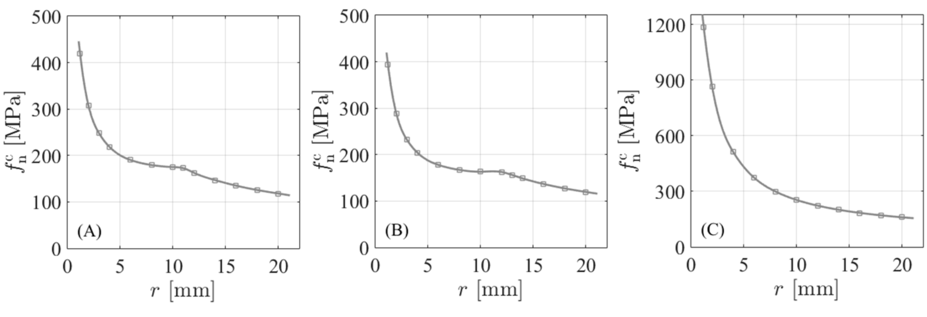

Fig. 7. Critical load for three types of boundary conditions: (A), (B), (C) according to Fig. 1, proportional scaling of the whole domain a/r=18.75

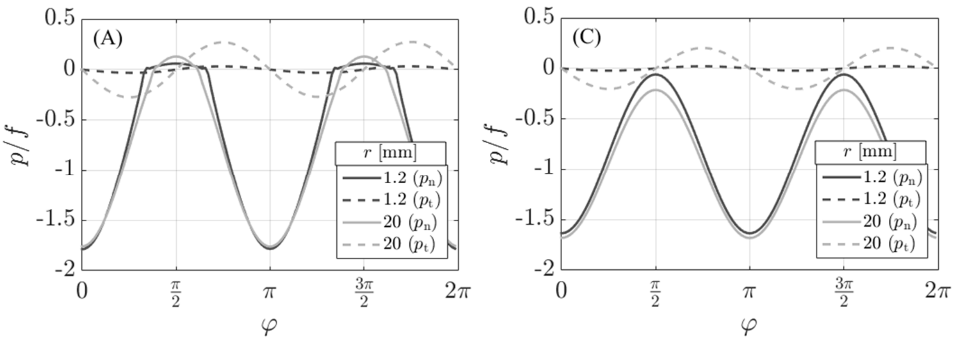

Fig. 8. Interface stresses for the constraints (A) and (C), a difference between small and big inclusions, proportional scaling, a/r=18.75

4. Conclusion

The influence of the steel reinforcement radius on crack initiation at the interface between the steel fibre and the concrete body of the structure was discussed using a numerical approach. The approach used a quasi-static delamination model for interface rupture based on an energetic approach with a cohesive zone interface. The obtained results confirm that the critical load which causes initiation of an interface crack strongly depends on sizes and ratios of the structural element dimensions. These geometric characteristics also affect the mode of the crack which appears at the interface. Nevertheless, this preliminary study provokes many questions about the interface properties or the applied load and their role in the discussed size effect: interfaces stiffnesses k, fracture energy G, used CZM model and its parameters, type of loading etc. A few such parametric studies will be analysed in the next future.

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CONCRETE NANOMODIFICATION WITH SELECTED NANOPARTICLES

The aim of the paper is to present the state of the art and technology in the field of concrete nanomodification. This new approach to the design and manufacture of materials by modifying their microstructure at the nanometric level is increasingly used also in the case of concrete. Owing to C-S-H phase changes concrete porosity and permeability can be reduced, which increases concrete durability. The improvement of concrete properties and the possibility of the manufacture of new building materials are the most important benefits of the impact of nanotechnology on construction. The paper describes the most commonly used nanoparticles in concrete technology, including nano-SiO_2, nano-TiO_2, nano-Fe_2O_3, Fe_3O_4.

Keywords: nanotechnology, nanomaterials, nanoconrete, nanopowders, nanopowders

1. Introduction

Nanotechnology is a new research area in building materials engineering. It is based mainly on physics, chemistry, engineering and biology. Owing to its intensive development the early 21st century is called the beginning of the era of nanotechnology. Nanometre (originating from the Greek word *nanos* meaning dwarf) is a length unit equal to one billionth of the metre 10^{-9}. Nanotechnology covers activities on elements smaller than 100 nm [1,2]. Nanotechnology is targeted at the use of properties of materials to obtain their improved physical, chemical and biological properties. One of the fields applying the achievements of this modern technology is construction. The use of nanomaterials and nanomodification has become highly popular in recent years. New materials (carbon nanotubes, among others) have been invented, however, also traditional materials such as concrete are being modified [3]. The strength and durability of concrete structures should be significantly improved compared with the traditional materials. Nanocement has a larger specific surface area. The very active

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nanoparticles are distributed evenly, which accelerates the cement hydration process with no detrimental impact on the material's final strength. Moreover, nanoparticles fill in the pores, which increases intermolecular forces, which, consequently, improves the cement microstructure and cement paste – aggregate interaction in concrete [4,5].

