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Budownictwo i inżynieria środowiska z 53 2009

MOHAMED AHMAD[1]

THE INFLUENCE OF ARTIFICIAL RESINS ON WATER RESISTANCE OF GYPSUM MATERIALS

ABSTRACT

The thesis presents the results of laboratory research concerning the influence of combined silicon and epoxy resin supplements on hydrophobic and water resistance features of gypsum materials. For comparative reasons, the first stage concerns the influence of singular modifying preparations. It has been proved that both silicon and epoxy resins introduced to gypsum compositions separately have advantageous influence on hydrophobic and water resistance features of modified gypsum materials. However, they do not protect these materials against destructive water effects. The second stage of the research concerned introducing two preparations that have different modifying characteristics simultaneously. The results prove that this research direction is justified and it allows to obtain gypsum materials with increased hydrophobic and water resistance features, which can be used in higher humidity places.

KEYWORDS: gypsum material, hydrophobia, water resistance

1. INTRODUCTION

Typical and technical disadvantage that limits the use of gypsum materials in building industry is their susceptibility to destructive water effects. Gypsum materials in the humid state prove lower strength up to 60-80%, have great absorbability up to 30-50% and considerable blurring effect during direct contact with water. Various research centres conduct studies on increasing hydrophobic and water resistant features of gypsum materials. The results derived so far do not provide complete and practical solution to the problem [1]. Technological processes such as vibration, compacting, ultrasound result in concentration of gypsum materials and decreasing their porosity, what leads to increasing strength and slightly decreasing their absorbability. Nonetheless, the malacia factor during squeezing remains quite low. Surface impregnation does not protect gypsum materials entirely against destructive humidity effects. Impregnated surface is susceptible to damages and unprotected material mass gets easily exposed. Inuring gypsum materials against water effects in entire mass guarantees durable and permanent protection of gypsum materials against destructive water effects [2]. This research direction consists in introducing supplements altering some gypsum materials features to mixing water suspension. Majority of modifying supplements constitute organic compounds which are artificial resins or their solutions. They only improve one

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feature at the expense of downgrading the other feature. Some supplements decrease absorbability, but at the same time they reduce strength of gypsum materials. Other supplements increase strength in dry and full water saturation state but they do not reduce absorbability. For the last few years there have been many researches conducted based on complex studies aiming at both increasing hydrophobic features and increasing water resistance of gypsum materials [3]. One of the research directions on increasing hydrophobic and water resistant features which does not require any additional technological processes is introducing two or more chemical modifying supplements to gypsum leavens or to mixing water of the suspension. The research results presented in the thesis significantly contribute to increasing hydrophobic and water resistant features of gypsum materials.

2. METHODOLOGY AND RESEARCH SCOPE

For comparative reasons, some physical and strength features of gypsum control samples are marked and presented in table 1. Marking of the features of gypsum samples are made due to norm requirements [4-6]. The scope of laboratory investigation embraced marking of strength against squeezing and bending in dry and humid state, absorbability and capillary absorption of water. On the basis of the obtained parameter values of strength, the malacia factor during squeezing and bending was calculated in accordance with norms [7]. Marking of these features were considered sufficient to assess the influence and effectiveness of modifying supplements to increase hydrophobic and water resistance features of gypsum materials.

Table1. Features of gypsum materials used in laboratory investigation

W/A	STRENGTH				MALACIA FACTOR		ABSORBABILITY
	R_s	R_{sm}	R_z	R_{zm}	K_s	K_z	N_m
	MPa						%
0,60	11,00	2,86	4,20	1,22	0,26	0,29	34
0,55	12,10	3,39	4,80	1,50	0,28	0,31	31
0,50	15,30	4,60	5,30	1,75	0,30	0,33	27
0,45	17,00	5,61	6,70	2,35	0,33	0,35	23
0,40	18,40	6,35	7,50	2,81	0,35	0,37	19

R_s - Strength against squeezing in the dry state; R_{sm} - Strength against squeezing in the humid state;
R_z - Strength against bending in the dry state; R_{zm} - Strength against bending in the dry state;
K_s - Malacia factor during squeezing; K_z - Malacia factor during bending; N_m - Mass absorbability.

3. CHARACTERISTICS OF THE SUPPLEMENTS USED IN LABORATORY INVESTIGATION

Silicon resins are multimolecular silicon materials built from silicon atoms linked with oxygen atoms and partially bound with carbon atoms. Thanks to their valuable features their application as special materials in various branches of economy is very wide. In building industry they are used for increasing external and internal water resistance of building materials. In laboratory investigation, silicon preparations Sarsil H-14/2 and Sarsil M-25 were used. Sarsil H-14/2 is the solution of methyl silicon in white spirit. It is used to improve hydrophobic features of brickworks, industrial buildings, livestock buildings, monument and historic buildings maintenance etc. Sarsil M-25 is water emulsion of silicon resin with small admixture of organic solvents. It is used to improve hydrophobic features of porous building materials. As for epoxy resins, Epidian 5 and Epidian 53 were used. Resin Epidian 5 belongs to fundamental unmodified epoxy resins which are 100 percent resins [8]. Epidian 5 can be a solid body in scale form of light yellow colour or half liquid body with great viscosity of yellow colour with brown hue. It can also be a liquid body with consistence of syrup of light yellow colour. In the unhardened state, Epidian 5 is a plastic material and its viscosity decreases with temperature rising. Specific consistency of epoxy resin Epidian 5 in hardened state amounts 1,19 g/cm^3, its strength against squeezing amounts 100-115 MPa, strength against bending amounts 80-120 MPa. Epidian 53 is liquid epoxy composition including thinner as a modifier. It is a lucid viscous liquid of colour ranging from yellow to dark brown. This composition gets hardened in room temperature, most often with hardener Z-1 or other depending on its expected direction of hardened material use. Epidian 53 has low viscosity and medium reactivity. It is used to supersaturate and impregnate various types of porous materials. One of the most important uses of Epidian 53 is cold process sticking of various building materials such as metal, glass, ceramics and connecting rigid constructions. Strength of epoxy resin Epidian 53 in hardened state against squeezing amounts 88,2-107,8 MPa, against bending, however, 68,6-78,4 MPa. Epoxy resins Epidian 5 and 53 are self-cured polymers. The process of hardening was conducted in room temperature (temperature in laboratory room). The hardening process lasts 7-10 days and after 48 hours the hardening rate amounts 80-90%.

4. THE INFLUENCE OF SILICON RESINS ON WATER RESISTANCE OF GYPSUM MATERIALS

Two silicon preparations were used during laboratory research, Sarsil H-14/2 and Sarsil M-25. These preparations were introduced directly to mixing water of the suspension. This is the great advantage of these modifiers from the practical building point of view. Contribution of silicon resin to gypsum composition amounted 0,25-1,25% with relation to gypsum adhesive mass. Small amounts of introduced silicon preparations are justified due to the fact that surplus of silicon is undesirable because they produce soluble in water potash salts. On the basis of the initial research of the influence of the silicon preparations on gypsum materials features it is proved that they hasten biding of gypsum leavens and increase plasticity and workability of gypsum mass. As an inhibitor of biding time, the citric acid was used, which does not decrease strength features of gypsum materials. 0,04% of the inhibitor - citric acid in relation to gypsum adhesive mass was added to mixing water of the suspension. The results of the investigation on the influence of silicon preparations on hydrophobic and water resistance features of gypsum materials are presented in table 2 and are depicted on charts (figures 1 and 2). According to the research, despite the fact that silicon resins have positive influence on the features of gypsum materials, they do not cause significant improvement of hydrophobic and water resistance features of modified gypsum materials (table 3). The highest increase of gypsum samples strength against squeezing in the dry and humid state was observed when introducing minimal amounts of silicon preparations 0,25-0,50% with relation to gypsum adhesive mass to gypsum leaven. When the amount of silicon resin in gypsum compositions was higher, the mechanical strength of modified samples decreased and the absorbability increased. The malacia factor while squeezing the modified gypsum samples is higher and amounts 0,5 (at 0,25% of Sarsil H-14/2). Mass absorbability, however remains large and amounts 17,31%.

Table 2. The influence of silicon resins on water resistance of gypsum materials

SUPPLEMENT		W/A	STRENGTH				MALACIA FACTOR		N_m	CAPILLARY ABSORPTION OF WATER After hours			
			R_s	R_{sm}	R_z	R_{zm}	K_s	K_z		1	3	6	24
%			MPa						%	cm			
Sarsil H-14/2	0,25	0,4	20,89	10,43	6,75	2,58	0,50	0,38	17,31	1,5	3,1	4,2	7,0
	0,50		20,55	9,81	5,55	2,05	0,48	0,37	18,98	1,9	3,3	4,9	7,8
	0,75		19,88	9,21	5,11	1,78	0,46	0,35	22,12	2,8	4,0	5,2	8,6
	1,00		18,34	8,21	4,83	1,64	0,45	0,34	23,51	3,1	4,3	7,3	9,3
	1,25		17,50	7,51	3,43	1,13	0,43	0,33	24,31	3,8	4,9	8,4	11,0
Sarsil ME-25	0,25	0,4	21,00	9,00	7,21	2,80	0,43	0,39	16,95	1,2	3,0	4,0	6,5
	0,50		20,80	8,60	5,89	2,22	0,41	0,38	18,45	1,7	3,2	4,7	7,6
	0,75		20,20	7,40	5,76	2,09	0,37	0,36	21,86	2,5	3,8	4,9	8,5
	1,00		20,00	7,20	5,59	1,86	0,36	0,33	22,35	2,7	4,2	6,0	9,3
	1,25		18,00	6,40	5,39	1,65	0,36	0,31	23,41	3,6	4,7	7,3	10,0

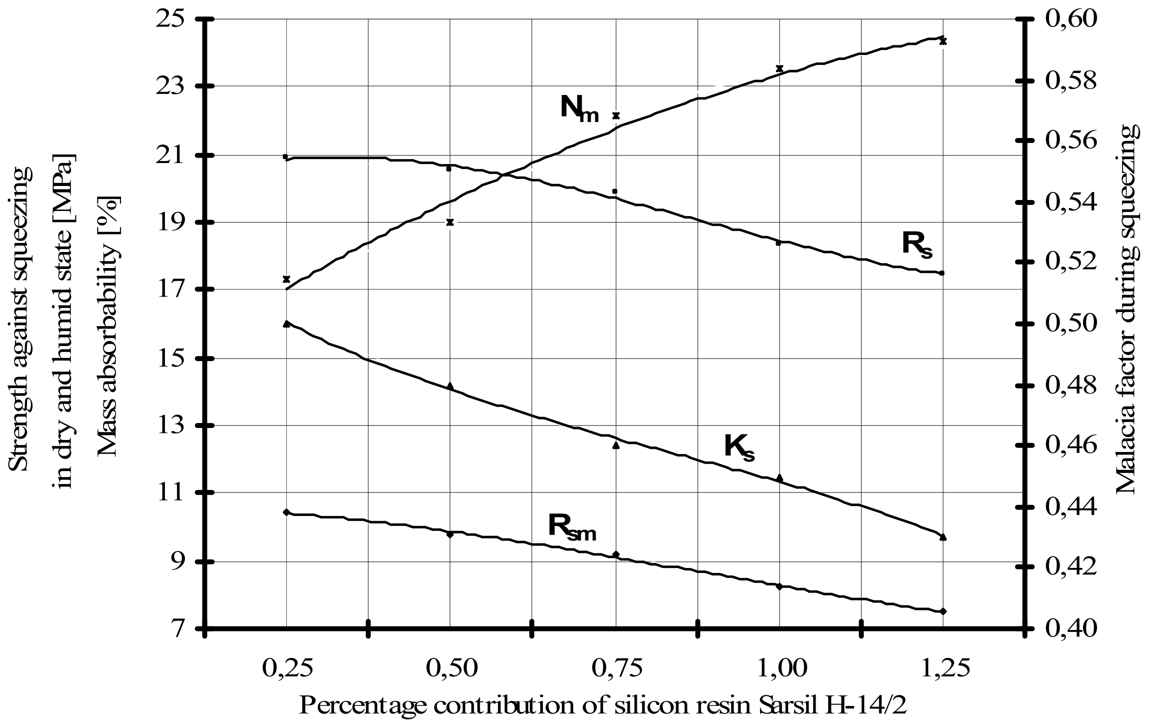

Figure 1. The influence of silicon resin Sarsil H-14/2
on water resistance of gypsum materials

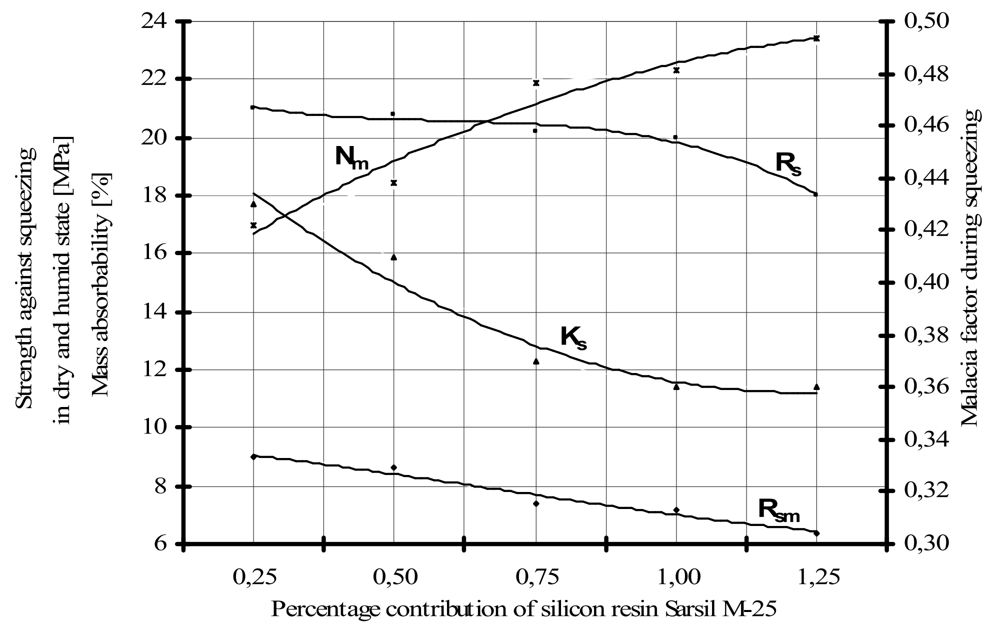

Figure 2. The influence of silicon resin Sarsil M-25
on water resistance of gypsum materials

Table 3. The effectiveness of using silicon resins to increase water resistance of gypsum materials

MODIFIER NAME AND CONTENT		W/A	STRENGTH INCREASE				ABSORBABILITY DECREASE
NAME	%		ΔR_s	ΔR_{sm}	ΔK_s	ΔK_z	
			%				
Sarsil H-14/2	0,25	0,4	13,53	64,25	42,86	2,70	8,89
	0,50		11,68	54,49	37,14	0,00	0,11
	0,75		8,04	45,04	31,43	-	-
	1,00		-	-	-	-	-
	1,25		-	-	-	-	-
Sarsil M-25	0,25	0,4	14,13	41,73	22,86	5,41	10,79
	0,50		13,04	35,43	17,14	2,70	2,89
	0,75		9,78	16,54	5,71	-	-
	1,00		8,70	13,39	2,86	-	-
	1,25		-	-	-	-	-

5. THE INFLUENCE OF EPOXY RESINS ON WATER RESISTANCE OF GYPSUM MATERIALS

Epoxy resins Epidian 5 and Epidian 53 were dissolved in acetone before use, in the appropriate proportions to increase its viscosity. The hardener Z-1 was used to harden these resins. The mixture of resin, hardener in acetone was dosed with mixing water of the suspension, mixing all the time. Next building gypsum was added. Supplements of the epoxy resins make forming of trabeculas difficult, due to the fact that when the amounts of these supplements in gypsum leavens get higher, mass workability gets worse and it becomes sticky. The research results of the influence of the epoxy resins Epidian 5 and Epidian 53 on hydrophobic and water resistance features of gypsum materials are presented in table 4 and depicted in charts (figure 3 and figure 4).

Table 4. The influence of epoxy resins on water resistance of gypsum materials

SUPPLEMENT		W/A	STRENGTH				MALACIA FACTOR		N_m	CAPILLARY ABSORPTION OF WATER After hours			
			R_s	R_{sm}	R_z	R_{zm}	K_s	K_z		1	3	6	24
%			MPa						%	cm			
Epidian 5	1,0	0,40	19,00	7,60	7,70	3,20	0,40	0,42	15,30	0,0	0,0	1,0	1,0
	2,0		19,50	9,40	7,90	3,80	0,48	0,48	14,20	0,0	0,0	0,9	0,9
	3,0		20,50	11,20	8,00	4,00	0,55	0,50	13,10	0,0	0,0	0,8	0,8
	4,0		24,00	16,80	8,90	6,70	0,70	0,75	11,40	0,0	0,0	0,5	0,6
	5,0		27,00	21,60	9,30	7,50	0,80	0,81	9,30	0,0	0,0	0,3	0,5
Epidian 53	1,0	0,4	20,00	11,00	8,00	4,00	0,55	0,50	11,00	0,0	0,0	0,8	0,8
	2,0		21,00	12,80	8,60	4,80	0,61	0,56	9,30	0,0	0,0	0,7	0,7
	3,0		23,70	15,40	9,00	6,00	0,65	0,67	8,70	0,0	0,0	0,4	0,4
	4,0		25,70	18,80	9,70	8,00	0,73	0,82	8,10	0,0	0,0	0,3	0,3
	5,0		28,30	24,60	10,50	9,50	0,87	0,90	7,00	0,0	0,0	0,2	0,2

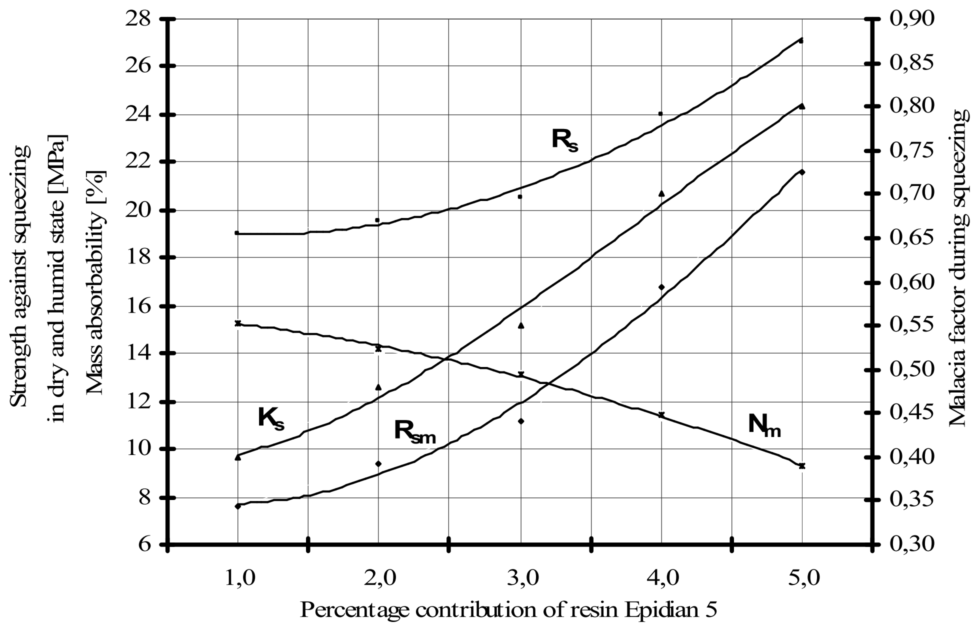

Figure 3. The influence of epoxy resin Epidian 5
on water resistance of gypsum materials

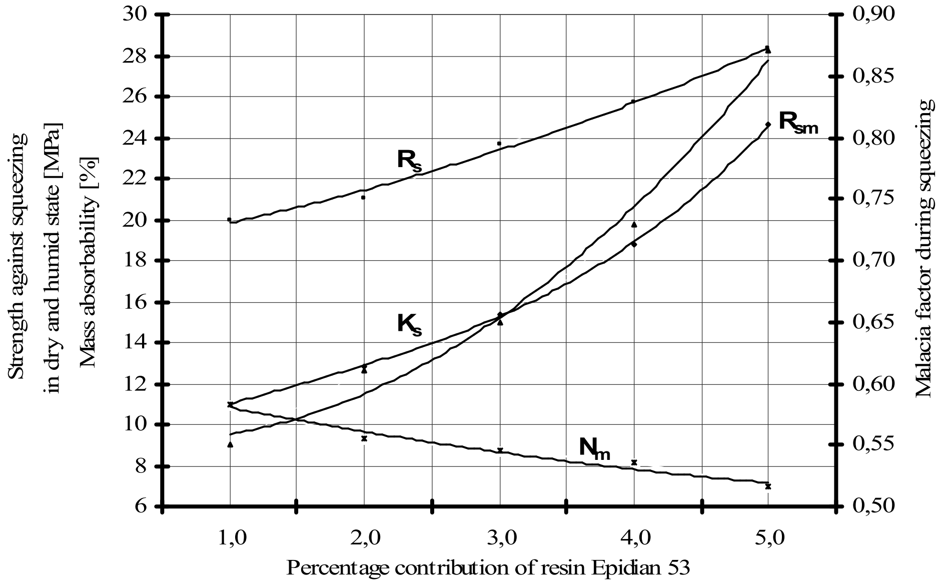

Figure 4. The influence of epoxy resin Epidian 53 on water resistance of gypsum materials

The results of the research proved that the more epoxy resins were introduced to gypsum leavens, the strength of the modified samples against squeezing and bending in dry and full water saturation state was increased. Strength of the gypsum samples modified with resin Epidian 5 against squeezing in dry state increases 3,26-46,73%, in the humid state however, 19,68-240,15% in relation to control samples made from building gypsum (table 5).

Table 5. The effectiveness of using epoxy resins to increase water resistance of gypsum materials

MODIFIER NAME AND CONTENT		W/A	STRENGTH INCREASE				ABSORBABILITY DECREASE
NAME	%		ΔR_s	ΔR_{sm}	ΔK_s	ΔK_z	
			%				
Epidian 5	1	0,4	3,26	19,68	14,28	13,51	19,47
	2		5,97	48,03	37,14	29,72	25,27
	3		11,41	76,37	57,14	35,13	31,06
	4		30,43	164,56	100,00	102,70	40,00
	5		46,73	240,15	128,57	118,91	51,05
Epidian 53	1	0,4	8,69	73,22	57,14	35,13	42,10
	2		14,13	101,57	74,28	51,35	51,05
	3		28,80	142,51	85,71	81,08	54,21
	4		39,67	196,06	108,57	121,62	57,36
	5		53,80	287,40	148,57	143,24	63,15

The malacia factor during squeezing amounts 0,80, during bending 0,81 and it is higher than the factor of control samples by 128,57 and 118,91%. Mass absorbability decreases to 15,3-9,3% and it equals 19,47-51,05% in relation to control samples absorbability. Better results were obtained with the use of epoxy resin Epidian 53. The strength increase of gypsum control samples against squeezing in the dry state amounts 8,69-53,80%, in the humid state 73,22-287,40%. The malacia factor during squeezing amounts 0,87, during bending 0,9. Mass absorbability of modified samples amounts 11-7% and it is lower than the absorbability of control samples by 42,10-63,15%. The better results obtained with resin Epidian 53 can be explained by the fact that this resin has lower viscosity what simplifies its regular distribution in gypsum leaven. Both epoxy resins Epidian 5 and Epidian 53 make capillary absorption of water more difficult. The level of humidity of the gypsum trabeculas put vertically in water after 24 hours does not exceed 1 cm.

6. THE INFLUENCE OF THE COMBINED SILICON AND EPOXY RESINS ON WATER RESISTANCE OF GYPSUM MATERIALS

Two modifiers of different gypsum material modifying features were introduced to gypsum leaven simultaneously. Sarsil H-14/2 and Sarsil M-25 were dosed by 0,25%, Epidian 5 and Epidian 53 were dosed by 1-5% in relation to gypsum adhesive mass. The results of the influence of silicon preparations Sarsil H-14/2 and Sarsil M-25 obtained before proved that optimal contents of these preparations in gypsum compositions amounts 0,25-0,50% in relation to gypsum adhesive mass. Higher contribution of these preparations causes negative changes of strength features of gypsum materials. Thus, it is necessary to state the appropriate correlation between contents of silicon and epoxy resin supplements and hydrophobic and water resistance features of gypsum materials. The variable content of epoxy resins was introduced to gypsum leavens with constant amounts of silicon resins. The results of the research are presented in table 6 and depicted in charts (figure 5 and figure 6).

Table 6. The influence of combined silicon and epoxy resin supplements on the water resistance of gypsum materials

<table>
<tr><td colspan="2" rowspan="2">COMBINED SUPPLEMENTS</td><td rowspan="3">W/A</td><td colspan="4">STRENGTH</td><td colspan="2">MALACIA FACTOR</td></tr>
<tr><td>R_s</td><td>R_{sm}</td><td>R_z</td><td>R_{zm}</td><td rowspan="2">K_s</td><td rowspan="2">K_z</td></tr>
<tr><td colspan="2">%</td><td colspan="4">MPa</td></tr>
<tr><td>Epidian 5</td><td>Sarsil H-14/2</td><td colspan="7">%</td></tr>
<tr><td>1</td><td rowspan="5">0,25</td><td rowspan="5">0,4</td><td>19,22</td><td>10,76</td><td>7,33</td><td>3,54</td><td>0,56</td><td>0,48</td></tr>
<tr><td>2</td><td>21,64</td><td>14,30</td><td>8,02</td><td>4,11</td><td>0,66</td><td>0,51</td></tr>
<tr><td>3</td><td>22,87</td><td>17,55</td><td>8,43</td><td>5,65</td><td>0,77</td><td>0,67</td></tr>
<tr><td>4</td><td>24,21</td><td>19,88</td><td>8,93</td><td>6,77</td><td>0,82</td><td>0,76</td></tr>
<tr><td>5</td><td>25,32</td><td>22,13</td><td>9,04</td><td>7,75</td><td>0,87</td><td>0,86</td></tr>
<tr><td>Epidian 53</td><td>Sarsil M-25</td><td colspan="7"></td></tr>
<tr><td>1</td><td rowspan="5">0,25</td><td rowspan="5">0,4</td><td>21,05</td><td>11,97</td><td>8,31</td><td>4,77</td><td>0,57</td><td>0,57</td></tr>
<tr><td>2</td><td>22,43</td><td>15,22</td><td>8,52</td><td>5,76</td><td>0,68</td><td>0,68</td></tr>
<tr><td>3</td><td>23,84</td><td>18,67</td><td>9,04</td><td>7,22</td><td>0,78</td><td>0,80</td></tr>
<tr><td>4</td><td>24,98</td><td>21,77</td><td>9,61</td><td>8,33</td><td>0,87</td><td>0,87</td></tr>
<tr><td>5</td><td>26,76</td><td>24,02</td><td>10,43</td><td>9,33</td><td>0,90</td><td>0,89</td></tr>
</table>

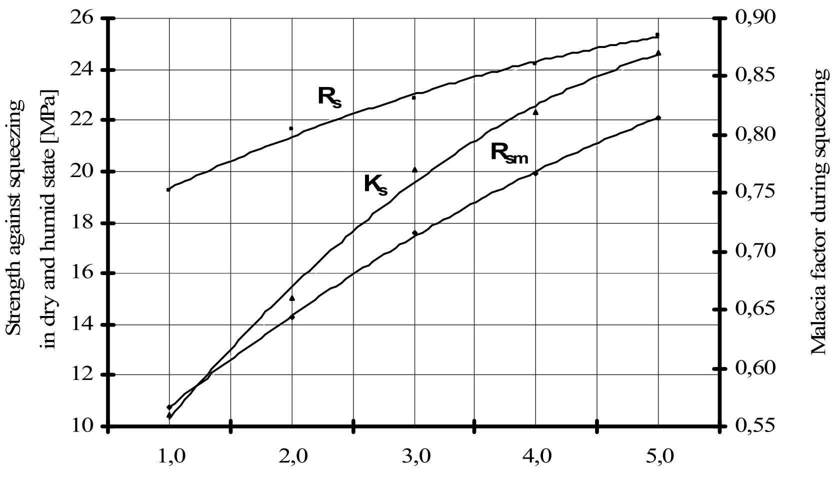

Figure 5. The influence of combined supplements Epidian 5 and Sarsil H-14/2 on water resistance of gypsum materials

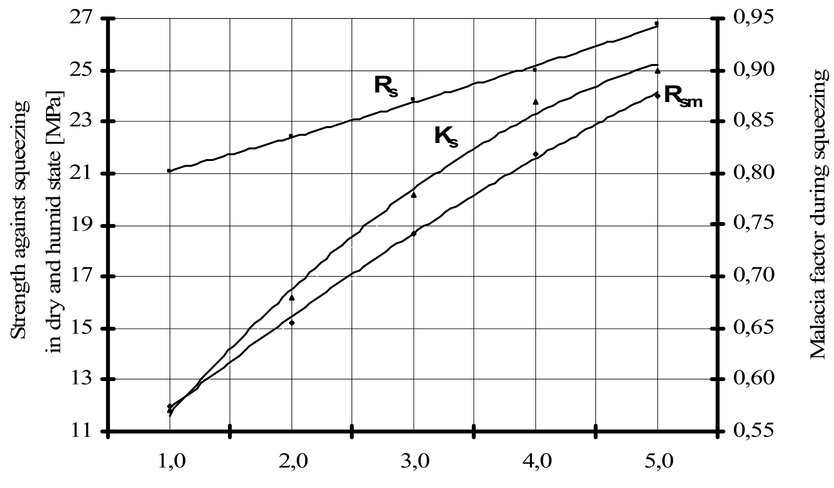

Figure 6. The influence of combined supplements Epidian 53 and Sarsil M-25 on water resistance of gypsum materials

The effectiveness of using combined silicon and epoxy resins to improve water resistance of gypsum materials is presented in table 7. Resin Epidian 5 at 0,25% of Sarsil H-14/2 causes increased strength against squeezing in the dry state of the modified gypsum samples by 4,45-37,60% in relation to strength of control samples, in the humid state, however, 69,44-248,50%. The malacia factor during squeezing increases by 60,00-148,57%. Resin Epidian 53 at 0,25% of Sarsil M-25 causes increased strength against squeezing in the dry state of the modified gypsum samples by 14,40-45,43% in relation to strength of control samples, in the humid state, however, 88,50-278,26%. The malacia factor during squeezing increases by 62,85-157,14%. Modified gypsum samples, when immersed in water for 24 hours did not reveal any mass increase what proves that they are hydrophobic. Modified gypsum trabeculas put vertically in water for 24 hours did not reveal any increase in humidity level. The features of modified gypsum samples of combined supplements of silicon resins Sarsil H-14/2, Sarsil M-25 and Epidian 5 and Epidian 53 prove that the research direction is justified and correct.

Table 7. Effectiveness of using combined silicon and epoxy resins to improve water resistance of gypsum materials

COMBINED SUPPLEMENTS		W/A	STRENGTH INCREASE			
			ΔR_s	ΔR_{sm}	ΔK_s	ΔK_z
%			%			
Epidian 5	Sarsil H-14/2					
1	0,25	0,4	4,45	69,44	60,00	29,72
2			17,60	125,19	88,57	37,83
3			24,29	176,37	120,00	81,08
4			31,57	213,07	134,28	105,40
5			37,60	248,50	148,57	132,43
Epidian 53	Sarsil M-25					
1	0,25	0,4	14,40	88,50	62,85	54,05
2			21,90	139,68	94,28	83,78
3			29,56	194,01	122,85	116,21
4			35,76	242,83	148,57	135,13
5			45,43	278,26	157,14	140,54

Silicon and epoxy modifiers have an effect on gypsum materials features in various ways. Silicon resins reduce absorbability and capillary absorption of water. Epoxy resins, however, improves strength of samples in the dry and humid state. Optimal selection of modifiers results in obtaining gypsum materials with great hydrophobic and water resistance features, which can be used in humid conditions.

7. SUMMARY

The research conducted proves that introducing combined modifiers of artificial resins to gypsum leavens results in obtaining gypsum materials with higher hydrophobic and water resistance features, which can be used in humid conditions. On the basis of the laboratory investigation, the correlation between contents of combined modifying supplements and gypsum material features can be stated.

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
Budownictwo i inżynieria środowiska z 53 2009

Yesim K. AKTUGLU[1]

CURRENT ISSUES OF CIVIL AD ENVIRONMENTAL ENGINEERING, IN SINGAPORE AND AUSTRALIA

ABSTRACT

In recent time, while time is running very fast, there have been lots of constructions, are being built in everywhere. Especially, Changi Airport Terminal 3, opened its doors in the beginning of 2008. It is not a huge structure to house huge spaces, but also it is an environmental issue of today. Singapore Flyer, the sister of London Eye with a height of 135m.,from a height of 165m. says welcome to the new arrivals to Singapore from Changi. In Little India, the La Salle Art School, shines also during the night time while going to Esplanade, but never reachable on time. There are several high rises residential in Singapore, Little India, besides the official skyscrapers in city center.

In Australia, Surfers Paradise is full with skyscrapers as residential for summer times. The airport in Brisbane welcomes to the passengers from Singapore, and makes them go to Sydney. The city pattern of Sydney is a real copy of London, with Hyde Park, and street names around. First stop is at Sydney Opera House, surely. Its architectural and structural solution, coming from a period 1957 and 1973, is still unique in the world. From inside of the Sydney Opera House towards Harbour Bridge during night time, gives enough idea what the Harbour Bridge looks like. Towards Darling Harbour, along the streets, a lot of skyscrapers take place as offices. Around Darling Harbour, the new environmental development, named King Street Wharf, announces about lots of addresses for eating during the day, defining the new name of enjoyment in Sydney. The convention center in Darling Harbour is ready for every exhibition. And the shopping center, nearby the convention center and the sea museum at the next side of the wooden bridge for monorail connection serve for the happiness of the people in Sydney. The New South Wales Museum, just across the Royal Botanical Gardens, is a great chance to meet with talented Australian artists and their products under a huge construction.

In the paper, the flow about the current issues of civil and environmental engineering in Singapore and Australia will be defined with their architectural and structural explanations.

KEYWORDS: Singapore Changi Airport Terminal 3, Singapore Flyer, London Eye, La Salle Art School in Singapore, Esplanade, Surfers Paradise, Brisbane Airport, Sydney Airport, Sydney Opera House, Hyde Park in Sydney, Harbour Bridge, Darling Harbour, King Street Development in Darling Harbour, the New South Wales Museum

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1. INTRODUCTION

It is very easy to understand the current issues of civil and environmental engineering, in Singapore and Australia, when somebody will arrive there. Nearly all connections to the airports of Australia is through Singapore Changi Airport Terminals, 1, 2, 3.

Both countries are in need of the additional effect of tourism activities, besides the business issues. For this reason, they have to build buildings, also attractive for the tourists, besides urgent structures as Harbour Bridge in the artistic manner.

2. SINGAPORE

Singapore is at the southern tip of the Malay Peninsula and it is an island city-state [1]. It has an area of 710.2km^2 [1]. It has an international airport, which is one of the most important airports to get the breaks during the flights for long distances in the world. This creates a big advantage to get an easy access to the country, needs attractions to make people enjoy and spend more time while they are in Singapore for break or for business, etc.

Fig. 1. Welcome of Singapore coming from Changi Airport with Singapore Flyer (photo taken by Yesim Kamile Aktuglu, in June 2008)

2.1 Singapore Changi Airport

When you arrive to Singapore Changi Airport Terminal 2, you immediately take care of the central ventilation issue. If it is summer time, the humidity outside of the airport should be very high. Then the central ventilation should make the guests be happy and relaxed, while the interior decoration of the airport giving that feeling, together with the advanced technology used for the facade supporting system, also for the cantilevered roof structure.

While departing from the Terminal 3, and while giving a break between the flights to Australia and Europe in Terminal 3, you immediately understand the magical points of the newly opened terminal, opened on January in 2008, with its green wall. It is designed by CPG Corporation and SOM, Skidmore, Owings and Merrill LP. The new terminal 3 is located directly opposite Terminal 2, with an area of 380.000 m2. Its roof solution named as “butterfly” roof architecture lets daylight go in [2].

The green tapestry at Terminal 3 at Changi Airport by Singapore Landscape Architect Tierra, has suspended I-beams, and stainless steel cable structure, spanning 300m. across the middle of the voluminous interior. They are covered with vines, creepers and epiphytes in a scale, never have been built here in past [3].

The environmental design of the Terminal 3 is enough sufficient for a passenger to spend time without any suffer, full with shops where all needs can be found, full with cafes, where there can have different tastes, and other enjoyment issues as wireless internet, etc. inside a natural green environment [4].

2.2 Singapore Flyer

When a huge space as Changi Airport welcomes you, the meeting moment with Singapore Flyer is a very attractive point in that conditions.

Singapore Flyer, the sister of London Eye, has a diameter 150m. for its wheel. Its giant Ferris Wheel is the largest in the world, up to the height of 165m. Its architects are Kisho Kurokawa Architects & Associates, and DP Architects with Arup for structural engineers [5]. Arup has learned lots of while they were preparing the conceptual design of London Eye in London for the celebration of 3rd millennium. The main difference between two wheels is London Eye is supported from one side only, and its rim is a triangular truss, while the spindle of Singapore Flyer is supported on both sides, and having a 2-D ladder truss for the rim structure [6].

From far away, it is easy to see the Singapore Flyer in the skyline of Singapore [7].

2.3 La Salle Art School

La Salle Art School is a shinning building complex in the city center of Singapore during the night time, because of its lights and its white color. It is designed by RSP Architects.

Six organically shaped buildings are having facets of glass inside and has an external cladding made of Stone and aluminium. And it has an innovative roof structure to cover the top level of the site area seamlessly. The whole complex is looking like a living sculpture [8]

It is a surpising moment to meet with La Salle Art School with the crystal cladding inside, and black cladding outside. The membrane structure over the buildings is having an end with a space truss [9].

3. AUSTRALIA

Australia is in the southern hemisphere. It is the world’s smallest continent and the world’s largest island [10].

The main meeting points in Australia are at the aiports. From Changi to Brisbane, then to Surfers Paradise area is through the airport hub. Then from Brisbane to Sydney, again the travel is from airport to another airport. Every time, it is experienced the huge areas with their structural solutions, for the function is same, the architectural solution is the same more or less.

Surfers Paradise is a summer area full with skyscrapers, starting from 30 storeys, to upper storeys. Additionally, there are individual houses near the canals, with their traditional characteristics.

Fig. 2. Sydney Opera House in front of the Harbour Bridge (photo taken by Yesim Kamile Aktuglu, in june 2008)

3.1 Sydney Opera House

One of the most attractive venues in Sydney is Sydney Opera House and its environment, not only the performances inside but also the life outside are very colorful moments to enjoy. The results of competition, opened in 1957 and won by Jorn Utzon, the Danish architect, arrived to a worldwide land mark composition in 1973 [11]. In these days, its more than 100 rooms are having performances mostly every day and night. Its unique architectural form supported with unique structural solutions from outside, and its unique inner solutions, again in the architectural manner and structural manner, the moments inside and outside are adding values to lives of the people, experiencing it. This is a vey important built example to understand the situation that if the architectural solution is totally different from the ones till that moment,the structural solution finally is arriving in coming years. And it lasts forever, while it gains money in every moment, with their performances, cafes, schools, etc [12].

3.2 Sydney Harbour Bridge

From everywhere, esp. from the foyer of the great hall inside the Sydney Opera House, the view of Sydney Harbour Bridge is more than great. Its openning time is in 1932 in single-arch form with a longest span for 503m. It has a width for 49m. It has a height of 139m. and it is 49m. high above the sea level [13]. For the touristic activities, it is a good bakground piece after Sydney Opera House from the end of Royal Botanic Gardens Area [14].

3.3 Darling Harbour

To visit the whole Darling Area, it is needed for lots of time. The Pyrmont Bridge with its monorail above, connects the two sides of the Harbour, since 1902, with its wooden structure. It is recorded that it is the largest swing span in the world and it was one of the first to be powered by electricity [15].

From the Pyrmont Bridge, the observation for the Darling Harbour gives the most detailed infos in the least time. At one side, there are the Sydney Aquarium and King Street Wharf, while at the other side, there are a huge Harbourside Shopping Centre, Australian National Maritime Museum, Sydney Convention and Exhibition Centre and others [16].

3.4 King Street Wharf

King Street Wharf has an area around five hectares. And it is a former maritime industrial area [17]. In these days, there are development activities for the places of old wharf structures, as residential, reail, hotel, tourist and charter boat activities, waterfront promenade and associated infrastructure [18].

The new structures to house the new development topics as cafes, restaurants are having an attractive structural steel solution for high rise buildings along the waterfront promenade [19].

3.5 New South Wales Art Gallery

The Art Gallery of New South Wales is coming from old times in1884 when the first purpose built gallery building was opened. Later on, in 1971, the area is enlarged rom 2000 to 4900 m2. During the refubishment period, the grey toned rough concrete was used to "blend" with the sandstone of the old building [20].

In these days, the more area as cafes and restaurants were added towards the sea with structural steel in cladding with glass [21].

4. CONCLUSION

As a conclusion, the built examples in Singapore and Australia, give an idea about the topic of their civil and environmental engineering level.

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INFLUENCE OF CONCRETE ADMIXTURES ON LOWERING THE CONCRETE MIX FREEZING POINT

ABSTRACT

The paper presents the 'anti-freezing' action of admixtures in the light of their influence on lowering the freezing point of fresh concrete. There have been presented the results of freezing point tests for selected admixtures to concrete mix in commercial concentrations, reduced to the concentrations applicable in dosing to a regular concrete mix, as well as the results of tests of the concrete mix with the use of the said admixtures. The results of the tests should be helpful in understanding which admixtures better support the protection of concrete that sets in low temperature conditions, and in particular – why the application of admixtures does not enable the abandoning of other protective activities at the early stage of concrete setting in minus temperatures.

KEYWORDS: concrete, concrete admixtures, freezing point, concrete casting in winter, concrete curing

1. INTRODUCTION

Application of anti-freeze admixtures is often thought to be the only and sufficient procedure to protect the hardening concrete in low temperature conditions or in winter. This is, unfortunately, a wrong belief and may lead to serious problems [1] or even a construction catastrophe [2]. Nevertheless, it results from some lack of knowledge with regard to the mechanisms of operation of anti-freeze admixtures or is an effect of begin misguided by the contents of the technical charts of some admixtures. The information: 'the admixture enables concrete casting down to the temperature of -4°C' – or even down to -10°C – is understood by many contractors (also concrete manufacturers) as a method to secure themselves against the negative consequences of fresh concrete freezing. They do not perceive or do not understand the cunningly added note: 'the application of the admixture does not release the contractor from complying with the principles of casting stipulated in the Instruction of the Building Research Institute (ITB) No. 282' [3]. This is a very important note, as it mainly requires the protection of the freshly cast concrete against freezing, until it reaches sufficient strength – regardless of whether there has been applied any admixture or not. This is also a specific protection for the manufacturer (distributor) of the admixture if the concrete shall freeze, which will result in negative consequences for the structure and demanding compensation by the contractor for damages on that account.

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The paper presents the influence of using admixtures on lowering the freezing point of fresh concrete. The results of the presented research shall surely help to understand why the application of admixtures does not protect concrete against freezing and why it does not allow for abandoning other protective activities at the early phase of concrete hardening in minus temperatures [4].