2. Mechanism of nanoparticles operation

Cement paste is composed of small grains of hydrated calcium silicate gel and large crystals of hydrated products of hydration. Between these there are nanopores and capillary pores which are the potential space for nanoparticles improving the properties of cement paste to deposit. However, high surface energy makes nanoparticles readily combine into aggregates, which hinders their uniform dispersion (especially in the case of significant amounts). In such conditions the formation of nanoparticles aggregates leads to the formation of voids, which has a negative impact on the mechanical properties of cement paste [6,7]. The mechanism responsible for the improvement of microstructure and strength of cement composites can be explained as follows. When a small number of nanoparticles is evenly dispersed in the cement paste, the cement hydration products begin to deposit on the nanaoparticles due to their high surface energy. During the hydration reaction they form conglomerates with nanoparticles as nuclei. Owing to their high reactivity the nanoparticles located in the cement paste will additionally support and accelerate cement hydration. Nanoparticles' uniform dispersion results in a suitable microstructure of the cement paste with uniformly distributed conglomerate [6,8].

3. Nanomodification of concrete

A large number of papers on the application of materials with nanoparticles in concrete and concrete modification has been published. Nanoparticles operate as heterogeneous hydrate nuclei accelerating the hydration reaction, as nanoreinforcement and as a nanofiller consolidating concrete microstructure, which reduces its porosity.

3.1. Modification of concrete with nano-SiO_2

From among materials in nanotechnology silicon dioxide (SiO_2) is by far the most popular (Fig. 1). Its wide application results from its general availability and specific properties of great utility for both industry and scientific research. It is stable in water and in elevated temperatures. The chemically inert silica reacts only with boiling concentrated aqueous solutions of KOH and NaOH, fused to K_2CO_3 and Na_2CO_3 and hydrogen fluoride or its aqueous solutions [9]. The atoms of nanoparticles can be arranged both at random (amorphous structure) or orderly (crystalline structure). They may be found in another material, for instance in metal alloys. In nanotechnologic applications silica is

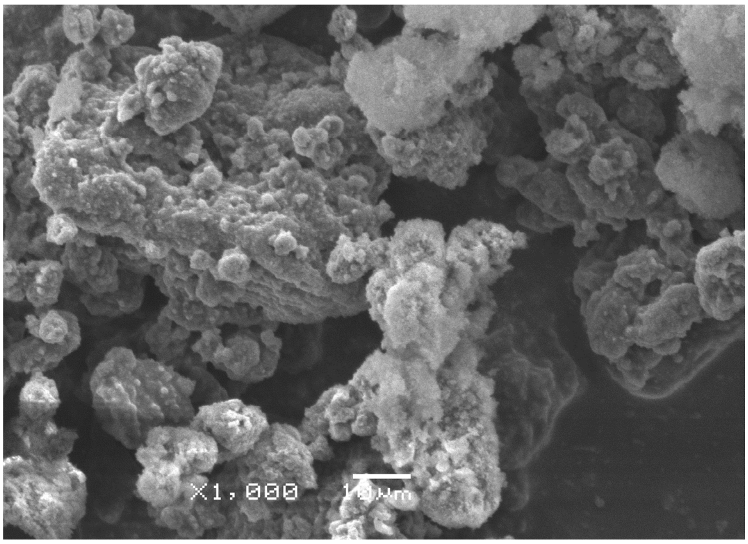

Fig. 1. SEM of SiO_2 nanoparticles

used for the production of various types of silica and hybrid nanostructures. The diameter of the most frequently obtained silica nanoparticles ranges from 5 to 1000 nm, and their specific surface area from 545 to 2.73 m^2/g. Silica nanoparticles are mainly found as precipitated amorphous silica, gels, sols, colloids and flame silica [4, 10–12].

The use of SiO_2 nanoparticles reduces porosity and permeability, which improves concrete durability. The properties fresh concrete to a large extent depend on the particle-size distribution (PSD) [14]. The effect of SiO_2 nanoparticles on the properties of concrete during the hydration reaction is shown in figure 2.

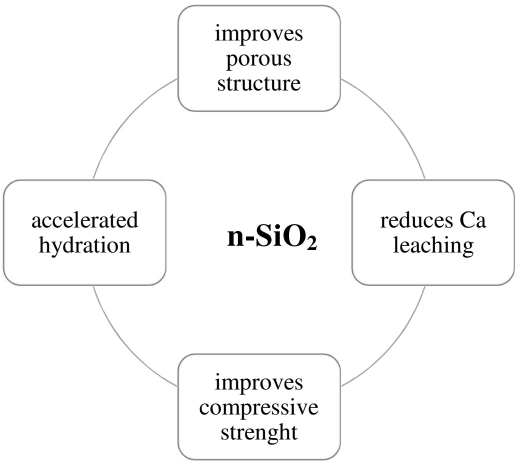

Fig. 2. Effect of nano-SiO_2[15] on concrete properties during hydration reaction

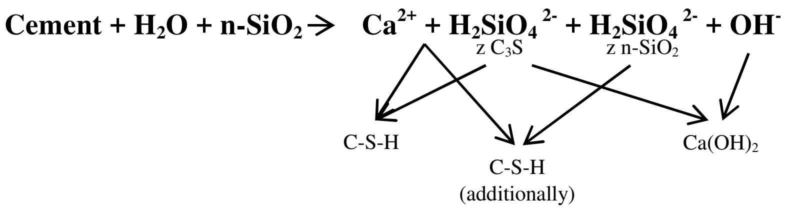

Fig. 3. Processes in cement paste after the introduction of nanosilica [15]

Micro- and nanoparticles of silica fill in the voids between cement grains. Nanosilica has much higher pozzolanic reactivity than silica fume. When cząstki nano-SiO_2 particles are added to cement grains, H_2SiO_4 in reaction with Ca^{2+} ions produces an additional amount of hydrated calcium silicates (Fig. 3). By filling in the pores, nanoparticles increase the intermolecular forces, which, consequently, improves cement microstructure and cement paste – aggregate interaction in concrete.