2. WHAT ARE ANTI-FREEZE ADMIXTURES?

The use of the word ‘anti-freeze’ in the name of admixture is not currently specifically justified, as in the present systematic introduced by the PN-EN 934-2 standard [5], such admixtures have not been defined. Such names are often met in technical literature, as well as in the previous version of the Polish Standard [6]. Anti-freeze admixtures were defined in the Polish Standard as:

‘... - products enabling the reaction of cement and water in minus temperatures; they are substances causing:

- acceleration of cement hydration heat release and concrete temperature increase;
- decrease of water freezing point in fresh concrete;
- decrease of the quantity of batch water maintaining the assumed consistency;
- entraining in the fresh concrete mix of a large number of microscopic air bubbles;

the admixtures are inorganic and organic compounds causing physical and chemical reactions in hydration processes.’

Thus, anti-freeze admixtures may be classified according to the standard [5] as accelerating binding, accelerating hardening, reducing or strongly reducing batch water and air-entraining.

The term ‘anti-freeze admixture’ may often be met in catalogues or technical charts of many admixture manufacturers. If such name is not used, then the following wording often appears in the technical description of the product: ‘the admixture enables concrete casting down to the temperature of -10ºC’ or ‘admixture enabling concrete casting at low temperatures’ or ‘accelerating admixture’.

Admixtures accelerating binding or hardening are mainly chemical compounds (salts) without chloride contents (as these are forbidden, particularly for reinforced concrete). They possess reaction accelerating properties and their main action is the decrease of batch water freezing point in concrete. In regular concrete batch water freezes at temperatures below 0ºC, which is a consequence of the concentration of the solution of salts being cement components. Adding to the concrete mix of an admixture, which is also a salt solution, results in increased concentration and further decrease of the freezing point.

The protective action of air-entraining admixtures brings about the same mechanism, which results in concrete freeze resistance. Actually, these are the same admixtures. Large quantity of microscopic air bubbles become the space for increased volume of batch water during freezing.

In the case of plasticizers and super plasticizers the quantity of water in the concrete mix is reduced. The protective effect is doubled. Firstly, it influences the increase of the “spare” strength, which may be lost as a result of hardening at low temperatures. Secondly, the change of batch water physical properties is of major importance – there is less water and, thus, the concentration is higher, which decreases water freezing temperature (similarly as in the case of salt admixtures accelerating the processes of binding and hardening). Additionally, the quantity of water that may freeze extending its volume decreases.

3. INFLUENCE OF ADMIXTURES ON DECREASING THE FREEZING POINT OF FRESH CONCRETE

According to literature (e.g. [3, 7]), batch water in regular concrete freezes at the temperature between -1ºC and -3ºC, as an effect of the concentration of the solution of salts being cement components. This results from additional introduction of a concentrated salt solution (in the case of accelerating admixtures) or decreasing the quantity of water (in the case of plasticizers and super plasticizers). According to manuals, this shall cause the lowering of the freezing point of batch water by further 1 to 3 degrees (e.g. [7]), resulting in the case of some admixtures even in -10°C freezing point [8]. Such information gives hope to the contractors or concrete manufacturers that the application of admixtures may fully secure concrete against freezing in the period of minus temperatures. Nevertheless, the data is too optimistic and may lead to specific workmanship errors.

The carried out research [9] of the freezing point of the very admixtures – or rather the solutions of admixtures at concentrations resulting from the proportion of dosing to cement (batch water) – shows that it is hard to expect such drastic decrease of batch water freezing point. Figure 1 presents temperature charts for four selected admixtures at commercial concentrations that have been chilled. Green line represents accelerating admixture (salt solution – the contents of the admixture is the trade secret of the manufacturer), which did not freeze at all in the researched interval of temperatures (temperatures down to -20°C were researched). Other admixtures, regarding the value of the expected significant freezing point decrease are, respectively: black line – mixture of accelerating admixture and super plasticizer

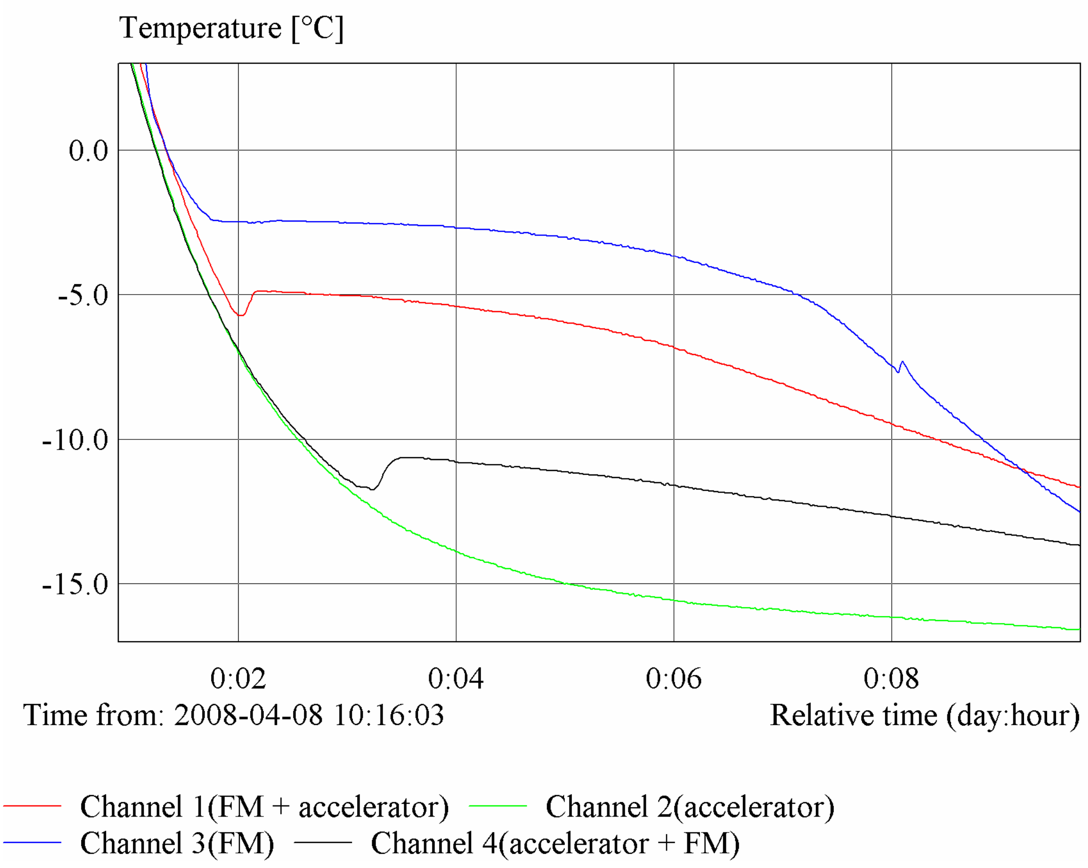

Figure 1. Freezing points of four selected admixtures to concrete mix at commercial concentrations.

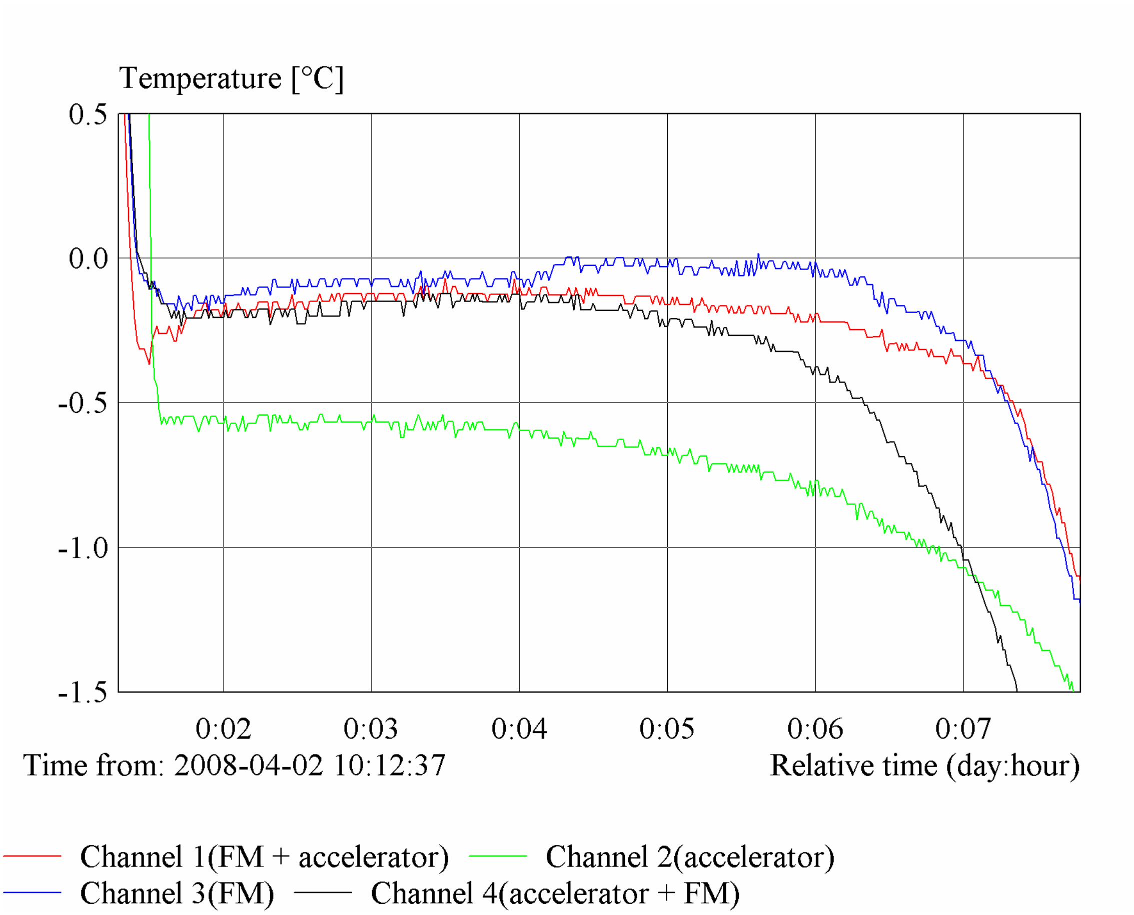

Figure 2. Freezing points for the solutions of four selected admixtures (the same as in Figure 1).

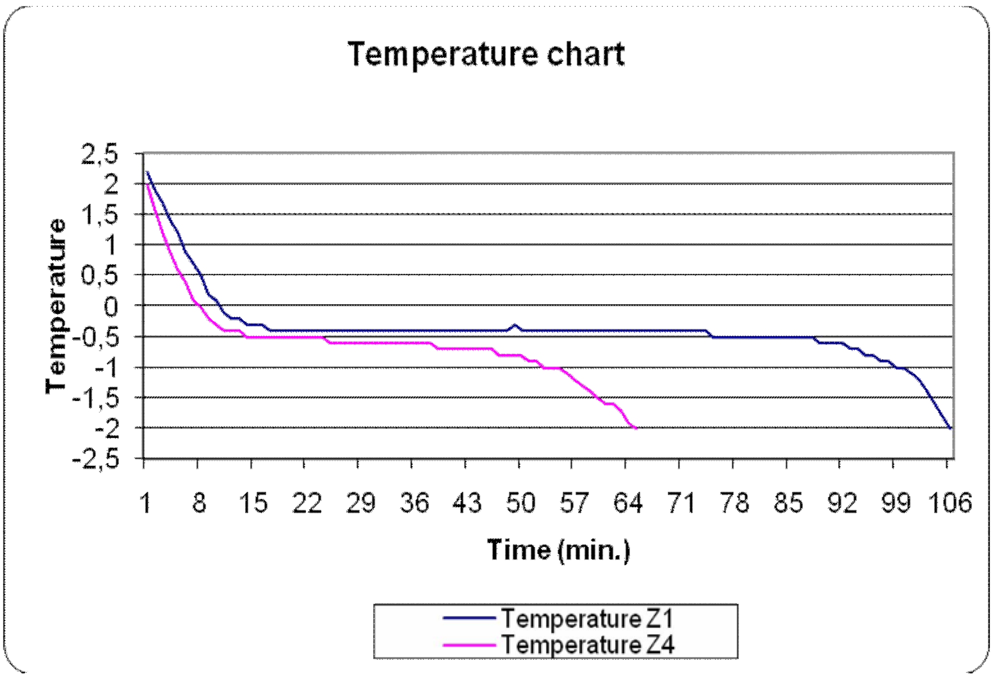

Figure 3. Freezing point difference between fresh concrete without admixture (Z1) and with accelerating admixture (Z4) [10].

(contents reserved by the manufacturer), red line – mixture of super plasticizer and accelerating admixture (proportions reverse to the preceding one, however, contents have also been reserved by the manufacturer), blue line – super plasticizer being a mixture of naphthalene and melamine sulfonates. Freezing points are, respectively: -11.7°C, -5.7°C and -2.5°C. Figure 2 presents temperature charts for the solutions of the same admixtures, however, at maximum concentrations suggested by the manufacturers (for concrete of water-cement ratio = 0.5). The solution of the accelerating admixture of such concentration freezes already at the temperature of -0.7°C, while the solutions of other admixtures freeze at higher temperatures, ranging from 0.4°C to -0.1°C. The combined influence on the freezing point resulting from the consequences of dilution of salts contained in the cement with the consequences of applying admixtures is presented in Figure 3. It shows the specimen measurement of the freezing point of fresh concrete without admixture (master) and the same concrete with accelerating admixture (the same as has been used in the tests presented in Figures 1 and 2). The difference (-0.4°C for master concrete and -0.7°C for concrete with admixture) is slight and does not confirm the possibility of considerable decrease of the concrete mix freezing point as an effect of applying admixture.

4. SUMMARY

It is generally expected that admixtures to concrete mix (particularly ‘anti-freeze’ admixtures) shall considerably decrease the freezing point of a concrete mix. The presented research results clearly show that the application of admixtures even in the maximum quantities specified in product technical charts does not considerably affect the parameter – finally, it has been confirmed that fresh concrete freezing point is at the most several decimal points of blow zero degree.

Due to the above comments regarding the results of the research, there become clear the recommendations imposed by the ITB’s Instruction [3] or the most novel document in the form of a European preliminary standard regarding concrete works – ENV 13670-1 [11], which absolutely forbid freezing of concrete that has been cast before it reaches the strength of 5 MPa. This shall be ensured by the adequate protection and curing of the freshly cast concrete, which also includes ensuring the minimal temperature of the cast mix required by the PN-EN 206-1 standard, namely +5°C, also in winter.

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RESEARCH OF REINFORCED-CONCRETE BEAMS DEFORMATIONS, RENEWED AFTER THE INFLUENCE OF AGGRESSIVE ENVIRONMENT

ABSTRACT

The article presents the experimental and theoretical research of reinforced-concrete beams deformations, damaged in the conditions of simultaneous influence of aggressive environment and loading with their further renewal. The peculiarity of this research is that during influence of aggressive environment reinforced-concrete beams were tested both under the action of loading and without it. The influence of aggressive environment and renewal took place in the conditions of the operating loading on the reinforced-concrete beams. Such a combination of influences allows to define the optimal method of research of the renewal process in reinforced-concrete structures and the influence of loading, at which the strengthening takes place on the efficiency of renewal and deformations of experimental samples. The experimental and theoretical values of deformations of reinforced-concrete beams which were renewed without loading and under the operating loading are obtained. Their comparison is carried out with the purpose of determining the optimal method of research of such structures.

KEYWORDS: reinforced-concrete, corrosion, renewal, simultaneous influence, stress-strain state, deformations.

1. INTRODUCTION

A lot of building structures have been already exploited for a long time in the conditions of influence of different aggressive environments, especially it is characteristic for industrial enterprises. It results in appearance of corrosive damages of constructions, therefore at present much attention is paid to the problem of such constructions strengthening, reconstruction and renewal of their load-carrying capacity. It is also necessary to take into account at the damaged structures renewal, that in the real time the process of renewal often passes in the conditions of influence of operating loading on the structures.

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The experimental testing of deformations of the renewed reinforced-concrete beams which were damaged in the conditions of simultaneous influence of high aggressive environment and loading is considered in this article. The peculiarity of this research is that the renewal of damaged constructions is carried out both under the action of loading and without it. Such a combination of influences allows to define the optimal method of research of renewal process of reinforced-concrete constructions.

A lot of scientists and researchers were engaged into the experimental research of corrosive influences on reinforced-concrete constructions [1-3]. However the analysis of the mentioned research works shows that the problem of influence of simultaneous action of loading and aggressive environment on the stress-strain state of reinforced-concrete constructions, as well as the renewal of bearing capabilities of such constructions is not studied enough [4].

The experimental research of durability of normal cuts of reinforced-concrete beams, damaged as a result of simultaneous action of the operating loading and aggressive environment for next renewal of their bearing capacity was examined in one of the previous articles [5]. Deformations of such constructions are examined in this article, namely the maximal beam's deflection, deformation of the outer fiber of the compressed concrete. For comparison, the renewal of reinforced-concrete constructions is also conducted without loading on a construction with the next test of the renewed beams to failure.

2. METHODS OF EXPERIMENTAL RESEARCH

For implementation of experimental research there were made the series of reinforced-concrete beams by sizes 2100×200×100 mm with the working armature 2∅14 A-III. The concrete strength of beams for 28 days was 41,0 MPa [5].

The normal and the renewed beams were tested to failure by two concentrated forces in the third of the beam span. Loading was put in steps. During testing the deflection, cracking, deformation of concrete and armature were fixed. The failure of beams took place as a result of yield point of armature in the zone of the highest bending moment with the next breaking up of concrete in the compressed zone. The strength of normal sections of beams, without corrosion damages, was equal to M_u^{exp} =24,81 кN·m (the beam of BZ-1.1).

The experimental tests of reinforced-concrete beams at the simultaneous action of aggressive environment and loading have been executed on the special device (fig. 1).

a) b)

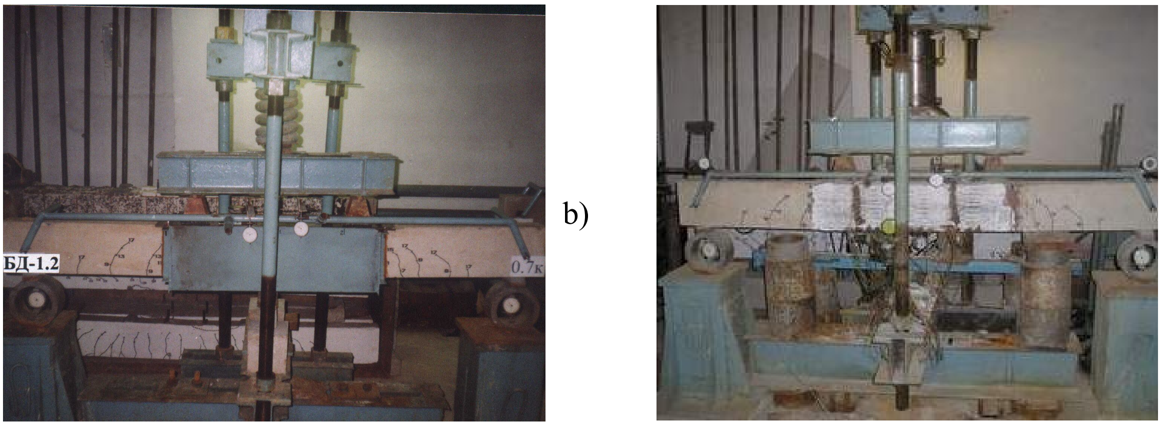

Fig. 1. The special device for the test of beams:
a) at the long-term action of aggressive environment and loading;
b) at the short-term test of the renewed beams

Two beams were loaded to the level 0,7 M_u^{exp} and after it to them was added the influence of aggressive environment (beams BDp-1.2-0.7к and BDp-1.3-0.7к). Other two beams were dipped only into the aggressive environment without the influence of loading (BDp-1.4-к and BDp-1.5-к). All beams were twins. For the faster action of aggressive environment on beams the special capacities with 10% solution of sulphuric acid were used. The acid concentration, to which the products of corrosion of beam materials were disposed, was under the permanent control.

For beams, loaded to the level $0,7M_u^{exp}$ deflection were increased nonlinear with a more noticeable increase after the 68-th day. Further, on the 81-th day from the beginning of the aggressive environment influence, the access of corrosive environment was stopped, and beams were prepared to strengthening. The section width was diminished on ~32...35 mm, the section depth diminished on 25...30 mm.

The strengthening of reinforced-concrete beams was carried out by the renewal with new fine grained concrete. At renewal of a section by such a method, of importance is tripping of construction existent concrete with the new strengthening concrete. For the choice of the best connecting layer separate experimental tests were conducted which allowed to choose the most effective material. It was Sika Monotop 610. The process of section strengthening and the bearing capacity renewal of reinforced-concrete beams took place without their unloading, under the operating load $0,7M_u^{exp}$. The surface of beams was cleaned out with the sandblast machine. Further on all of beams the structural framework from a steel wire ∅1,2 mm was set (four longitudinal bars with transversal hopes with a step 60 mm). Then the connecting layer of Sika Monotop 610 was inflicted and renewed in the section of beams by concrete. After 28 days the renewed beams were tested by short-term loading to failure.

3. RESULTS OF THE EXPERIMENTAL RESEARCH

According to the accepted testing method, the descriptions of deformations were determined at the moment of time, when there was a loss of beams bearing capacity after the armature yield. Basic descriptions of beams deformations were deformations of the concrete compressed area and beams deflection in the middle of span.

For beams loaded to the level of $0,7M_u^{exp}$ the increase of concrete deformations after loading was 130×10^{-5} and deflection was 7,3 mm (fig. 2).

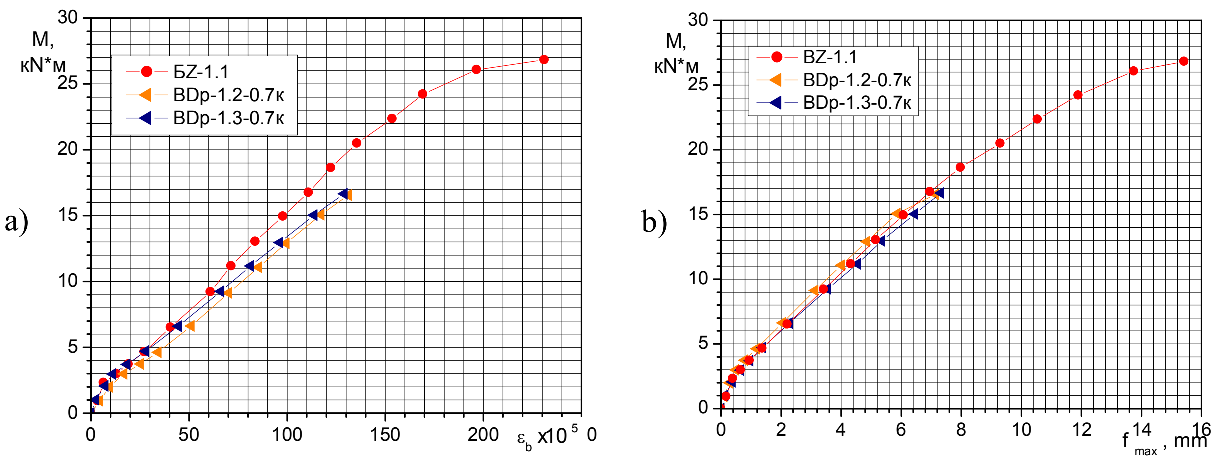

Fig. 2. Deformations of experimental beams before the corrosion influence:
a) deformations of concrete; b) deflection

During a period between loading of beams to the $0{,}7M_u^{exp}$ and beginning of influence on them of the aggressive environment, there is an increase of deformations of the compressed concrete (~20×10^{-5}) and deflection (~0,45 mm). It is explained by the display of concrete plastic deformation and armature stress relaxation, which are present at first moments after constructions loading. Deformations and deflection of beams are stabilized further.

The access of the aggressive environment to the beams of BDp-1.2-0.7к and BDp-1.3-0.7к was halted, shutting out the beginning of armature corrosion. The graphs of concrete deformations change as well as beams deflection in H_2SO_4 environment during 81 days are presented on fig. 3. At the moment of halting of the aggressive environment influence the concrete deformation was 225×10^{-5}, 229×10^{-5} for the beams of BDp-1.2-0.7к and BDp-1.3-0.7к accordingly. Beams deflection were – 12,56 mm, 12,86 mm accordingly.

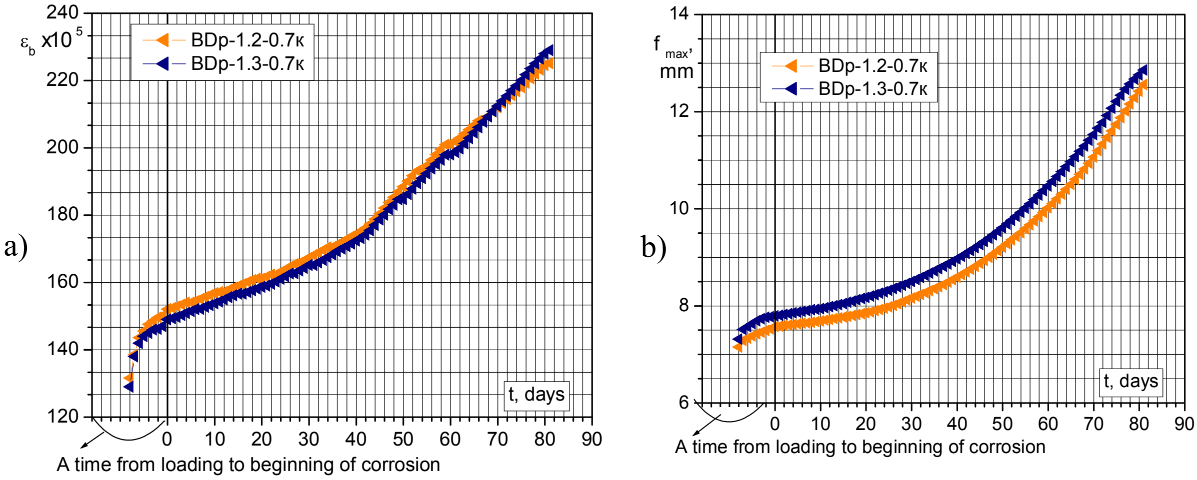

Fig. 3. Deformations of experimental beams in the environment of H_2SO_4:
a) deformations of concrete; b) deflection

After the stay of all beams in the aggressive environment they were strengthened and tested under the short-term loading to failure. Sizes of concrete deformations and deflection at the moment of bearing capacity loss at yield of armature were 254×10^{-5} and deflection 13 mm (BDp-1.2-0.7к and BDp-1.3-0.7к), and 184×10^{-5} and 10 mm (BDp-1.4-к and BDp-1.5-к).

The difference in values is explained by the presence of long-term action of loading (BDp-1.2-0.7к and BDp-1.3-0.7к), at which micro cracking in the concrete, caused by corrosion, become the stress raisers, which result in the increase of concrete deformations and deflection. Comparisons of the deformations graphs of upper edge of the compressed concrete and maximal deflection of beams are presented on fig. 4.

Experimental values of concrete deformations of the compressed zone and deflections at the moment of bearing capacity loss, as well as their comparison to the theoretical values of deflections according to the Ukrainian norms [6] are presented in table 1.

It should be noticed, that in this case the method of norms allows with sufficient accuracy to determine deflection of the renewed beams, tested by short-term loading. Divergence between theoretical and experimental values make -5,8%...11,6%, toward understating or overstating of theoretical values.

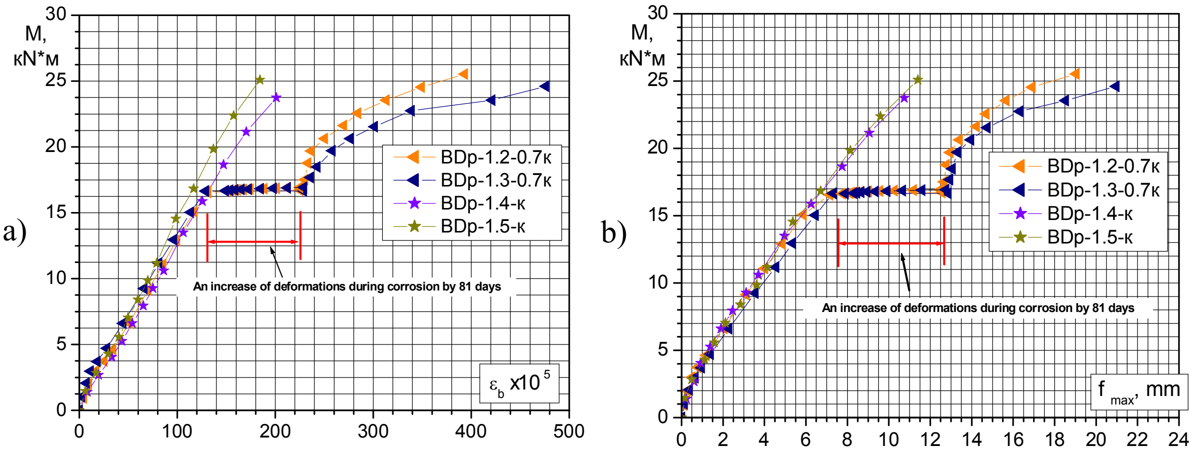

Fig.4. Deformations of the renewed experimental beams after the long-term test loading to failure: a) deformations of concrete; b) deflection

Table 1. Deformations of experimental beams, renewed after the influence of aggressive environment and tested to failure

Identification of beams	Experimental values: Bending moment	Experimental values: Concrete deformations $\varepsilon_b\times10^{-5}$		Experimental values: Beams deflection f ,mm		Theoretical values of deflection according to norms	$\frac{f_1^{exp}}{f_1^{norm}}$
	yield of armature M_u^{exp} кNm	before loading	at M_u^{exp} $\varepsilon_b\times10^5$	before loading	at M_u^{exp} f_1^{exp} мм	at M_u^{exp} f_1^{norm} mm	
BDp-1.2-0.7к	20,07	225,1	242,0	12,56	13,13	13,94	0,942
BDp-1.3-0.7к	20,15	228,9	266,6	12,86	13,61	14,01	0,971
BDp-1.4-к	23,15	0	193.1	0	10,49	8,39	1,250
BDp-1.5-к	24,04	0	174.5	0	10,71	8,99	1,191

The analysis of results shows that on the values of concrete deformations and deflection the decisive influence is caused by the presence or absence of the long-term loading during the stay in the aggressive environment. In spite of the fact that the bearing capacity of the beams with normal section of BDp-1.2-0.7к and BDp-1.3-0.7к was less (M_u^{exp}=20.07….20.15 kNm) than the beams of BDp-1.4-к and BDp-1.5-к (M_u^{exp}=23.15...24.04 kNm) the deformations of these beams were considerably larger. That is, the final value of concrete deformations of upper edge to the section and deflection depend on the presence of the long-term loading before the beginning of short-term loading test of the renewed beam.

The analysis also shows, that the method of design norms at the calculation of deformations of renewed reinforced-concrete elements allows with the satisfactory accuracy to determine the value of deflection.

At the experimental research of beams, renewed after the corrosion without loading and at the action of loading with the next short-term test to failure, the distribution of deformations on the section height on different stages of loading were tested. For the beams of BDp-1.4к and BDp-1.5к, renewed after corrosion without the action of loading on all stages of short-term test, the linear distribution of deformations with more intensive increase near to failure was noticed. At the moment of armature yield the deformation of the concrete compressed of renewed beams was 185×10^{-5} and 201×10^{-5} and for the beams of BDp-1.5к and BDp-1.4 к accordingly.

For beams, increased after the corrosion at the long-term action of the initial loading $0{,}7M_u^{exp}$ there was noticed a more intensive increase of deformations on the level of strengthening of the concrete upper edge section in comparison with the increase of deformations of concrete on the level of the edge of old damaged section.

Thus at the moment of achievement of armature yield M_u^{exp} deformations at the level of edge of old damaged section equal $229\ldots243\times10^{-5}$. At the level of concrete higher edge of sections strengthening for the same beams the deformation equals $25\ldots47\times10^{-5}$. The analysis of the obtained results shows that deformations values at the level of old damaged concrete exceed ε_{bu}, that corresponds to the concrete strength R_b. At the level of the concrete higher edge deformations were considerably lower. These witnesses about fact that the exploitation of characteristics of concrete strengthening is used not fully enough. Thus it is necessary to take the mentioned facts into account at the analysis of the stress-strain state of the reinforced-concrete beams increased after corrosion.

4. CONCLUSIONS

Consequently, as a result of the performed research, it is possible to draw a conclusion, that the method of research of deformations parameters of reinforced-concrete constructions for providing of research adequacy requires the account of influence of simultaneous action of loading at strengthening of constructions.

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
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SEVERAL DEFINITIONS OF ZERO-ENERGY BUILDINGS DEPENDING ON CALCULATION PROCEDURE AND BOUNDARY CONDITIONS

ABSTRACT

Depending on the country, geographic situation, weather conditions, measuring, there are different definitions of zero-emission or zero-energy buildings. First there must be decided if the building is on the grid or not. Afterwards, guided by the calculation, the on-the-grid buildings can be divided in four groups: net zero site energy use, net zero source energy use, net-zero energy cost, net zero energy emissions. With every definition, you have some specific systems, also depending on different boundary conditions. Every building needs to have his own monitoring system so they can see if they are a zero-energy building or not. Belgium and Poland are the two countries which we compared. They are not so similar on the used systems for renewable energy. Mostly, it depends on what the government advice, what they're giving their support to. Still, in the end, the user makes the choice.

KEYWORDS: zero-energy, building, definition, monitoring

1. INTRODUCTION

In Europe 40% of our energy use is consumed in buildings, more than by industry or transport. Keeping our homes comfortable uses a lot of energy. Almost half of the average home's energy consumption is used for heating. Another 17 % is used for water heating, 6 % for cooling rooms, and 5 % for refrigeration. Almost one-fourth of the energy used in homes is used for lighting and appliances. There are different types of energy sources. 40% of hot energy use is extracted from petroleum. The world supply decreases every day with 85 million barrels (1000 barrels/sec). The supply is not endless. Experts predict that the Peak Oil is around 2030. (Peak oil is the point in time when the maximum rate of global petroleum extraction is reached, after which the rate of production enters terminal decline).

Second biggest used energy source is natural gas. Natural gas is a fossil fuel formed from plant matter over the course of millions of years. It is a finite resource and thus considered to be a non-renewable energy source. The world gets almost 1/4 of its energy from natural gas. The consumption of natural gas has nearly doubled in the last 30 years. The third biggest use

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and also the last fossil fuel is coal. Coal has the most widely distributed reserves; it is mined in over 100 countries, and on all continents except Antarctica. The largest reserves are found in the USA, Russia, Australia, China, India and South Africa. Not all the mines are discovered yet, so they don't know exactly when the reserves will be ended. After the fossil fuel, uranium is most used in the world. The world's top producers are Canada (28% of world production) and Australia (23%). Other major producers include Kazakhstan, Russia, Namibia and Niger. With 3.3 million tones, all 436 world-wide operating nuclear power plants can be supplied for several decades. Conclusion**:** all the resources are running out. Therefore, more renewable energy sources must be used. As seen in graph 1, they are now only responsible for 7% of total energy use.

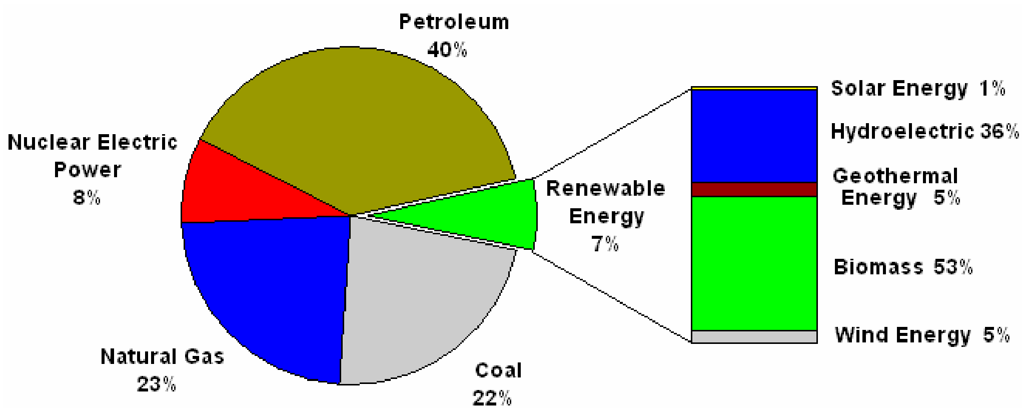

Figure 1. The share of every energy source in the world [1]

2. THE COMMON MEANING OF ZERO-ENERGY BUILDINGS

A zero energy building (ZEB) or net zero energy building produces at least as much energy as it consumes on an annual basis, with zero carbon emissions annually. This design principle is gaining interest as renewable energy is a way to cut greenhouse gas emissions. Advantages: reduce of energy prices by using more insulation, increasing comfort due to more-uniform interior temperatures, reducing total net monthly cost of living, higher resale value,… Disadvantages: initial costs can be higher; few designers or builders have the necessary skills to build ZEBs, climate-specific design may limit future ability to respond global warming, … The most cost-effective energy reduction in a building usually occurs during the design process. Sunlight and solar heat, prevailing breezes, and the cool of the earth below a building, can provide day lighting and stable indoor temperatures with minimum mechanical means. Z.E.B.'s are optimized to gain passive solar heat and also shading. Combined with the thermal mass of the building, that should stabilize diurnal temperature variations throughout the day. Figure 1 shows the steps to achieve a ZEB. First choose the right orientation of your building to have sunlight on the right places en then induce renewable energy by using wind- and solar power. Follow this tree steps to achieve a ZEB: first reduce the energy demand to 15 kWh/m²: airtight building, min. 30 cm of well-chosen insulation, heat recovery by heat pumps and well chosen ventilation systems, double layered glass and energy-efficient equipments inside. Then use an efficient system for heating and hot water and good blinds to prevent overheating. Finally generate the remaining energy with renewable energy sources. [2]

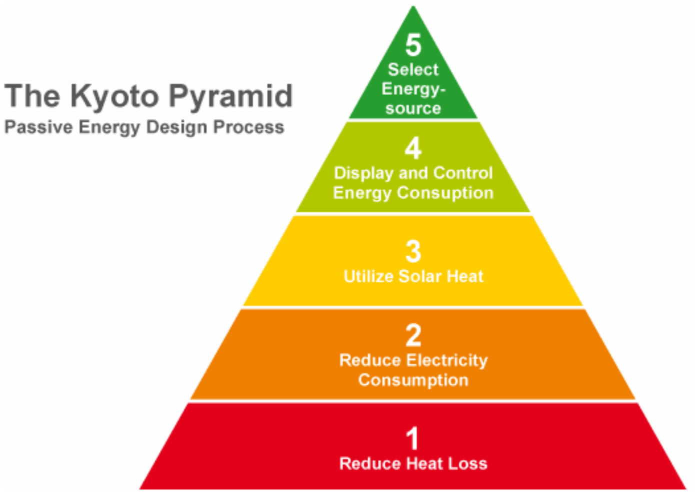

Figure 2. The important steps to reach a passive design and eventually a ZEB [3]

3. DIFFERENT DEFINITIONS OF ZEB –BUILDINGS

3.1. Off-the grid: stand alone ZEB

Off-grid homes are autonomous; they don't rely on municipal water supply, septic tank, natural gas, electrical power grid or similar utility services. Electrical power can be generated on-site with renewable energy. Renewable energy sources capture their energy from existing flows of energy, from solar power, wind power, wave power, hydropower, bio-fuel such as anaerobic digestion, and geothermal power. On-site water sources can include a well, stream, or lake. Depending on the water source, this may include pumps or filtration.

Advantages: increase security, less CO_2-emission, independent of the energy market prices and reduce environmental impacts by using on-site resources (sunlight, rain,...).

Disadvantages: Growing all of your own food is more time-consuming; autonomous living can require sacrificing lifestyle choices and personal behavior, some find it isolating.

There are some systems used for the water need off-the grid: greywater systems (using wastewater to water plants and flush toilets), composting toilets, a well, a solar still osmosis (to distillate water). To generate electricity solar cells and wind turbines are used. For heating and cooling they use passive solar design, a Trombe wall (sun-facing wall that uses thermal mass), earth sheltering, concrete core activation and heat recovery ventilation. Finally to have warm water, the use of a solar boiler, cogeneration and hot water recycling is required. [4]

3.2. On the grid [5]

3.2.1. Net zero site energy use

In this type of ZEB (mostly used in the USA), the amount of energy provided by on-site renewable energy sources is equal to the amount of energy used by the building.

Generation examples include photo voltaic cells or solar hot water collectors. A limitation of a site ZEB definition is that the values of various fuels at the source are not considered. For example, one energy unit of electricity used at the site is equivalent to one energy unit of natural gas at the site, but electricity is more than three times as valuable at the source (no losses of transportation). For all-electric buildings, a site ZEB is equivalent to a source ZEB. For buildings with significant gas use however, a site ZEB will need to generate much more on-site electricity than a source ZEB. It can be easily verified through on-site measurements.

3.2.2. Net zero source energy use

The 'zero primary energy building' or 'zero energy source building' recognizes that the off-site generation of energy, particularly electricity, is very inefficient. Typically only around 35% of the energy used in a normal fossil fuel power plant is converted to electricity, with the remainder lost as waste heat. Further losses accumulate during electricity transmission. Because of this, in order to meet the definition of zero primary energy use, the amount of electricity exported must be substantially higher than the amount of energy registered on the electricity meter. To calculate a building's total source energy, both imported and exported energy are multiplied by the appropriate site-to-source energy factors. To make this calculation, power generation and transmission factors are needed.

3.2.3. Net zero energy cost

Net zero energy cost relates to the price of energy. In such a building, the cost of purchasing energy is balanced by income from sales of electricity to the grid of electricity generated on-site. Whether this balance can be preserved over the medium to long term is subject to changes in energy prices. The money received for the exported electricity will have to compensate energy, distribution, peak demand, taxes, and metering charges for electricity and gas use. In wide-scale implementation scenarios, this definition may be ineffective because service rates will change dramatically. For commercial buildings, a cost ZEB is typically the hardest to reach, and is very dependent on how a utility credits net electricity generation and the utility rate structure the building uses.

3.2.4. Net zero energy emissions

Outside Canada and the US, a net zero energy building is usually defined as one with zero net energy emissions, also known as a 'zero carbon building' or 'zero emissions building'. Under this definition the carbon emissions generated from on-site or off-site fossil fuel use are balanced by the amount of on-site renewable energy production. This includes not only the carbon emissions generated by the building in use, but also those generated in the construction of the building and the embodied energy of the structure. The net zero emissions ZEB definition has similar calculation difficulties. Many of these difficulties are related to the uncertainty in determining the generation source of electricity.

4. SUSTAINABLE BUILDING IN DIFFERENT COUNTRIES

4.1.IPD Environment Code

The IPD Environment Code was launched in February 2008; it sets a new global standard for measuring the environmental performance of buildings. They are responsible for 20% of global CO_2 emissions. Therefore, the company will be asked to supply more detailed and higher quality information about their estate's environmental performance. The code covers a wide range of building types (from offices to airports). [6]

4.2. Rating system worldwide

Many countries have developed their own standards of energy efficiency for buildings such as EPB (Belgium), LEED (Brasil, Canada, India, Mexico & USA), Green Star (New Zealand, Australia & South Africa), BREEAM (The Netherlands and the UK),… Poland doesn't have its own system yet, but it's working on it.

5. ZEB IN POLAND AND BELGIUM

5.1.Belgium

In Belgium, renewable energy covers 6% of the energy use. Lately, the renewable energy production is increasing while the energy use stays stable. Most of the renewable energy comes from biomass (55%), followed by wind energy (17,2%) and biogas (15,4%). The government is stimulating well-isolated homes, placing PV-cells, the use of high-efficiency glass and condensation boilers,... by giving subsidies and taxes reduce. The use of biomass as energy source, heat pumps and condensing boilers is gaining interest. [7]

5.2. Poland

In Poland, mostly used renewable energy sources are: water energy, then wind energy, biomass power and then biogas power. But the percentage of use such energy sources are still small. [8]

5.3. Comparison

Belgium and Poland are quite similar lands, the weather conditions are almost the same. The biggest differences are that Belgium is ten times smaller, has a higher density and also a smaller coastline. Most used renewable energy system for countries like Belgium and Poland are PV-cells, solar boilers, heat pumps & windmills (these last two are more used in Belgium than in Poland at the moment). If the government promotes renewable energy more, people will use more and more RES. They also place common energy systems, so more people can have the profits of a system without paying a huge amount of money.

6. MONITORING

Knowing how your building uses energy is the key to optimize your resources and improving energy efficiency. Residential buildings have carbon or energy footprints (residential and commercial buildings cover for example 40 % of the U.S. energy market). If the cause of the footprint is found, there can be something done about it. Therefore you need monitoring of your energy use and energy savings. Formula for success: energy efficiency + renewable energy + monitoring & visualization = sustainable high performance buildings. Different monitors used are carbon footprint monitor (that you can use for you whole living style or just for your home), sun flow monitor, … Even with user-friendly systems, such as a energy flow program for iPod, the energy flow of your house can be monitored. Off course, every RE system must be checked and improved frequently so the yield stays good.

6.1.Monitoring equipment

A data logger is often used because it records data over time or in relation to location either with a built in instrument or sensor or via external instruments and sensors. For measuring temperature, the use of thermocouples (TC), thermistors, resistance temperature detectors, and integrated circuit temperature sensors are recommended. Temperature measurements for building energy evaluation purposes are less difficult than with many engineering applications because accuracy requirements are typically not as tight, environmental conditions are not extreme, and response times can be longer. The simplest method for measuring the solar radiation is with a pyranometer, which measures the total solar radiation on a surface. Luminance is a measurement of radiation as sensed by the human eye. Photodiodes are the most common sensors for luminance measurements. The relative humidity and the gas flow are also monitored. [9]

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
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EVALUATION OF REINFORCEMENT BONDING IN FOAM CONCRETE

ABSTRACT

The presented study is devoted to experimental research of the reinforcing bar and foam concrete bonding phenomenon and the estimation of its durability. It is set that the reason of early destruction of the reinforced foam concrete beam structures is the inefficient reinforcing bar and foam concrete bonding. The authors present a method of calculation of bonding stresses on the edge of bar contact with the array of foam concrete, which are defined from the condition of the accepted rectangular distribution diagram of stresses on the length of anchoring. Two types of diagrams are made for usage within the offered engineering method of calculation of bonding strength of both smooth and ribbed reinforcement with foam concrete of different density. The way of improvement in safety of the reinforced foam concrete constructions is specified through the use of anchors.

KEYWORDS: foam concrete constructions, reinforcement bonding, bonding strength, anchoring.

1. INTRODUCTION

The well-grounded study and production of economic and heat-retaining building materials (e.g. non-autoclaved foam concrete) and constructions became the crucial question in the conditions of modern world economic crisis. The public policy of energy savings development is aimed at the decrease of energy resources consumption, the reduction of gross domestic product power-consuming and the raise of national production competitive strength by means of backing up of efficient building materials production. In order to realise this policy there was approved The Programme of cellular concrete production development and their usage in construction for years 2005-2011.

A foam concrete has good heat and sound insulating properties, it's much cheaper than a heavy-weight concrete and it has enough compressive strength. But there is one problem while using it as a structural material. It can't bear tensile load. Such problem in heavy-weight concrete is solved by properly installed steel reinforcement and that problem is studied widely. It's logically to assume that we can use similar solution in our case of foam concrete usage. Such composite material was necessary for builders and its appearance was caused by

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this necessity. The studies in the field of reinforced cellular concrete were also held, but we can't consider them to be exhausting. The researches of Dobrynin E. [1], Trambovets`kyi V. [2], Mileykovska K., Levin N., Makarychev V. [3] were fragmentary and the phenomenon of bonding was not fully examined.

2. THE REINFORCED FOAM CONCRETE BEAM TESTS

The modern investigations of reinforced non-autoclaved foam concrete structures started at Lviv Polytechnic National University in year 2005. They have been holding till now but their beginning was caused by the test of beam structures [4]. The test procedure consisted of manufacturing and examination of eight experimental beam specimens. The experimental foam concrete beams were tested by one span hinged beam scheme with one hinged (immovable) support and one roller support. The concentrated loads were put on one third of the span to create a pure bending zone. The loading of specimens was held in steps with 7-10 minutes intervals on each stage. There were carried out thorough visual inspection of all beam edges with the purpose of crack detection on each stage. There were also measured bending flexure of the beams and deformations of the foam concrete.

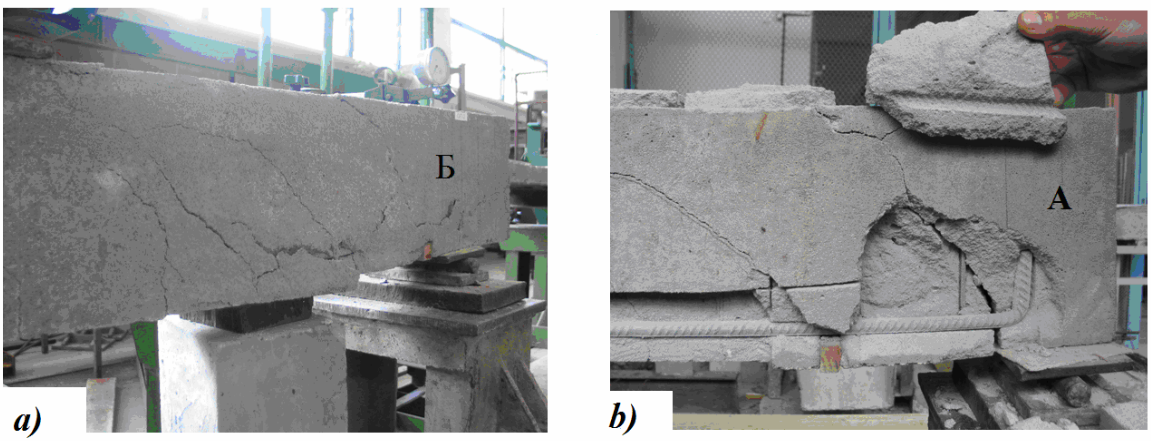

Fig. 1. The B-1 reinforced foam concrete beam:
a) the crack propagation near support Б;
b) the foam concrete and reinforcement bonding loss.

The observations and tests analysis of all experimental beams showed that crack initiation began on the firsth—second stage of loading by the occurrence of horizontal and inclined cracks near supports. The process of cracking was often accompanied by sound effects. Cracks were spreading instantly at the increase of loading.

The conducted researches resulted in such conclusion: as the character of destruction (example of B-1 beam, showed on Fig. 1., is characteristic for all beams) and relative deformation diagrams testify, destruction of test specimens took place as a result of insufficient foam concrete and reinforcement bonding.

3. EXPERIMENTAL PART OF BONDING INVESTIGATION

The reinforced foam concrete beam testing results compelled the authors to consider a steel bar and foam concrete bonding phenomenon in detail, to investigate the features of transmission of tensions between these materials. During 2005-2009 authors' team studied different kinds of reinforcement and foam concretes. The main attention was concentrated on heat-insulating structural non-autoclaved foam concrete, its density varied within the limits of 600-1000 kg/m³. Besides density the cube compressive strength of foam concrete was considered as another of its base characteristics. The cube strength determination was conducted on 15 cm cube specimens made simultaneously with basic specimens from the identical batches. There was applied conversion factor 0.95 while using 10 cm cubes (according to Notice 2 Tab. 5 GOST 10180-90 [5]). The character of cubes destruction is shown on Fig. 2. During investigation the cube strength of foam concrete varied in such limits: 0.673—1.6 MPa.

Fig. 2. The character of specimen’s destruction for determination of foam concrete cube strength.

Current investigation of bonding used foam concrete prism specimens with imbedded steel bar; the test specimens were molded in steel form with 15x15 cm cross-section for ribbed reinforcement and 10x10 cm for the usage of smooth bars; the load was put to the unrestrained bar end besides the end of foam concrete prim and pulled out this bar from the test specimen. Investigated length of anchoring varied from 10 to 60 centimeters.

The authors focused their attention on two types of the reinforcing bars: 1) smooth bars 3, 4 and 10 millimeters in diameter; 2) ribbed bars with 8 and 10 millimeters in nominal diameter with traditional spiral cross-section accordingly to GOST 5781-82* and crescents-shaped cross-section accordingly to DSTU 3760:2006. During investigation there were applied three schemes of test execution. They are shown on Fig. 3.

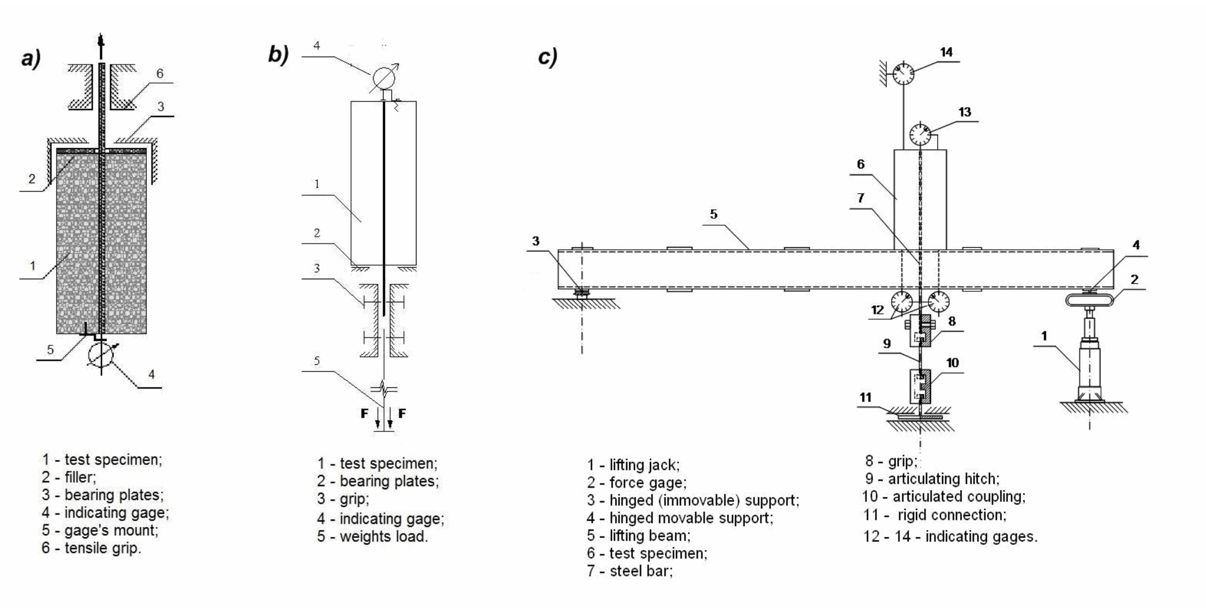

Fig. 3. The schemes of test execution:
a) on tearing machine; b) with manual loading; c) on a lifting beam.

At the realization of author's experimental researches there were used different devices and instruments, as specifies the marks on Fig. 3. The methods and consistency of performed tests are presented in previous author's publications [6,7].

In materials of this article the results of 75 experimental pull-out take place.

4. MODELLING AND ANALYSIS OF OBTAINED RESULTS

The transmission of stresses from reinforcing bar to foam concrete occurs as a result of three phenomena: 1) mainly, due to the bearing of foam concrete by the ribs on reinforcement, 2) friction of steel and foam concrete created by squeezing of foam concrete after its shrinkage and 3) adhesion of foam concrete and reinforcing steel. All these interactions between bars and concrete result in origination of bonding stresses in our case. We propose to evaluate them in the following way.

The reinforcing bar is modeled as a cylinder bar is it smooth or ribbed. Created forces are replaced by tensions on the nominal surface of contact with the foam concrete array.

In the scientific literature the tangent component of these stresses is traditionally termed the relative "bonding stresses". After the investigations of steel-to-heavy-weight-concrete bonding there was determined a diagram form of bonding stresses distribution. Fig. 4. shows these diagrams. It's inconveniently to use such curve of tensions distribution in the engineering calculation of necessary length of anchoring because it'll use integration during process.

We propose to substitute the real bonding stresses distribution diagram for a rectangular one as it's made in the case of heavy-weight concrete usage. The magnitude of averaged bonding stresses changes depending on considered length of anchoring zone. The aforesaid simplification also: 1) facilitates the testing performance by excluding the necessity of a steel bar straining measurement on the edge of contact (mentioned measurement is necessary while using a real bonding stresses distribution diagram) and 2) do not create the visible damage to the reliability of conducted calculations.

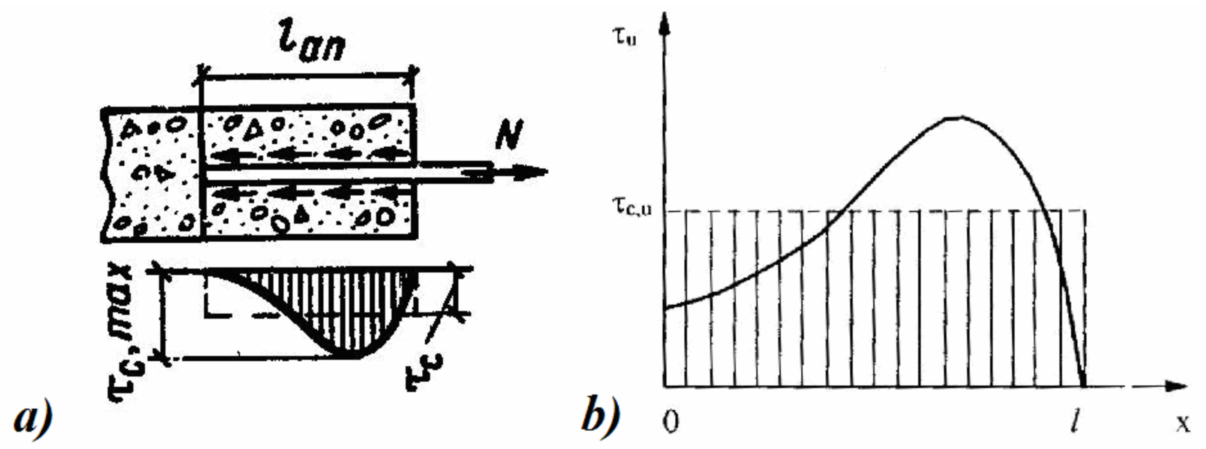

Fig. 4. The transmission of bonding stresses:
a) the relative bonding stresses by Baykov V. and Sygalov E [8],
b) bonding stresses by Shmukler V. [9];

The normal component of bonding stresses (commonly termed in scientific literature as the "transversal pressure") creates a sufficient thrust effect for destruction of specimens, as we can see on Fig. 5. Displayed of Fig. 5. a) and Fig. 5. b) cracks appeared in the 50cm and 60cm test specimens with 3cm foam concrete protective cover.

Fig. 5. The view on results of thrust effect in action:
a, b) specimens splitting under the action of transversal pressure.

We may notice similar splitting also on Fig. 1. a). The longitudinal cracks which appear as a result of foam concrete and steel bonding destruction provoke inclined cracks propagation near supports and early destruction of beams.

The beginning of cracking is to be observed at 0,1—0,2 mm slipping of the bars in foam concrete array and happens only with the usage of ribbed reinforcement. This fact also attracts attention to the imperfection of proposed minimal thickness of protective cover which is set at 25 mm by the paragraph 5.6 of current building code [10]. We suggest that this statement of our national building code should be supplemented with the statement of obligatory valuation of protective cover crack-resistance in case of ribbed reinforcement usage.

5. SUGGESTIONS ON ENGINEERING METHODS OF REINFORCEMENT BONDING STRENGTH VALUATION

In the foundations of reinforcement bonding strength valuation we lay the equation which connects averaged bonding stresses with tension load N and the area of contact edge $l_{an}\pi d$, where d – a diameter of the bar:

$$\overline{f_{an}} = N / l_{an}\pi d \tag{1}$$

The numeric data, obtained after testing, and connecting the length of anchorage zone and the relevant magnitude of bonding stresses at the destruction point are placed on two coordinate planes for two considered cases respectively. Fig. 6. shows the model of nomogram in case of ribbed reinforcement usage.

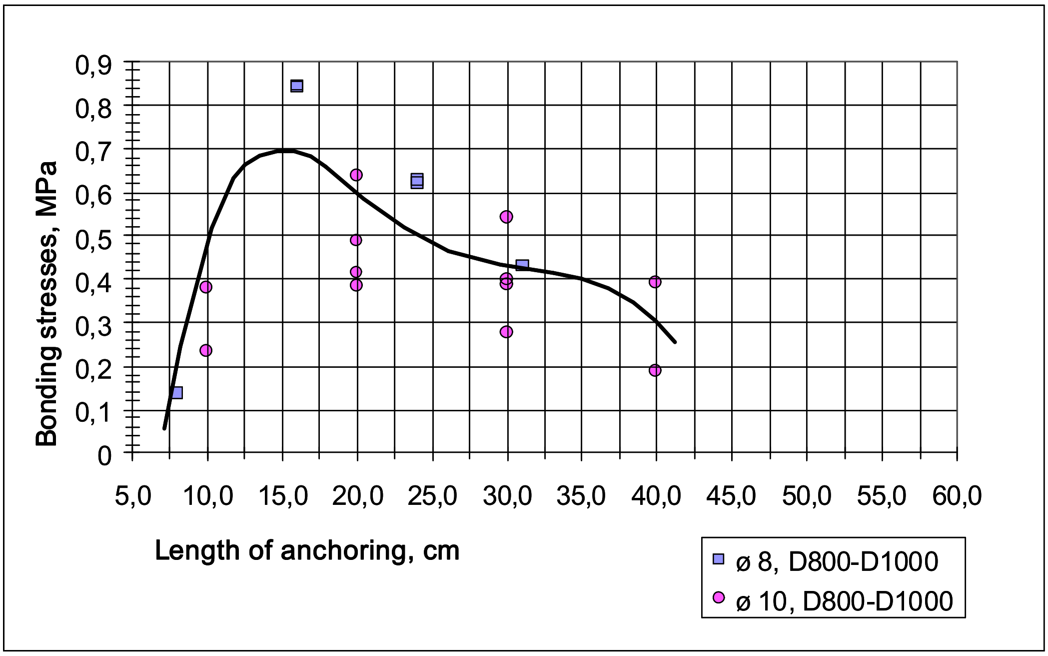

Fig. 6. The diagram for bonding stresses evaluation in case of ribbed reinforcement usage in grade D800-D1000 foam concrete.

The polynomial trendline showed on Fig. 6. is obtained indirectly by summarization of all testing results. Its curve resembles the real bonding stresses distribution diagram showed on Fig. 4. This fact confirms the similarity of mechanism of bonding stresses transmission in foam concrete and heavy-weight concrete and additionally justifies the appropriateness of rectangular stresses distribution diagram usage during the test execution.

In design calculation by the usage of diagram on Fig. 6. and Eqn. 1. we can determine the value of tensile load N which the foam concrete of given density can take up by bonding with the bar of given diameter d on the testing length of anchoring l_{an}.

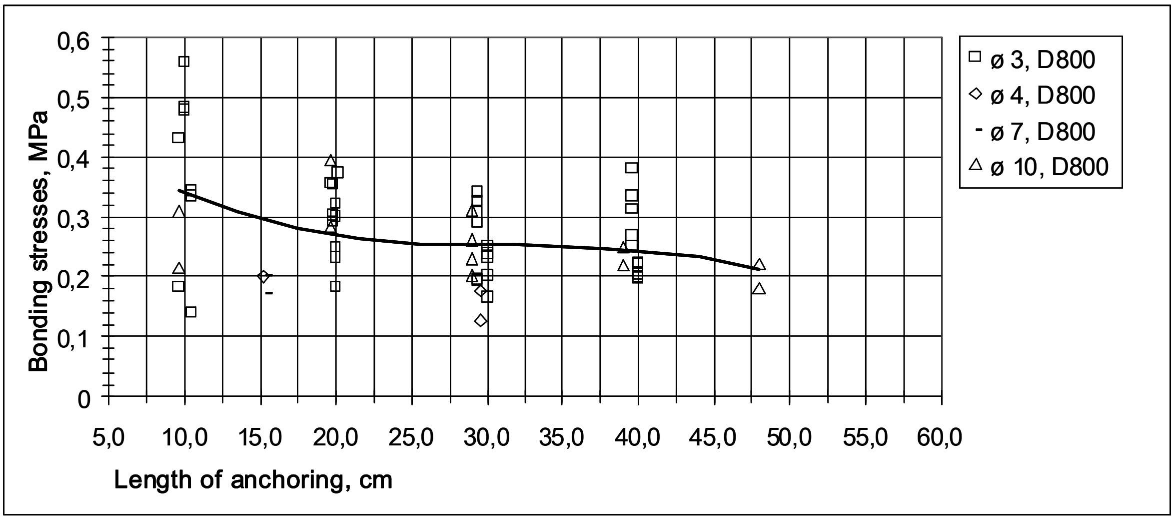

Fig. 7. The diagram for bonding stresses evaluation in case of smooth reinforcement usage in grade D800 foam concrete.

The diagram in case of smooth reinforcement usage seems to be like one, displayed on Fig. 7.

Presented on Fig. 7. curve shows that the mechanism of smooth reinforcement and foam concrete bonding differs from ribbed reinforcement and foam concrete bonding but the well-stratified empirical data after the 20 cm length of anchorage allows the usage of this diagram for preliminary estimation of bonding strength.

6. CONCLUSIONS

Performed analysis of foam concrete beam structures destruction and the results of own experimental research of foam concrete and reinforcement bonding allow the authors to draw a conclusion that the designing of reinforced foam concrete structural bearing members is possible. The main trouble spots of these structures are hidden in high danger of longitudinal cracks origination and the destruction of inclined cross-section after excessive crack opening. The cause of both these dangers lays on insufficient bonding between tension reinforcement and foam concrete near supports and slipping of the bars in bores. Presented authors' study answers to one of the main questions in reinforced foam concrete structures design which sounds like that: "What load can bear an anchored tension reinforcing bar near support in design inclined section"? In case when sufficient strength of anchorage can't be reached only by bonding there appears the necessity of anchors usage. The study of anchors work in foam concrete structures will be the next stage of our research.

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MODIFICATION OF EPOXY MORTARS BY A PET HYDROLYSATE

ABSTRACT

This paper describes a method of obtaining as well as selected properties of epoxy mortars. The addition of poly (ethylene terephtalate) hydrolysate (PET) as a modifier of epoxy resins (EP) resulted in an improvement in selected properties of the mortars in comparison with the properties of mortars based on unmodified resins. The application of PET waste materials in the modification of polymeric binders in mortars is also advantageous for economical and ecological reasons.

KEYWORDS: polymer mortars, epoxy resins, recycling, PET hydrolysate, chemical modification

1. INTRODUCTION

In recent years, significant progress has been observed in technical solutions in building and constructional materials. Attention is paid in order that these materials be both ecological and energy-efficient. Besides cement concretes, concretes modified with polymers are also used, in which both cement and polymer act as binders [1-3]. Also applied are resinous concretes, containing only a polymer binder [4-6].
Concretes modified with polymers are characterized by an increased mechanical strength in comparison with cement concretes, by increased adhesion to various bases and also by greater durability. The drawback of these materials is their higher cost of production.
At present, great interest has been noted in the application of recyclates and plastic wastes in the modification of concretes. Plastic recyclates can be also applied as additional fillers in cement concretes [7-12] and in the production of new polymer binders for concretes [13-16].

There exists the problem of the utilization of wastes from poly (ethylene terephtalate) - the commonly-known polyester applied in, for example, the production of drinks packaging [17]. PET hydrolysis products are used in the early stages of the manufacture of polyurethane, unsaturated polyesters, and also in the modification of epoxy and phenol resins [18-23].

The aim of this paper was the modification of epoxy resin by PET hydrolysate, its use as a binder in resinous concrete, and the determination of selected properties of that concrete.

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2. EXPERIMENTAL PROCEDURE

2.1. Materials

1) Epidian 5 - epoxy resin manufactured by 'Nowa Sarzyna' Chemical Plant, Poland (Table 1)
2) PET hydrolysate (in dipropylene glycol), obtained in a laboratory (Table 2), identified by GPC chromatography FT-IR and NMR spectra (Figs. 2-4).
3) Triethylenetetraamine (TETA) from 'Nowa Sarzyna' Chemical Plant, Poland.
4) Standard sand with granularity as shown in Fig.1, containing at least 98% SiO_2.

In all compositions, the ratio of sand to the resinous binder (EP + PET + TETA) was 4:1, and the ratio of hardener to the pure epoxy resin was 1:10.

Table 1. Properties of epoxy resin

Name of resin	Density [g/cm^3]	Viscosity (25^{o}C) [mPa s]	Molecular weight [g/mol]	Epoxy number LE [val/100g]
Epidian 5	1,17	30000	450	0,49

Table 2. Properties of PET hydrolysate

Density (23^{o}C) [g/cm^3]	Melting temperature [^{o}C]	Form	Hydroxyl number [mgKOH/g]	Molecular weight	
				$\overline{M}_n$ [g/mol]	$\overline{M}_w$ [g/mol]
1,30	78 ÷ 82	semi-solid wax	515	404	849

Table 3. Mixing proportions of mortar compositions

Test series	EP resin : PET (m/m)
EP + PET 0%	100:0
EP + PET 5%*	95:5
EP + PET 10%	90:10
EP + PET 15%	85:15

*The abbreviation 'EP + PET 5%' indicates mortar containing 95% EP and 5% PET modifier. Mortar samples were made using standard sand of grain composition as presented in Fig. 1.

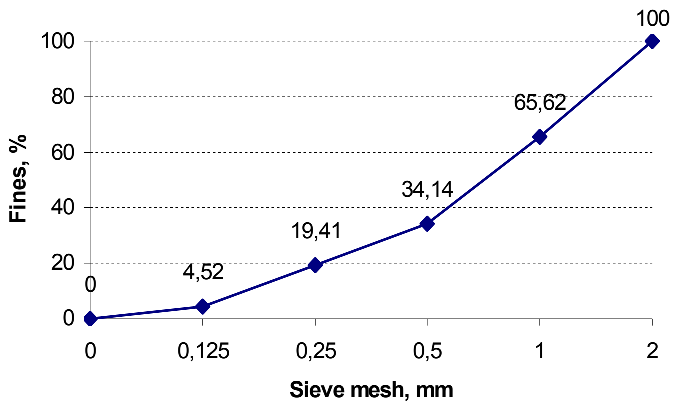


Fig.1. Granulation of standard sand

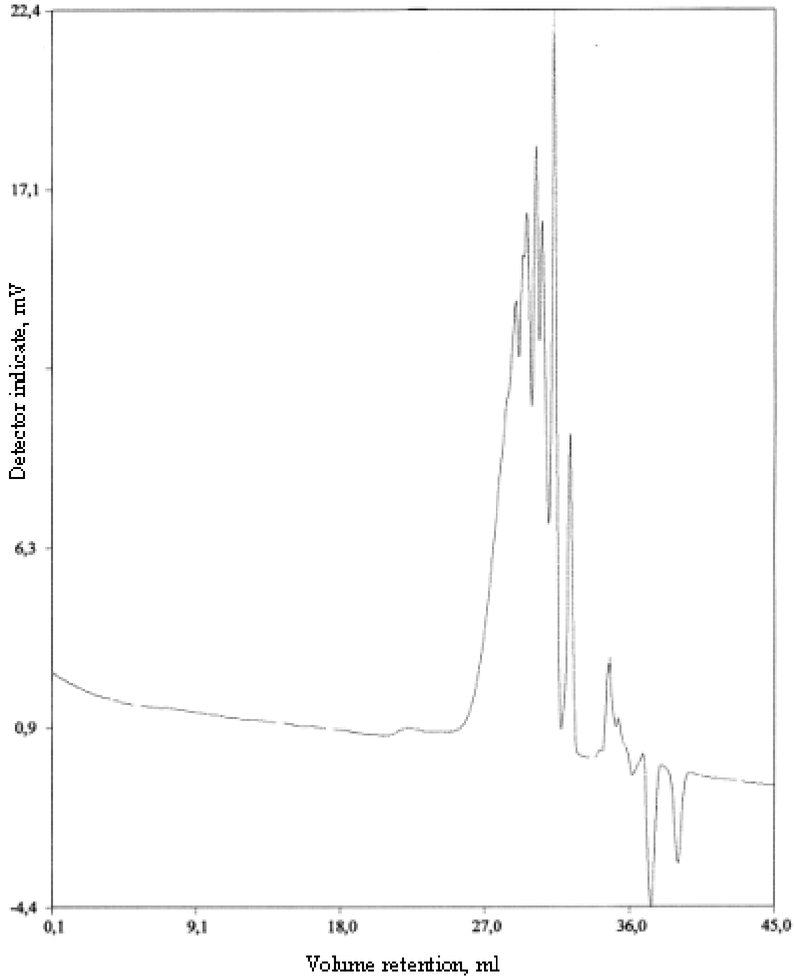


Fig. 2. GPC chromatogram of PET hydrolysate

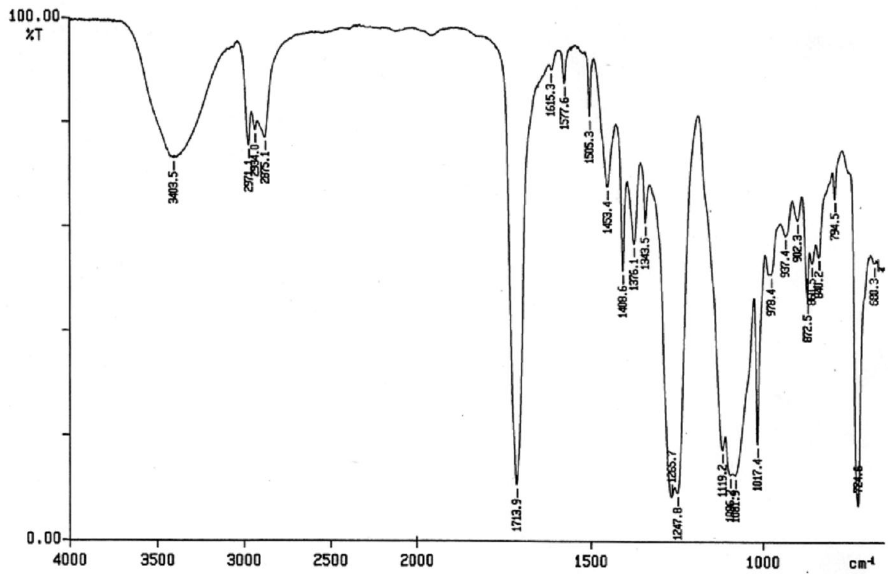


Fig. 3. FT – IR spectrum of PET hydrolysate in dipropylene glycol

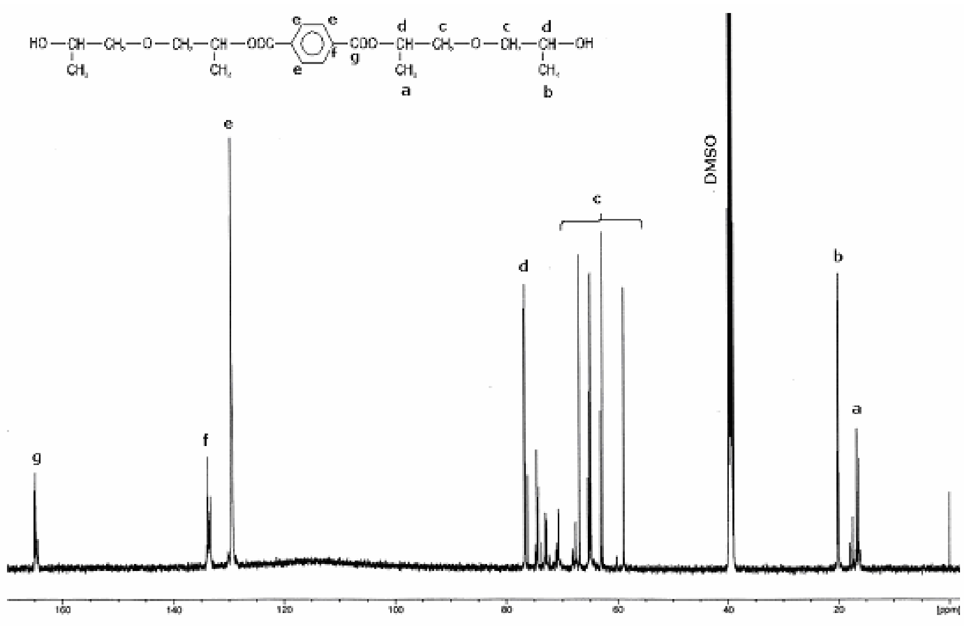


Fig. 4. C^{13}NMR spectrum of PET hydrolysate

2.2. Specimen preparation

Appropriate amounts of the epoxy resin and the modifier were weighed in a beaker with an accuracy of ± 0.01 g and were mixed with a glass rod for uniformity.

After mixing, the constituent ingredients were heated for 60 minutes at a temperature of 80 ± 1 ^{o}C in order to allow the reaction of the functional groups of both components; later, these were cooled in desiccator.

The mortar samples were prepared using a laboratory mixer. The resinous compositions prepared earlier were mixed with standard sand maintaining the same mixing time and rotation speed of the mixer.

The finished mortar was placed in steel forms of dimensions 40x40x160mm. Specimens were prepared for the investigation of mechanical properties, water absorption and chemical resistance. These specimens were left for 7 days at room temperature for hardening and seasoning.

2.3. Characterization of Measurements

The hardened samples were subjected to investigation of flexural and compressive strength according to standard PN-B-04500: 1985. The results are presented in Figs. 5 and 6. The samples were also examined for hardness with a Brinell hardness tester, in accordance with standard PN-EN ISO 2039-1: 2004 (Fig.7).

The next investigation was the testing of water absorption of the obtained hardened mortars. This was tested over 7 days at a temperature of 23 ± 1°C in accordance with PN-EN 206-1: 2003. The results are presented in Fig. 9.

The specimens were exposed, over a period of 14 days, to the following aggressive media:

- Nitric acid 10%
- Sulfuric acid 10%
- Hydrochloric acid 10%
- Sodium chloride 10%

- Ammonium hydroxide 10%
- Sodium hydroxide 1%

The investigation was conducted in accordance with standard PN-EN ISO 175: 2002; the results obtained are shown in Fig.10.

In order to determine the structure of the obtained mortars, an investigation was performed with a Joel SEM 5500 - LV scanning microscope. The structures of mortars with and without the addition 10% PET hydrolysate modifier are depicted in Figs. 11 and 12.

3. RESULTS AND DISCUSION

The results of investigations of mechanical properties are presented in Figs. 5-7.

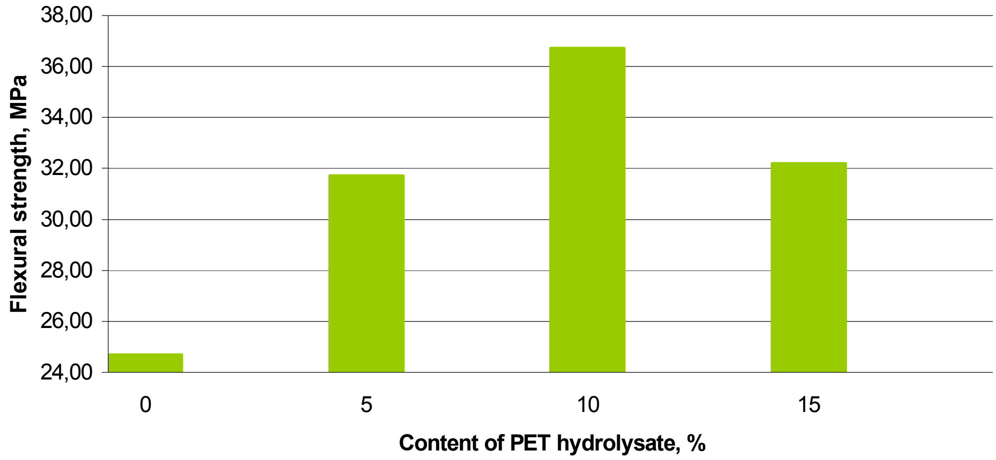


Fig. 5. Flexural strength of epoxy mortars versus modifier content

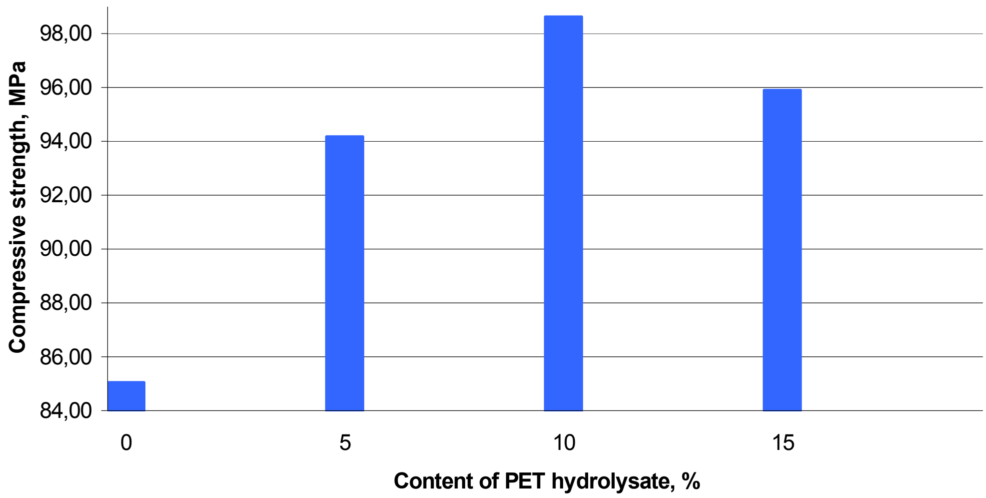


Fig. 6. Compressive strength of epoxy mortars versus modifier content

Fig. 5 depicts the influence of PET hydrolysate on the flexural strength of modified epoxy mortars in comparison with unmodified mortars.

Modified mortars have a flexural strength in the range of 32-37 MPa - this is 33-51 % greater than for the unmodified mortar.

The compressive strength of modified mortars is similarly increased; 96-99 MPa for modified mortars compared with 85 MPa for the unmodified mortar (Fig. 6). The greatest increase in strength is achieved with the addition of 10% PET hydrolysate.

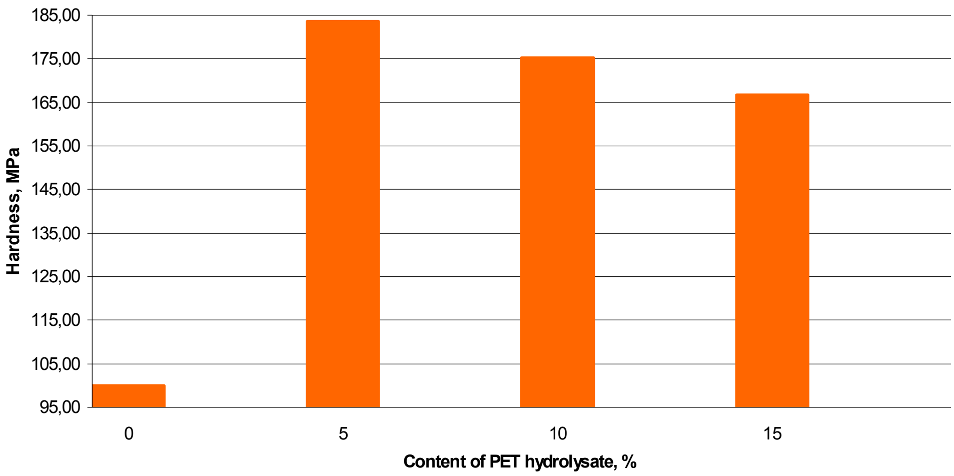
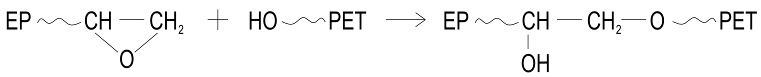


Fig. 7. Hardness of epoxy mortars versus modifier content

The influence of the addition of PET hydrolysate on the hardness of epoxy mortars was also examined. Greater values of hardness were observed for modified mortars in comparison to those unmodified (Fig. 6). Maximum hardness (184 MPa) was achieved in the sample with a modifier content equal to 5%. A greater content of modifier resulted in a small reduction in hardness.

The advantageous influence of PET hydrolysate addition on mortar properties may be related to the chemical reaction between the functional groups of the epoxy resin and PET modifier during heating at 80 ± 1°C. It may be depicted by the following equation:

(1)



The possibility of such a reaction is confirmed by analysis of FT-IR spectra for the EP + PET 10% composition (Fig. 8).

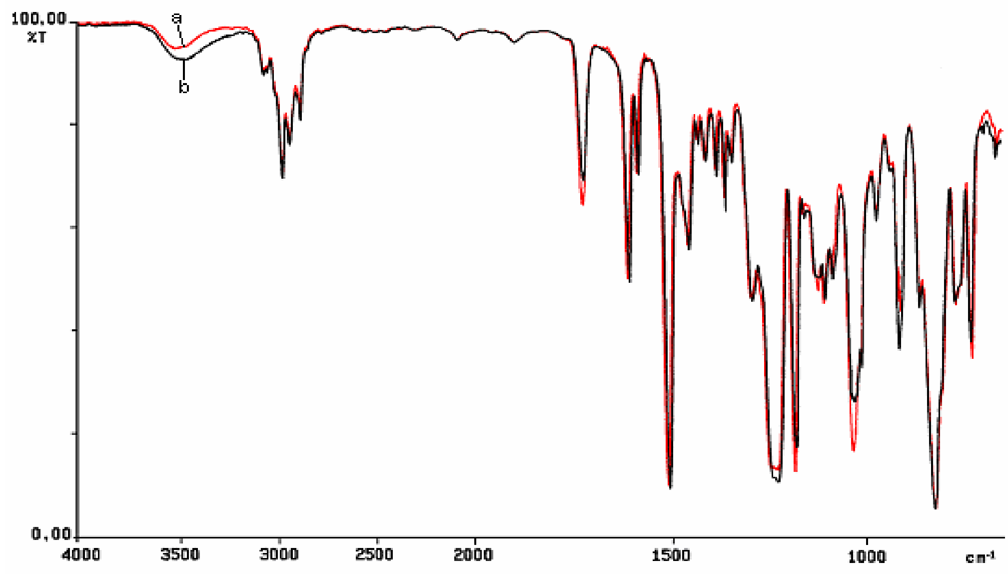


Fig. 8. FT-IR spectrum of EP + PET 10% composition:
a- composition after 2hrs heating at 80 ± 1 ºC;
b - composition without heating.

In Fig. 8, reductions in intensity may be observed in the 930 cm^{-1} band characteristic of the epoxy group and in the 1030 cm^{-1} band specific to the $-CH_2OH$ group in the PET hydrolysate. Simultaneously observed is an increase in intensity of the 1181 cm^{-1} band specific to the etheric group ($-CH_2-O-CH_2-$) and in the 1030 cm^{-1} band characteristic of the second order -OH groups.

The investigation of water absorbability indicated that this is relatively small - just over 0.18% (Fig. 9). All compositions exhibit an increase in water absorbability during the first 3-4 days, whilst later it remains constant.