The use of nanosilica primarily improves concrete compressive strength. The admixture of nanosilica instead of silica fume to cement makes the paste denser and the hydration reaction faster. The investigations have proved that the hydration of cement with a high fly ash or slag content is faster with the admixture of only 1% nanosilica. On the other hand, However, the use of 2% shortens the beginning and end of the bonding on the one hand, but, on the other hand, increases concrete compressive strength after three to seven day curing. In combination with nanoferrite, nanosilica improves also concrete compressive, bending and tensile strengths, as well as modulus of elasticity [12,15–16].

3.2. Modification of concrete with nano-TiO_2

Titanium oxide IV (TiO_2) was discovered by William Gregor in Cornwall in 1791. It is composed of a titanium atom and two atoms of oxygen. It is an odourless, white and grey solid body. In nature TiO_2 is found in three different crystalline structures: as rutile, anatase and brookite. On industrial scale TiO_2 is produced from titanium ores or ilmenite ($FeTiO_3$). TiO_2 is commonly known as titanium white or titania [17, 18].

Titanium nanooxide is mainly used in the production of photocatalytic cement Owing to its photocatalytic properties, under the influence of UV radiation and in precipitation water environment on the surface of concrete, TiO_2 accelerates decomposition of hazardous substances. Cement with titanium oxide nanocrystalline content also has superhydrophilic properties, which means that a surface containing this binding agent in its composition has self-cleaning properties. An important aspect is that when used as a catalyst, titanium oxide is not worn during the processes, consequently, air purification phenomenon is continuously renewable and long-term. One of the best known objects built with the use of TiO_2 nanoparticles is the Jubilee Church in Rome (Fig.4) [10,17].

Fig. 4. Jubilee Church in Rome [13]

Titanium oxide nanoparticles are used in self-compacting concrete. The results of research done by Nazari and Riahi confirm that the admixture of TiO_2 nanoparticles improves the compressive strength of self-compacting concrete, as a result of accelerated hydration of cement and reduces concrete porosity [18–19].

3.3. Modification of concrete with nano-Fe_3O_4 and nano-Fe_2O_3

The most promising nanomaterials include nano-Fe_3O_4 (nanomagnetite) (Fig. 5). The research on the application of iron oxides (Fe_2O_3, hematite in particular) in cement composite materials has proved that these materials favourably affect their mechanical properties and microstructure. They improve compressive and bending strengths and reduce their overall porosity. Moreover, the use of nano-Fe_2O_3 very favourably affects the properties of self-compacting concrete [11]. Researchers confirm that the use of iron oxides in the dispersed phase can be important in the future production of high performance concrete. The research on the effect of Fe_3O_4 on the protective properties of concrete has indicated satisfactory results. However, due to high surface energy of iron oxides, these particles tend to agglomerate. When a large number of these is used, they may lead to the reduction of concrete strength [15]. Some researchers claim that the use of nano-Fe_3O_4 in small amounts (up to 0.3% by weight in relation to cement) may improve the mechanical properties of composites and pores structure. Some other researchers maintain that the admixture of 1.5% nano-Fe_3O_4 by weight improves concrete compressive strength, reduces chlorides penetration and water absorption in the cement matrix. Also, the research on the impact of iron oxides on the properties of cement paste in elevated temperatures has indicated promising results [20].

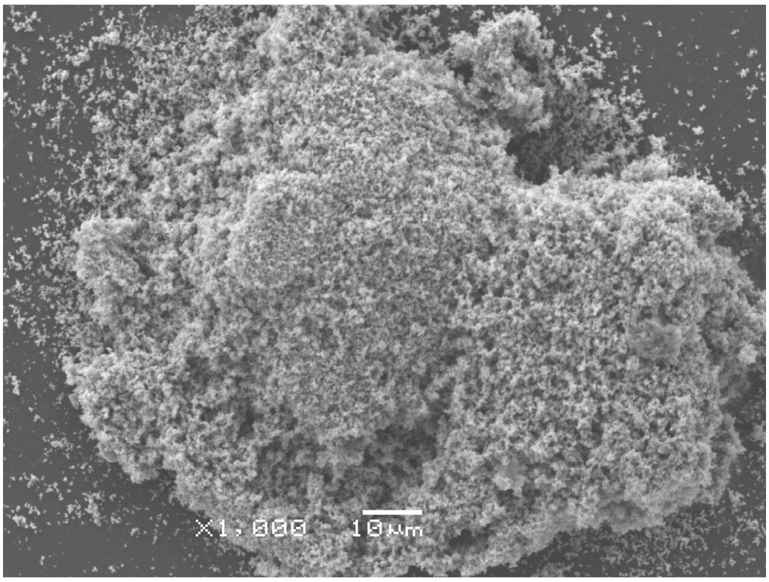

Fig. 5. SEM of nano-Fe_3O_4

Research has shown that Fe_3O_4 [20]:

- does not significantly affect fresh mixture consistency provided the nanoparticles content does not exceed 5% of cement by weight,
- operates as a filler of the microstructure by reducing the overall porosity, thus increasing the composite's density,
- does not affect the cement hydration rate,
- the use of too large amount of Fe_3O_4 nanoparticles results in local agglomerations, which, in consequence, has an unfavourable effect on the mechanical properties of cement composite materials,
- the admixture of 3% by weight of Fe_3O_4 most favourably affects the properties of cement composite materials.

4. Conclusions

In spite of being by far one of the newest areas of science, nanotechnology is becoming more and more widely spread. The majority of phenomena used to be investigated at the macroscopic level, but the progress in science has enabled nanoresearch. In construction industry nanotechnology is used on a large scale. The materials that are nanomodified include a commonly known material, i.e. concrete. Owing to its wide applicability it is appreciated not only for its mechanical properties, but also as a cutting-edge architectonic material. An enormous emphasis is placed on improving its strength and physical parameters. The admixture of nanoSiO_2, nanoFe_3O_4 and nanoFe_2O_3 greatly improves concrete compressive strength. NanoTiO_2. enjoys a wide range of applications. It is used for bacteria destruction in the environment of low intensity UV radiation, odour elimination from closed spaces and in self-cleaning surfaces. Compared with traditional concrete, nanoconcrete is considered as more durable, of higher strength and with smoother surface. The admixture of nanoparticles to cement paste, mortar or concrete improves their functional characteristics and technological parameters. It should be remembered that research in this field is only beginning to unfold [4,6].

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ANALYSIS OF THE RESULTS OF THE DESIGN PROCESS PERFORMED ON THE HISTORIC ARCHITECTURAL STRUCTURE BASED ON THE EXAMPLE OF THE EXTENSION OF HISTORICAL DIDACTIC COMPLEX IN RZESZÓW

The article analyses the issue of the extension of the historical architectural didactic complex in Poland. The conditions of the building before the design process was described and the adopted design solutions were analyzed. Design works allowed to arrange form of facades. The results of the work is an additional didactic and administrative space. The article analyses the results of cooperation between local delegation of Regional Office of Protection of Historical Monuments and the architect, which allowed to extend the possibilities of historical use of the object in combination with the newly design part.

Keywords: historical architectural complex, architectural expansion, design in historical architectural structure

1. Introduction

Historical architectural buildings demonstrate designers creative aspirations, but also in many cases there are not changed their function despite the passage of years. The necessity of the growth of urban centers is forced by permanent demographic growth. Requirements of the needs of a specific destination have changed over the years, which means that technical conditions and room programs also have changed. The Act on the Protection of Monuments and the Care of Monuments defines the scope of works that can be performed with the preservation monument [1]. One of the treatments that allows historic building to adapt to new requirements in the expansion of the building. The issue of adaptation historical building to modern requirements has been many times described in literature [2–5],

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but there were no tools allows to analyze designed solutions. Registration in the monuments register allows to ensure proper protection of the historical building, but also imposes the need to cooperate between the designer and the local unit of the Protection of Monuments Office. The obligation to consult design solution with the conservation office imposed on the architect is undoubtedly a burden, but also positively affects to the final result, which do not blur the original design solution, but ensure the reference to the historical buildings. Designing in historical areas requires detailed historical analyzes, which allows full understanding of the context of place. An example of such works may be the project of expanding historical building located in Jałowego street in Rzeszów (Figure 1).

Fig. 1. View on the analyzed building with extended part

2. Characteristic of the analyzed building

The school complex of Presentation Sisters is located at the crossroads of Cieplińskiego Avenue and Jałowy Street in Rzeszów. Both historically and nowadays, the buildings have a didactic function. The approximate date of construction is 1880 (Fig. 2) [6]. After World War II the building was used by the military Pedagogical University, since 1972 it was the seat of the Rzeszów branch of the Maria Curie-Skłodowska University in Lublin, and since 1996 it has been the seat of the General Secondary School, which in subsequent years was transformed into the Complex of General Education Schools run by the Congregation for the Presentation of the Blessed Virgin Mary [6].

Since the moment of its creation, the building have been renovated and extended, e.g. in 2003 a gymnasium was built. Systematic changes in the architectural structure caused chaotic form of frontage of building on the side of

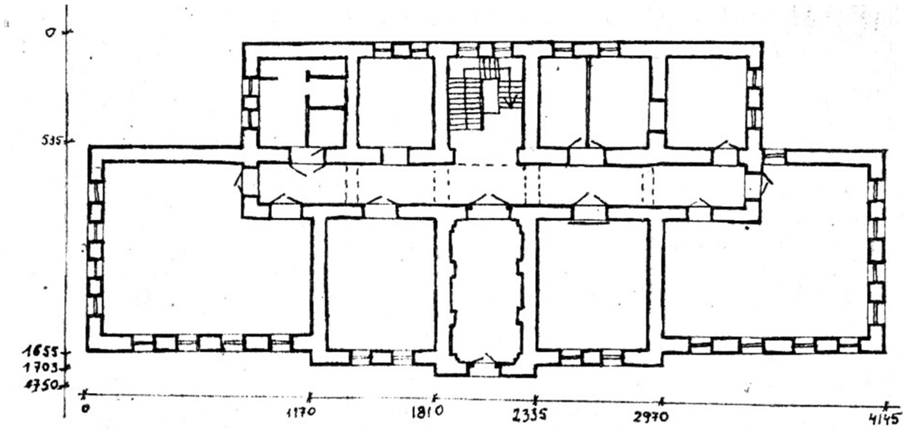

Fig. 2. Plan of the oldest part of the didactic building constructed in 1880.
Source: Archive of the Provincial Office for the Protection of Monuments Delegation in Rzeszów, Poland

Ciepliński Avenue (Figure 3). The design of the extension of the architectural complex simultaneously allowed to calm down the stormy form of the building and created additional spaces for didactic, administrative and recreational purposes. Prior to the design process, a technical expertise was created, which showed no sign of damage, signs of subsidence or displacement of foundations exceeding the permissible values [7].