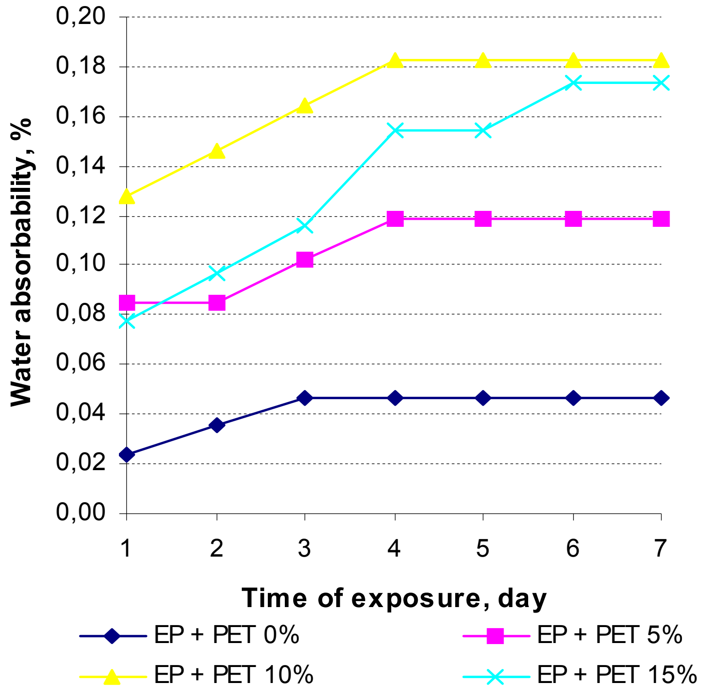


Fig. 9. Water absorbability of epoxy mortar samples

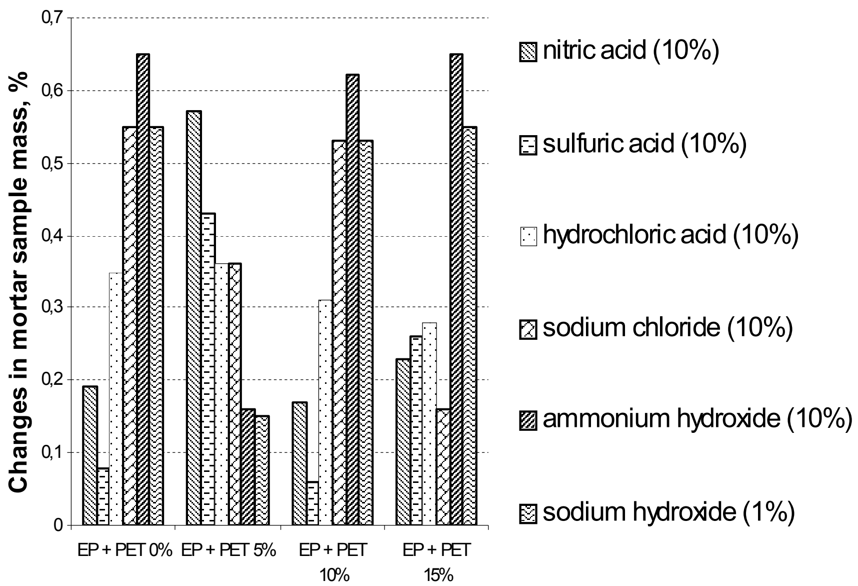


Fig. 10. Changes in mortar sample mass after 14-day exposure to selected aggressive media at 23 ± 1°C

The influence of six aggressive media on mortar samples was also examined. The percentage mass changes are depicted in Fig. 10.

All samples displayed a very good chemical resistance. The smallest changes in mass were observed for samples immersed in 10% hydrochloric acid (Fig. 10) - these mass changes did not exceed 0.4%. Very good results were also obtained for samples exposed to 10% sulfuric acid (Fig. 10) and 1% sodium hydroxide (Fig. 10).

In order to determine the microstructure of the mortars, photographs were taken of sample fractures with a differential scanning microscope at 250x magnification.

Analysis of the photographs (Figs. 11 and 12) clearly indicates filler grains of varying granularity and a continuous phase of polymer matrix. Also visible are small quantities of air bubbles. It appears that a polymer matrix with a modified epoxy resin is smoother and more homogeneous than one with an unmodified resin.

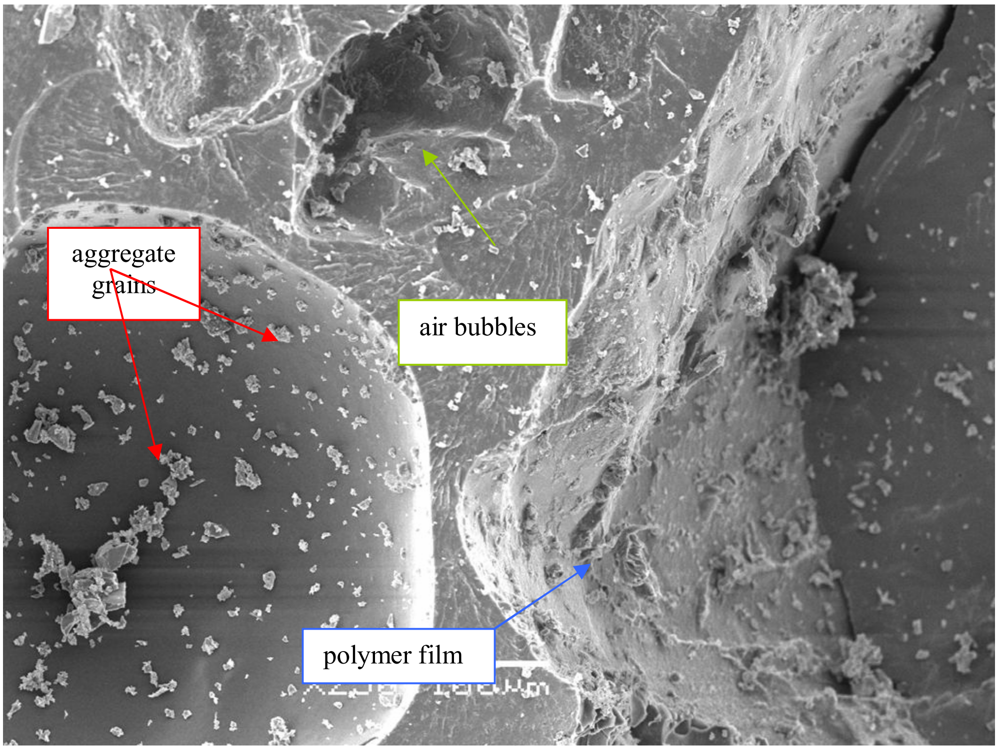


Fig. 11. SEM observations of EP + Pet 0% mortar sample (250x, marker = 100 μm)

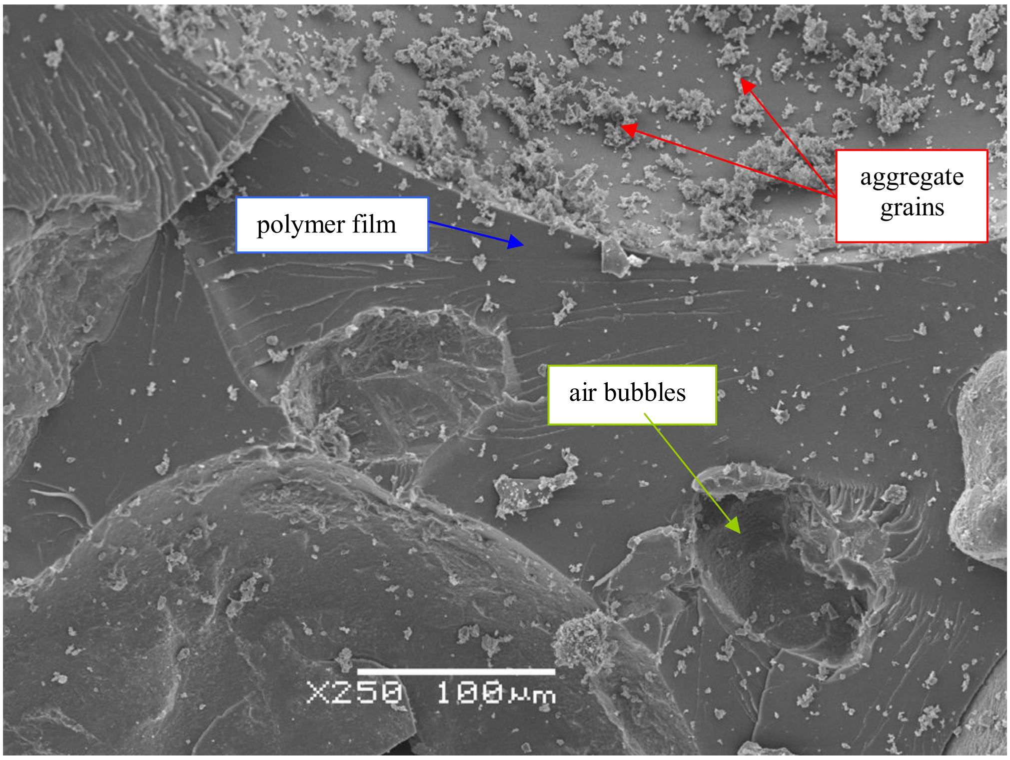


Fig. 12. SEM observations of EP + PET 10% mortar sample (250x, marker = 100 µm)

4. CONCLUSIONS

Based on the investigations performed into the mechanical, physical and chemical proprieties of the obtained modified epoxy mortars, the following conclusions may be drawn:

- Mortars based on modified epoxy resin demonstrate a greater compressive and flexural strength in comparison to mortars based on unmodified resin.
- Modified epoxy mortars are characterized by a higher hardness than unmodified mortars.
- Epoxy mortars modified by PET hydrolysate have a low water absorbability (in the range of 0.08-0.18%) and a very good chemical resistance to selected corrosive media.
- PET hydrolysate may be applied as a modifier of epoxy resins used as binders in polymer concretes. The modifier improves selected properties of the concretes and at the same time reduces their cost of manufacture.
- The application of PET hydrolysate in polymer mortars is a partial solution to the problem of the utilization of wastes of this plastic material.

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
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PROBABILISTIC MODELING OF TIMBER MEMBERS

ABSTRACT

Safety assessments of structures built in the past require taking into consideration many different parameters. These parameters are mostly random and describe materials characteristics, actions and history of actions. The load bearing capacity of timber structures decreases with time. It depends on the type of load and timber. Strength reduction effects, referred to as creep-rupture effects, due to long term loading at high stress ratio levels are known for many materials. Timber materials are highly affected by this reduction in strength with duration of load. Characteristic values of load duration and load duration factors will be calibrated by means of using probabilistic methods. Three damage accumulation models will be considered, that is Gerhards model, Barret & Foschi's and Foshi & Yao's model. The reliability will be estimated by means of using representative short- and long-term limit states. Time variant reliability aspects will be taken into account using a simple representative limit state with time variant strength and simulation of whole life time load processes. Non-destructive laboratory tests of timber strength will be applied to update the level of safety. The Bayes theory and methodology will be applied to calculate the aposteriori level of timber members parameters and safety.

KEYWORDS: timber structures, creep rupture effects, safety factors, reliability.

1. INTRODUCTION

During the last decades structural reliability methods have been further developed, refined and adopted and are now at a stage where they are being applied in practical engineering problems. Furthermore, basic knowledge concerning the actions on structures and timber material characteristics has improved due to increased focus, better measuring techniques and international research co-operation. This knowledge has now reached a level where it enables designers to take into account uncertainties in material properties and actions in assessing the load carrying capacity, serviceability and service life of timber structures and connections. Most building codes national and international, are based on a probabilistic safety approach. The code formats are deterministic with connections to reliability design achieved through failure probability, partial safety factors and characteristic values. Partial

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safety factors are calibrated for standard cases against probabilistic analyses for similar cases. The condition for calibration that is the probabilistic analysis and deterministic code should be fulfilled the same safety requirements. Hence, the required safety is usually not accomplished by using probabilistic theories in everyday designing process.

Timber is rather complex building material. The timber material characteristics depend on the specific wood species, the geographical location where the wood has been grown and furthermore on the local growing conditions. Timber is an orthotropic material and it consists of high strength grains which are predominantly orientated along the longitudinal axis of a tree. Material characteristics – the ultimate bending stress and the bending stiffness depend on the orientation of the moment axis to the grain direction. Irregularities in regard to grain direction, knots, fissures become highly decisive for load bearing capacity of a timber structural element. Only in very small test specimens it is possible to avoid these irregularities – with so-called clear wood specimen it is possible to investigate the orthotropic material properties of timber.

Timber materials are highly affected by reduction in strength with duration of load in time. Therefore design of timber structures utilizes a strength reduction factor to reduce characteristic short-term strength. The Eurocode for timber structures EC 5 [1] refers to this strength modification with a load duration factor, k_{mod}. Traditionally, the load duration factor is determined empirically by experience on timber structures. But there are probabilistic methods connected with damage accumulation models to estimate factor k_{mod} as well [2], [3], [4], [5]. The load duration factor k_{mod} is defined in EC 5 as a factor which takes into account the effects of load duration and ambient climate on the strength parameters of structural timber members. Three damage accumulation models are taken under consideration, namely the models by Gerhards , Barrett & Foschi and Foschi & Yao.

The mechanism leading to the reduction in strength of a timber member under sustained load is a creep rupture. This could arise from propagation of voids in the microstructure of the timber at a stress level lower than the short-term strength. A number of models of creep rupture, involving a damage state variable (similar to that used in the analysis of metal fatigue), have been proposed to assess damage accumulation in wood structural members subject to loading histories, typically modeled as renewal pulse process. The basic model has the following form :

$$d\alpha/dt = F[\sigma(t)] \tag{1}$$

where t – time, α - the damage state variable which ranges from 0 (no damage) to 1 (failure),the function F(.) has two constants that must be determined from test data, and $\sigma(t)$ – the ratio of the applied stress to the failure stress under short-term ramp loading.

2. MATERIAL PROPERTIES

2.1 Basic properties

The reference properties of structural timber are:

- the bending strength $f_{\mathrm{m,k}}$ in [MPa]
- bending modulus of elasticity $E_{0,\mathrm{mean}}$ in [MPa],both measured on short–term standard test specimens described in EC 5 [1]
- timber density ρ_k [kg/m^3],

measured according to EC 5 [1].

Relationship between other timber properties and reference properties are shown in table 1. The reference material properties are sensitive to the deviations from the standard test conditions of cross section in situ (i.e. at any generic point in time and in space) can be estimated as:

Bending strength in situ, $f_{m,\alpha}$:

$$f_{m,\alpha} = \alpha_s(H(s,\omega,\tau,T))\, f_{m,k}. \tag{2}$$

Bending modulus of elasticity in situ $E_{0,\alpha}$:

$$E_{0,\alpha} = E_{0,mean} \;/(1 + \delta_s\,(H(s,\omega,\tau,T))) \tag{3}$$

Timber density in situ $\rho_{k,\alpha}$:

$$\rho_{k,\alpha} = \rho_k\,. \tag{4}$$

Where $H(s,\omega,\tau,T)$ is the exposure of structure to loads s, humidity ω and temperature τ, in the time [0,T]. $\alpha_s(H(.))$, δ_s (H(.)) are a strength and modulus of elasticity modification functions ,in general defined for a particular set of exposures. Other material properties are estimated based on reference material properties. Expressions for the expected values *Ex*[.] and coefficient of variation COV[.] are given in Table 1.

Relations are derived for standard test specimen properties tested under reference conditions. However, it is assumed that the relations can be used at any level, i.e. for components of any size and for other climate and load conditions.

Table 1. Relation reference properties – other properties

Property	Expected value *Ex*[X]	Coeficient of variation COV[X]
Tension strength parallel the grain $f_{t,0,k}$	$Ex[f_{t,0,k}] = Ex[f_{m,k}]$	$COV[f_{t,0,k}] = 1.2COV[f_{m,k}]$
Tension strength perpendicular to the grain $f_{t,90,k}$	$Ex[f_{t,90,k}] = 0.015Ex[f_{m,k}]$	$COV[f_{t,90,k}] = 2.5COV[f_{m,k}]$
Modulus of elasticity parallel to the grain $E_{0.05}$	$Ex[E_{0.05}] = Ex[E_{0,mean}]$	$COV[E_{0.05}] = COV[E_{0,mean}]$
Modulus of elasticity perpendicular to the grain $E_{90,mean}$	$Ex[E_{90,mean}] = Ex[E_{0,mean}]/30$	$COV[E_{90,mean}] = COV[E_{0,mean}]$
Compression strength parallel to the grain $f_{c,0,k}$	$Ex[f_{c,0,k}] = 5Ex[f_{m,k}]^{0.45}$	$COV[f_{c,0,k}] = 0.8COV[f_{m,k}]$
Compression strength perpendicular to the grain $f_{c,90,k}$	$Ex[f_{c,90,k}] = 0.007Ex[\rho_k]$	$COV[f_{c,90,k}] = COV[\rho_k]$
Shear strength $f_{v,k}$	$Ex[f_{v,k}] = 0.2Ex[f_{m,k}]^{0.8}$	$COV[f_{v,k}] = COV[f_{m,k}]$
Shear modulus G_{mean}	$Ex[G_{mean}] = Ex[E_{0,mean}]/16$	$COV[G_{mean}] = COV[E_{0,mean}]$

2.2 Probabilistic distributions of basic variables

The distribution type and recommended coefficient of variation COV of the basic material properties are given in table s 2 and 3.Indicative values of the correlation coefficient matrix are shown in table 5.

The distribution parameters can be determined with the information given in Tables 1 and 2. If additional information about the material property of interest of the considered population becomes available, e.g. inform of test data, this information should be integrated using a Bayesian updating.

Table 2. Probabilistic models for reference properties

Property	Distribution	COV
Bending strength $f_{m,k}$	Lognormal	0.25
bending modulus of elasticity $E_{0,mean}$	Lognormal	0.13
timber density ρ_k	Normal	0.10

Table 3. Probabilistic models for other material properties

Property	Distribution
Tension strength parallel the grain $f_{t,0,k}$	Lognormal
Tension strength perpendicular to the grain $f_{t,90,k}$	Weibull
Modulus of elasticity parallel to the grain $E_{0.05}$	Lognormal
Modulus of elasticity perpendicular to the grain $E_{90,mean}$	Lognormal
Compression strength parallel to the grain $f_{c,0,k}$	Lognormal
Compression strength perpendicular to the grain $f_{c,90,k}$	Normal
Shear strength $f_{v,k}$	Lognormal
Shear modulus G_{mean}	Lognormal

Table 4. Correlation coefficient matrix

	$E_{0,mean}$	ρ_k	$f_{t,0,k}$	$f_{t,90,k}$	$E_{0.05}$	$E_{90,mean}$	$f_{c,0,k}$	$f_{c,90,k}$	G_{mean}	$f_{v,k}$
$f_{m,k}$	0.8	0.6	0.8	0.4	0.6	0.6	0.8	0.6	0.4	0.4
$E_{0,mean}$		0.6	0.6	0.4	0.8	0.4	0.6	0.4	0.6	0.4
ρ_k			0.4	0.4	0.6	0.6	0.8	0.8	0.6	0.6
$f_{t,0,k}$				0.2	0.8	0.2	0.5	0.4	0.4	0.6
$f_{t,90,k}$					0.4	0.4	0.2	0.4	0.4	0.6
$E_{0.05}$						0.4	0.4	0.4	0.6	0.4
$E_{90,mean}$							0.6	0.2	0.6	0.6
$f_{c,0,k}$								0.6	0.4	0.4
$f_{c,90,k}$									0.4	0.4
G_{mean}										0.6

The values in Table 4 are quantified by judgment -COST E24 [6], such that 0.8 – high correlation, 0.6 – medium correlation, 0.4 low correlation, 0.2 – very low correlation

2.3 Damage models

Damage models are used to mathematical describes the long term strength reduction as a function of stress level and duration of loading. In this paper tree damage models are fitted against data obtained on Nordic structural timber subjected to constant loading. The characteristics of the three damage models are that α is defined as the degree of damage, i.e. $\alpha = 0$ stands for no damage and $\alpha = 1$ stands total damage or failure.

2.3.1 Gerhards model

The damage accumulation presented by Gerhard [7] is

$$\frac{d\alpha}{dt} = \exp(-A + B\frac{\sigma}{fo}) \quad . \tag{5}$$

Where A and B are constant, σ is the stress and fo is short term strength of member.
Solution of equation (5) is

$$\frac{\sigma}{fo} = \frac{A}{B} - \ln\frac{10}{B}\log t = a - b\log t\,, \tag{6}$$

where

$$a = \frac{A}{B} + \varepsilon \qquad b = \frac{\ln 10}{B}\,, \tag{7}$$

and ε models the model uncertainty related the model in (5). ε is assumed to be Normal distributed with expected value equal 0 and standard deviation μ.
Assuming constant load and considering f as the residual strength the solution to (6) is:

$$\frac{f}{fo} = \frac{1}{B}\ln(1 + (1-\alpha)(\exp B - 1))\,. \tag{8}$$

This expression is used when simulating the damage due to load duration.

2.3.2 Barret and Foshi's model

The damage accumulation model proposed by Barret and Foschi [8] has the following expression:

$$\begin{aligned} \frac{d\alpha}{dt} &= A(\frac{\sigma}{fo} - \eta)^B + C\alpha\,; \qquad \frac{\sigma}{fo} > \eta \\ \frac{d\alpha}{dt} &= 0\,; \qquad \frac{\sigma}{fo} \le \eta \end{aligned} \tag{9}$$

Where A , B and C are constants, σ is the stress, fo – short term strength and η is a threshold ratio.

Assuming constant load and $\alpha=1$, solution of equation (9) is [9]:

$$\frac{\sigma}{fo}=(\frac{A}{C}(\exp(Ct)-1)^{\frac{-1}{B}}+\eta=a(\exp(\exp(b)t)-1)^{c}+\eta, \tag{10}$$

where:

$$a=\exp(\ln((\frac{A}{C})^{\frac{-1}{B}})+\varepsilon); \qquad b=\ln C; \qquad c=-\frac{1}{B}, \tag{11}$$

ε - describes uncertainty of the model (9).

Supposing that f is residual strength:

$$\frac{f}{fo}=\eta+[(1-\alpha)(1-\eta)^{B}]^{\frac{1}{B}}. \tag{12}$$

2.3.3 Foschi and Yao's model

En extended version of equation (9) has been proposed by Foschi and Yao [10]:

$$\begin{aligned}&\frac{d\alpha}{dt}=A(\frac{\sigma}{fo}-\eta)^{B}+C(\frac{\sigma}{fo}-\eta)^{D}\alpha; \qquad \frac{\sigma}{fo}>\eta\\&\frac{d\alpha}{dt}=0; \qquad \frac{\sigma}{fo}\leq\eta\end{aligned} \tag{13}$$

so is initial short term strength, σ is the stress and η is threshold ratio. A, B,C and D are constants.

For short term load [11] and assuming that rate of loading is large and C small, solution of equation (13) is as follows:

$$A=\frac{k(B+1)}{fo(1-\eta)^{(B+1)}}, \tag{14}$$

$$t_f=\ln(\frac{\sigma}{k}+\frac{1}{C(\frac{\sigma}{k}-\eta)^{D}}\ln(\frac{1+\lambda}{\alpha_o+\lambda}))+\varepsilon, \tag{15}$$

t_f – time until failure.

$$a_o=(\frac{\frac{\sigma}{fo}-\eta}{1-\eta})^{B+1}; \qquad \lambda=\frac{k(B+1)}{Cfo(1-\eta)^{D}}(\frac{\sigma}{fo}-\eta)^{B-D}, \tag{16}$$

ε - presets uncertainty of the model (13).

Assuming that f is the residual strength we can write[12]:

$$\frac{f}{fo} = \eta + (1-\eta)(1-\alpha)^{\frac{1}{1+B}}. \tag{17}$$

3. CONCLUSIONS

Duration of load research has been briefly reviewed, with particular emphasis on the work used as the basis for the time effect factors in limit states conditions [13]. A simplified approach to cumulative damage analysis using order statistic was then presented. A number of approaches to analyzing the reliability of timber members including duration of load behavior were examined.

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ANALYSIS OF BONDED CONNECTION FOR HYBRID STEEL-GLASS BEAM

ABSTRACT

In the last few years, due to the intensive progress and research in the field of glass structures, mechanical properties of glass have been distinctively improved and possibilities of use glass for a load carrying elements are now advanced. Different types of hybrid constructions, consisting of glass and other material, are analyzed or even newly developed focusing on an optimal structural interaction between both materials. The new hybrid steel-glass beam consists of steel flanges, glass web and bonded connection between them. A key aspect of this development is the selection of the suitable adhesive. Different types of adhesive with a different mechanical and deformation properties were chosen for the instant small-scale tests. Clear idea of the behavior of the adhesive layer and knowledge of the shear and tension strength of whole connection, are the first steps, needed to be done to successful modeling and investigation of the whole hybrid steel-glass beam under increasing load.

KEYWORDS: hybrid, steel-glass beam, glued connection, adhesive, material properties, analytical model, FE modelling

1. INTRODUCTION

The aim of the current research is to develop and investigate the behavior of the new transparent structural element, in respect to architectural, static-structural and fabrication criteria, which shows a high stiffness, load carrying capacity and robustness in comparison with a pure glass beams or fins. Hybrid beams with glass web and flanges from different materials are currently the subject of the interest in a lot of research centres around the Europe [1], [2], [3]. This paper deals with new hybrid steel-glass beam which is consisted of glass web and steel flanges assembled together by the adhesive layer. These beams will be used mainly as members of a transparent roof structures or as a supporting fins for a large-area glass facades.

In glass structure engineering and design, in general, tension strength of the glass determines the load carrying capacity of whole element. Glass is a material, which shows brittle behaviour. Pure glass beams always fail suddenly and without extensive previous warning. Therefore, except the higher stiffness, load carrying capacity and robustness, one of

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the main advantages of hybrid beams are the possibility to achieve more ductile behaviour of whole system. Stiff member (flange) works as a consumer of the break energy even after the first cracks in the glass pane are visible. This phenomenon is called residual carrying capacity and it is required because of the safety. From previous investigations came out, that this residual carrying capacity depends on the type of used glass.

2. ANALYTICAL MODEL OF HYBRID BEAM

Well known solution by Möhler was applied to this problem to describe the behaviour and stress distribution along the cross section of the beam with semirigid shear connection between the web and flanges under increasing load. This method considers the adhesive as a linear elastic material, and therefore it can serve only as a simplified method for the hand calculation, or as a control device. Assume the simplest connection between steel and glass – direct connection, see Fig. 1.

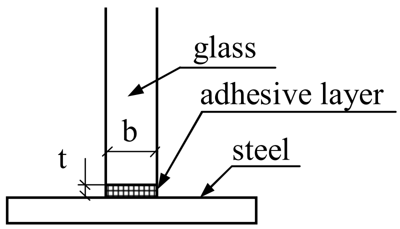

Fig. 1. The simplest glued steel-glass connection

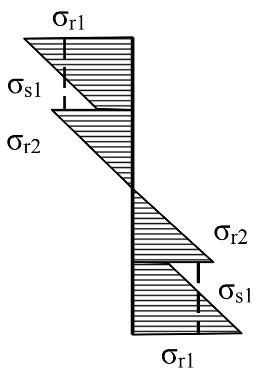

Fig. 2. Normal stress distribution along the cross section in accordance with Möhler theory

Than, average stiffness of the connection along its length K_K is given by the Eqn. 1, where $G_{K\,is}$ the shear modulus of the adhesive:

$$K_K = G_K \cdot \frac{b}{t} \,. \tag{1}$$

Effectivity factor γ and effective moment of inertia $I_{Y,eff}$ of double symmetric cross section are given by the following Eqn. 2 and 3:

$$\gamma = \frac{1}{1+k}, \tag{2}$$

$$I_{y,eff} = 2 \cdot I_S + n \cdot I_G + 2 \cdot \gamma \cdot A_S \cdot z_S^2 \,, \tag{3}$$

where

$$k = \pi^2 \cdot \frac{E_S \cdot A_S}{l^2 \cdot K_K} \,, \tag{4}$$

where E_S is the Young's modulus of steel [MPa], A_S is the area of one steel flange [mm^2], l is the span of the beam [mm], I_S is the moment of inertia of one steel flange [mm^4], I_G is the moment of inertia of the glass web [mm4], z_S is the vertical direction between the centre of gravity of the steel flange and centre of gravity of whole cross section [mm]. Factor n is equal to $n = Eg/Es$, where E_G is the Young's modulus of glass [MPa].

Values of the normal stress distribution along the cross section drawn in Fig. 2 can be obtained from following formulas:

$$\sigma_{r1} = \pm \frac{M_y}{I_{y,eff}} \cdot \left(\gamma \cdot z_S + \frac{t_f}{2} \right), \tag{5}$$

$$\sigma_{s1} = \pm \frac{M_y}{I_{y,eff}} \cdot \gamma \cdot z_S, \tag{6}$$

$$\sigma_{r2} = \pm \frac{M_y}{I_{y,eff}} \cdot \frac{h_G}{2} \cdot n, \tag{7}$$

where M_y is the maximal bending moment caused by external load [Nmm], t_f is the thickness of the steel flange [mm] and h_G is the height of the glass web [mm] . Finally, shear stress within the adhesive layer is calculated (simplified) by Eqn. 8:

$$\tau = \frac{Q \cdot \gamma \cdot z_S \cdot A_S}{I_{y,eff} \cdot b}, \tag{8}$$

where Q is the maximum shearing force caused by external load.

3. GLUED CONNECTION BETWEEN STEEL AND GLASS

Definitely, for this type of joint, glued connection is much more useful than bolted connection mainly because of better (more uniform) distribution of the stress along the contact. Adhesive layer itself also provides a protection from direct contact between steel and glass, which always has to be avoided. Different variations of the detail of connection between the glass web and steel flange were designed and investigated with different adhesives, see Fig.3.

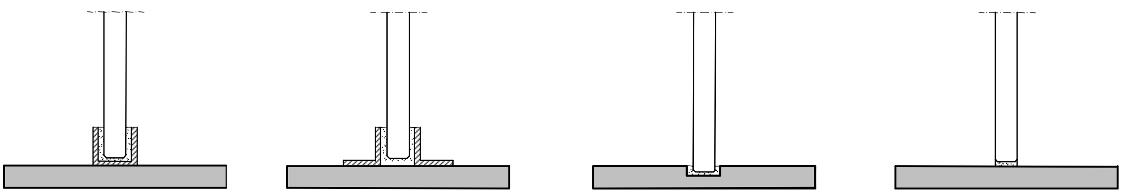

Fig. 3. Designed details of connection between glass web and steel flange

As can be seen in the Fig. 3, additional profiles are the U profile or a pair of L profiles at each flange. These profiles will be welded to the flanges. There is also a possibility to connect the web with the flange without any helpful profile by using direct connection or a small channel cut to the flange, see Fig. 3 on the right hand side. In general, whole glued joint has to be stiff enough to provide an optimal interaction between both materials, but the adhesive also has to provide a flexible reaction of the joint to avoid possible cracks, caused by different temperature elongation of glass and steel.

3.1 Hand calculation of adhesive layer

Simple linear theories can be used for approximately calculations of the average shear stress within the adhesive layer or to obtain the average deformations of overall joint. The shear-strain relation of adhesives can be simplified by bi- or tri-linear diagram for polyurethane adhesives, sometimes by linear diagram for epoxy resins [4]. According to this theory, bonded joint can be modelled as a bedding, see Fig. 4, with three linear stiffness's – normal stiffness (k_1) and two shear stiffness's (k_2 and k_3). These stiffness's, Eqn. 9 and 10, are determined by the Young's modulus, shear modulus and the joint's thickness.

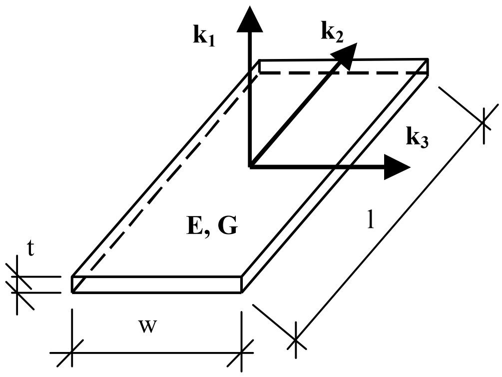

Fig. 4. Stiffness's of the adhesive layer in 3 defined directions

Stiffness's of the adhesive layer can be expressed as

$$k_1 = \frac{E_a}{t_j} , \tag{9}$$

$$k_{2,3} = \frac{G_a}{t_j} , \tag{10}$$

where k_l is the normal stiffness [N/mm^3], $k_{2,3}$ are the shear stiffness's [N/mm^3], E_a is the Young's modulus of the adhesive [MPa], G_a is the shear modulus of the adhesive [MPa], t_j is the thickness of the joint [mm].

The average shear stress is equal to force applied to joint area, see Eqn. 11. The shear strain can be calculated with Eqn. 12 as an angle of the adhesive layer slope.

$$\tau = \frac{F}{A_j} , \tag{11}$$

$$\gamma = \arctan\left(\frac{v}{t_j}\right) , \tag{12}$$

where τ is the calculated average shear stress [MPa], F is the applied force [N], A_j is the joint area [mm^2], γ is the shear strain [-], v is the shear deformation [mm], t_j is the thickness of the joint [mm].

3.2 Different type of adhesives and their behaviour under loading

Adhesive connections offer a lot of benefits in comparison with bolted connection. First of all it is the uniform stress distribution in the connection, ability to assemble thinner materials or to assemble different materials together as well as aesthetic appearance and reduction of self-weight. Depending on width and stiffness of the adhesive, glued connection can compensate different thermal deformation of connected materials. The strength of adhesive connection can be influenced by a lot of factors and depends on the mechanical properties of adhesive, type and load duration, geometric shape of the joint, surface quality and quality of implementation, as well as environmental influences like UV light, moisture and temperature. Failure of the glass-adhesive-glass joint can occur by one of following modes:

- tensile failure of the glass pane,
- shear failure of adhesive – cohesive failure,
- adhesive slip or plucking failure – failure at the glass-adhesive interface.

Adhesives can be divided according to their modulus of elasticity and shear modulus into flexible, elastic (i.e. silicones, modified silicones and polyurethanes) and rigid (i.e. epoxy or polyester resin, acrylate). Rigid adhesives have extremely high strength but very low elongation in comparison with elastic adhesives which have elongation more than 150%. Practical application of different types of adhesive is appear from material properties and their behaviour under the loading. Elastic adhesives are suitable for linear bonded joint and dynamic and short-term loads, rigid adhesive are much more appropriate for point connection and long-term load, i.e. self-weight. Behaviour of the adhesives under the loading follows from their molecular structure, [5]. Adhesives are polymers which consist of simple monomer units chained to macromolecules. Molecular structure can be linear, branched, cross-linked or intertwined. Silicone or polyurethane have long cross-linked polymer chains in comparison with high cross-linked polymer chains of acrylic adhesive, polyester or epoxy resins. Under external loading it is possible to recognise three type of deformation of adhesive:

- elastic deformation which is reversible,
- time dependent viscoelastic deformation, also time dependent reversible,
- time dependent viscoplastic deformation which is irreversible.

4. EXPERIMENTAL PROGRAM

Wide range of the adhesives with different mechanical and deformation properties is involved to the experimental program. Set of the adhesives starts with a very stiff epoxy resin and goes down to very flexible silicone. Material tests, Fig. 5, of all used adhesives have been arranged in accordance with [6].

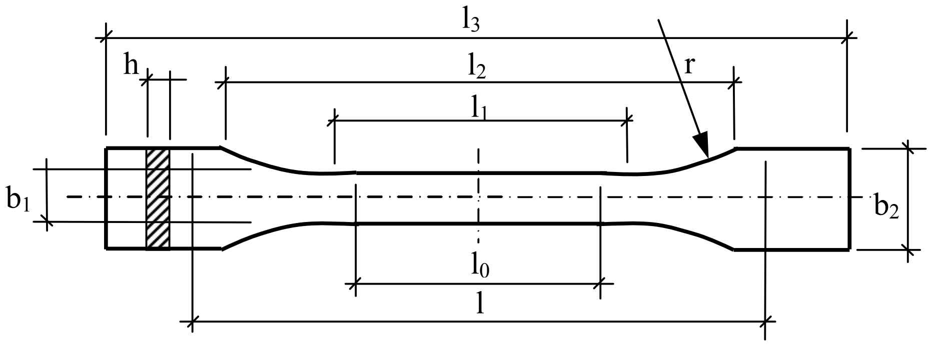

Fig. 5. Test specimen for material tests

Important data like a real tension strength, stress-strain diagram, elongation at break, Young's modulus and Poisson's ratio of each adhesive will be obtained from them and serve as an input data for nonlinear FE material models of the adhesives.

Next experimental set, instant small-scale tension and shear tests of the steel-glass connection are currently in progress at CTU Prague. Tension test specimen consists of the small glass pane assembled to steel targets; see Fig. 6 on the left hand side. Shear test specimens are arranged by the scheme in the Fig. 7 on the right.

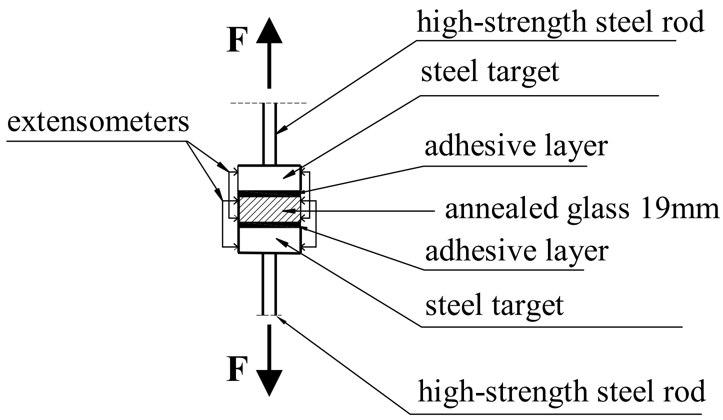

Fig. 6. Tension test specimen

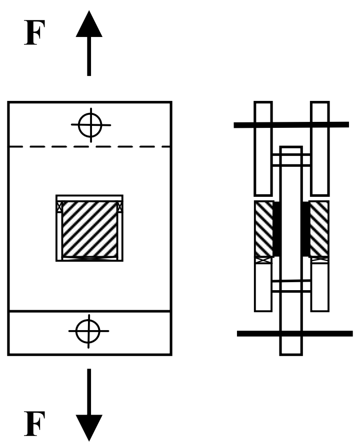

Fig. 7. Shear test specimen

Ultimate tension and shear carrying capacity of the steel-glass connection and the ratio between stress and deformation of the joint will be obtained from these connection tests. Results of these experiments will serve as a device for calibration of the simple FE models of the glued connections, see Fig. 8, including adhesive layer with nonlinear properties obtained from previous material tests.

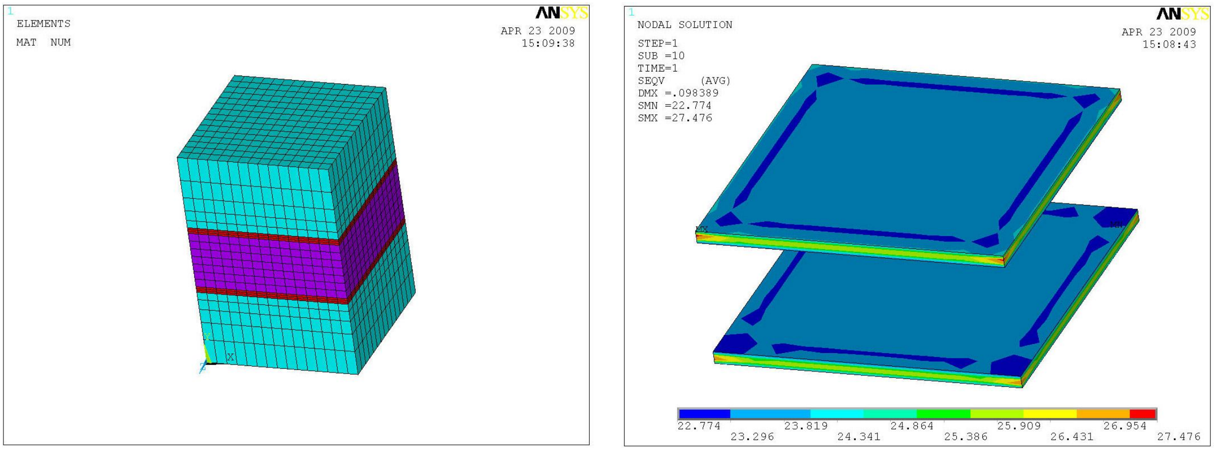

Fig. 8. FE model of simple tension connection tests

Verified models of the adhesive layer can be than involved to the complex FE model of the hybrid beam, Fig. 9, and parametric study will be carried out in accordance with the results of the full-scale tests of the beam under 4 point bending, arranged by the scheme in Fig. 10.

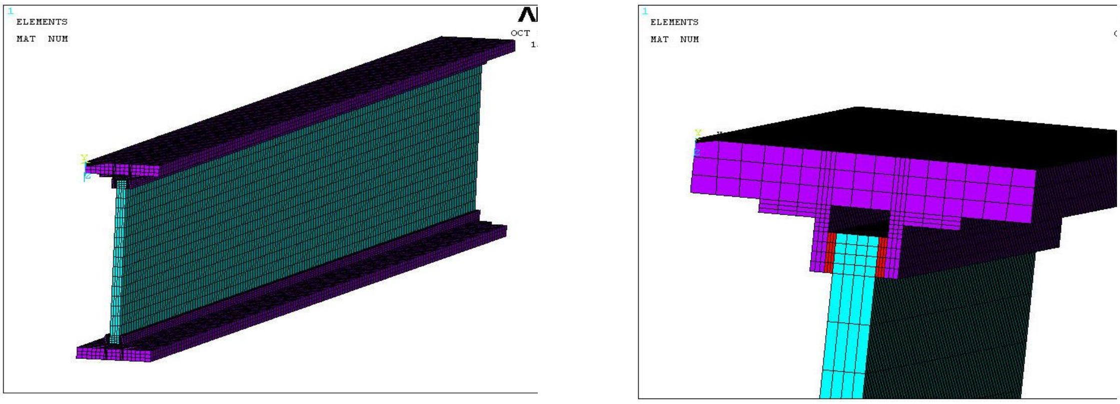

Fig. 9. Complex FE model of the hybrid beam made in software ANSYS

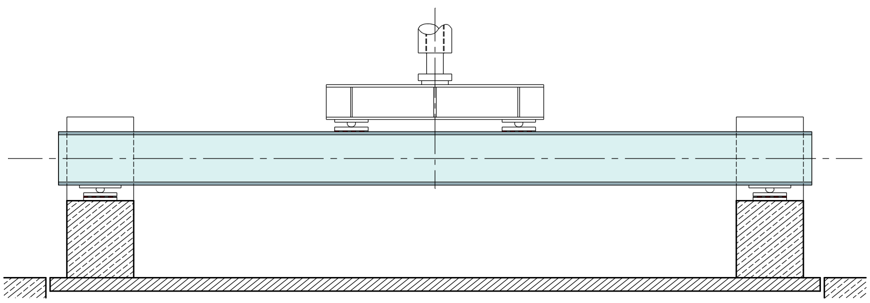

Fig. 10. Set-up of the full-scale experiment of the hybrid steel-glass beam

5. CONCLUSIONS

Described research and numerical analysis should better the knowledge about the exact behaviour of the hybrid structures including the influence of the semirigid glued connection. Experiments will also prove the accuracy of the simplified calculation methods for this problem. Generalized results of the experiments and numerical studies will help to create guidance, how to safe and economically design such a structural element like a hybrid steel-glass beam is.