Fig. 3. View from Ciepliński Avenue before the design process shows the chaotic form of analyzed building. Source: Archive of MWM Building Design Group design studio, Rzeszów, Poland

3. Expansion concept

The development conditions for the extension of the analyzed building are described in planning permission issued by Mayor of the City of Rzeszów among others from the side of Jałowy Street a line of development was determined as continuation of existing face of the historical building and from the side of Cieplińskiego Avenue the designer was obliged to maintain basic elements of the designed building parallel to the edge of the roadway and the maximum height of the extension was also determined. Due to the chaotic form of the historical building from the eastern side, the designer decided to created extension based on two main directions. One of them is parallel to the Cieplińskiego Avenue, and the second one is parallel to the path leading to nearby "Park of Unity" (Figure 4). On the ground floor and the first floor there are classrooms and administrative rooms, while on the top floor there is a classroom and a utility terrace. Also on the ground floor there are utility rooms and an atrium. The main entrance from the side of Jałowego street has not been changed. From the side of Cieplińskiego Avenue an emergency exit has been designed. The extension of the historical building is a design challenge for both the architect and the constructor. The structural design must not affect the historic elements, but economical efficient is also important issue. The main structural system has been designed partly as reinforced concrete and partly as steel structure [7].

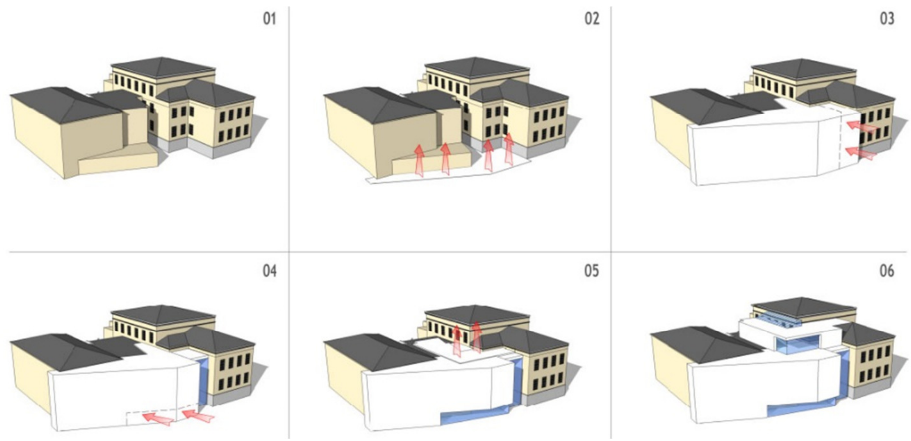

Fig. 4. Diagram of the creation of the designed part.
Source: Archive of MWM Building Design Group design studio, Rzeszów, Poland

4. Architectural analysis

Assessment of the impact of the adopted design solutions in urban space is a complex issue due to a large numbers of factors influencing the final perception of the solid. In order to examine the impact of the adopted architectural solutions the analysis has focused on separating the elements forming whole geometry of the solid e.g. analysis of the contour points of the façade. This methodology allowed to analyze the separated features of the architectural complex and then refer the conclusions to the context of the place.

Designing the face of the wall parallel to the axis of the roadway positively influenced to the perception of the architecture solid. The designer decided to reduce the number of edges of east elevation. The values of the distances between edges of elevation from Jałowego street (which has not been changed during the design process) is from 5.4 m to 11.8 m with average value equal to 8.36 m. The values of the distance between the façade edges from the Ciepliński Avenue before the design process were from 1.6 m to 11.2 m with the average value equal to 4.47 m. After the extension the values of the façade edges distance are from 3.6 m to 11.8 m with average of 7.05 m. The designed extension caused a situation where the values of the edge distances of both elevation are similar, which gives the impression of a harmonious architectural complex (Figure 5).

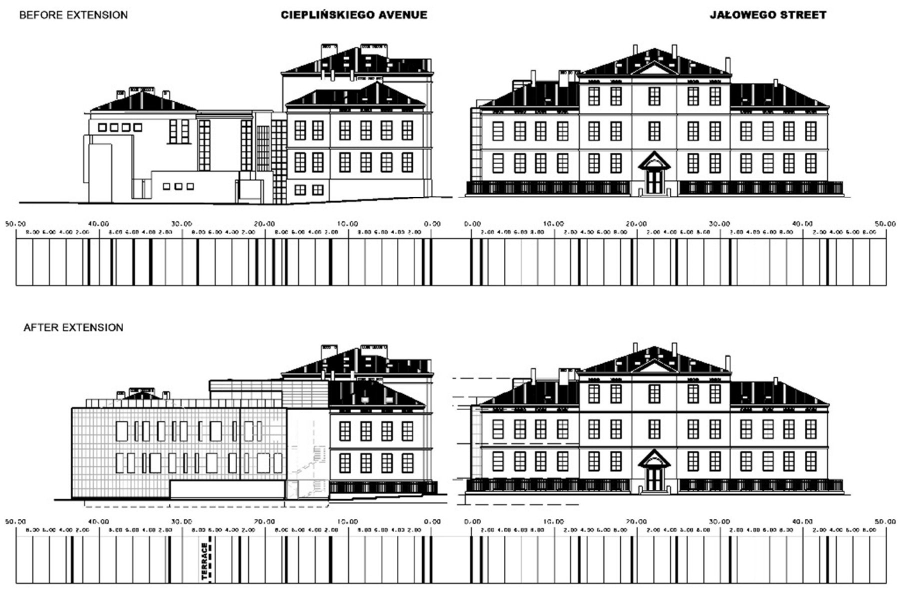

Fig. 5. Scheme presenting the analysis of façade edges in a historic architectural complex before and after extension. The study was created based on materials from the archives of MWM Building Design Group design studio

An analysis of the density of points forming the contours of the façade at Cieplińskiego Avenue and Jałowego Street has also been developed. The east elevation before the design works had an inconsistent form, which is shown on heatmap diagram (Figure 6). The design process limited the number of edges and contour points. The design caused the accumulation of points at the contact of the historic part with the extension part, while in the south-west direction (where the “Park of Unity” is located) the amount of contour points were reduced to a minimum, which calmed down the façade at Cieplińskiego Avenue.

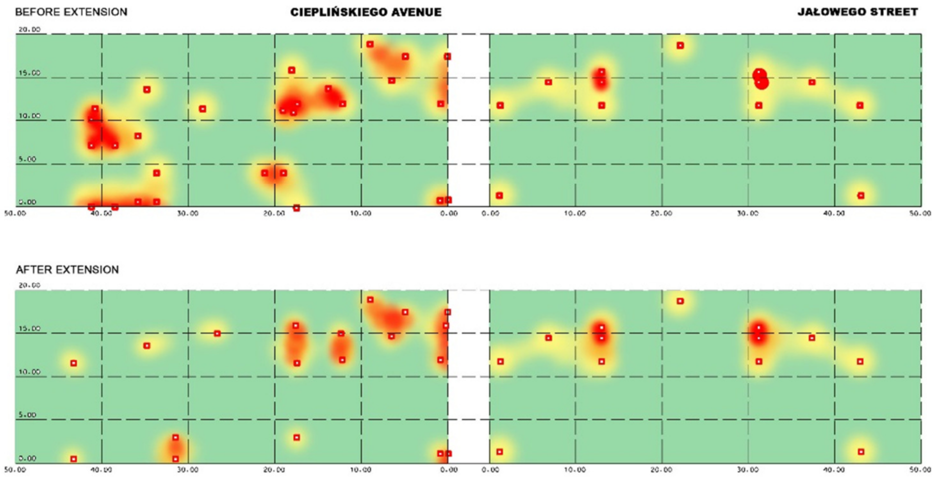

Fig. 6. Scheme showing the heatmap of density of contour points for the analyzed facades before and after the design process. The study was created based on materials gather from archives of MWM Building Design Group design studio

5. Conclusion

Constantly developing urban centers oblige designers to adapt buildings to new or changed needs with simultaneous maintaining historical buildings in the least changed character. The outcome of the design work in the historical architectural structures is influenced by numerous factors e.g. the development context, legal conditions or the competences of the local conservation office. Design process in the historical objects should guarantee the durability of the adopted solutions with the least possible interference in the preserved elements. The analyzed design process of the extension confirms the assumptions of the necessity of cooperation between designers and the conservation office in every stage of the execution of the investment. The works carried out allowed to expand usability of the historical building.

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THE POSSIBILITIES FOR IMPLEMENTING RAINWATER MANAGEMENT MEASURES IN TUKE CAMPUS

All of the buildings in TUKE campus are connected on water main as only one source of water. There is no building with alternative source of water for non-potable uses so that potable water is used for drinking purposes as well as all others activities (flushing toilets, cleaning..). Drainage solutions of the TUKE campus are in traditional way too. The buildings situated in TUKE campus have a classical drainage system for rainwater runoff consist from traditional direct channelling of surface water through networks of pipes to sewer system except two buildings - PK6 and PK5 which have a drainage system for rainwater runoff designed through the infiltration facilities – infiltration shafts. This paper describe a big potential savings of potable water by the use of rainwater in TUKE campus as well as the big potential for "green" drainage solution – infiltration in TUKE campus.

Keywords: drainage, infiltration, rainwater harvesting, savings

1. Introduction

As the process of growing population and the consequent urban growth is unavoidable in urban regions, resulting in non-riverine flooding due to impermeable surface, low infiltration and high runoff [1], RWH will be a significant water source [2].

By introduction of Rainwater management system it provides a lot of benefits for urban areas:

- reduced volume of paved surfaces runoff discharged to the sewer system,
- improved resilience of the existing water supply system to drought and independent source of water,
- reduced drinking water treatment costs,
- reduces costs for water supply and discharge costs,
- in generally – better life condition and educational factor [5,6].

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In the case of school-type buildings potential of water savings replaced by rainwater is significantly higher what result from absent of purposes as a showering, bathing, laundry, etc. The most volume of potable water of school buildings in TUKE campus is consumed for flushing toilets which is precisely the most suitable purpose for use of rainwater [3,4].

2. Rainfall conditions and rainwater harvesting potential

2.1. Basic information

For the determination of theoretical volume of rainwater we need also data of rainfall intensity. The resource that provides us information about the rainfall intensity is rain gauge and is located on the roof of University Library.