6. ACKNOWLEDGMENT

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
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COAL FLY ASH PROCESSING FOR ITS UTILIZATION IN BUILDING MATERIALS PREPARING

ABSTRACT

The fly ash from coal combustion processes is a waste very attractive for recycling in the building materials preparing. Presently, a small portion of coal burning products is being utilized for this purpose in Slovakia. Aimed at a wider recycling and exploitation of coal fly ash as siliceous and aluminous substance in the building materials production, the pre-treatment of fly ashes is designed for their quality improvement. This paper presents the results of mechanical and chemical activation of coal fly ash regarding the utilization in concrete and cements production. The investigation of physico-mechanical properties of concrete specimens based on mechanically and/or chemically activated coal fly ash as a partial cement replacement (25 wt. %) after 28- and 90 days of hardening showed that the addition the coal fly ash activated under optimal conditions improves the compressive strength of these composites in comparison to concrete with non treated coal fly ash.

KEYWORDS: coal fly ash, utilization, mechanical and chemical activation, concrete

1. INTRODUCTION

Coal combustion products (CCPs) are formed with the production of electricity in coal-fired power plants all over the world. The production of these CCPs is increasing worldwide by the higher demand for electricity due to growing population and economic development [1].

CCPs include combustion residues such as boiler slag, bottom ash and fly ash from different types of boilers as well as desulphurisation products like spray dry absorption product and FGD gypsum. The CCPs are mainly utilized in the building material industry, in civil engineering, in road construction, for construction work in underground coal mining as well as for recultivation and restoration purposes in open cast mines.

Approximately 33 % of the total fly ash amount produced in Europe is used as cement raw material, as constituent in blended cements and as addition for the production of concrete. By this they are a main constituent of the cement or they are replacing a part of the cement necessary for the production of concrete [2].

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The changes in the fly ash quality brought about by the introduction of advanced combustion and clean coal technologies have had a significant impact on the use of this waste kind in concrete. For the improvement of fly ash quality for given application purpose are used suitable treatment processes. Based on our previous works [3-5], mechanically and chemically activated coal fly ash added to cement mixtures notably influences the development of the compressive strength of composite.

Recently, an alkaline activation as a simple method for the design geopolymer concrete mixtures was proposed. An alkaline liquid is used to react with the silicon and aluminium in a source material (by-product materials such as fly ash) to produce alternative binders to Portland cement. The chemical reaction under alkaline condition on Si-Al minerals results in an inorganic polymerization product called geopolymer [6]. Its chemical composition is similar to natural zeolitic materials, but the microstructure of geopolymer is amorphous. According to [7], geopolymeric materials have a wide range of applications in the field of industries such as in the automobile and aerospace, non-ferrous foundries and metallurgy, civil engineering and plastic industries. The type of application of geopolymeric materials is determined by the chemical structure in terms of the atomic ratio Si/Al in the polysialate (Tab. 1).

Table 1. Applications of Geopolymeric Materials Based on Silica-to-Alumina Atomic Ratio

Si/Al ratio	Applications
1	- Bricks - Ceramics - Fire protection
2	- Low CO_2 cements and concretes - Radioactive and toxic waste encapsulation
3	- Fire protection fibre glass composite - Foundry equipments - Heat resistant composites, 200ºC to 1000ºC - Tooling for aeronautics titanium process
>3	- Sealants for industry, 200oC to 600oC - Tooling for aeronautics SPF aluminium
25-35	- Fire resistant and heat resistant fibre composites

Fly ash has been successfully used to manufacture geopolymer concrete when the silicon and aluminum oxides constituted about 80% by mass, with the Si/Al ratio of about 2. The content of the iron oxide usually ranged from 10 to 20% by mass, whereas the calcium oxide content was less than 5% by mass. The carbon content of the fly ash, as indicated by the loss on ignition by mass, was as low as less than 2%.

This paper is focused on the study of mechanical and alkaline activation influence on compressive strength development of products based on the partial cement replacement by coal fly ash.

2. MATERIAL AND METHODS

The Portland cement (CEM I 42,5) and two different coal fly ash types from classical incineration of brown coal fly ash originating from Slovakian power plant ENO A in Novaky were used as raw materials. Granulometric composition of two different coal fly ashes is given in Tab. 2. The particle size analysis was carried out by dry sieving down to 500 μm on standard series of sieves.

Table 2. Granulometric composition of used coal fly ashes

Fraction [μm]	Mass yield [wt.%]	
	CFA I	CFA II
> 500	0	8.57
500 - 250	4.10	9.77
250 - 125	24.59	47.67
125 - 71	14.62	17.73
71 - 40	19.84	7.13
< 40	36.12	9.13

Tab. 3 summarizes chemical composition of coal fly ashes in form of stable oxides. The content of components was determined by AAS analysis (VARIAN, Austria). Total amount of SiO_2 and Al_2O_3 was 82.13% (CFA I) and 77.85% (CFA II). The presence of crystalline phases was detected by X-ray diffraction (XRD) analysis on diffractometer DRON 2,0 with goniometer GUR-5 (Technabsexport, Russia). The following minerals as major components present in coal fly ashes are quartz, mullite, hematite, albite (CFA I) and quartz, mordenite, mullite, andaluzitte (CFA II).

Table 3. Chemical composition of used coal fly ashes

Component	Content [%]	
	CFA I	CFA II
SiO_2	62.53	58.95
Al_2O_3	19.6	18.90
CaO	2.89	4.29
MgO	1.90	-
Fe_2O_3	8.07	3.4
TiO_2	-	0.62
LOI*	2.38	3.89

* LOI - loss of ignition

Mechanical and chemical activation of coal fly ashes was used to improve their properties in accordance with standard (STN EN 450) with aim to partial replace the cement in concrete by activated products. Mechanical activation was focused on increase of coal fly ash fineness (CFA I) and chemical activation was aimed for improvement of coal fly ash pozzolanic activity by zeolitization (CFA II).

Mechanochemical activation of coal fly ash I was carried out by dry milling (without and with using solid NaOH) and wet milling (5 M aqueous NaOH solution) in laboratory vibratory mill (0,25 and 0,5h). By optimization of milling conditions the products were prepared, where the particle size analysis was carried out on the laser granulometer Helos/LA with dry dispersion unit Rodos 11 SR (Sympatec GmbH, Germany).

Alkaline activation of coal fly ash II was realized in autoclave at 130°C and pressure of 160 kPa in 5 M NaOH solution during 5 hours at solid/liquid ratio of 0,5. Coal fly ash II/cement mixture was alkaline activated under normal pressure at variation of temperature (120-260ºC) and reaction time (12-36h).

Coal fly ash/cement mixtures with 25 wt.% cement replacement were prepared with starting, milled and alkaline activated coal fly ash at solid/liquid ratio of 0,5. The compressive strengths of bodies (40mm x 40mm x 160mm) after 28 and 90 days hardening were measured. The value of relative strength K_R was expressed as the ratio of compressive strength of coal fly ash/cement sample to compressive strength of comparative concrete.

3. RESULTS AND DISCUSSION

The relative compressive strength values of 28 and 90 days hardened composites based on coal fly ash I milled under various conditions are presented in Tab. 4. As it can be seen, compressive strength values of two samples M1 and M5 (cement replacement by dry milled coal fly ash samples at 0,25 and 0,5h) exceed values required by STN EN 450 after 28 and 90 days of hardening (78,3% and 86,0% after 28 days; 90,1 % and 85,4% after 90 days). The high compressive strength values are connected with coal fly ash fineness and mainly with activity of new-created surface area of particles at milling (Tab. 4). The particle surface activity is determining factor of cement mixture behavior in beginning phase of hardening. Components presenting in coal fly ash are hydrated faster due to particle surface activity what contributes to improvement of consolidation of composite structure and increase in compressive strength of composite [8, 9].

Table 4. The relative compressive strength values K_R of composites after 28 and 90 days hardening based on partial cement replacement by the coal fly ash I milled under various conditions and fraction under 20 μm (ΔR-20) and mean particle diameter (dm) of milled products

Sample	Milling					K_R [%]		ΔR_{-20} [%]	d_m [μm]
	0,25 h	0,50 h	dry	wet	with NaOH	28 days	90 days	Fraction under 20 μm	Mean particle diameter
M0	starting coal fly ash					24,6	25,3	8,9	109,3
M1	●	-	●	-	-	78,3	90,1	37,9	39,8
M2	●	-	●	-	-	31,8	39,3	-	-
M3	●	-	●	-	●	39,2	38,4	40,6	41,4
M4	●	-	-	●	●	40,8	44,9	55,8	23,6
M5	-	●	●	-	-	86,0	85,4	47,8	29,0
M6	-	●	●	-	-	38,6	41,7	-	-
M7	-	●	●	-	●	41,7	54,5	50,6	30,1
M8	-	●	-	●	●	45,8	67,3	55,9	25,2
Mc	comparative sample					100	100	-	-

Tab. 5 illustrates the calculated values of the relative compressive strength of composites based on alkaline activated coal fly ash II (A1-A3) and its mixture with cement (U1-U6) after zeolitisation in autoclave. These relative compressive strengths of coal fly ash II/cement mixtures are compared with relative compressive strength of starting and milled samples of natural zeolite (Z1 - Z3).

Samples A3, U4 and U5 prepared with coal fly ash zeolitization and its mixture with cement in autoclave exceed the values required by STN EN 450 (75% and 85% of comparative sample compressive strength) after 28 and 90 days of hardening. The high compressive strength values in accordance with data published in [10] are connected with the formation of amorphous aluminosilicates - zeolitic precursors (phase N-S-A-H similar to CSH) and/or partially amorphous and crystalline zeolitic phases on surface of coal fly ash particles in dependence upon alkaline activation conditions and coal fly ash quality. The formation of a three-dimensional aluminosilicates chain constitutes favorable conditions for structure development in concrete matrix [11].

Table 5. The relative compressive strength values K_R of composites after 28 and 90 days hardening based on the coal fly ash II (A1-A3) and its mixture with cement (U1-U6) alkaline activated in autoclave and natural zeolite (Z1-Z3)

Sample		A1	A2	A3	U1	U2	U3	U4	U5	U6	Z1	Z2	Z3	Ac,Uc,Zc
K_R [%]	28 days	24,6	36,8	76,6	61,7	72,3	60,8	77,0	100	63,9	55,2	72,7	74,5	comparative sample
	90 days	25,3	37,8	105,1	67,9	74,7	60,7	86,3	103,9	63,1	53,5	68	69,1	

Tab. 6 shows identified crystalline phases in starting coal fly ash and alkaline activated products.

Table 6. Identified crystalline phases in starting coal fly ash and alkaline activated products

Crystalline phase	Sample			
	Starting coal fly ash	Alkaline activated coal fly ash (A3)	Alkaline activated coal fly ash/cement mixture	
			U4	U5
Quartz	●	●	●	●
Kristobalite	●	●	●	●
Mullite	●	-	-	-
Andaluzite	●	-	-	-
Kyanide	●	-	-	-
Kaolinite	●	-	-	-
Hematite	●	●	-	-
Magnetite	●	-	-	-
Illmenite	●	-	-	-
NaP1 zeolite	-	●	●	-
Phillipsite	-	●	-	-
Albite	●	●	-	-
Mordenite	●	-	-	-
Illite	●	-	-	-
Analcim	-	-	●	●
Hydroxy-sodalite	-	-	●	●
Zeolite Na-Pc	-	-	-	●

The relative compressive strength values of composites based on the milled natural zeolite are lower than the standard requirements. The study of alkaline activation of coal fly ash/cement mixtures under normal pressure conditions by variation of temperature (120-260ºC) and reaction time (12-36h) showed that higher temperature leads to the low compressive strength of hardened composite (they don't correspond with requirements of standard STN EN 450) [12].

4. CONCLUSION

Concretes prepared with addition of mechanical and alkaline activated coal fly ash after 28 and 90 days of hardening reached the relative compressive strength values required by standard STN EN 450. The compressive strengths correspond to concrete strength classes C 20/25 and C 25/30 according to standard STN EN 206. The obtained results show that the chosen treatment methods of coal fly ash have a positive influence on compressive strength development of composites.

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PHOTOVOLTAIC SYSTEMS IN SLOVAKIA

ABSTRACT

The current energetic crisis, which affect Europe and Slovakia, evoked amount of serious questions concerning the dependence on fossil fuels and the possibility of their replacement by alternative energy sources in the future. One from the potential alternative source is solar energy and its immediate conversion to electricity by using photovoltaic cells.

Nowadays, Slovakia finds its position at the end of various tables pointing to the installed capacity of photovoltaic panels in the European Union.

Article describes a brief overview of photovoltaic development in the world and suggests potential uses in the future. Article is directly related with building up a center of excellence for renewable energy sources and related progressive indoor technology, which is involved with Faculty of Civil Engineering, Technical University of Košice. One from the several objectives resolving in this project is the transformation of solar energy into electrical energy through photovoltaic cells. Part of the thesis is intended to compare legislative and economical conditions in Slovakia and abroad.

KEYWORDS: photovoltaic cells, energy, solar, building.

1. INTRODUCTION

Solar radiation can be used in several ways. One possibility is passive solar architecture, where the sunlight is used as efficiently as possible, using the actual architectural design of the building. Another of the possibilities of using solar radiation are solar collectors that convert solar energy to thermal energy, and last but not least, is the conversion of solar energy to electrical energy through photovoltaic cells, which will be detailed addressed in this contribution.

2. CURRENT SITUATION IN SLOVAKIA

In our geographical latitudes, the solar energy has the largest potential of renewable energy sources (RES) in Slovakia, which corresponds to 54 038 TWh. Of this total share is technically exploitable potential 9 450 GWh, from that electricity is 1 540 GWh/year. In Slovakia, the potential use of solar energy is higher than for countries (Czech Republic,

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Germany, the Netherlands), where the potential is lower but the number of installations of PV systems far exceeds Slovakia.

Important role in the field of photovoltaic in Slovakia plays a forthcoming law on the promotion of RES. Other important standards include the legal strategy of higher utilization of RES and energy security of the Slovak Republic. Price regulation performs the Regulatory Office for Network Department (ÚRSO).

The price of electricity produced from renewable energy sources is determined as a fixed price in Slovak crowns for mega-watt-hour for the period of 12 years, since the introduction of assembly devices into service as follows:

From solar energy:

1. for equipment put into service until 31. December 2004: 390 €/MWh
2. for equipment put into service from 1st January 2005 to 31st December 2008: 425 €/MWh
3. for equipment put into service from 1st January 2009: 448 €/MWh .

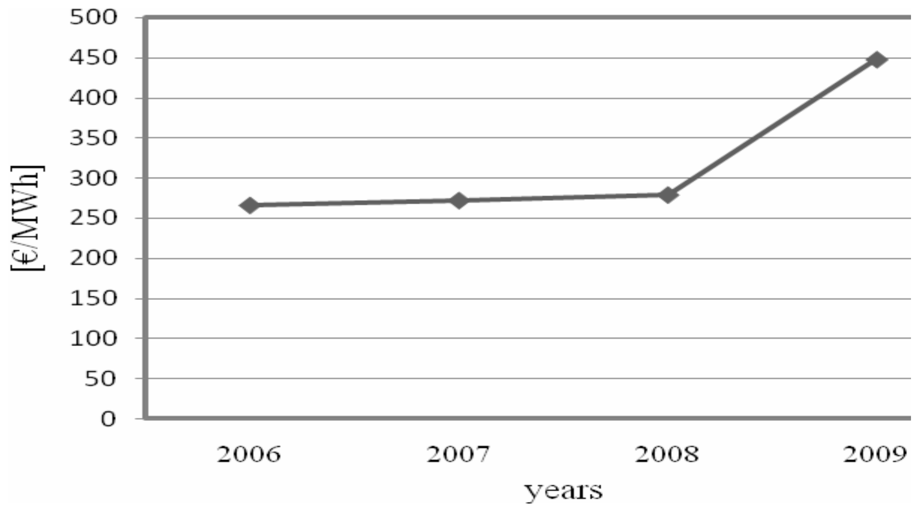

Fig. 1: Graph of purchasing prices (Sk/MWh)

Compared to last years, it is a significant move forward, because in the last year was the price set at 279 €/MWh (8 410 Sk/MWh) in 2007 it was 272 €/MWh (8 200 Sk/MWh) and in 2006 it was 266 €/MWh (8 000 Sk/MWh). From these numerical statements is evident more than 60% price increase for the repurchase of electric power produced by photovoltaic cells as compared to last year (Fig. 1) [4].

Table 1. Production of electricity in 2015 [1]

Source / Year	**2010 [GWh]**	**2015 [GWh]**	**Difference [GWh]**
Small water power plants	350	400	100
Biomass	410	650	340
Wind power plants	300	900	350
Biofuel	180	300	220
Geothermal energy	0	40	40
Photovoltaic cells	0	10	10
Sum	**949**	**2 300**	**1 060**

By the year 2015, the aim is to produce from RES 7% of the total electricity consumption. Estimated production of various forms of RES in 2015 compared to 2010 is shown in the table (Tab. 1) [1].

Slovakia is in the European Union at the tail end of the various statistics and scales about installations of photovoltaic panels and the actual use of electricity (Tab. 2).

Table 2. Installed power capacity per person (Wp/pers.) [2]

Country	Wp/Pers.	Country	Wp/Pers.	Country	Wp/Pers.
Luxembourg	51,2	Greece	0,82	Ireland	0,1
Germany	46,5	France	0,77	Hungary	0,03
Spain	11,74	Sweden	0,68	Bulgaria	0,02
Austria	3,49	Belgium	0,59	**Poland**	**0,02**
Netherlands	3,34	Denmark	0,57	Romania	0,01
Cyprus	2,25	Czech rep.	0,39	**Slovakia**	**0,01**
Italy	1,71	Slovenia	0,32	Lithuania	0,01
Portugal	1,68	Great Britain	0,29	Latvia	0
Finland	0,95	Malta	0,24	Estonia	0
				EU	**8,49**

3. APPLICATION OF PHOTOVOLTAIC SYSTEM IN MODEL CONDITIONS

Proposal of photovoltaic system is transferred to the model administration building in which they were applied theoretical knowledges and the various calculations necessary to design the system. Photovoltaic panels are installed on a flat roof building using a metal structure under the 45 ° angle oriented toward the south.

Input parameters:

Q_{max} = 130 kW – maximum heat power input of the building

θ_i = 20 °C - required computing temperature (average)

θ_e = -13 °C - required computing temperature (average)

$\theta_{e,pr}$ = 4 °C – average external air temperature

d = 223 – length of the heating period

Period of service in the summer - 1 830 hours (cooling)

Table 3. Electric energy need

Daily energy demand					
number	**appliance**	**amount**	**service hours [h/d]**	**input power [W]**	**energy need [kWh/d]**
1	computer	100	10	150	150
2	printer	40	2	50	4
3	lamp	550	12	15	99
4	refrigerator	5	24	200	24
5	cooling box	2	24	200	9,6
6	television	5	10	50	2,5
7	microwave	10	1	1000	10
8	projector	1	1	100	0,1
9	cashier	4	10	20	0,8

When the total consumption of electricity in the administration building played one of the most important tasks, are electrical appliances. The daily need of electricity for appliances in the building is constructed around the value of 300 kWh/day. Considering 250 working days, the total annual need for electricity is 75 000 kWh/year. In the table (Tab. 3) is daily energy needs for electric appliances in evaluated building.

In the next table (Tab. 4) and figure (Fig. 2) is annual consumption of energy need for every type of energy consumption in the office building.

Table 4. Annual electricity consumption

Type of energy consumption	kWh/year
electric appliances	75 000
heating	206 389
cooling	95 160

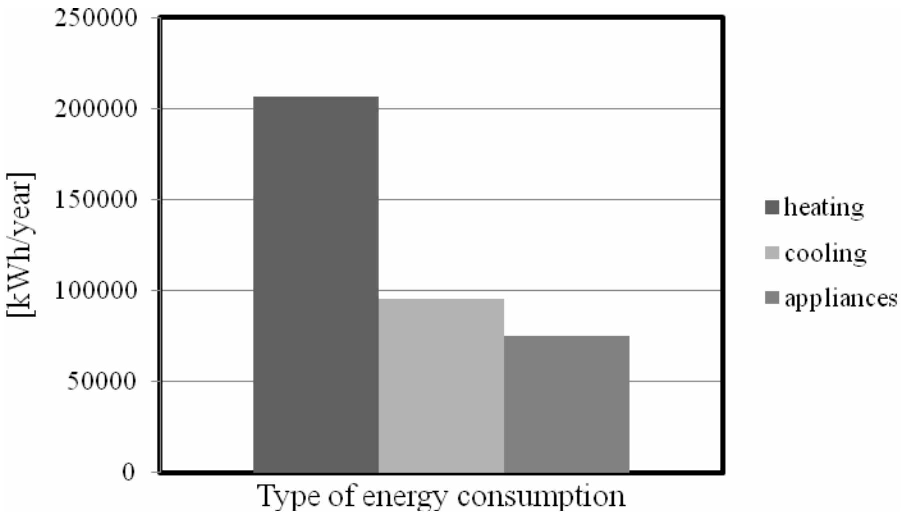

Fig. 2. Annual energy consumption [kWh/year]

4. PHOTOVOLTAIC SYSTEM DESIGN

For a model office building is designed photovoltaic system with a direct connection to the electricity grid (grid-on) (Fig.3). This is the most widely used system of connected photovoltaic panels. The wide application of the PV system is in areas where there is sufficient coverage of electricity distribution network. The system does not need any battery because of the electricity produced is either consumed by appliances in the building or the excess electric energy is supplied to the electricity grid. When this involvement is expected to return earlier system.

In the design of photovoltaic system was used 203 pc. of photovoltaic panels Solartec SG 72-175/24, composed of 72 pieces monocrystaline 5" silicon Si.

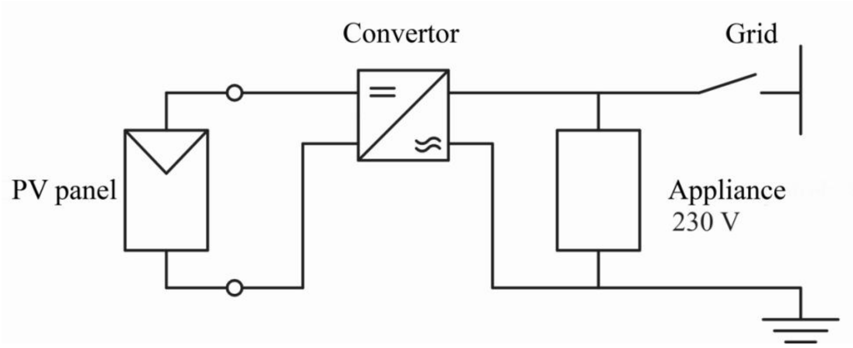

Fig. 3. Direct connection to the grid

In the next table (Tab. 5) and figure (Fig. 4) is calculated percentage coverage of electric power made by photovoltaic system for each type of energy consumption in the office building.

Table5. Percentage electric power coverage

Energy need coverage	**[%]**
heating	16,51
cooling	35,82
electric appliances	45,45

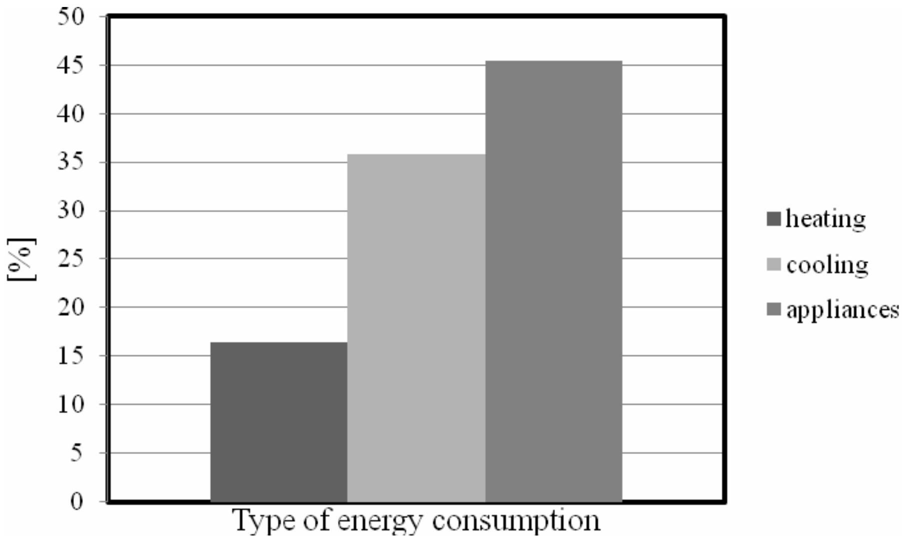

Fig. 4: Energy needs coverage [%]

5. ECONOMIC RETURN OF PHOTOVOLTAIC SYSTEM

Price of one piece of the PV panel is 830 € (25.000 Sk). The investment in the purchase of 203 pieces of PV panels at a price of 830 € (25.000 Sk) will be 168.460 € (5.075.000 Sk). Total investment of the photovoltaic system is about 182.566 € (5.500.000 Sk) (including transport, installation, frame, etc.). The Slovak Republic is obligated to buy electricity produced using renewable energy sources. Price for photovoltaic energy is set to 0,488 €/kWh (13,5 Sk/kWh). The price of electricity from the power plants is fixed at 0,11 €/kWh (3,32 Sk/kWh). This implies a profit 0,338 €/kWh (10,18 Sk/kWh). With the produced electricity 34.083,7 kWh/year, it is saved 11.879,4 € (357.878,85 Sk). Financial return of the PV system is calculated for 16 years.

6. PERSPECTIVE CAPITALIZING OF PHOTOVOLTAIC SYSTEM IN FUTURE

The following graph (Fig. 5) shows the evolution of the price of photovoltaic cells in recent years. This indicates a price reduction of photovoltaic panels but also increases its performance. From this graph it is clear that this continuing trend, the price will decline and performance rise, is the prospect of using the system in the future.

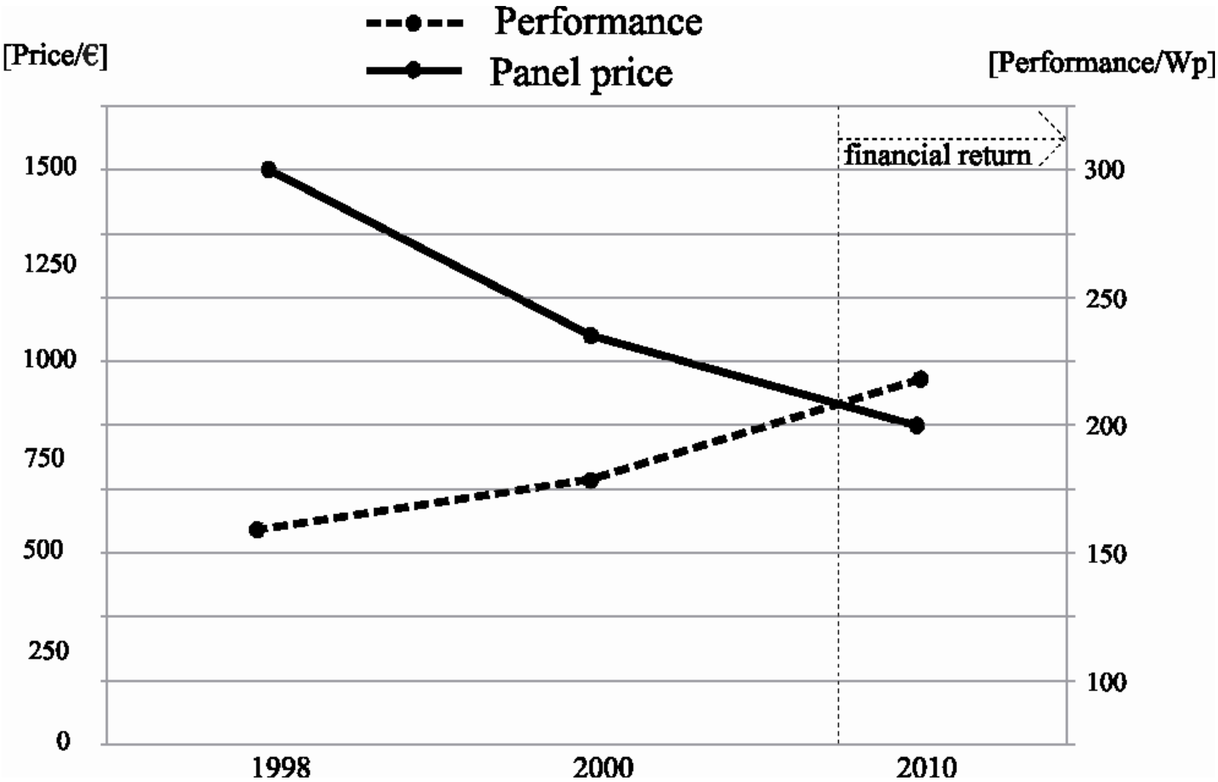

Fig. 5: Capitalizing of PV system

7. CONCLUSION

In this paper are presented legislative standards and decrees, that are valid in Slovakia. There is also described the calculation version of the photovoltaic system with the economic return and its perspective capitalizing.

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
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THE PROPERTIES OF CEMENT CONCRETES MODIFIED WITH SUPERPLASTICIZERS

ABSTRACT

The properties of grouts, mortars and concrete mixtures with four different superplasticizers have been investigated. We determined the plasticizing and setting time of cement, the plasticizing and shrinkage of mortars, the consistency change of concrete mixtures as a function of time using the slump and flow tests, and the compressive strength and water absorption of cement concrete. The investigations carried out show the significant impact of superplasticizers on the plasticizing of cement grouts and on the shortening of their setting time. The shrinkage of mortars depends on the type of superplasticizer. Furthermore, the consistency change as a function of time depends on the type of superplasticizer and indicates the possibilities of lengthening the time of workability maintenance of mixtures with those superplasticizers. The linear relationship was found between the values of the slump loss and the slump flow as a function of time. The superplasticizers used significantly lower the water absorption of concrete and increase the quality of those concretes by 1-2 classes as compared with the concrete without admixture. The investigations into cement concrete with superplasticizer indicate the possibilities of regulation of concrete mixtures consistency and selecting the adequate mean of transportation and placing, as well as some cement savings.

KEYWORDS: superplasticizer, paste mortar, concrete mixture, slump loss, flow test, concrete

1. INTRODUCTION

The basic functions of plasticizing and high-range water reducing admixtures used in cement materials are following [1-14]:

a) the reduction of *W/C* ratio value while maintaining the initial consistency of concrete mixture constant,
b) the reduction of cement and water content without worsening the fluidity of concrete mixture and while sustaining the same value of compressive strength as for the control concrete,
c) the change of concrete mixture consistency into more fluid while maintaining W/C ratio constant (no changes in basic content of concrete mixture).

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The objective of investigations undertaken was to determine the properties of grouts, mortars, concrete mixtures and concretes with four different superplasticizers of SNF type, denoted as P1, P2, P3 and P4.

2. TEST AND THE RESULTS

2.1 Materials

The superplasticizers SNF in liquid from four different producers were used:

P1 - recommended in the quantity of 0,2÷3,45 % by weight of cement,

P2 - recommended in the quantity of 0,2÷3,45 % by weight of cement,

P3- recommended in the quantity of 0,4÷2,50% by weight of cement,

P4 - recommended in the quantity of 0,2÷2,30% by weight of cement.

Cement CEMI 32,5R Górażdże used in the experiments corresponds to the requirements of standard [16] in accordance with the compressive strength after 2 and 28 days of hardening.

The following aggregates were used:

a) the gravel 8-16 mm from Mietków mine,
b) the gravel 2-8 mm from Mietków mine,
c) the sand 0-2 mm from Mietków mine.

The tap water was used.

The mineral aggregates were experimentally combined into an optimum aggregate composition – characterized by maximum compactness and minimum amount of water absorbed by the aggregate. The void determined for the compacted aggregates c_a=26,5 %.

2.2 The properties of cement used

The testing of cement binding time was curried out on pastes with consistency [15]. Superplasticizers P1, P2, P3 and P4 were added together with mixing water from 0,5-3,0% according to the weight of cement. Figs 1-4 reflect the relationship between the amount of removed water (ΔW) and the amount of superplasticizers (P1, P2, P3, P4).

The setting time of cement was tested using pastes characterized by standard consistency [15]. The results of test by Vicat method are presented in Tab. 1.

2.3 The properties of mortars with superplasticizers

Five mortars were made with the use of sand 0-2 mm Mietków: four with the superplasticizer (P1, P2, p3 and P4) and a control one. The cement mortars were prepared in the mass proportion 1:3 of cement sand. The consistency of fresh mortar was „5" according to Novikov's slump. Since the applied superplasticizers act in the plasticizing way into the batch of fresh mortars, the amount of make-up water decreased by 12 to 27% depending on the type and quantity of preparation keeping the initial consistency on the control mortar. Queen closure samples of 40×40×160 mm with metal pins were shaped to determine the shrinkage with the method of Graf-Kaufmann [16].

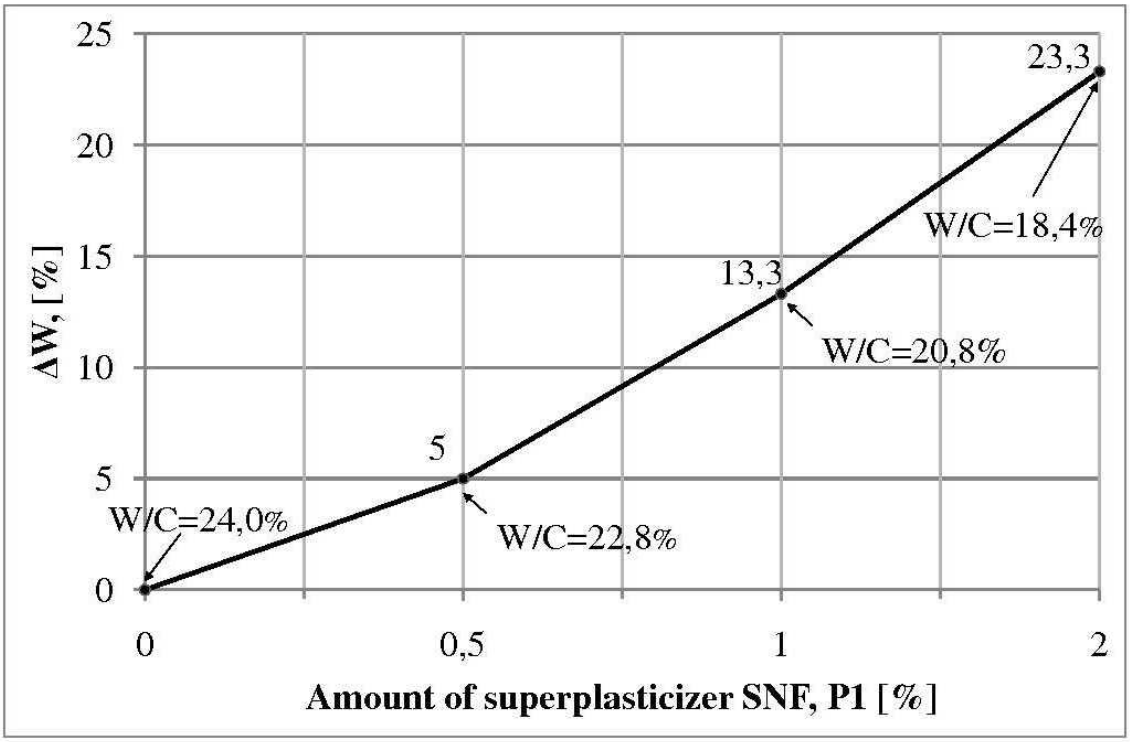

Fig. 1. The superplasticizer P1 influence on water reduction in standard paste

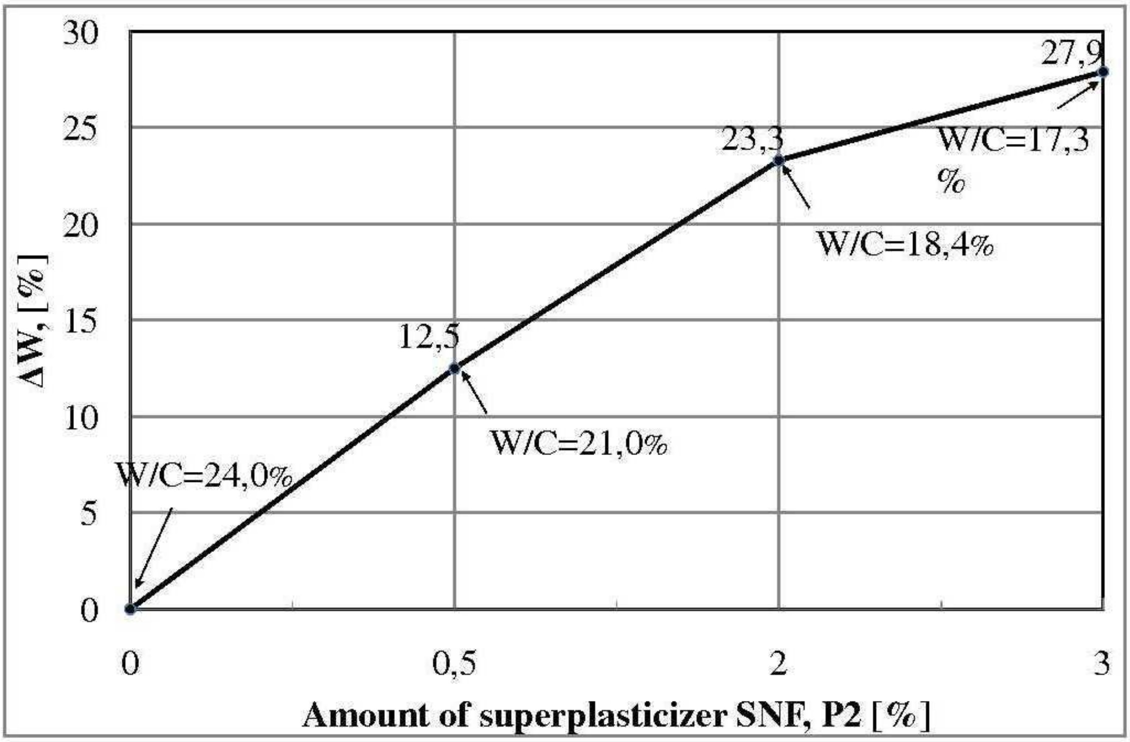

Fig. 2. The superplasticizer P2 influence on water reduction in standard paste

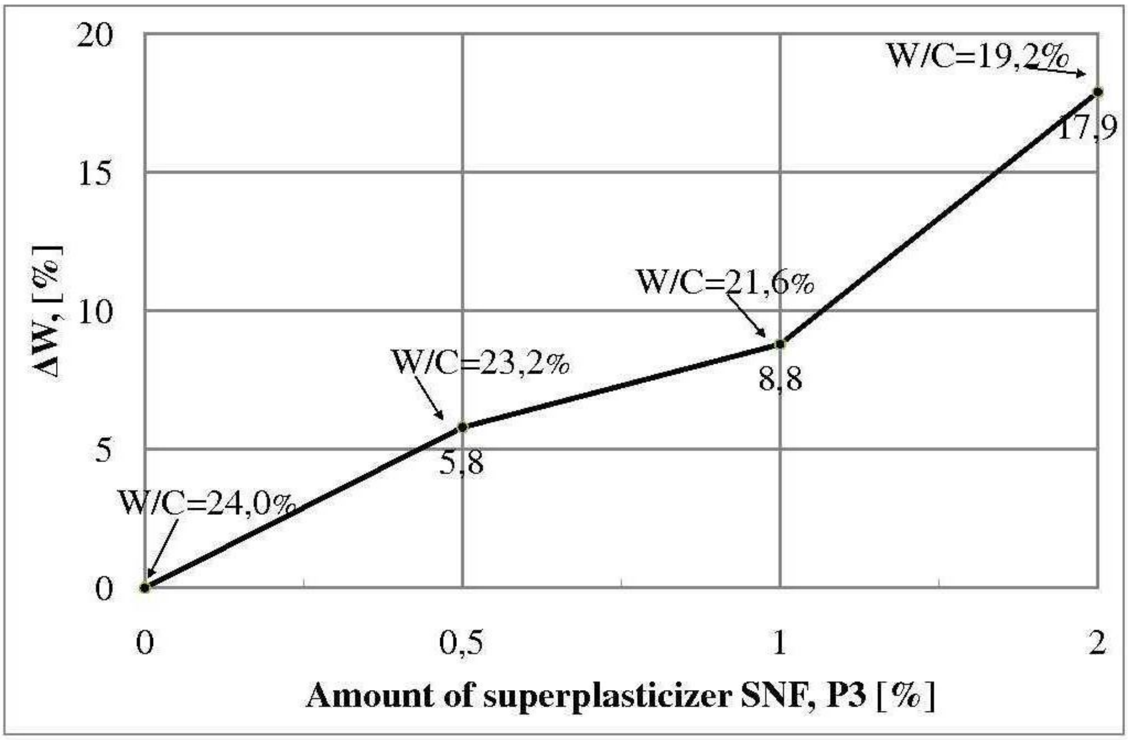

Fig. 3. The superplasticizer P3 influence on water reduction in standard paste

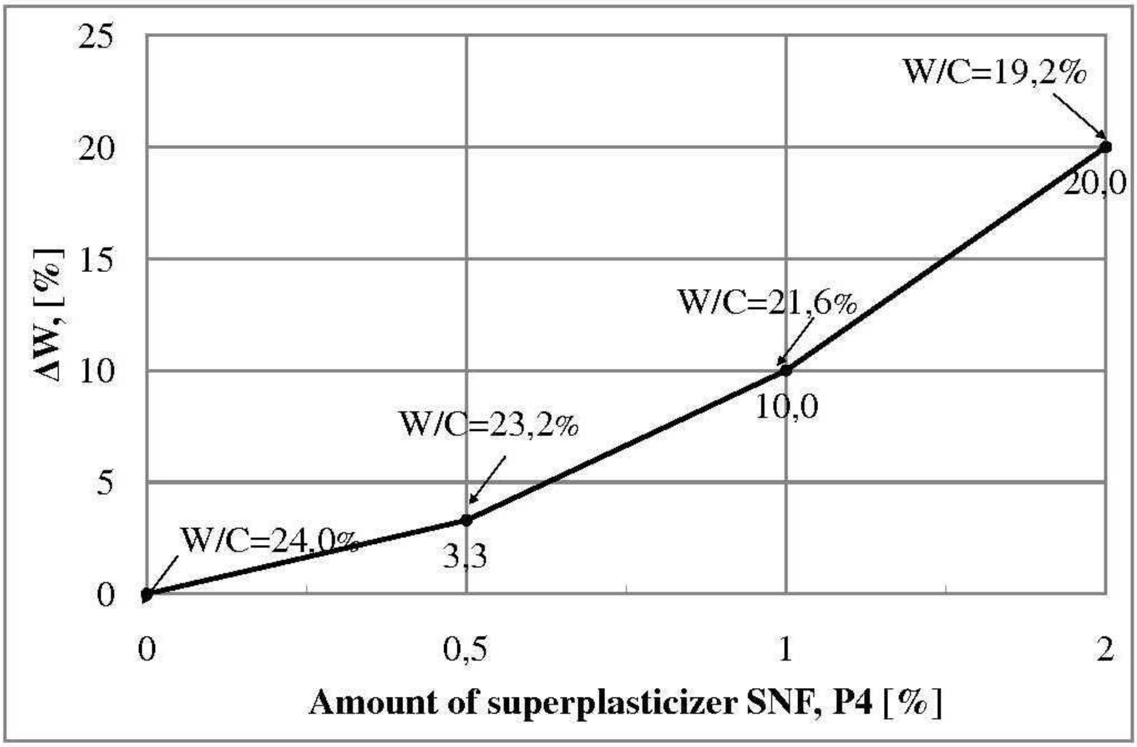

Fig. 4. The superplasticizer P4 influence on water reduction in standard paste

Table 1. The plasticity and the setting time of standard pastes with superplasticizers

Superplasticizer	The amount of superplasticizer (% by weight of cement)	W/C, [%]	ΔW, [%]	Setting conditions [hours h, minutes']		
				initial t_i	final t_f	setting time t_s
-	0,0	24,0	0,0	1h00'	4h30'	3h30'
P1	0,5	22,8	5,0	0h20'	5h50'	5h30'
	1,0	20,8	13,3	0h30'	9h20'	8h50'
	2,0	18,4	23,3	0h40'	13h20'	12h40'
P2	1,0	21,0	12,5	0h30'	5h50'	5h20'
	2,0	18,4	23,3	0h40'	6h20'	5h40'
	3,0	17,3	27,9	0h50'	6h50'	6h00'
P3	0,5	22,6	5,8	0h20'	6h10'	5h50'
	1,0	21,9	8,8	0h20'	10h00'	9h40'
	2,0	19,7	17,9	0h20'	10h50'	10h30'
P4	0,5	23,2	3,3	1h10'	4h40'	3h30'
	1,0	21,6	10,0	0h40'	5h10'	4h30'
	2,0	19,2	20,0	0h10'	6h00'	5h50'

The values of shrinkage of the mortars hardening in the laboratory conditions (temperature of 18˚C (±2˚C) and a relative air humidity of 65% (±5%)) are given in Tab. 2.

Table 2. The shrinkage of cement mortal with superplasticizers

Superplasticizer	The amount of superplasticizer (% by weight of cement)	W/C	Shrinkage, [mm] -days-						
			2	3	4	7	10	14	28
-	0,0	0,61	0,167	0,229	0,292	0,354	0,479	0,510	0,677
P1	1,2	0,50	0,000	0,042	0,083	0,229	0,333	0,406	0,552
P2	2,0	0,44	0,000	0,177	0,208	0,271	0,375	0,458	0,531
P3	1,0	0,52	0,024	0,042	0,073	0,094	0,240	0,302	0,385
P4	1,0	0,54	0,125	0,500	0,583	0,729	0,813	0,854	0,958

2.4 The properties of concrete mixtures with superplasticizers

15 concrete mixtures were made using CEMI 32,5R and natural aggregate: 3 of them were control concrete mixtures with the consistency of S1-S3 and with the respective cement amount of 320, 360 and 400 kg/m^3, and 12 of them were concrete mixtures with superplasticizers P1, P2, P3 and P4 in an adequate quantity. We determined the consistency change of concrete mixture as a function of time using the slump and flow tests. The repeated dosing of superplasticizers and applying them in the same amount as in the first dosage after the workability loss of concrete mixtures, increases significantly their fluidity. With 320 kg/m^3 of cement and addition of superplasticizers P1, P2, P3 and P4, the slump loss and flow equal respectively to: 90 and 440, 140 and 397, 110 and 477, and 130 and 460 mm. With 360 kg/m^3 of cement and addition of superplasticizers P1, P2, P3 and P4, the slump loss and flow account respectively to: 80 and 457, 120 and 460, 130 and 466, and 130 and 470 mm. Finally, with 400 kg/m^3 of cement and addition of superplasticizers P1, P2, P3 and P4, the slump loss and flow amount respectively to: 60 and 460, 70 and 493, 105 and 453, and 130 and 520 mm.

Table 3 shows the changes in the consistency of concrete mixtures specified with two standard methods referred to as a function of time. In addition, the linear relationship between the values of slump loss and flow as a function of time was examined [17]. The coefficient of Pearson linear correlation r fall within the range of 0,89-0,99 reflecting the relatively strong relationship between the results of concrete mixtures consistency determination within two standard methods [15-17].

2.5 The properties of concrete with superplasticizers

The following properties of concretes outlined in table 3 (the amount of cement – 320, 360 and 400 kg/m^3 and consistency – slump ~100 mm, flow~450 mm) were determined: compressive strength and water absorption after 28 days of hardening in the laboratory condition. The specimens used for all determinations were crude in a climatic chamber a temperature of 18˚C (±2˚C) and a relative air humidity of 95% (±5%). For each type of concrete the compressive strength was determined using 6 cube specimens with a 150 mm side [15]. The average compressive strength values are shown in Fig. 5(the amount of cement – 320 kg/m^3). In addition, the dispersion of results in the form of standard deviation was calculated.

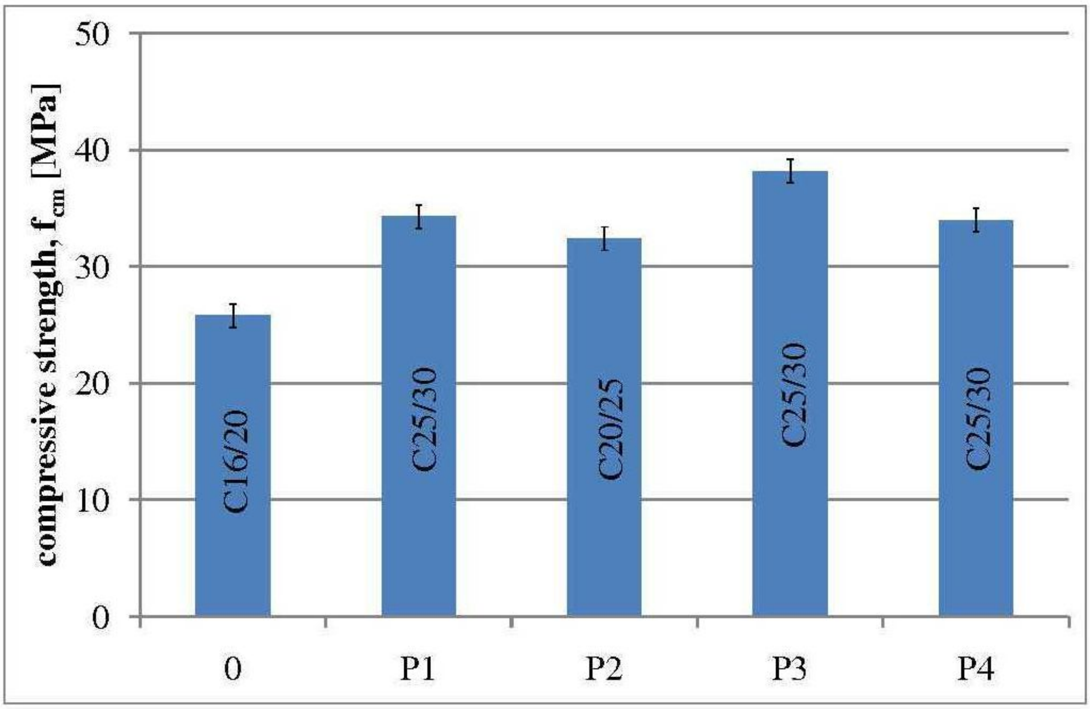

Fig. 5. Compressive strength by concrete with superplasticizer (C=320 kg/m^3)

Further, the water absorption was determined using cube specimens with a 100 mm side, five for each type of concrete. The water absorption by concrete was given in Fig. 6 (the amount of cement – 360 kg/m^3).

Table 3. The changes in consistency of concrete mixtures versus time

Superplasticizer	The amount of superplasticizer (% by weight of cement)	The amount of cement, [kg/m³]	Consistency of concrete mixtures, a) slump, S [mm], b) flow, F [mm] -time, t [min]-								
			5'	10'	40'	70'	100'	130'	160'	190'	220'
-	0,00	320	S 100	55	30	15	15	10	0	-	220
			F 433	416	362	357	353	342	331	-	-
P1	0,47	320	S 100	50	40	20	15	10	5	0	-
			F 453	430	387	380	360	350	347	340	-
P2	0,47	320	S 100	70	40	15	0	-	-	-	-
			F 447	427	390	383	350	-	-	-	-
P3	0,56	320	S 100	70	25	15	10	5	0	-	-
			F 480	437	400	393	357	350	350	-	-
P4	0,63	320	S 100	60	20	15	10	5	5	5	0
			F 447	410	380	370	370	360	337	320	303
-	0,00	360	S 100	50	30	15	10	0	-	-	-
			F 446	418	357	351	340	340	-	-	-
P1	0,36	360	S 100	40	30	15	15	10	5	0	-
			F 473	430	390	380	360	357	354	350	-
P2	0,42	360	S 100	55	25	15	5	0	-	-	-
			F 453	427	367	363	353	347	-	-	-
P3	0,52	360	S 100	50	25	15	10	10	10	5	0
			F 433	393	363	360	350	347	347	340	340
P4	0,66	360	S 100	60	20	10	10	5	5	0	-
			F 450	423	383	380	373	367	337	333	-

continued on the next page

-	0,00	400	S	100	45	25	20	10	5	0	-	-
			F	438	392	366	353	337	321	306	-	-
P1	0,28	400	S	100	30	20	15	10	10	5	0	-
			F	480	386	386	380	367	350	347	347	-
P2	0,38	400	S	100	30	20	15	10	5	0	-	-
			F	447	420	367	363	353	347	343	-	-
P3	0,50	400	S	100	60	35	10	10	5	5	5	0
			F	427	393	383	357	353	353	347	347	343
P4	0,69	400	S	100	60	30	20	15	5	0	-	-
			F	453	413	370	352	333	297	287	-	-

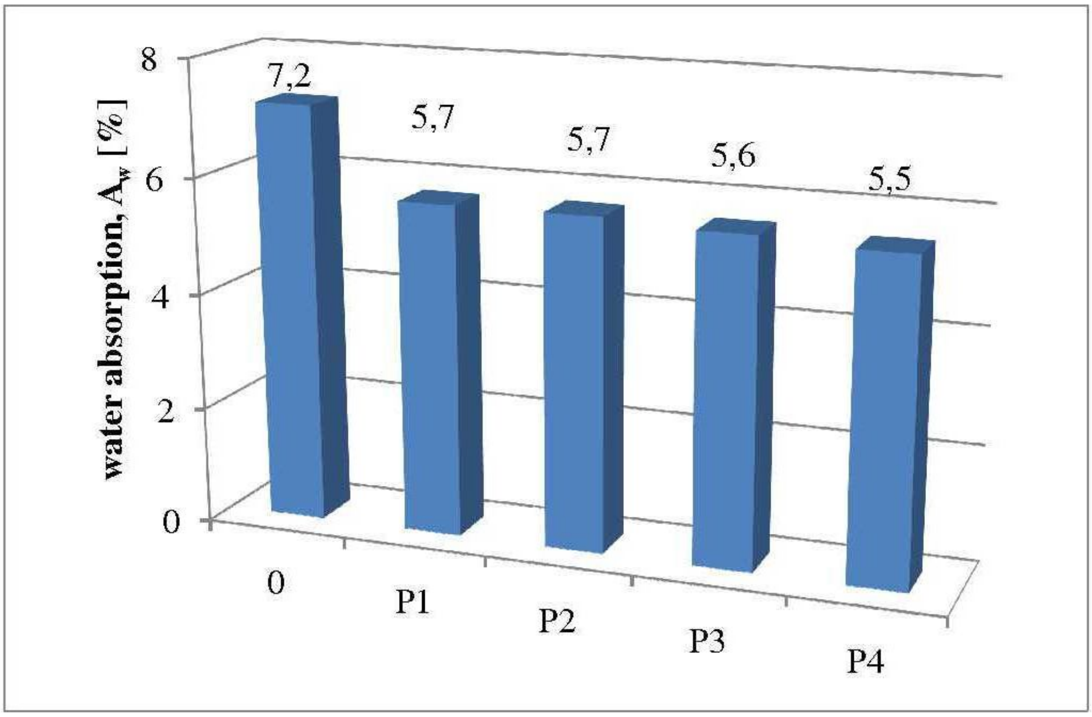

Fig. 6. Water absorption by concrete with superplasticizer (C=360 kg/m^3)

3. CONCLUSIONS

From the test carried out on material cement with superplasticiers SNF the following conclusions can be drawn.

From the research of grout plasticizing and consistency of concrete mixtures we can conclude that the most compatible with CEMI 32,5R Górażdże are superplasticizers P1 and P2. Those admixtures added in the amount of 0,5÷3% of cement mass allow to reduce the amount of water in the standard grout to 28%. The smaller quantities of those superplasticizers are needed to achieve the consistency ~ 100mm accordingly to the slump loss and ~ 450 mm accordingly to the slump flow than the two other superplasticizers P3 and P4.

The superplasticizers shorten the time from initial setting within 10-50 minutes and extend the time until final setting by 10 to 530 minutes. It depends on the chemical structure of superplasticizer.

The adding of superplasticizers P1, P2 and P3 reduces the shrinkage of mortars after 7 days by 18% to 44%, and after 28 days by 23% to 73% compared to control mortar. The modification with superplasticizer P4 increases the shrinkage of mortars after 7 and 28 days by 106% and 42%, respectively.

The superplasticizers affect the workability of concrete mixtures over time. The repeated dosage of those admixtures and applying them in the same amount as in the first dosage, significantly increases the fluidity of concrete mixtures by changing their consistency from S1 to S3 (slump test) and from F1 to F3 (flow test). It depends on the quantity of cement and the type of superplasticizer SNF.

The relatively strong linear relationship between the values of slump loss and flow as a function of time was observed. The coefficient of Pearson linear correlation is within the range 0,89-0,99.

The reduction of the amount of water in the concrete mixtures with superplasticizer, while maintaining the consistency of control concrete mixture, increases the compressive strength of concretes by 1-2 classes compared to the non modified concrete. It is possible also to save a certain amount of cement, maintaining the class of control concrete [5].

The water absorption of concretes modified with superplasticizers is by 20-23% lower as compared to the control concrete.

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OPTIMIZATION OF NATURAL ILLUMINATION AND THE SAVINGS OF ENERGY OF EDUCATIONAL ROOMS CONCERNING RECONSTRUCTION OF NATIONAL UNIVERSITY „LVIV POLYTECHNIC" BUILDINGS

ABSTRACT

A problem of windows reconstruction, that necessary to do with observance of operating normative requirements became actual. Low level of resistance of air permeability is conducted to significant periodic and non-uniform receipt in a premise cold and warm air during a year. However, high level of natural illumination without engineering means of regulation creates light discomfort in the rooms.

There is possible to reach optimization of conditions of natural illumination and the saving of energy in educational rooms due to rational reduction of windows size.

KEYWORDS: illumination, temperature, microclimatic aspect, thermal resistance, comfort

1. INTRODUCTION

One of the oldest academic technical schools in Europe and the first in Ukraine was technical academy in Lviv which has opened the doors in 1834, and later became National university "Lviv polytechnics". The academy has begun the full life after construction of the main case in 1874-1877. Technical academy developed. Built new cases in which building and architectural tendencies of the then Europe were displayed. It was the first stage of formation of the educational environment of academy. Buildings had good heat - protective properties and the sufficient area of windows which provided good illumination of audiences, laboratories, reading rooms. After the Second World War have been constructed many cases which carry to the second stage of formation of an educational zone.

On this stage are characteristic educational buildings of skeleton type with thin concrete panels, and tape windows with double glasses in one frame. Windows provide natural illumination which exceeds normative (300lux) and have low thermal resistance (0.39 $(m^2 x ^\circ C)/Wt$).

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The age of new cases comes nearer to 40 – 50 years. As the result a problem of their reconstruction, that necessary to do with observance of operating normative requirements and recommendations [1, 2, 3], became actual. Under a problem of reconstruction it is necessary to distinguish both constructive and microclimatic aspect which concern illumination and thermal mode.

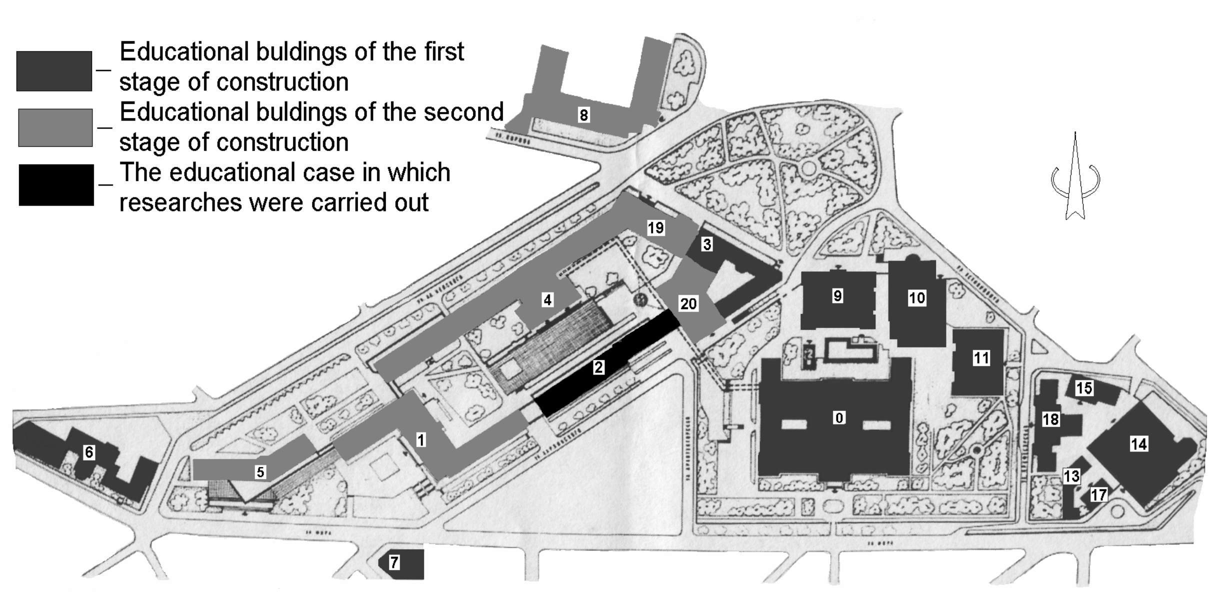

Fig. 1. The scheme of the general plan of an educational zone NU „ Lvov polytechnics”.
0 - the Main case; 1 – the educational case with an assembly hall with 1500 places; 2 - the case of Institute of construction and engineering of an environment, Institute of a geodesy; 3 – the educational case №3; 4 – the case of Institute of economy and management, Institute of applied mathematics and fundamental sciences; 5 – the educational case of Institute of computer sciences and information technologies, 6 – the educational case №6; 7 – the case of faculty of foreign languages; 8 - the educational case №8; 9 – Institute of chemistry and chemical technologies; 10 - the educational case №10; 11 – Institute of telecommunication, radio and electronic technics; 12, 13 - educational cases; 14 – Institute of mechanics and transport; 15, 16, 17 – educational cases; 18 – scientific library; 19 – technical library; 20 – student's club and a dining room.

2. PROBLEM

The educational buildings constructed in post-war years are characterized by greater size of windows. It conducts to greater losses of heat during the winter period of year and to greater receipts of heat in premises during the summer period. The low level of resistance of air permeability is conducted to significant periodic and non-uniform receipt in a premise cold and warm air during a year. The high level of natural illumination without engineering means of regulation creates light discomfort in the rooms, and greater losses of heat during the winter period and receipt of heat during the summer period creates thermal discomfort.

3. HYPOTHESIS

Authors suppose optimization of conditions of natural illumination and the saving of energy in educational rooms is possible to reach due to rational reduction of size of the windows. Researches of the formulated hypothesis were spent in four directions:

A. Natural measurements of illumination of educational rooms ;
B. Experimental researches of model of an audience on installation " the artificial sky ";
C. Graphic calculation of illumination according to A.M.Daniljuk's normative method [1];
D. Heat technical calculation of premises with the changed size of the windows

Researches were spent by the method of the comparative analysis for following variants of the sizes of the windows:

- Existing position (the basic variant) (figure 2.a) – area of windows F_{win}=22 m^2, it does $.\frac{F_{win}}{F_{wal}} = \frac{22}{39,6} \cdot 100\% = 55,5\%$ from area of a wall;
- reduction of the area of windows by partial closing from two parties– area of windows F_{win}=17,7 m^2, it does $.\frac{F_{win}}{F_{wal}} = \frac{17,7}{39,6} \cdot 100\% = 44,6\%$ from area of a wall;
- reduction of the area of windows by partial closing from four parties (figure 2.b) – area of windows F_{win}=13,4 m^2, it does $.\frac{F_{win}}{F_{wal}} = \frac{13,4}{39,6} \cdot 100\% = 33,8\%$ from area of a wall;
- variants of reduction of the area of windows by partial closing top (figure 2.c) and bottom (figure 2.d) parts by horizontal strips, thus the area of windows makes F_{win}=16, 5 m^2, it does $.\frac{F_{win}}{F_{wal}} = \frac{16,5}{39,6} \cdot 100\% = 41,6\%$ from area of a wall.

4. RESEARCHES

Natural illumination of rooms through the windows constantly changes. Therefore natural illumination characterizes a relative parameter – factor of natural illumination. This factor, *e*, expresses the attitude of illumination in room in a point of gauging E to simultaneous illumination of external point E which is on a horizontal plane shined by light of all firmaments.

$$e = \frac{E_{in}}{E_{ex}} \cdot 100\% \qquad (1)$$

Natural researches were spent in a typical educational room for 50 places of 2-nd educational case in the summer period of year with the cloudy sky, at absence of students. Windows of the room are focused on northwest. The area of a floor is equal F_{fl} =12mx6m =72m^2 (12mx6m). In this room there are four windows with double glasses the coupled binding. The sizes of one window are equal: 2,75 x 2,0 m. It is necessary to note, that in the others premises of the educational institution, despite of their purpose, are similar windows. Thickness of glass is 3 mm; the distance between glasses is equal 40mm.

In the room there are certain 35 settled points placed on the height of a conditional working surface, that it is accepted equal to the height of student's school desks (0,8m above a floor).

Measurements of factor of natural illumination were spent by means of measuring device.

As the gauge it is used the photodiode. The gauge moved on settlement points at a level of a conditional working surface.

Under the influence of natural illumination in the photodiode there is an electric current which force approximately proportional to intensity of an irradiation. Photocurrent measured by the microampermeter with accuracy 0,5 %. The measurements of natural illumination by means of the photodiode the attitude of illumination of a point in a room to illumination of an external point is replaced by the attitude of indications of the microampermeter in electric units without their translation in lux.

For an experimental research of natural illumination the model of the room has been made in scale 1 :15, at preservation of geometrical and lighting parameters. Researches were spent on installation « artificial sky » which represents a dome with diameter 3,6м with 36 fixtures on perimeter [4].

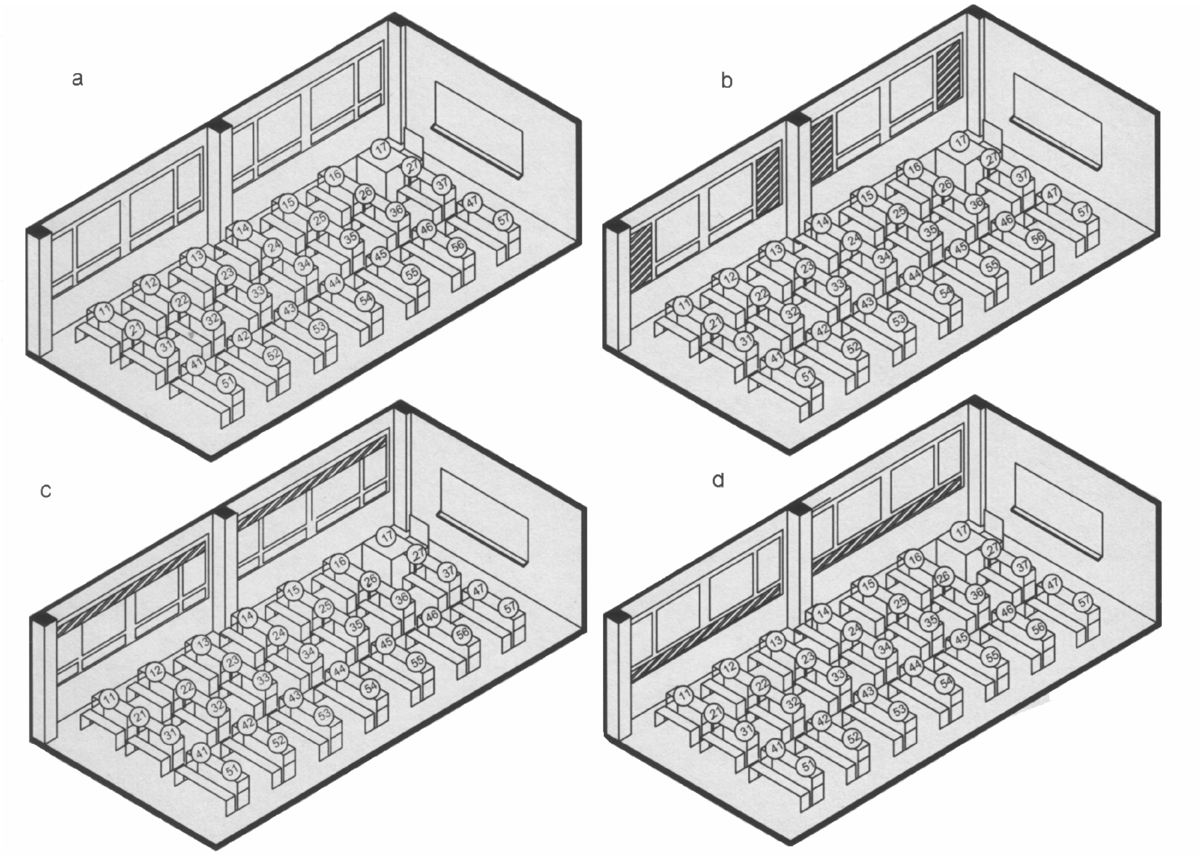

Fig. 2. The axonometrical scheme of the investigated audience with points in which measurements were spent: a – an existing variant of the sizes of windows; b – a variant of partial closing from four parties; c – reduction of windows by means of horizontal strips placed in the top part; d – reduction of windows by means of horizontal strips placed in the bottom part ; 11 … 57 – the first figure means number of row, the second – number of a point of measurement in a row.

Graphic calculation of illumination was spent by a normative technique [1] for a characteristic cut of a room. Thus A.M.Daniljuka's schedules were imposed on the plan and a cut of an educational room with following calculation of factor of natural illumination.

Results natural and experimental researches, and also results of graphic calculations are processed in the form of schedules of dependence factor natural illumination from depth of the room for points 16 … 56 (Figure 3).

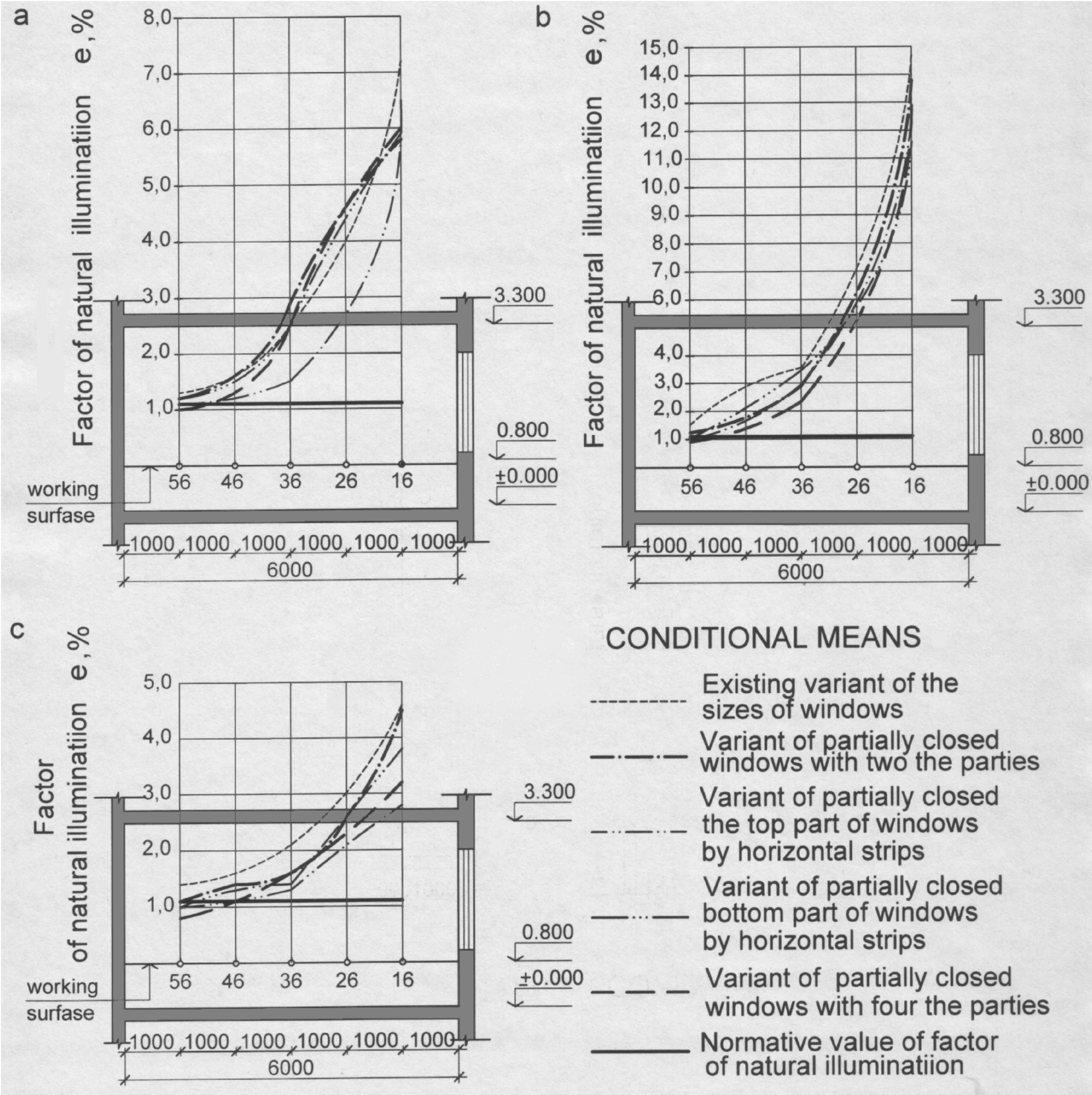

Fig. 3. Dependence of factors of natural illumination from depth of an educational room.
a - results of measurements of illumination of a premise in natural conditions; b - results of graphic calculation; c - results of measurements of illumination the educational rooms made on geometrical model;

On schedules identical laws which testify to correctness of methods of research are looked through. On the basis of the received results of researches of natural illumination it is possible to draw a conclusion, that the best are variants the reduction of window area by partial closing bottom (figure 2. d) and reduction by partial closing from two parties.

For heat-technical point of view the best is the variant with the minimal total thermal losses. Such there is a variant of lateral closing of each of two windows (figure 2 b). At this variant, total thermal losses will decrease on 13,5 % in comparison with an existing variant of the windows.

The reduction of the size of the windows by means of horizontal strips in the bottom part, thermal losses will be reduced to 12,4 %.

The reduction of the size of the windows by means of vertical strips from four parties, thermal losses will be reduced to 10.7 % concerning the existing size of windows. It is the least effective variant with heat technical point of view.

On the basis of lighting technical researches and heat technical calculations it is possible to draw a conclusion, that the optimal is a variant of reduction of the windows by means of horizontal strips placed in the bottom part (increase in height of a window sill).

5. CONCLUSION

- Constructed in 50 – 80 years of the last century educational cases of National university « Lviv polytechnics » do not answer conditions of light and thermal modes through greater sizes of the windows and low thermal resistance of protecting designs.
- Natural and experimental researches of natural illumination of a typical classroom, and also graphic calculations, have shown, that the area of windows can be reduced on 20 – 25 %;
- The analysis of the results of researches at different variants of change configuration and the area of the windows have shown, that the most uniform illumination is reached by closing the bottom part of the windows;
- Optimization of natural illumination by the reduction of the area of the windows will not worsen uniformity of clarification and simultaneously will reduce the losses of heat during the winter period of a year by 12,4 %.

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APPLICATION OF ALTERNATIVE FUELS FOR CEMENT PRODUCTION

ABSTRACT

In this publication a possibility of thermal utilization of alternative fuels in cement industry is presented. The replacement of natural fuels with secondary materials in the process of organized waste recycling is the most significant step in this direction. One should emphasize the concept of "organized recycling", i.e. executed consciously and above all with the earliest possible segregation of initial secondary materials. Such segregation must not only lead to separate of different materials, but should also be applied to secondary materials for the production of alternative fuels. Only then it will be efficient as regards both environmental protection and economics. The goal of lowering production costs is one of two main factors stimulating activities aimed at replacing the natural fuel used in the cement production process with alternative fuels obtained from combustible wastes. The second factor is environmental protection, for instead of undergoing potentially hazardous storage; wastes can be duly treated and rendered harmless in a useful way, with the total consumption of their energy. Cement kilns, in which combustion processes reach temperatures up to 2000°C, are one of the several industrial installations ensuring the effective and environmentally friendly combustion of alternative fuels.

KEYWORDS: alternative fuel, combustible wastes, thermal utilization, cement kiln

1. INTRODUCTION

High power-consuming of Portland cement clinker production, as well as constant rise in prices for fossil fuel, results in power use of alternative fuel from combustible industrial and communal wastes becomes one of main tendencies of cement industry heading toward production effectiveness increase. Considerable interest of cement industry to combustible wastes is conditioned, on one hand, by aspiration for cement production decrease in value,

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and on the other hand – by need of complicated large-tonnage waste utilization problem solving, which threaten environment.

Usage of rotary cement kilns is the most expedient for communal and industrial wastes utilization. At the same time, production of high-quality cement is a foreground job for cement plant. This means that not all wastes are burnt on cement plants. Alternative fuel from wastes must have properties which allow its usage in cement kilns. To such characteristics belong: physical state, calorific value, chemical composition, toxicity, ash content and its chemical composition, moisture content, homogeneity, possibility for reprocessing and transportation, density.

Putting wastes into cement kiln should not cause operating irregularities or influence negatively on cement quality. For normal cement kiln operation it is also important to calculate quantity of alternative fuel which will substitute natural fuel, regarding its calorific value, correctly. If fuel calorific value is low, necessity of required temperature maintenance in burning zone will be a limiting factor of its utilization. At higher alternative fuel calorific value (at the level of coal dust) presence of components, which lower cement clinker quality (for example, of heavy metals) and worsen consumption properties [1-3], can be a limiting factor.

Alternative fuel requirements are conditioned by production specificity and cement clinker properties, as well as economic use conditions in production process. First of all, fuel quality is characterized by its chemical composition, particularly, by signs of chemical elements additives content. Apart of main chemical elements, which form clinker minerals, cement clinker contains a number of elements, which are impurities in fuel or raw material. Taking process of cement clinker burning into account, alternative fuels with high calorific value, which is equivalent to natural fuel caloricity, is the most effective. Mean caloricity of industrial wastes in Europe varies in wide range – 8-10 MJ/kg [4, 5]. Combustion heat depends on combustible organic matter content. In case, when fuels have high caloricity, there are no limits for method of its putting into cement kiln. Calorific value of most fuels is lower in practice. Fuel caloricity of approximately 10 MJ/kg is considered as a lowest limit of alternative fuel calorific value, at which fuel energy is efficiently used in clinker burning process. Fuel caloricity is agreed with supplier and can vary in preset close limits. Burning of fuel with lower caloricity is possible but cost for its utilization exceeds its energy value for cement plants.

Alternative fuel must be delivered for burning in such a form which makes its feeding to the kiln possible. In some technologies for waste neutralization material dispersion is used, and coarse fraction is crushed. It allows energy intensity decreasing of all neutralization process. Too high content of fine fraction (0-10 mm), which contains ash and other mineral components, can sufficiently improve waste fuel properties. Instead, components of big size (over 100 mm) as a rule, need grinding.

Taking engineering process of clinker burning into account, different combustible wastes can be used in cement kilns. Depending on waste type and on cement kiln type, appropriate alternative fuel can be prepared, which will allow getting economical effect and will not cause environment threat increase. High temperature in the kiln and possibility of alternative fuel feeding in different places, allow burning almost any fuel. Though, taking into account temperature conditions, which are to be provided in cement kiln, waste calorific value must exceed 15 MJ/kg, to get high-quality clinker and required economic effect from alternative fuel use. Usage of alternative fuel with lower calorific value will cause kiln production decrease, unit heat use and quantity of emission gases increase. It doesn't concern wastes, which partially substitute raw material and fuel, as, for instance, coal production waste. In that case minimal calorific value is not technological limited, and its size testifies about additional economical effect. One more important criterion, which affects the possibility to use alternative fuel of different types, is process of cement production (wet or dry process) and kiln design [6-8].

During secondary material selection for alternative fuel production one must be guided by such criteria as accessibility (cheapness, availability in region, where it is planned to use alternative fuel, supply regularity), and homogeneity of physical-chemical parameters of secondary fuel materials for alternative fuel production.

2. RESULTS AND DISCUSSION

"Ivano-Frankivskcement" JSC is cement producing factory which is working since 1964. Yearly sale of finished cement products is approximately 700 thousand tons. Cement making process is high power-consuming (about 60% of costs is for fuel and electric power). Problem of cement production power intensity reduction is one of the primary tasks of cement industry and "Ivano-Frankivskcement" JSC in particular. In this connection in 2002 the factory (first in Ukraine) has changed to solid fuel of clinker firing (coal). In July 2008 kiln No. 3 was shifted to dry process of Portland cement clinker production (also first in Ukraine). At that, specific fuel consumption in kilns No. 1, 2 of wet process production in average is 220 kg/t of clinker, and of dry process kiln No.3 - 100-110 kg/t.

During recent 5-years on "Ivano-Frankivskcement" JSC in process of Portland cement clinker burning, different types of alternative fuels are introduced and for given period its volume reaches 15%. Alternative fuel is woodworking and oil-refining industry waste, peat, processed tires and mixes on its basis. Solid alternative fuel (sawdust, wood chipboard waste, peat) is delivered by motor transport to the storage. With the help of autoloader components are supplied to the tray, and then through inclined transporter to the screen, where coarse waste is separated. Further fuel is loaded to a drying drum with the help of belt conveyor, and then after drying in a cyclone, it is fed to dry fuel storage. Oil production and processing waste (oil-slime) are stored in the storage, equipped with heating system. From alternative preparation room, fuel and mixes by motor transport are transported to bins or to buffer store. From bins fuel is supplied to a star feeder with the help of conveyor system. From the feeder it is supplied to the rotary kiln by the system of air ducts. If there's no need of predrying, fuel can be supplied directly to the bins or to the buffer store. In the buffer store mixing of different fuel types is carried out (for example of peat and wood chipboard).

The most important characteristic of all fuel types is its caloricity. In the process of solid fuel adaptability evaluation, combustion heat, moisture, the volatile combustible matter content, ash content, granularity, spontaneous combustion and ability to maintain burning process are also considered. Sawdust caloricity sufficiently depends on moisture content W, %: thus, at W=5% calorific value Q=17,7 MJ/kg, and at W=40% calorific value decreases to Q=10,2 MJ/kg. So sawdust usage with moisture content more than 40% is economically unpractical.

In consideration of dry process kiln No. 3 productivity is 100 t/h of clinker at clinker formation heat consumption 3,3 MJ/kg during coal usage as main fuel (Q=25,6 MJ/kg), fuel saving under the condition of 10 % substitution with alternative fuel is 12,7 kg of coal per 1 ton of clinker. Similar is calculation of peat amount, which is necessary for substitution of 10% of coal at moisture content 7,5% and calorific value Q=14,9MJ/kg, is 21,9 kg. For mixes of wood chipboard waste: peat (1:3) with moisture content W=7,5% (peat Q=14,9 MJ/kg, chipboard Q=17,1 MJ/kg) at mix caloricity Q =15,5 MJ/kg coal substitution is 21,8 kg. For mix wood chipboard/oil-slime (1/1), which is necessary for substitution of 10% of coal (at moisture content 10%, oil-slime Q =25,2 MJ/kg, chipboard Q=16,6 MJ/kg), calorific value of Q mix is 20,9 MJ/kg, respectively coal substitution is 15,6 kg per 1 ton.

Secondary types of fuel can be used either in main burner of the rotary kiln or in secondary burner in raw meal calciner. Considering high temperature during clinker burning (about 1450ºC), which is necessary for reasons of quality, and required excess air, ideal conditions for alternative fuel burning are provided in the main burner. As temperatures, which are necessary for calcination, not necessarily must be so high, it is possible to use low-caloric or lump types of fuel in calciner secondary burner. On a plant of dry process production ("Ivano-Frankivskcement" JSC) together with above mentioned main methods of alternative fuel supply for burning in rotary cement kiln No.3, hot disk reactor method of secondary fuel supply is implemented (Fig. 1).

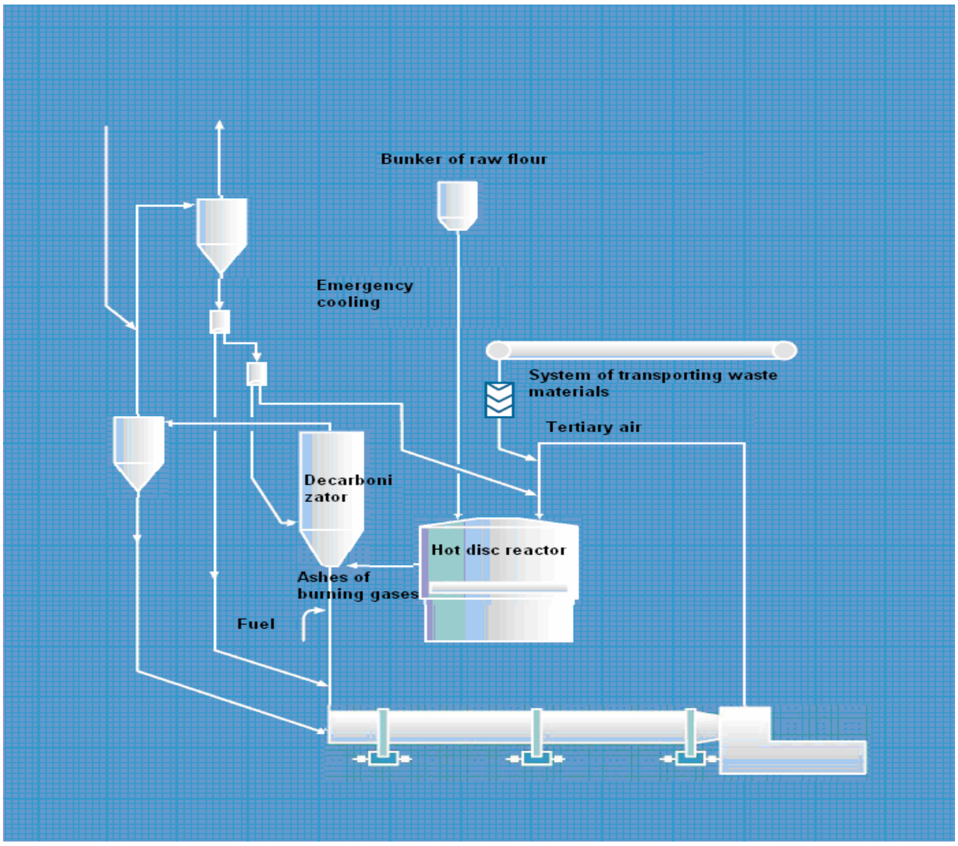

Fig. 1. Places of putting in different types of alternative fuels in cement kiln of dry process production

When using alternative types of fuel it is necessary to control temperature regimes of burning in order to avoid reducing atmosphere occurrence in kiln. In this case, clinker obtained from alternative fuel use is characterized by the most nonhomogeneous structure because of insufficient ash mixing with burned material. Chemical fuel under burning, which cause reducing atmosphere in the kiln, is accompanied by calcium alumoferrite decay because of reducing of iron to metal. Partial destruction of alite crystals which contain Fe_2O_3 in the form of solid solution and occurrence of CaS mineral in clinker are also observed. Clinker color is changing from normal green-black to brown at that.