Rain gauge is joined with its own concrete foundation using a steel rod. Flat roof helped us fixing the rain gauge into horizontal position which is the first condition for receiving correct data. We use recording heated rain gauge for all year round measuring. There are known unheated rain gauges as well used for limited part of year when the temperatures aren't so low. Heated rain gauge is used for measuring liquid precipitation (rain) and solid precipitation (snow) as well. Rain gauge is made of stainless material. Rain gauge's round catchment area is 200 cm^2 and its function is based on tipping bucket mechanism. Tipping bucket is located inside the rain gauge body right under the funnel outlet. Rain or snow fall down the funnel outlet into the divided bucket. The bucket does not move until it is filled with calibrated 0.2 mm amount of water, then it tips and second half of bucket can be filled with rain water. When the bucket tips it empties the liquid from the half of the bucket into a drainage hole. Tipping bucket is made of plastic with very thin layer of titanium and it is hanged on stainless steel axial holder. Tipping continues according to the length of rainfal [5,6,8].

Figure 1 represents the measured monthly rainfall totals during our research. Data are presented for the period August 2011 to December 2014.

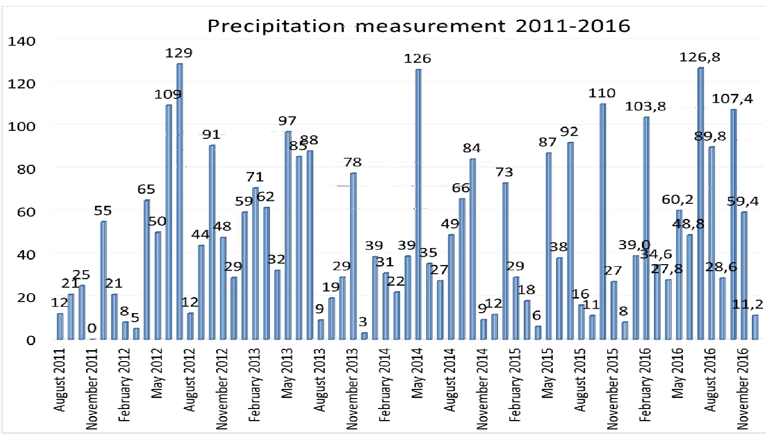

Fig. 1. Measured monthly rainfall totals during August 2011 – December 2016

Figure 2 represents view of the Technical University of Kosice campus site in Kosice-city. Blue rectangles indicate school buildings for all faculties of Technical University of Kosice.

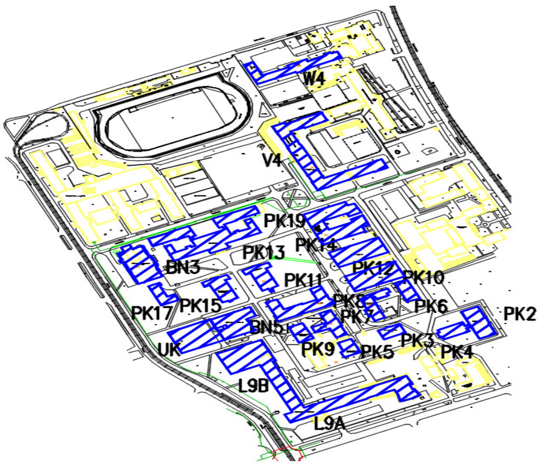

Fig. 2. TUKE campus

A planned situation of rainwater management in TUKE campus consider about replacing of traditional rainwater drainage into the sewage system by the use of rainwater in the school buildings. All of school buildings respectively the roofs of these buildings in TUKE campus (figure 3) represent a potential source of rainwater for non-potable purposes especially for flushing toilets [7].

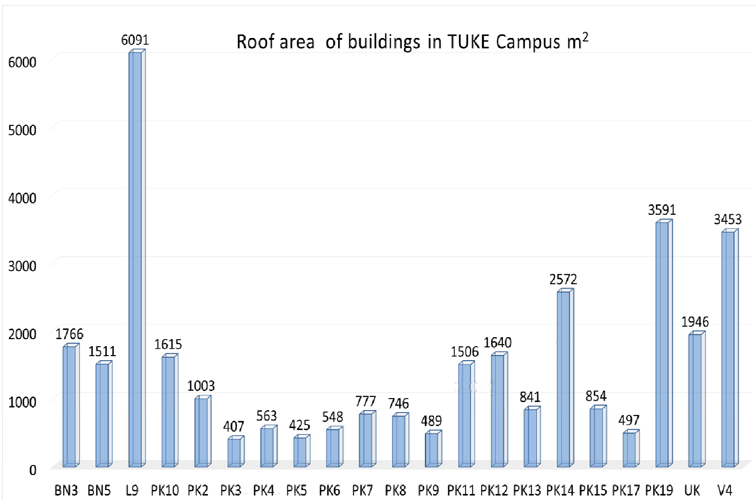

Fig. 3. Roof areas (m2) of buildings in TUKE campus as o potential source for RWH

According our measurements of monthly rainfall totals, figure 4 represent theoretical monthly volumes of collected rainwater from roof areas of buildings in TUKE campus. Data are presented for the period August 2011 to December 2016.

Fig. 4. Theoretical monthly volumes of collected rainwater from roof areas of buildings in TUKE campus during our research August 2011 – December 2016

2.2. Measured data

We have started our research and own measurements in scope of stormwater quantity and quality parameters at the campus of Technical University of Košice within the project relating to the management of stormwater. The objects of research represent two infiltration shafts in the campus of TU Kosice that were made before the start of our research. These infiltration shafts represent drainage solution for real school building PK6 and all of the runoff rainwater falling onto the roof flows into these underground shafts [6,10]. Data of rainwater inflow provide us information about real rainwater volumes from roof construction of PK6 building.