Noncombustible fuel – ash affects the process of clinker burning, as it accumulates on material and changes its chemical composition. Ash from alternative fuel burning can contain: 3…10% Al_2O_3, 10…60% SiO_2, 1…10% Fe_2O_3, 5…10% CaO, 0,5…4% MgO, 5…30% SO_3. Since SiO_2, Al_2O_3, Fe_2O_3 prevails in content of secondary fuel material inorganic part, when ingress of ash on Portland cement clinker, its saturation factor decreases and silica module increases. Ash, which forms after peat burning, has different chemical composition, represented in general by next oxides: 30% SiO_2; 9% Al_2O_3, 7% Fe_2O_3, 46% CaO, 1% MgO, 3% SO_3, 0,6% K_2O, 0,08% TiO_2. That's why it is necessary to take ash amount into account during raw meal calculation to receive clinker with preset composition [7].

Tyre rubber is a valuable hydrocarbon raw material: its mean calorific value is 26 MJ/kg. Initial ignition mean temperature of tyre samples is 280ºC. According to differential thermal data, in temperature interval 350-500 ºC, main physicochemical transmutations in tyre burning take place accompanied by considerable mass change, which forms 60 mass % of overall loss during firing. An intensive endothermic effect at the temperature 475ºC corresponds to this process. Ash content in processed tyre samples is 5,84 mass %. Ash can have a certain effect on mineralogical composition of Portland cement clinker, which forms in process of raw material firing in cement kiln. Ash chemical composition is characterized by high content of ZnO and SO_3, which forms 42,0 and 26,0 mass % respectively. At the same time, heavy metals content in ash causes an additional problem. Thus, heavy metals are immobilized in clinker minerals structure. That's why tyre use in cement production has a significant advantage over its use for power generation, where the problem of ash utilization after waste burning is still not solved.

Ash difractogram analysis from processed tyre firing (Fig. 2) is evidence that ash examined is characterized by intensive lines of villemite (Zn_2SO_4) (d/n= 0,277; 0,260; 0,218 nm). During clinker firing on alternative fuel, ash which is added to raw material bench, influence on main clinker characteristics. Findings of chemical and mineralogical clinker content investigation is evidence that there is some calcium oxide content decrease (from 64,83 to 63,23 mass %), silicon oxide (from 21,45 to 21,09 mass %), aluminum oxide (from 4,59 to 4,58 mass %), iron oxide (from 3,99 to 3,86 mass %) and SF (from 0,93 to 0,92) in clinker, which is fired on alternative fuel. That is why when raw material bench calculating one proceed from increased SF value in re-calculation for its decrease in clinker to required limit.

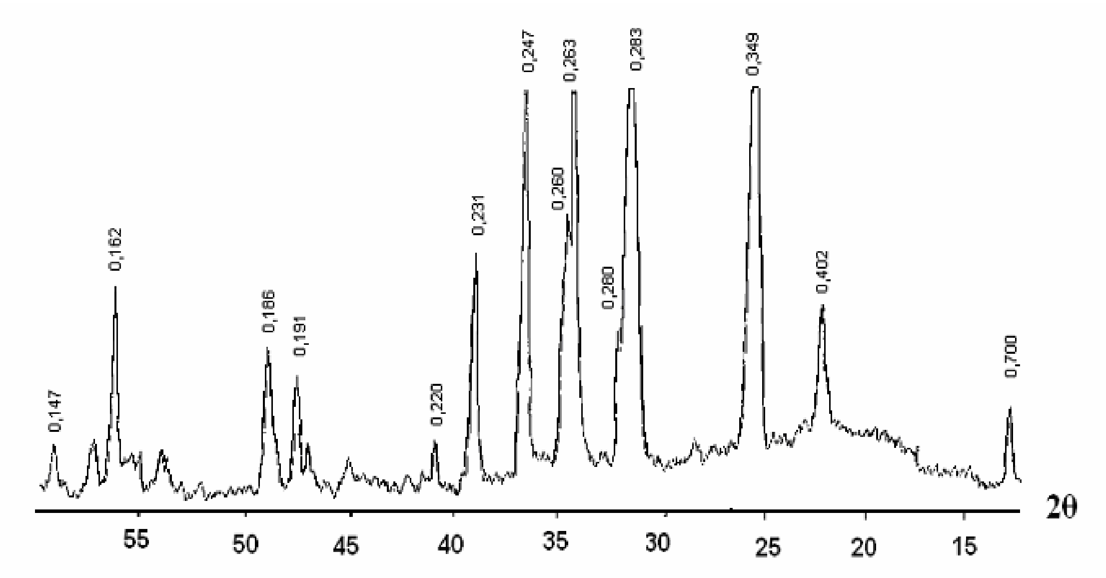

Fig. 2. Ash difractogram analysis from processed tyre firing

Optimization algorithm allows technical, economical and ecological properties analysis of alternative fuel and mathematical model development, which is the basis for alternative fuel properties modeling as well as for its process expert evaluation. In case of bicomponent solid alternative fuel usage (SAF) quantitative composition of wood chipboard waste (X1 - 5…25 mass %) and sawdust (X_2 – 10…40 mass %) are chosen to be an optimization criteria.

For SAF development optimum waste quantity selection, whose physicochemical parameters would correspond to norms, method of orthogonal centrally-composite design was used. Calorific value, sulfur content, chlorinity, lead, cadmium, arsenic, zinc and manganese content were chosen to be optimization criteria. On the basis of graphical interpretation of how sawdust and wood chipboard quantity affect the alternative fuel properties, it was determined that at the content of wood chipboard with quantity of 25 mass % and with sawdust quantity of 40 mass % , calorific value of SAF reaches 21,5 MJ/kg, which exceeds normative value by 6,5 MJ/kg. (fig. 3). Isoline analysis of elements content like sulfur, chlorine, lead, cadmium, arsenic, zinc, manganese indicate that its quantity in SAF does not exceed the normative value.

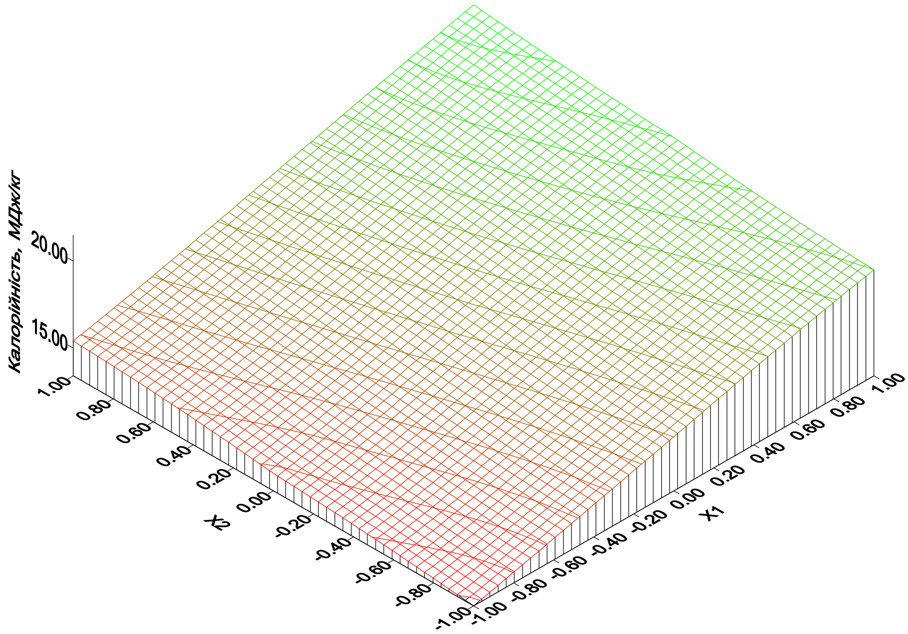

Fig. 3. Isoline analysis of SAF calorific value

Mix, obtained after mixing SAF, meets the requirements of calorific value and particular chemical elements content (table 1). It is determined by physic mechanical test of Portland cement on the basis of clinker fired on natural gas and mixed fuel (gas +alternative fuel), that strength of such Portland cement after 7 and 28 days of hardening does not differ a lot.

Table 1. Physicochemical analysis of SAF

Fuel type	Calorific value, MJ/kg	S*	Cl*	Pb	Cd	As	Mn	Zn
		mg/kg						
SAF	21,5	0,18	0,17	3,90	0,05	1,17	6,00	14,00
requirements	>15	<1	<1	<10	<1,5	<2	<10	<50

* - mass %

Energetic use of combustible waste in cement industry allows utilization of wide industrial and communal waste range in comparison with different utilization technologies (trash burning on combustion plants or its conversion to chemical industry goods), and it also brings to risk level decrease, connected with toxic polychlorinated dibenzo-twain-oxides and polychlorinated dibenzofuran emissions. Insertion of water-borne wastes sludge into rotary kiln during Portland cement clinker firing solves problem of such wastes utilization, and also brings to raw material resource economy during cement production [4-7].

Thus, chemical and mineralogical composition analysis of Portland cement clinkers, burned with experimental developed samples of alternative fuel, testifies that alternative fuel ash additive does not substantially influence on the processes of clinker formation in a cement kiln, and heavy metals which are contained in a negligible quantity in the products of combustion, are immobilized in the structure of Portland cement clinker minerals. Thus, necessary physico-mechanical properties of cement on the basis of clinkers, got with the use of alternative fuel are provided. Modern technologies of Portland cement clinker production using alternative fuel ensure traditional fuel costs decrease for Portland cement clinker production to 10-20%, which allows creation of progressive model in cement industry regarding clean production, and finds integrated solution for raw material and fuel-energy economy, providing for different industrial and domestic waste processing. This permits to deliver environment from significant waste amount, decrease greenhouse gasses emissions and toxic chlorine organic compounds to the atmosphere, as well as gives significant advantages to cement industry.

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THE DURABILITY PREDICTION OF DETERIORATING REINFORCED CONCRETE MEMBERS

ABSTRACT

The features of durability prediction and concepts of time-dependent reliability analysis of deteriorating reinforced concrete structures subjected to extreme gravity and lateral actions as rectangular pulse renewal processes are discussed. New methodological approaches to a time-dependent survival probability analysis and durability prediction of reinforced concrete members are considered. The safety margin of particular members is modeled as a random finite sequence. The effect of coincident recurrent extreme actions on their survival probabilities is analyzed. The instantaneous and long-term survival probabilities of particular and structural members of deteriorating structures are considered. It is recommended to calculate these probabilities by the unsophisticated analytical method of transformed conditional probabilities. The technical service life as a quantitative durability parameter of deteriorating structural members is studied.

KEYWORDS: survival probability, probabilistic durability, technical service life.

1. INTRODUCTION

The analysis of time-dependent reliability of deteriorating particular and structural members as their design on durability is indispensable in order to predict premature destructions or failures of load-carrying structures and to avoid economic and psychological losses and accidents. The durability of deteriorating materials and structures is the ability to preserve their efficiency for a long period of time with planned repairs and rebuilding taken into account. Higher durability requirements are applied to structural members when their erection, routine or preventive maintenance and strengthening or replacements require great efforts.

In many cases, the durability assessment and prediction of deteriorating structures subjected to short-term recurrent random extreme variable actions becomes one of main design task of engineers and users. An artificial deteriorating of materials caused by aggressive environmental actions and time-dependent reliability of structural members can be evaluated by several methods including some simplified and practical procedures based on diverse methodological features.

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The conservative durability assessment of deteriorating structures is based on implicit recommendations and is related with long-term experience. However, a wide range of applied durability issues can be neither formulated nor solved within deterministic analysis approaches. For the definition of the procedures of durability prediction of members, macro-, meso-, and micro- level design may be used. Using simplified calculation models, the meso-level (or material engineering level) may be recommended as a real design for durability predictions of concrete structures.

JCSS [1] presents a probability-based durability formulation taking into account non-stationary processes of ageing material and member strengths, external actions and loads resulting in reliability and durability calculations of structures. Vrowenvelder [2] and COST Action C12 [3] recommend to base their prediction analysis on rather complicated considerations of uncertainties of variable degradation processes of structures and their extreme action effects. In all cases, it should be expedient to base the time-dependent reliability analysis of structural members on the concepts of the intended service period as it is presented in CEB Bulletin 238 [4] and of the life span assessment recommended by ISO 2394 [5] and Melchers [6]. According to these concepts, not only the performance but also the survival probability values of structures are time-dependent random variables. In this case, the probability-based design on the durability and long-time reliability of structures are to a large extent similar. Besides, only probability-based approaches allow us explicitly predict basic and additional uncertainties of analysis models caused by inherent random variability, insufficient data and impressive knowledge of corrosion parameters.

According to Rackwitz [7] and fib Bulletin 34 [8], it is possible to introduce probabilistic approaches in design practice of deteriorating structures. However, the engineering modeling of time-dependent survival probabilities of reinforced concrete structures subjected to aggressive environmental actions and extreme live and climate loads are still unsolved. In many cases, the safety assessment and prediction of deteriorating structures subjected to short-term recurrent extreme variable actions becomes one of main design task of structural engineers.

This paper deals with some new methodological concepts on probability-based modeling of members of deteriorating reinforced concrete structures subjected to permanent loads and recurrent single or joint extreme service and climate actions.

2. DYNAMIC ANALYSIS MODEL

2.1 The resistance of deteriorating structural members

The structural members (beams, slabs, columns, joints) of buildings and civil engineering works are represented in design practice by their particular members (normal or oblique sections, connections) for which the only possible failure mode exists. The robustness and safety requirements of design codes should be satisfied for all particular members of structures. Structural members may be treated as auto systems of two or three not fully correlated particular members. However, multicriteria failure modes and survival probabilities of structural members under different limit states may be objectively assessed and predicted only knowing structural safety parameters of particular members.

The artificial ageing and deterioration of materials caused by aggressive environmental actions is very dangerous for load-carrying structures. Aggressive actions induced by concrete carbonation, chloride penetration and other chemical or physical attacks are the basic factors of deteriorating reinforced concrete structures. A corrosion protection of steel reinforcement depends on the density, quality and thickness of concrete covers and cracking intensities.

As it is known, aggressive actions induced by concrete carbonation, chloride penetration, chemical and physical attacks are the main factors of deterioration processes of reinforced concrete structures. It is expedient to divide the life cycle of deteriorating concrete structures as their physical and mechanical degradation processes [1] into the initiation, t_{in}, propagation, t_{pr}, and attack, t_{at}, phases (Fig. 1). A length of the initial phase, t_{in}, is a random variable depending on a feature of degradation process, environment aggressiveness and a quality of protected covers. The invulnerability of reinforced concrete structures may be characterized by the duration of this phase. When the degradation process of members is caused by intrinsic properties of materials, the phase $t_{in}\approx 0$. The corrosion initiation period comes to an end when a carbon dioxide front reaches a surface of reinforcement or a concentration of chloride ions becomes equal to its effective level. The resistance of members in initiation period, R_{in}, is presented as a fixed stationary random process. Its numerical values are random only at the beginning of process.

The propagation period, t_{pr}, is delayed for structures protected by coats. The structural safety limit state concerns the safety of people and the structure subjected to aggressive processes and extreme loadings separating a desired state of the structure from an adverse unreliable state.

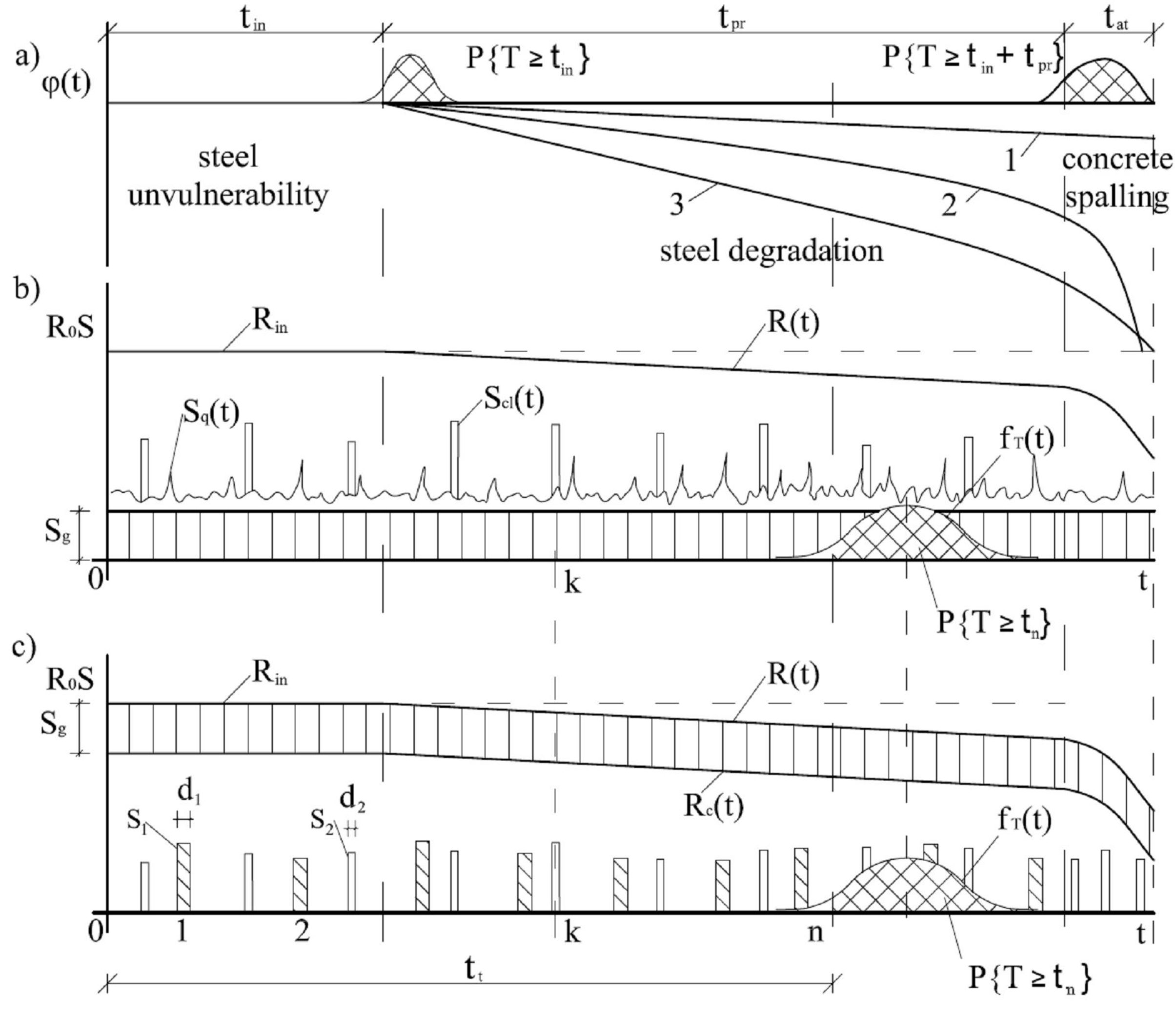

Fig.1. Degradation function φ(t) (a), actual (b) and conventional (c) dynamic models for time-dependent reliability analyse 1- unloaded members, 2 – loaded columns, 3 – loaded beams

The resistance of members in this period is treated as:

$$R(t) = R_{in}\varphi(t) = R_0\varphi(t) \tag{1}$$

where $\varphi(t)$ denotes the deterioration function depending on the rate of a resistance decrease induced by artificial ageing and degradation of materials.

The deterioration function of particular members for corrosion affected structures may be presented in the form:

$$\varphi(t) = 1 - \alpha(t - t_{in)})^{\beta} \tag{2}$$

here β depends on the degradation mechanism, defines a non-linearity of this function and α is a random deterioration factor. For typical degradation mechanism of a reinforced concrete members exposed to corrosion actions and sulphate attacks, the parameter β is close to 1.0 and 2.0 [9, 10].

The concrete spalling process during an attack phase is related with the concentration of corrosion products in normal and longitudinal cracks of members. These cracks are indesirable due to bond-anchorage losses and cross-section diminishes of reinforcing steel bars (rebars).

2.2 Extreme actions and their effects

Action effects of structures are caused by permanent loads g, sustained $q_s(t)$ and extraordinary $q_e(t)$ components of live loads, snow loads s(t) and wind, surf or seismic actions w(t). Extraordinary components $q_e(t)$ represent all kind of live loads which do not belong to sustained ones, i.e. $q_e(t)= q(t)- q_s(t)$.

According to Rosowsky & Ellingwood [11], the annual extreme sum of sustained and extraordinary live load effects $S_q(t)$ caused by $q_s(t)$ and $q_e(t)$ may be modelled as an rectangular pulse renewal process described by Type I (Gumbel) distribution of extreme values with the coefficient of variation $\boldsymbol{\delta}q=0.58$ and the mean $S_{qm}=0.47S_{qk}$, where S_{qk} is its characteristic value. The duration d_q of this annual extreme effect is fairly small and equal to 1-14 days for merchant and 1-3 days for other buildings [1].

It is proposed to model annual extreme snow and wind climate action effects by a Gumbel distribution with the mean values equal to $S_{sm}=S_{sk}/(1+k_{0.98}\boldsymbol{\delta}S_s)$ and $S_{wm}=S_{wk}/(1+k_{0.98}\boldsymbol{\delta}S_w)$, where S_{sk} and S_{wk} are the characteristic (nominal) values of action effects and $k_{0.98}$ is the characteristic fractile factor of these distributions [1,2,5,12]. The coefficients of variation of snow and wind extreme loads depend on the feature of geographical area and are equal to $\boldsymbol{\delta}s=0.3\text{-}0.7$ and $\boldsymbol{\delta}w=0.2\text{-}0.5$.

The durations of extreme floor and climate actions are: d_q=1-14 days for merchant and 1-3 days for other buildings, d_s=14-28 days and d_w=8-12 hours. Renewal rates of annual extreme actions are equal to λ=1/year. Therefore, the recurrence number of two joint extreme actions during the design working life of structures, t_n in years, may be calculated by the formulae:

$$n_{12} = t_n(d_1 + d_2)\lambda_1\lambda_2 \tag{3}$$

where $\lambda_1 = \lambda_2 = 1/t_\lambda$ are the renewal rates of extreme loads.

Thus, the recurrence numbers of extreme concurrent live or snow and wind loads during t_n=50 years period are quite actual and equal to n_{qw}=0.2-2.0 and n_{sw}=2.0-4.0. The bivariate distribution function of two independent extreme action effects may be presented as their conventional joint distribution function with the mean $S_{12m}=S_{1m}+S_{2m}$ and the variance $\boldsymbol{\sigma}^2 S_{12}= =\boldsymbol{\sigma}^2 S_1+ \boldsymbol{\sigma}^2 S_2$ [13].

3. THE SURVIVAL PROBABILITY OF PARTICULAR MEMBERS

3.1 The safety margin of particular members

According to probability-based approaches (design level III), the time-dependent safety margin of deteriorating particular members exposed to extreme action effects may be defined as their random performance process and presented as follows:

$$Z(t) = g[\boldsymbol{X}(t), \boldsymbol{\theta}] = \theta_R R(t) - \theta_g S_g - \theta_q S_1(t) - \theta_q S_2(t) \tag{4}$$

Here **X**(t) and **θ** are the vectors of basic and additional variables, representing respectively random components (resistances and action effects) and their model uncertainties. The mean values and standard deviations of additional variables are: θ_{Rm}=0.99-1.04, $\boldsymbol{\sigma}\theta_R$=0.05-0.10 [14] and $\theta_{gm}\approx 1.0$, $\boldsymbol{\sigma}\theta_g \approx 0.1$ [1].

According to ISO 2394 [5] and EN 1990 [15] recommendations, Gaussian and lognormal distribution laws is to be used for member resistances. The permanent actions can be described by a normal distribution law [5, 15]. Therefore, for the sake of simplified but quite exact probabilistic analysis of deteriorating members, it is expedient to present Eq. (4) in the form:

$$Z(t) = R_c(t) - S(t) \tag{5}$$

Here

$$R_c(t) = \theta_R R(t) - \theta_g S_g \tag{6}$$

is the conventional resistance of members the bivariate probability distribution of which may be modeled by a Gaussian distribution,

$$S(t) = \theta_q S_1(t) + \theta_q S_2(t) \tag{7}$$

is the conventional bivariate distribution process of two stochastically independent annual extreme effects with the mean $S_m(t)=\theta_q S_{1m}(t)+\theta_q S_{2m}(t)$ and the variance $\boldsymbol{\sigma}^2 S(t)=\boldsymbol{\sigma}^2(\theta_q S_1(t))+\boldsymbol{\sigma}^2(\theta_q S_2(t))$.

Inspite of analysis simplifications, the use of continuous stochastic processes of member resistances considerably complicates the durability analysis of deteriorating structures exposed to intermittent extreme gravity and lateral variable actions along with permanent ones. The dangerous cuts of these processes correspond to extreme loading situations of structures. Therefore, in design practice the safety margin processe (5) may be modelled as a random geometric distribution and treated as finite decreasing random sequence:

$$Z_k = R_{ck} - S_k, \; k=1, 2, \ldots, \; n-1, n, \tag{8}$$

where $R_{ck}=\varphi_k\theta_R R_{in}-\theta_g S_g$ is the member conventional resistance at the cut k of this sequence (Fig. 1) and n is the recurrence number of single or coincident extreme action effects, S_k, is given by Eq.(7).

When extreme action effects are caused by two stochastically independent variable actions, a failure of members may occur not only in the case of their coincidence but also when the value of one out of two effects is extreme. Therefore, three stochastically dependent safety margins should be considered as follows:

$$Z_{1k} = R_{ck} - S_{1k}\,,\ k=1, 2,\ldots,n_1, \tag{9}$$

$$Z_{2k} = R_{ck} - S_{2k}\,,\ k=1, 2,\ldots,n_2, \tag{10}$$

$$Z_{3k} = R_{ck} - S_{12k} = R_{ck} - S_{1k} - S_{2k}\,,\ k=1, 2,\ldots,n_{12}, \tag{11}$$

where the number of sequence cuts n_{12} is calculated by Eq. (3).

3.2 Instantaneous and long- term survival probability of particular members

In many cases, the conventional resistance $R_{ck}=R_k-S_{gk}$ and the action effect $S_k=S_{1k}+S_{2k}$ of members may be treated as statistically independent variables. Thus, the instantaneous survival probability of members at k-th extreme situation, assuming that they were safe at the situations 1,2,…,k-1, is calculated by the formula:

$$\boldsymbol{P}_k = \boldsymbol{P}\{R_{ck} > S_k \exists k \in [1, n]\} = \int_0^{\infty} f_{R_{ck}}(x) F_{S_k}(x)dx \tag{12}$$

Here $f_{R_{ck}}(x)$ is the density function of member resistance R_{ck} and

$$F_{S_k}(x) = \exp\left[-\exp\left(\frac{S_{km} - x}{0.7794 \times \boldsymbol{\sigma} S_k} - 0.5772\right)\right] \tag{13}$$

is the cumulative distribution function of the action effect S_k the mean and standard deviation of which are S_{km} and $\boldsymbol{\sigma}S_k$.

Usually, the decreasing stochastically dependent instantaneous survival probabilities of members form series systems. The time-dependent survival probability of members as series systems may be calculated by Monte Carlo simulation and numerical integration methods. However, it is more reasonable to use the unsophisticated simplified but quite exact method of transformed conditional probabilities [13].

When the resistance of members is a non-stationary process, the long-term survival probability of particular members may be written in the form:

$$\boldsymbol{P}_i = \boldsymbol{P}_i\{T \geq t_n\} = \boldsymbol{P}\left\{\bigcap_{k=1}^{n}(Z_k > 0)\right\} = \prod_{k=1}^{n}\boldsymbol{P}_k\left[1 + \rho_{n,n-1\ldots1}^{x_n}\left(\frac{1}{\boldsymbol{P}_{n-1}} - 1\right)\right] \times \ldots$$
$$\times\left[1 + \rho_{k,k-1\ldots1}^{x_k}\left(\frac{1}{P_{k-1}} - 1\right)\right] \times \ldots \times \left[1 + \rho_{21}^{x_2}\left(\frac{1}{P_1} - 1\right)\right] \tag{14}$$

Here P_k is the instantaneous survival probability by (12):

$$\rho_{kl} = Cov(Z_k, Z_l)/\boldsymbol{\sigma}Z_k \times \boldsymbol{\sigma}Z_k \approx \varphi_k\varphi_l / \left[1+\boldsymbol{\sigma}^2\left(\theta_{q_i}S_i\right)/\boldsymbol{\sigma}^2\left(\theta_R R_{ck}\right)\right] \quad (15)$$

is the coefficient of auto correlation of rank safety margin components, $Cov(Z_k,Z_l)$ and $\boldsymbol{\sigma}Z_k$, $\boldsymbol{\sigma}Z_l$ are an auto covariance and standard deviations of these components. The correlation factor of deteriorating series system elements or decreasing sequence cuts may be expressed:

$$\rho_{k,k-1\ldots1} = \left(\rho_{k,k-1} + \ldots + \rho_{k1}\right)/(k-1) \quad (16)$$

The bounded index "x_k" of correlation factors of random sequence cuts may be expressed as:

$$x_k \approx \left[(4.5+4\rho_k)/(1-0.98\rho_k)\right]^{1/2} \approx \left[8.5/(1-0.98\rho_k)\right]^{1/2} \quad (17)$$

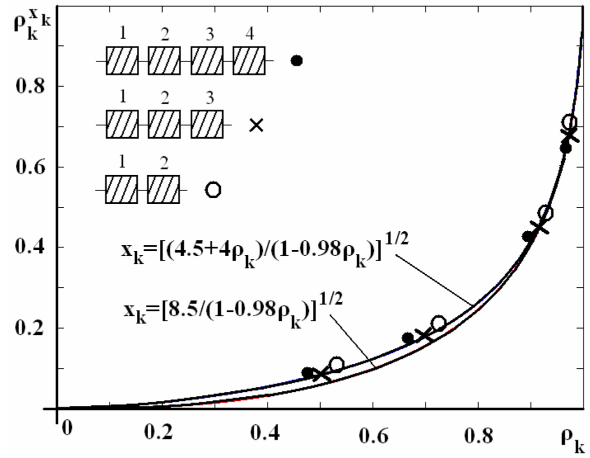

Fig 2. The indexed correlation factor $\rho_k^{x_k}$ of series systems versus its basic value ρ_k

Taking into account three safety margins from Eqs. (9)-(11), the integrated survival probability of particular members as series stochastic systems may be calculated by the formula:

$$\begin{aligned} P_s &= P_s\{T \geq t_n\} = P\{Z_1(t) > 0 \cap Z_2(t) > 0 \cap Z_3(t) > 0\} = \\ &= P_1P_2P_3\left[1+\rho_{3,21}^{x_3}\left(\frac{1}{P_2}-1\right)\right] \times \left[1+\rho_{21}^{x_2}\left(\frac{1}{P_1}-1\right)\right] \end{aligned} \quad (18)$$

where the ranked survival probabilities $\mathbf{P}_1 > \mathbf{P}_2 > \mathbf{P}_3$ of members exposed to different extreme load effects are expressed by Eq. (14); ρ_{21} and $\rho_{3,21}=0.5(\rho_{31}+\rho_{32})$ are the coefficients of rank cross-correlation of the safety margins defined by Eqs. (9)-(11).

The survival probability of members may be introduced by the generalized reliability index

$$\beta = \Phi^{-1}(\boldsymbol{P}_s) \quad (19)$$

where $\Phi(\bullet)$ is the cumulative distribution function of the standard normal distribution tabulated in statistics texts. According to Eurocode EN 1990 [15] and other international codes or standards, the target reliability index β_{min} of members depends on their reliability classes by considering the human life, economic, social and environmental consequences of failure or malfunction. For persistent design situations during 50 year reference period, the values of β_{min} are equal to 3.3, 3.8 and 4.3 for reliability classes RC1, RC2 and RC3 of structural members. The value of β_{min} for particular members should be not less.

4. TECHNICAL SERVICE LIFE PREDICTION FOR STRUCTURAL MEMBERS

4.1 The survival probabilities of structural members

The technical service life as the lifetime at preset target reliability index of deteriorating reinforced concrete members is the period for which it can actually perform according to the service requirements based on an intended purpose without major repairs [3]. In any case, the technical service life, t_t, of members comes to an end before the beginning of concrete spalling process at an attack phase (Fig. 1).

The minimum values for reliability index β_{min} associate with the structures or structural members. Therefore, the durability prediction of structures should be considered for beams, columns, slabs, piles, joints and other structural members as autosystems representing their multicriteria failure mode due to various action effects and responses of particular members. A necessity to use autosystem models in design practice is illustrated in Fig. 2.

According to the method of transformed conditional probabilities, the total survival probability of structural members as series, parallel and mixed autosystems may be respectively calculated by the Equations:

$$\mathbf{P}_{ser} = \mathbf{P}_{ser}\{T \geq t_n\} = \mathbf{P}\{Z_1 > 0 \bigcap Z_2 > 0\} = \mathbf{P}_1\mathbf{P}_2\left[1 + \rho_{12}^{x}\left(\frac{1}{\mathbf{P}_{1/2}} - 1\right)\right] \tag{20}$$

$$\mathbf{P}_{par} = \mathbf{P}_{par}\{T \geq t_n\} = \mathbf{P}\{Z_1 > 0 \bigcup Z_2 > 0\} = \mathbf{P}_1 + \mathbf{P}_2 - \mathbf{P}_1\mathbf{P}_2\left[1 + \rho_{12}^{x}\left(\frac{1}{\mathbf{P}_{1/2}} - 1\right)\right] \tag{21}$$

$$\mathbf{P}_{mix} = \mathbf{P}_{mix}\{T \geq t_n\} = \mathbf{P}\{Z_1 > 0 \bigcup Z_2 > 0 \bigcap Z_3 > 0\} = \mathbf{P}_{par}\mathbf{P}_3\left[1 + \rho_{3,21}^{x}\left(\frac{1}{\mathbf{P}_{3/par}} - 1\right)\right] \tag{22}$$

Here $\mathbf{P}_{1/2}$ and $\mathbf{P}_{3/par}$ are the greater value from the probabilities $\mathbf{P}_1$, $\mathbf{P}_2$ and $\mathbf{P}_3$, $\mathbf{P}_{par}$ calculated by Eqs. (18) and (21); $\rho_{3,21} = 0.5(\rho_{12} + \rho_{13})$ is the coefficient of rank cross-correlation.

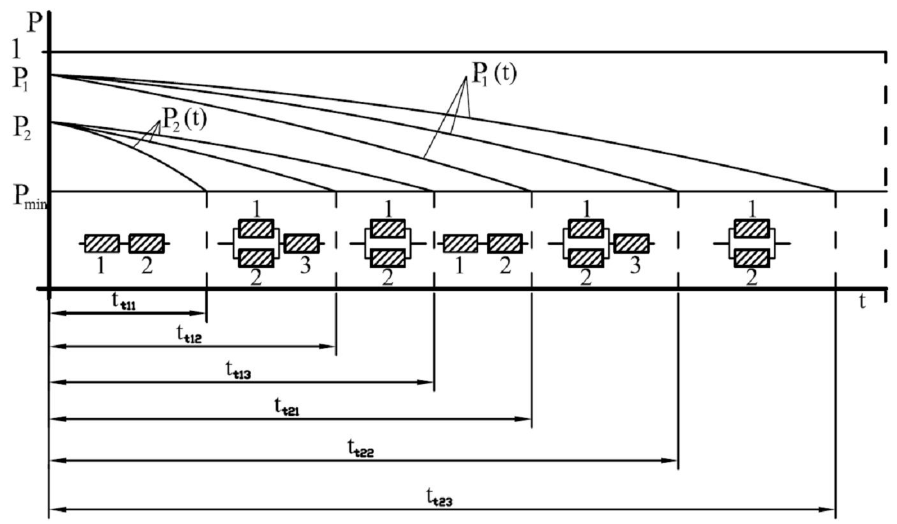

Fig 3. Effect of autosystem types and initial survival probabilities on the technical service life t_t of structural members

The technical service life t_t as a quantitative durability parameter of deteriorating structural members may be calculated from Eqs. (20)-(22) when $t_n=t_t$. The computation is iterated until the value t_t corresponds the target probability condition:]

$$\beta = \Phi^{-1}\left[\mathbf{P}(T \geq t_t)\right] = \beta_T \tag{23}$$

4.2 Numerical illustration

We consider the structural safety prediction of deteriorating single reinforced concrete beams of a flat roof resisting to bending moments M_g and M_S caused by dead and snow loads. The corrosion of reinforcing steel bars (rebars) is caused by internal chloride ions. Therefore, an initiation corrosion period $t_{in}\approx 0$.The statistical parameters of their initial resistances are: $(\theta_R R_0)_m=R_{0m}=474$ kNm, $\boldsymbol{\sigma}^2(\theta_R R_0)=2808(\text{kNm})^2$. The deterioration function is: $\varphi=1-\alpha t^{\beta}$, where $\beta=1.0$, $\alpha_m=0.00125$.

The statistical parameters of bending moments are: $(\theta_m M_g)_m=M_{gm}=140$ kNm, $\boldsymbol{\sigma}^2(\theta_g M_g)=\boldsymbol{\sigma}^2 M_g=155$ $(\text{kNm})^2$, $(\theta_m M_s)_m=M_{sm}=60$ kNm, $\boldsymbol{\sigma}^2(\theta_g M_s)=\boldsymbol{\sigma}^2 M_s=576$ $(\text{kNm})^2$.

According to Eq.(18), the long-term survival probability and reliability index of roof beams for 30 year reference period are: **P**=0.9994 and $\beta=3.24$.

5. CONCLUSIONS

The probabilistic reliability prediction as one of the main design tasks is indispensable in order to guarantee a time-dependent performance of deteriorating reinforced concrete structures subjected to recurrent extreme episodic actions and to avoid unexpected sudden failures. The strategy of this prediction should be based on the concept that not only a performance but also a safety margin of their particular members is time-dependent random variables.

In this case, it is not complicated to predict time-dependent reliability indices of deteriorating particular members (sections, connections) as series systems by simplified but rather exact probability-based approaches. The safety margins of members the resistances of whose are considered as non-stationary processes, should be presented as random finite decreasing sequences. The cuts of these sequences correspond to extreme loading situations of structures. The survival probabilities of particular members may be calculated by the simplified method of transformed conditional probabilities, when they are exposed to one and two extreme action effects, respectively.

The durability prediction of deteriorating reinforced concrete structural members (beams, columns, slabs, piles) should be based on their technical service life concept and reliability index analysis from Eq. (23). The presented methodology on durability prediction of structural members may help engineers having a minimum appropriate skill and experience to calculate the technical service life of deteriorating reinforced concrete members.

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ZESZYT NAUKOWY POLITECHNIKI RZESZOWSKIEJ Nr 265
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OPTIMIZATION OF PROPERTIES OF CLAY SOLUTIONS ON THE BASIS OF THE UKRAINIAN BENTONITES

ABSTRACT

The article is devoted to the development of Ukrainian betonites use technology at no-dig lying of engineering communications by a horizontal-directed boring method, via special solutions upgrading and laying technology designing.

As a result of implementation of experimental research experimentally-statistical models are got. These models characterize influence of the varied factors on the technological rates of the special solutions on the basis of the Ukrainian bentonites: density of clay solutions, filtration-loss quality, and thickness of clay filter cake, viscosity, and yield point. The analysis of technological properties of such solutions modified by chemical additions is carried out.

On results of experimental research the choice of optimum compositions of the special solutions on the basis of the Ukrainian bentonites is carried out. A mathematical modeling was used for this purpose.

KEYWORDS: Ukrainian bentonites, horizontal directional drilling, optimization.

1. INTRODUCTION

A contemporary city requires constant development of underground communications. One of the most effective methods for construction of utility networks is a horizontal directional drilling (HDD). A special clay solution is to be used without ditching.

The correct choice of a drilling mud and use of special reagents are very important. It influences the process of HDD, the arrangement of the cored hole. It serves the time minimization and prevents accidents related to the sustainability of drillable ground. The right selection will provide high quality, reasonable cost and environmental safety.

However in Ukraine there is a problem because the spread of highly efficient method for trenchless laying of utilities by HDD is limited. Special mud powders – bentonites that are intended for use in this method cost much. They come into the Ukrainian market from the Czech Republic, France and even from the United States [1].

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But Ukraine has its own deposit of bentonite clays suitable for use in the trenchless technology. However the properties of Ukrainian bentonites have not been studied yet.

Drilling mud is characterized by a sufficiently large number of technological parameters, the analysis of which must be labor-intensive. Basing on research experience, a well-established method of experimental and statistical modeling was applied in the work [2].

The theory of planning experiments is used in order to reduce the number of experiments [2, 3].

The experiments were performed on the 18 point plan given below.

The content of bentonites varied during the studies, (X_1 = 51±5g at 1dm^3 mud powder) as well as the number of two types of chemical additives. Dosage of additives Prim-PLUS produced by Polish company HEAD was within X_2 = 0.7±0.7%. The amounts of additives FD-134 of French production varied within X_3 = 0.5±0.25%.

As part of the study process basic properties of clay solutions obtained from the Ukrainian deposits of clay, modified with chemical additives, were analyzed. These studies concluded optimization of special solution compositions based on the Ukrainian bentonites. The method of mathematical modeling was chosen as the most appropriate one.

Some of these solution compositions are considered below.

2. THE DENSITY OF CLAY SOLUTIONS (G)

Experimental and statistical model of changes in the density of the clay solution is of the form Eqn.1.

$$G = 1.026 - 0.004x_1 + 0.002\,x_1^2 - 0.002x_1x_2 \quad \bullet \quad \bullet \quad \bullet \quad + 0.001x_1x_3 + 0.002x_3 \quad \bullet \tag{1}$$

As shown in Fig. 1 (obtained from Eqn.1), changes in density of the clay solution mainly occurs due to changes in the amount of bentonite clay, which determines the slurry yield.

Fig. 1 shows, that additive FD-134 and additive PRIM PLUS don't have a significant impact on the change in density of the solution. But it must be born in mind that the maximum value of density of the clay solution, equal to 1.035 kg/m3, is achieved by maximum doses of both additives. The minimum density is reached at x_2 = 1 and x_3 =- 1.With increasing dosage of additives PRIM PLUS the density of clay solution increase slightly.

3. WATER LOSS (WL)

Based on the results of the experimental values of water loss, in the course of a mathematical treatment, an experimental-statistical model of the investigated impact analysis on its value was obtained (Eqn. 2.).

$$WL = 18.22 + 3.53x_1 + 3.87x_1^2 \quad \bullet \quad + 4.03x_2x_3 + 1.9x_2 + 4.73x_2^2 + 3.3x_1x_3 \quad \bullet \quad \bullet \tag{2}$$

Applying this model, a graph was constructed (Fig-2), which is reflecting the target variations through the use of solution modifying additives and through amount changes of bentonite powder.