Table 1 summarizes the measured monthly rainfall totals with corresponding theoretical volumes of collected rainwater and comparison with real volumes of collected rainwater from our measurements. Data are presented for the period April 2012 to December 2015. (data are present from April 2012 because at that time began measuring of the flow from all roof area of the building PK6 and precipitation measurements simultaneously) (Notice: August 2012 without data due to equipment failure) [11].

Table 1. Monthly rainfall totals with corresponding theoretical volumes of collected rainwater and real amount of rainwater from roof of PK6 building (548 m^2)

Month	Rainfall (mm)	Theoretical volume from 548 m^2(m^3)	Real volume from 548 m^2(m^3)
April 2012	65	35.6	26.7
May 2012	50	27.4	18.9
June 2012	109	60.0	40.8
July 2012	129	70.6	49.6
August 2012	12	6.7	-
September 2012	44	24.0	17.9
October 2012	91	49.6	36.5
November 2012	48	26.1	16.9
December 2012	29	15.8	12.1
January 2013	59	32.6	19.9
February 2013	71	38.8	23.5
March 2013	62	33.8	22.8
April 2013	32	17.6	11.8
May 2013	97	53.2	30.6
June 2013	85	46.8	30.2
July 2013	88	48.2	36.6
August 2013	9	4.9	3.8
September 2013	19	10.5	8.9
October 2013	29	15.9	13.7
November 2013	78	42.5	38.4
December 2013	3	1.6	1.3
January 2014	39	21.2	10.9
February 2014	31	17.0	12.4
March 2014	22	12.1	8.3
April 2014	39	21.3	13.3
May 2014	126	69.2	44.9
June 2014	35	19.4	12.6
July 2014	27	15.0	13.9
August 2014	49	26.7	20.8
September 2014	66	35.9	-
October 2014	84	46.1	-
November 2014	9	5.0	4.1
December 2014	12	6.4	4.7
January 2015	73	40.0	22.9
February 2015	29	15.9	8.9
March 2015	18	9.9	4.8
April 2015	6	3.3	2.1
May 2015	87	47.7	19.9
June 2015	38	20.8	11.0
July 2015	92	50.5	23.3
August 2015	16	8.8	3.9
September 2015	11	6.0	4.1
October 2015	110	60.3	35.6
November 2015	27	14.8	7.9
December 2015	8	4.4	2.2

3. Potential for runoff infiltration

As was mentioned above we have started our research and own measurements in scope of stormwater quantity and quality parameters at the campus of Technical University of Košice within the project relating to the management of stormwater. The objects of research represent two infiltration shafts in the campus of TU Kosice that were made before the start of our research. All measured data during our research show, the total infiltration of rainwater runoff inflow into the infiltration shafts from roof of PK6 building take place at the same time of duration of rainfall events, respectively very short-time after.

This represents a high infiltration rate of this infiltration shafts. It is given by the coefficient of infiltration of soil at the bottom of shaft. If we compared size of area for infiltration of runoff with other types of infiltration facilities (for example infiltration boxes) this size is several times smaller against other types of infiltration facility. But the infiltration coefficient of surveyed infiltration shafts $k_f = 1.10^{-3}$ m/s [3] ensures safe disposal of surface runoff [6,9,10]. The maximum water level at the infiltration shaft A, measured during the research period is 2.48 m, which is cca 1/2 filling depth of infiltration shaft A and maximum water level at the infiltration shaft B, measured during the research period is 1.31 m, which is less than 1/3 filling depth of infiltration shaft B too. Maximum water levels during a research are shown in figure 5. (notice: part of roof construction was unconnected what result to lower volumes of rainwater from year 2015).

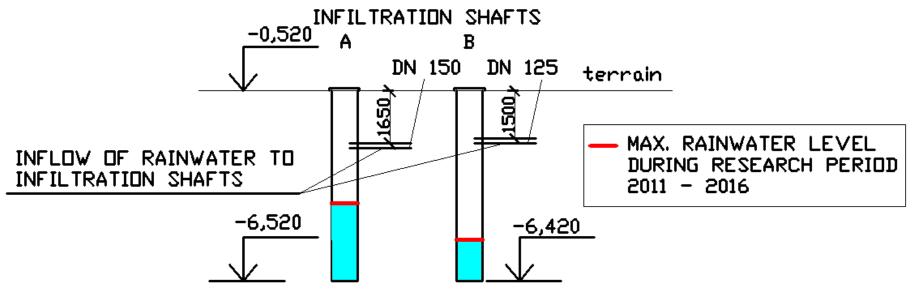

Fig. 5. Infiltration shafts and maximum water levels during the research

Application of RWH systems in TUKE campus has a big potential from environment, technical and financial point of view. Rooftop RWH potential in the TUKE campus as well as appropriate conditions for infiltration of rainwater is promising and could be applied on a larger scale for later beneficial use. Rainwater management in educational type of building has not only financial benefits but also educational and ethical benefits. Education of students leads to awareness of value of potable water and would avoid of wasting precious potable water which is used for example - for flushing toilets in our society. Further studies are recommended to quantify the economic value and environmental impact related to the application of RWH in the study area.

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