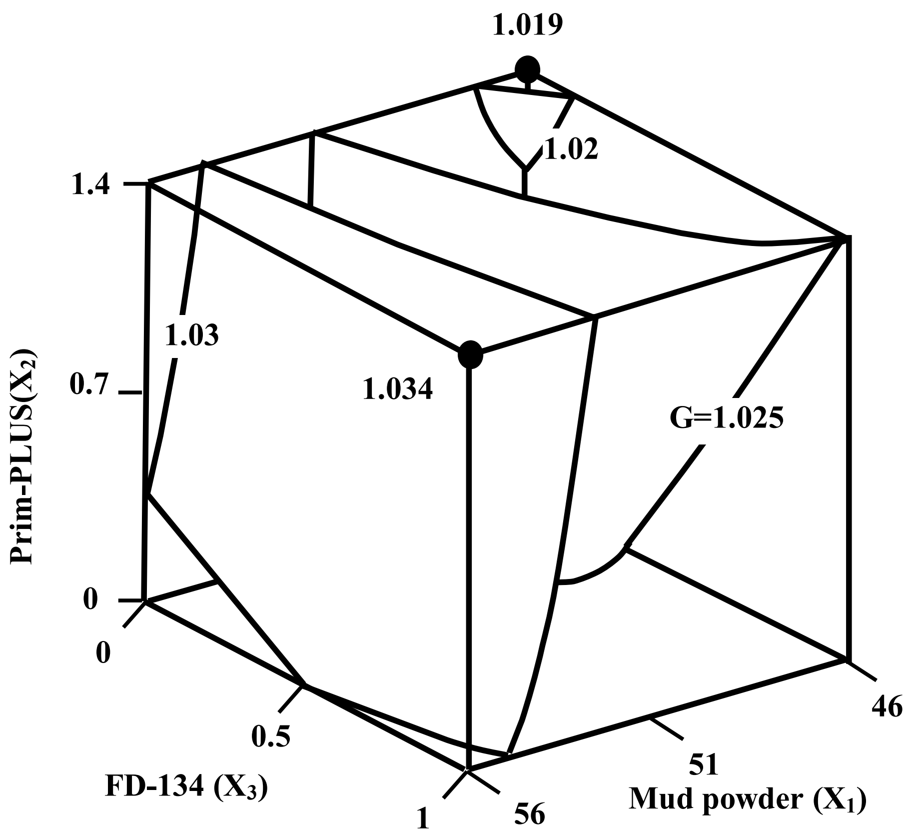

Fig. 1. The density variation of clay solutions

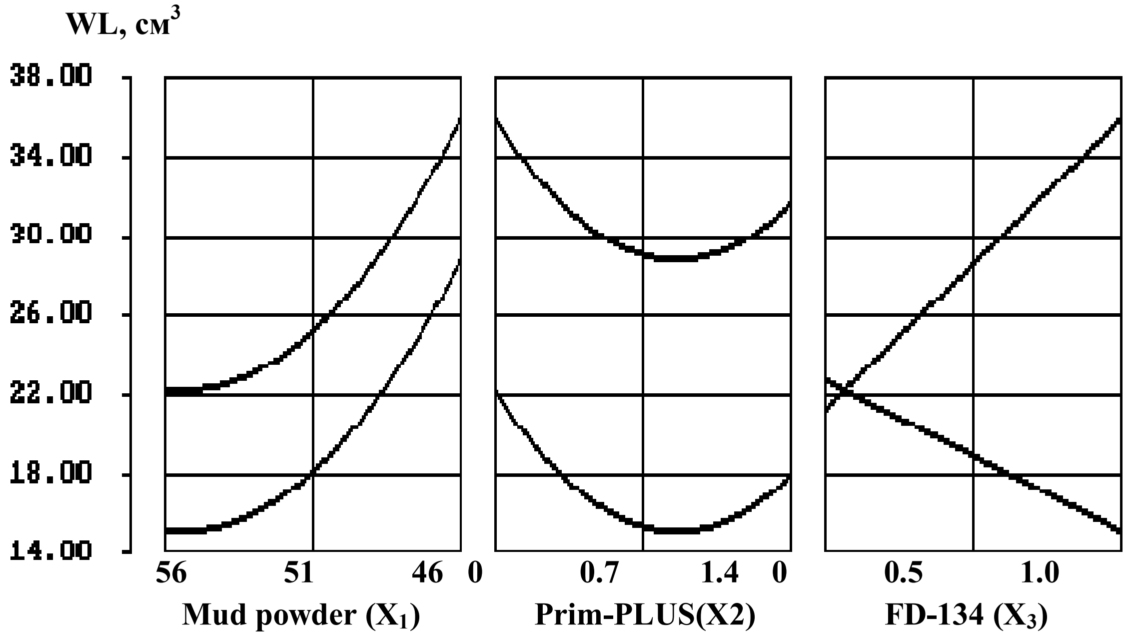

Fig. 2. Change of water loss

One can see from the Fig. 2. that the first change of water loss occurred with decreasing of the water quantity from 56g (x_1 = - 1) to 51g (x_1 = 0). The minimum target of water loss change their coordinates drastically. If the amount of mud powder was equal to 56gr. (x_1 =- 1,) the lowest index of water loss was achieved when the amount of additive FD-34 = 1% of the mud powder mass (so by x_3=1). If the amount of mud powder was equal to 51gr. (x_1 = 0), these values are already achieved without entries of additive FD-34 I (x_3 = -1).

As the amount of mud powder decreases from 56 to 46gr, the maximum target of water loss moves from the area, in which the additive FD-34 is absent (x_3 = -1), to the region where x_3 = 1 (FD-34 - 1%). Therewith the target of water loss is increasing too.

4. RELATIVE VISCOSITY (RV)

Analyzing the results of the relative viscosity test by the Marsh funnel, it proved that the variations of all the three investigated factors impact the property in different ways. The experimental and statistical model (Eqn. 3) received by processing of experimental results is given below.

$$\begin{aligned} RV = 143.3 &+ 7.2x_1 - 19.7x_1^2 - 8x_1x_2 - 11.8x_2x3 \\ &+ 22.2x_2 - 12.6x_2^2 \quad \bullet \\ &+ 8.6x_3 \quad \bullet \end{aligned} \tag{3}$$

The most significant increase in relative viscosity occurs due to increase in dosing rate of the additive Prim-PLUS. Within the range it changed the investigated property thrice.

As the two-factor reliability model shows, the additive Prim-PLUS exercises the highest influence on the relative viscosity of the solution (Fig. 3).

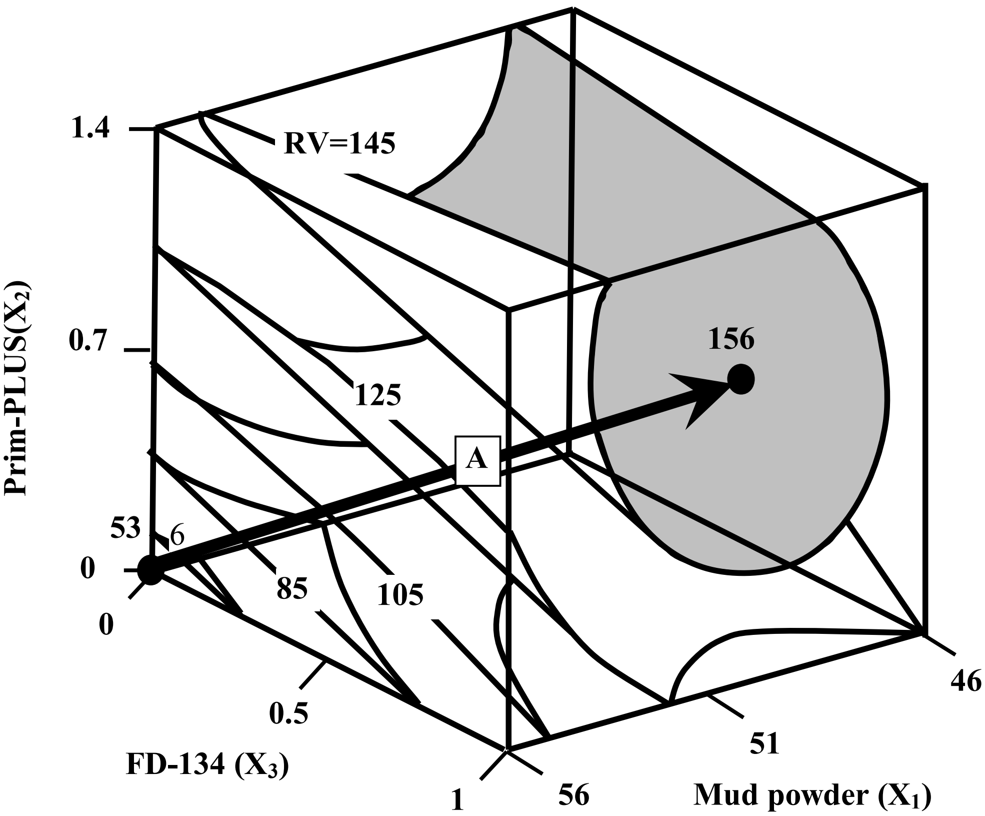

Fig. 3. 3D- Model of relative viscosity variations.

When the number of mud powder is equal to 51g ($x_1 = 0$), the relative viscosity index is increased from 100 to 150 seconds. However, the maximum values of the index were achieved in dosing rates Prim-PLUS (X_2) which are equal to approximately 70% of maximum input (Fig. 3).

The changes in amount of mud powder have a significant impact on relative viscosity (Fig. 3). As you can see from obtained three-factor reliabilities (Fig. 3), the smaller is the amount of mud powder, the higher is the passage speed of the solution through the Marsh funnel. Considered all, the variability of the relative viscosity index is smooth-tempered (average line A in Fig. 3.).

An important technological ability of solutions based on Dashukovskiy mud powders must be noted. The described behavior of viscosity changes, as well as its maximum and minimum values but don't change during the time required for making use of the solution. This time was taken as 24 hours in the research. Experiments were performed 3 times. First time the relative viscosity was determined immediately as soon as the mortar was prepared. Two other measurements were made after 12 and 24 hours. The data obtained were virtually identical (a difference of no more than 10%). Comparing the results, a conclusion was made, that this important technological ability remains stable during the taken time period. It must be a sufficient period of time to carry out the whole usual process of drilling.

5. THE RATIONAL CHOICE OF SOLUTIONS

Due to various influences of the additives on the technological abilities of the solutions on the basis of Ukrainian bentonites, there is a need to make trade-off compound and technological decision. Mathematical modeling seems to be the most appropriate method. In Ukraine there are no requirements of governing the properties of bentonite solutions for trenchless pipelaying by HDD. That's why the limits for the optimization of technological and operational properties of the solutions on a base of Ukrainian bentonites were taken by the reference to the ones by the French production.

So that the following range of limits were taken:

- the sieve residue 071K (SA) must be less than 1%;
- density of the clay mud (G), not more than 1.03 kg/m^3;
- relative viscosity (RV), at least 29 and not more than 145s;
- yield (point Yelda) (YP), at least 15 MPa×s;
- the thickness of mud cake (TK), less than 1mm
- water loss (WL) - no more 25 cm^3

The search of solutions meeting these requirements was performed with the aid of three-factor graph (Fig. 4.) in the COMPEX-99 which had been developed at the Odessa State Academy of civil Engineering and Architecture.

The graph shows the iso-surfaces of three index properties. These are relative viscosity ($29 \leq RV \leq 145$s), water loss ($WL \leq 25$ cm) and the density of the clay solution ($G \leq 1.03$ kg/m3). Targets of the remaining three properties meet the requirements taken as relative ones of the solutions.

The optimal compositions we find within the boundaries of the three iso-surfaces. As the one can see from the Fig. 4., not less than 51 grams of mud powder and not more than 1% of the powder mud's mass of additives Prim-PLUS must be entered. Amount of entries of the additives FD-134 may vary throughout the range of doses, but just by dosage of two other factors - 50% of the maximum permissible.

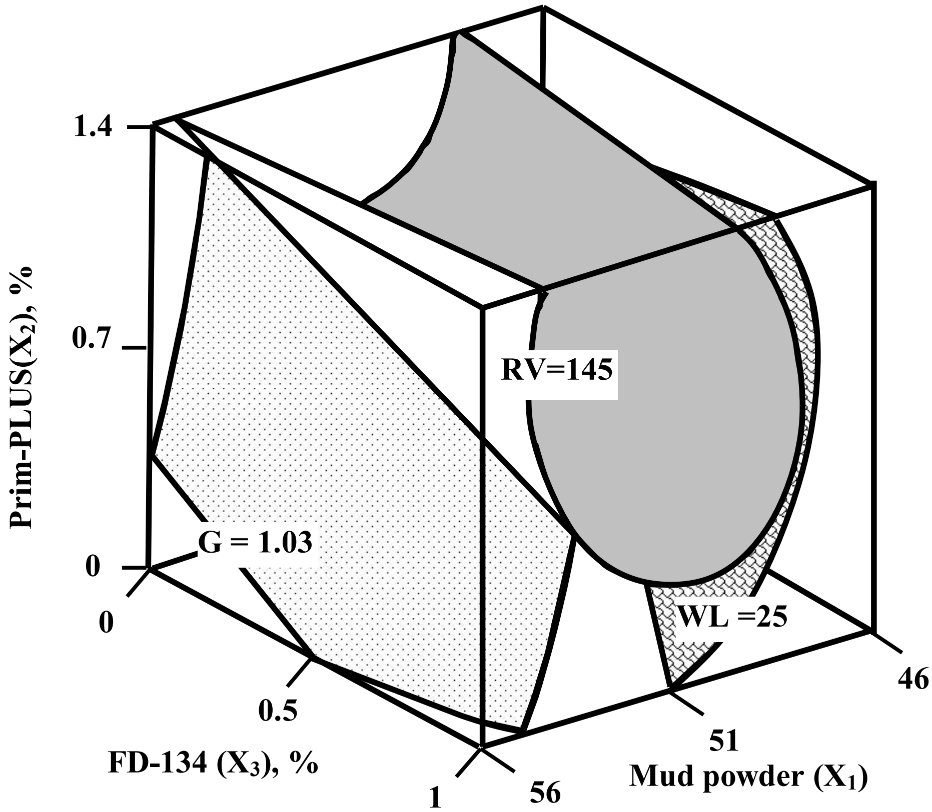

Fig. 4. Region of solutions limited by iso-surfaces of relative viscosity (29≤RV≤145c), water loss (WL ≤25 cm^3)and solution density (G ≤ 1.03кг/м^3).

6. SUMMARY

A solution with any amount of powder mud and additives within the boundaries of the iso-surfaces on the graph can be used for the construction of the utility networks by a horizontal directional drilling (HDD).

Clay solution optimization, which results are shown in Fig.4, proposed the following ratio of mixture (for 1dm3 of water):

- mud powder amount - 51gr;
- amount of additives Prim-PLUS - 0.7% of the mud powder mass (0.36gr);
- the number of additive FD-134 - 0.5% of the mud powder mass (0.26gr).

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EFFECTIVENESS OF COMPLEX MODIFICATORS IN CONCRETE

ABSTRACT

The paper is devoted to obtain high performance concrete with complex modificators and investigation of its exploitative properties. It was shown, that improved characteristic of concrete are achieved by optimal granulometric composition of fine, coarse aggregates and complex modificators plastifying and accelerating on the basis of sulphonaftalinphormaldegides, polycarboxylates and highly soluble electrolytes, such as sodium tiosulfate and rodanide. The optimal compositions of complex modificators due synergic action of their ingredients allowed to obtain concrete with improved properties durability and provided efficiency in monolithic building.

KEYWORDS: concretes, complex modificator, polycarboxylate, high strength, flowability, durability

1. INTRODUCTION

The new generation concrete ("High Performance Concrete") production, application and perfection are actual problems of theory of modern concrete. The accelerated development of the construction concrete technologies has been justified by new engineering challenges related to the civilization development and by the insufficient durability of concrete structures on global scale. For all building materials in the 21 th century, environmental considerations are becoming as important as the performance of the product itself. Service life or durability is a significant part of the environmental consideration [1,2].

In point of view of economy and high durability the high performance concrete production and use have significant practical importance. High strength and durability of such concrete is useful in difficult environmental influence and load even at the high flowing of concrete mixtures. High performance concretes heavy demand following characteristics: good workability (workability trades P4-P5) for at least 60 min; high compressive strength at early

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and later ages; high durability (frost, corrosion, air resistance). Important technological and economic advantages of new generation concrete are by comparison to traditional concrete.

Summarized results of research by many scientists show that the most radical way to increase in of binder systems effectiveness and concrete high performance lies the chemical activation not only of cement, but also concrete by complex modifiers of polyfunctional action. Admixture as component of concrete composite even of a very insignificant percentage may contribute a lot to its technical properties. The role of additions, as mean of the technological regulation, is increased to adequately growth of their influence efficiency on cement hydration and structure formation, properties of concrete mixtures and concrete [1]. Complex additions have a few reagents of different nature and mechanisms of action, each of which carries special function. Main forming principle of complex additives compositions is achieved necessary effects in technological and economic aspects consideration of compatibility with cements [2].

Modern effective superplasticizer is polycarboxylate, which is characterized by high plasticizing action, especially at low water/cement ratios (<0,35) due to electrostatic and steric effects [3, 4]. However many known chemical admixtures (surface-active substances) harshly reduce periods of Portland cement setting or essentially slow down its hardening. One of the main directions in the chemistry and cements technology development is the creation of rapid-hardening Portland cement binders which allow intensifying hardening of concrete strength increase. In order to intensification of binder materials using hardening accelerators such as inorganic electrolytes is effective. Strong activation agents of cement hardening by alkali compounds is due to hydration processes acceleration, change of morphology nature of newly formed formation and stone structure reinforcement [5, 6]. Because of synergy alkali admixtures significantly improve plastificating properties of such surface-active substances.

In this paper results of investigation of cement and concrete with complex chemical admixture of pacifying and accelerating action on the basis of surface-active substances and inorganic electrolytes such as sodium tiosulphate and rodanide are shown.

2. MATERIALS AND METHODS OF TESTING

Technology of high strength concrete is based on management of their structure formation in all production stages. High-quality binder materials, complex chemical modifiers of concretes structure and properties, active dispersion mineral components and microfillers are used for this purpose.

Portland-slag cement CEM II/A-S and Portland-composite cement CEM II/A-M were used. Complex chemical admixture of of plastifying and accelerating action consist with alkali metals salts (sodium tiosulphate and rodanide) and plasticizes (sulphonaftalinphormaldegides, polycarboxylates). Fly ash and technical microsilica (SiO_2 content is higher 90%) were used as microfiller. Natural sand (fineness modulus of 1,4-1,5), crushed stone sifting out (fraction 2,5-5 mm) and two coarse aggregates (fractions 5-10 mm and 10-20 mm) were applied.

Physico-mechanical tests of cements and concrete with complex modificators were carried out according to usual procedures. The evaluation of the properties of modificating cements was carried out through a flowing and compressive strength tests. Test methods also were included to evaluate fresh concrete properties (workability, slump-loss, bleeding and material segregation), hardening and strength development. The parameters of pore structure, density, water оглинання, drying shrinkage, corrosion, air and freeze-thaw resistance were determined as durability properties.

3. RESULTS AND DISCUSSION

Important properties of cements used in concrete investigations are listed in tab. 1. The 28-day compressive strengths of these test cements achieved value for trade 400 (class 32,5). The water demand to achieved standard stiffness is varied from 25,5 % to 24,0 % depending on the particle distribution and composition of cement. Setting times of used cement correspond to standard.

Table 1. Physic-mechanical properties of cements (GOST 310.1-4)

Cement type	Blain, $м^2$/кг	Reduce on sieve 008, %	Water demand, %	Setting time, h-min		Compressive strength, MPa, in age, days		
				initial	final	2	7	28
CEM II/A-S	315	9,8	25,5	2-40	4-10	12,3	28,4	42,8
CEM II/A-M	268	12,0	24,0	3-20	4-30	11,5	27,6	41,8

The results obtained from the flowing of cement with sulphonaftalinphormaldegide (SP) and polycarboxylate (PC) types plasticizers are shown that these admixtures are characterized by high reological effectiveness. Tab. 2 indicates that at constant water demand polycarboxylate-based admixture was more effective than sulphonaftalinphormaldegide type superplasticizers. Thus, flowability of fine-grained concretes mixture with admixture significantly increased from 115 to 210 mm (SP) and to 260 mm (PC). The significantly plasticizing action of polycarboxylate-based superplasticizers is caused by electrostatic and steric effects. Electrostatic effect is caused by the adsorption of anionic superplasticizer, increasing of negative surface charge of cement particles and stabilized cement paste suspension. The steric effect of polycarboxylate is provided due to increasing side chain length of polymers.

At the same time cement systems plastificated by superplasticizers need no only reological effectiveness, but hydration activity, which determine strength of materials at early and later time. The results of investigations show that using of polycarboxylate type plasticizer increase 28-day compressive strength of mortars 1,5 times.

Table 2. The influence of different kind superplastisizers on fine-grained concretes properties (Cement:sand ratio 1:2, cement - CEM II/A-S)

№	Kind and amount of admixture	Flowing, mm	Compressive strength, MPa, in age, days		
			3	7	28
1	-	115	4,1	12,8	20,0
2	0,5 mass. % NF	210	5,8	10,2	26,0
3	0,5 mass. % PC	260	7,9	20,8	35,0

Significant practical interest has modifying of cement system by complex chemical admixture of polyfunctional action on the surface-active substances and inorganic electrolytes basis for obtaining binders materials with high service properties. Complex admixtures on the basis of superplasticizers and highly soluble electrolytes provide increasing plasticity of cement composition at less water content and provide high strength due to formation homogenous micropore structure. The reological effectiveness or complex modifiers „surface-active substance+electrolyte” is caused by adsorption of surface-active substance on cement particles and by their deflocculating due to electrostatic and steric effects.

The effectiveness increasing action of complex modifiers on basis of polycarboxylate and reduceing of clinker content is achieved by fly ash adding to Portland cement. In results of fine-grained concrete with composite cement CEM II/A-M with complex modifiers testing (fig. 1) are established that flowability of modifying fine-grained concretes increased to 190 mm. At same time water demand of modifying cement is decreased 37,5% due to significant water reduce effect. The early strength (2 days) of mortar was 29,0 MPa (W/C=0,25). 28-days strength of fine-grained concrete with complex admixtures of plastifying and accelerating action was 49,8MPa. Applied cements corresponded to plastificate rapid-hardening Portland cement.

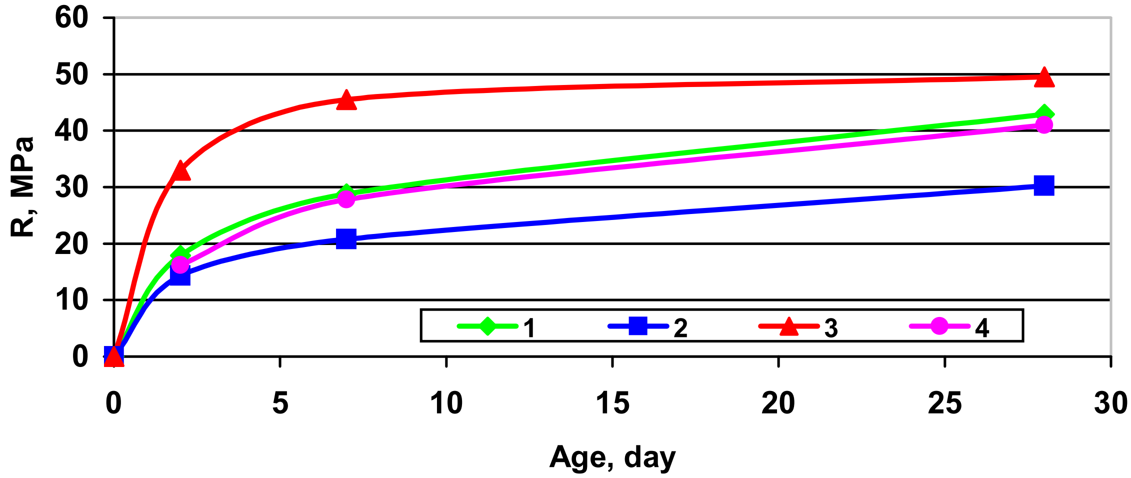

Fig. 1. Results of strength test of fine-grained concretes: 1 – without admixture (W/C=0,39, flowability 110 mm); 2 – 1,5 mass.% PC (W/C=0,4, flowability 190 mm); 3 – 1,5 mass.% PC (W/C=0,25, flowability 110 mm); 4 – 1 mass.% PC (W/C=0,29, flowability 170 mm)

The correlation of steric factor and anionic activity of polycarboxylates in complex with highly soluble electrolytes are provided due to high effectiveness polyfunctional admixtures. Using of complex modifying on polycarboxylate basis improve composition of rapid-hardening Portland cement by higher trade strength due to significant water demand W/C=0,4 – superplasticized cement system.

The role of fine and coarse aggregates in forming of concrete properties, especially high strength concrete grow with mineral additions content increase in cement. For providing of concrete optimal structure according to durability criterion is obligatory composition of fine and coarse aggregates in accordance with requirements of DIN 1045. It was be established that polyfractional aggregate consisted from, mass.%: sand - 25, crushed stone sifting out 2,5-5 mm – 9, coarse aggregate fraction 5-10 mm - 21and fractions 10-20 mm - rest.

Concretes on the basis of polyfractional aggregate (cement CEM II/A-S content was about 450 kg/m^3) without and with complex modifiers were produced and test carried out on fresh and hardened concrete (fig.). It was shown that providing high workability of concrete mixture (sump level 25 cm – class P5) is achieved at w/c ratio 0,56. Fig shows that, at a given sump level, using of complex chemical admixture permits reducing of water demand of concrete mixture from 0,56 to 0,39. It was established that modifyind concrete mixture (flowability trade is P5) bleeding was lower than 0,1%, material segregation was twice reduced. The results of workability test in time show that good workability of modifying concrete mixture was at least 3 hours.

The mixing water functions providing hydration process of cement constituents and giving the cement paste plasticity required for placement. In construction practice concrete generally have water/cement ratios between 0,3-0,7. Quantity of water corresponding to w/c ratio between 0,23-0,4 is necessary for completion of hydration reaction, but more water is generally required to achieved adequate workability. The water excess is the reason for the formation of voids, capillary pores, which reduce the strength and increase the permeability of the hardened cement paste or concrete to penetrating liquids and gases.

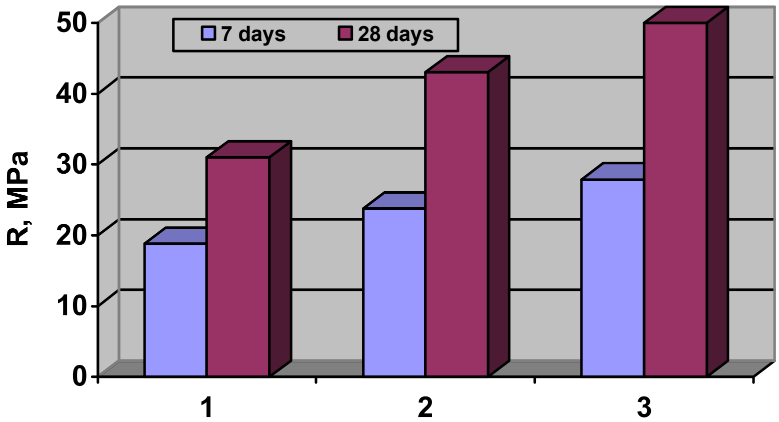

Fig. 2. Influence of complex additive on concrete compressive strength: 1 – without additions; 2 – 1 mass.% PK, 3 - 8 mass.% complex additive

Fig. 2 shows development of compressive strength of concrete without addition and with complex organic-mineral additive produced from high-flowing concrete mixture. As expected, the strength of concrete without addition was lower at early and later time than modifyied concrete, caused by excess water. The early compressive strength of concrete with complex polycarboxylate-based admixture is increased 1,6 time. The 28-day compressive strength was in a range of 51,2 MPa, which responsible to class B40. At that time 28-day compressive strength was only 27,9 MPa.

Porosity and pore size distribution are important for practically all properties of concrete that are relevant to durability. Harmful effects find their way into the building material from pore system. The using of complex chemical admixture and can have a crucial influence on the porosity and hence on the durability of concrete. The pore structure parameters of concrete are measured at 28 days in according with GOST 12730.4 (tab. 3).

The volume water devour of concrete with complex chemical modifiers (W_v), which characterized open capillary porosity, is decreased 2,2 times. When using technical microsilica as component of concrete water devour differed only slightly from that without any admixture. The results of investigations of concretes pore structure indicates, that adding of modifiers can regulate integral and differential porosity parameters (indicator of average pore size – λ_1 and indicator of pore similarity – α) of materials. Thus, indicator of average pore size of modifying by complex polyfunctional admixture concrete is decreased 3,5 times compared with concrete without admixture. The using of complex chemical admixture and fine filler is allowed to improve of material pore structure – decrease of average pore size, quantity of macropores and increased of pore similarity.

The plastifying concretes are characterized by high density (Wm<2%) and frost-resistance (higher F300). The air resistance of modifying concrete after 100 cycles of moistening-drying is increased in twice. The coefficient of corrosion resistance of concrete, modified complex additions, at high flowability was 0,89, at water demand diminishing of modified concrete it was - 0,95.

Table 3. The concrete mixture properties and basic parameters of concrete pore structure

Kind and amount of addition	W/C	Sump, sm	Water devour, vol. %	Indicator of average pore size, λ_1	Indicator of pore similarity, α
-	0,56	25	8,9	8,45	0,21
1 mass.% PC	0,39	27	6,5	1,74	0,26
8 mass.% complex additivon	0,39	27	4,1	1,14	0,5

4. CONCLUSION

Using of chemical additions is one of the most universal and accessible methods of concrete properties management. The principles of composition technology of high performance concrete with high values of flowability and compressive strength at early and later age are established, based on the synergistic action of polyfunctional complex modifiers on the basis of polycarboxylates and high-soluble electrolytes.

Test results of properties of modifyied concrete show that using of complex modifiers of plastifying and accelerating action on the basis of polycarboxylate provides high flowability (trade P5) and strength class B40.

Use of complex modifiers on the basis of polycarboxylate extends possibilities of application of the high flowing concrete mixtures in practice of monolithic building, especially at making of densely reinforced structures of difficult shape. The physical-chemical modifying by polyfunctional complex admixtures becomes the basic trend of improves of monolithic concrete and reinforced concrete up-to-date.

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DRY BUILDING MIXTURES ON THE BASIS OF GYPSUM-FREE PORTLAND CEMENT

ABSTRACT

In this publication theoretical principles for obtaining gypsum-free Portland cement (GFPC) modified by organic-mineral additives were investigated. Gypsum-free Portland cement was improved by rational selection of the chemical additives which act as hardening accelerators, plasticizers and setting retarders. The hydration and structure formation processes as well as building-technical properties and running ability were established. The influence of alkaline metals carbonates as hardening accelerators on pH of a liquid phase, hydration degree in the alkaline systems as well as features of plasticizers action in constitution complex chemical additives were investigated. The possibility of speed regulation of GFPC early structure formation by introduction setting retarders which are replacing the effects of gypsum was determined. There were investigated the characteristics of the phase structure and microstructure of the artificial cement stone on the basis of gypsum-free Portland cement modified by the complex chemical additives and fine-dyspersated lime stone. Early structure formation is characterized by hydroaluminate hardening type with formation of metastable carbonate replaced by AF_t–phase followed by structurally active hexagonal AF_m–phase $C_4A\cdot CO_2\cdot 11H_2O$.

KEYWORDS: gypsum-free Portland cement, dry building mixtures, organic-mineral additives, rapid-hardening.

1. INTRODUCTION

Development of effective rapid-hardening gypsum-free Portland cement for dry building mixtures for complex of characteristic building-technical properties achievement is a construction materials science perspective area.

Based on knowledge about gypsum-free Portland cement as a "clinker - calcite included additive – accelerant – plasticizer - retarding admixture" system, influence of each component on structure formation process was consequently analyzed and its part as basic component at work stone properties formation included in dry building mixtures (DBM).

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2. RESULTS OF THE INVESTIGATION

Hardening acceleration at gypsum-free Portland cement (GFPC) system and getting a dense and strong stone is provided by mandatory addition of alkali metals carbonates to GFPC system. Carbonate anion active structure formation part during cement minerals hydration is well known. But influence of cation type composed of accelerant alkali additives is not fully analyzed. Taking this on consideration researches of acceleration action efficiency were made for alkali metals carbonates with different nature of cation in row $K^+ \rightarrow Na^+ \rightarrow Li^+$.

Liquid phase composition formed during cement and water interaction is important regulatory parameter of physical-chemical processes appeared during hardening of binders with alkali included additives. Kinetics of binder dissolve and value of supersaturation appeared at "cement – liquid phase chemical composition" system, determine structure formation process character and cement stone strength synthesis.

Solution pH in suspensions (W:S=10:1) was defined at early stages to study hydration processes features of gypsum-free cement compositions with alkali included additives.

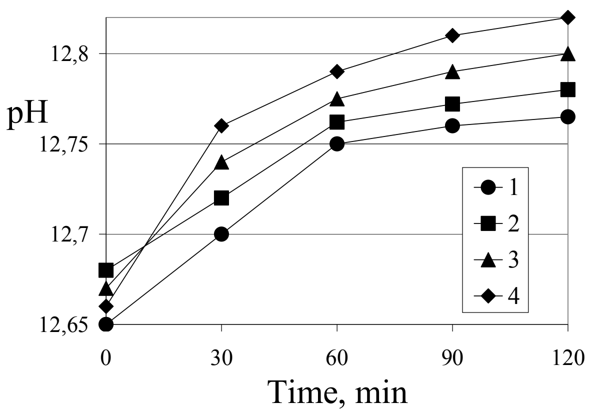

Fig. 1. 1,5 mass % alkali metals carbonates influence on liquid phase pH of gypsum-free cement systems (95 mass % of clinker + 5 mass % of limestone + 0,5 mass % LST): 1- without additive; 2 - K_2CO_3; 3 - Na_2CO_3; 4 - Li_2CO_3

Liquid phase pH analysis shows that results of exchange reactions between $Ca(OH)_2$ and R_2CO_3 with presence of fine dust limestone as active mineral additive, increase of hydrogen indication could be seen immediately at the hydration start moment for all analyzed compositions (Fig. 1). But after 30 minutes and further cement suspension with Li_2CO_3 additive indicate maximum pH level. This could be explain as lithium carbonate not used for interaction with calcium carbonate increase pH level during double carbonate connections formation and support effective hydration reactions acceleration and GFPC hardening at normal conditions.

Besides alkali metals carbonates influence on pH level during analysis of acceleration action mechanism in cement systems attention should be given to extremely high electronegativity of K^+, Na^+, Li^+ cations and its strong polarizing possibility. Therefore gypsum-free Portland cement hydration process at alkali included systems carried out at the background of alkali cations high polarization coefficient.

Positively charged alkali metal cation helps to intake electrons which form O-H connections during the interaction with electronegative oxygen atom in the water molecule due to positive electrostatic charge. As a result hydrogen atoms reach higher positive charge value, O-H connection became weak and proton separate from water molecule easier. This change of energy characterizes by O-H connection and which appear under cation action could be symbolically named as “interaction energy”. As smaller this energy is as easier proton H^+ to separate. We found that “interaction energy” decreases in $K^+ \rightarrow Na^+ \rightarrow Li^+$ row.

Next diagram of alkali metals carbonates interaction with rapid-hardened GFPC minerals was suggested (Fig. 2) based on these views about clinker minerals hydration.

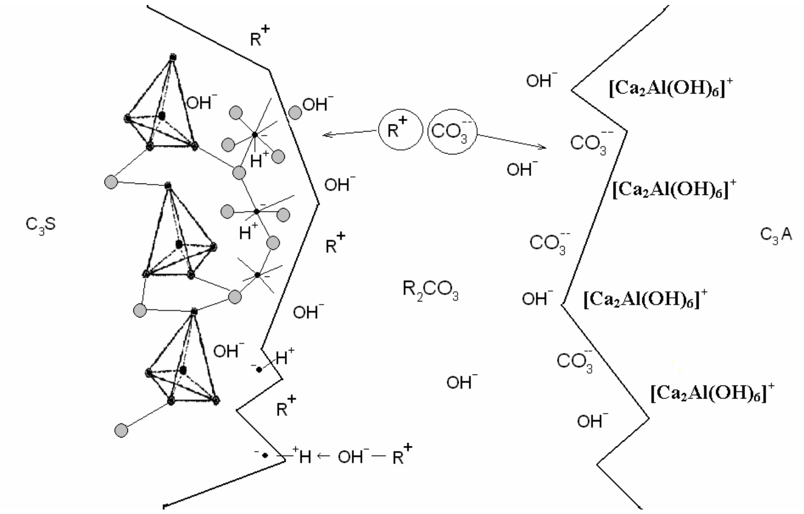

Fig. 2. Diagram of alkali metals carbonates interaction with rapid-hardened gypsum-free Portland cement minerals

During alkali metal carbonate hydrolysis CO_3^{2-} anion connect with C_3A and R^+ cation accelerate C_3S hydration by supporting protonization of undersaturated oxygen atom at the mineral structure and increasing pH of the system. Hydration phase structure formation based on calcium hydroxide octahedral layers which serve as a matrix for SiO_4-tetrahedrons condensation in hydrosilicates and formation of structure elements $[Ca_2Al(OH)_6]^+$ which form the base of AF_m-phases defining “genetic code” of harden cement system and provoke hexagonal crystals symmetry.

So due to hydrogen indication increase and energy estimation alkali cation part at harden system it is seen that Li_2CO_3 additive at GFPC system provides maximum acceleration of hydration processes.

Reaching of tight and strong is provided by obligatory inclusion of anion active plasticizing component to gypsum-free Portland cement system. Casein – effective plasticizing-retarding natural origin agent became to be popular at DBM technology for the last time. List of features was stated during analysis of casein part at gypsum-free Portland cement system: differently from regular Portland cement it doesn’t show retarding properties and have a low plasticizing effect. This could be explained by blockage of surface development in such cement dispersions at extremely fast free-gypsum Portland cement thickening and brings the opportunity of influence on water need to minimum.

Stated that for the effective plasticizing properties appearance of casein and other analyzed additives-plasticizers normal processes operation of early free-gypsum Portland cement structure formation are needed. Additives of boric 0,5-1,03 mass% and tartaric 0,05-0,1 mass% acids were included to complex chemical additive (CCA) composition with this purpose to provide necessary gypsum-free Portland cement thickening process retarding and create optimal conditions for plasticizing agents action based on hard dissolve screen of calcium borates $Ca(BO_3)_2{\cdot}4H_2O$ and tartrates $CaC_4H_4O_6{\cdot}4H_2O$ layers formation on the cement particles and its buffer action on pH level. As seen from analyses made at the free-gypsum

Portland cement system which include hardening accelerator as mandatory component, negative influence of boric and tartaric acids on further structure formation process and strength of free-gypsum Portland cement is not stated (tab. 1).

Table 1. Physical-mechanical properties of modified free-gypsum Portland cement * for dry building mixtures

No.	CCA composition, mass %				NC, %	Thickening period, h-min		Compression strength, MPa, at period, days		
	Accelerat Li_2CO_3	Plasticizer	Retarder H_3BO_3	$C_4H_6O_6$		start	finish	2	7	28
1	0,5	-	1,0	-	22,0	0-58	1-27	47,8	64,1	83,1
2	0,5	0,5% LST	1,0	-	19,0	2-15	3-05	49,6	74,1	116,5
3	0,5	0,5% C-3	1,0	-	18,5	1-47	2-45	49,9	75,0	117,3
4	0,5	0,5% Melment	1,0	-	18,5	2-05	2-57	48,7	75,9	119,7
5	0,5	0,5% Melflux	1,0	-	18,0	2-10	2-53	52,3	77,1	120,4
6	0,5	0,5% Casein	0,5	-	17,0	0-45	0-50	59,4	82,3	128,4
7	0,5	-	-	0,1	22,0	0-17	0-27	46,7	68,2	90,1
8	0,5	0,5% Melflux	-	0,1	18,0	1-56	2-34	55,8	80,3	126,6
9	0,5	0,5% Casein	-	0,1	17,0	1-10	1-45	63,6	84,6	135,4

* - 95 mass% of clinker + 5 mass % of limestone

Taking on consideration alkali character of hardening accelerator and presence of thickening retarder special activities of plasticizing agent were analyzed at work as a CCA component for free-gypsum Portland cement and for the first time experimentally shown possibility of usage not only traditional lignosulphonates (LST) but plasticizers of other known types: sulfonaphthaleneformaldehyde (C-3), sulfomelamineformaldehyde (Melment) polycarboxylated (melflux) and also casein.

By researches made in free-gypsum Portland cement system stated additive character of CCA components actions and synergistic occurrences as well. So due to synergistic gain of casein action with Li_2CO_3 presence, necessary pH level at liquid phase is reached what supports casein proteins active hydrolysis to more simple proteins, aminoacid residuals and phosphoric groups with further consecutive protein structure development which act as surface-active substances. This provides receiving of most effective lithium-casein plasticizer in analyzed row.

During analysis of phase composition and microstructure of carbonate free-gypsum Portland cement and modified CCA at different hydration and hardening stages, stated that intensive process of early structure formation provides by creation of ethrengythotype metastable AFt phases $Ca_6[Al(OH)_6]_2 \cdot 24H_2O \cdot [(CO_3)_3,(BO_3)_3 \cdot 2H_2O]$. Hexagonal high based calcium hydroaluminates C_4AH_{13} with $CaCO_3$ presence changes to low based AFm phases at $C_4A \cdot CO_2 \cdot 11H_2O$ alumina hydrocarbonates state structure formation part of which grow during time. Crystallized portlandite $Ca(OH)_2$ amount is decreased during all time of hydration, hydrosilicate phase represented in big amount. Cement stone microstructure characterized by uniformity at high stage of hydrate phases dispersion (Fig. 3).

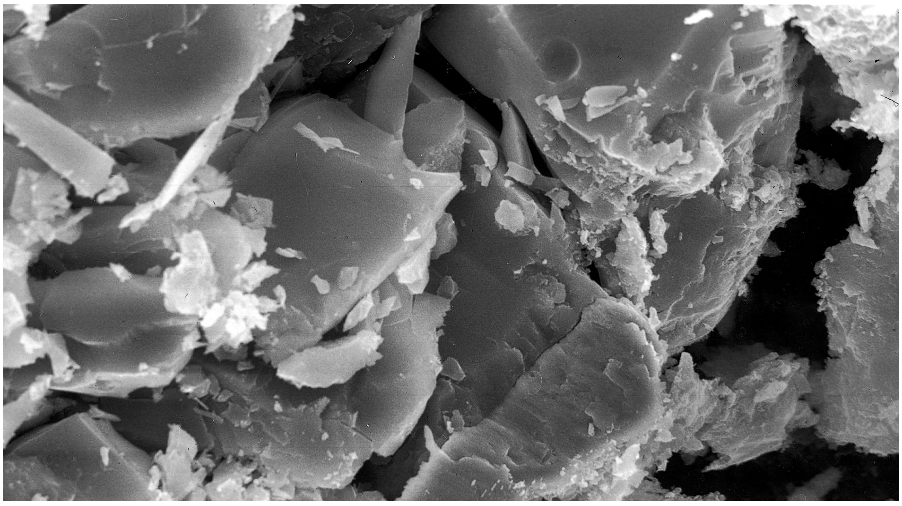

Fig. 3. Cement stone microstructure based on gypsum-free Portland cement with CCA hydrated during 7 days at normal conditions (x9000)

It is necessary to indicate mineral additive $CaCO_3$ active structure formation part composed of free-gypsum Portland cement. It appears in additional amount of hexagonal AFm-phases creation which provides good internal connection between stone compounds as a result of epitaxial mound. Besides it as a “fine dust” effect result, $CACO_3$ particles spread grains of harden system what accelerates hydration processes, helps in stone consolidation and rise of its strength at increased actual w/c and hydration products rejection.

Mentioned microstructure, regulated intensive process of early structure formation with Aft-phases appearance, phase ratio features characterized by increase of stable hexagonal carbonate included AFm-phases amount together with high dewatering effect provides reaching of tight, strong cement stone based on carbonated gypsum-free Portland cement with developed CCA.

3. CONCLUSION

Therefore possibility of free-gypsum Portland cement usage for dry building mixtures by modifying it with poly functional additives based on alkali metal carbonates, hydrophilic surface-active substances and nonsulphate action thickening retarders (tartaric and boric acids) with 5-20% of fine dispersed calcium carbonate presence is confirmed. This provides a directed structure formation processes control and allowed to get rapid-hardened dry building mixtures for intensive construction technologies.

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MAIN GOAL - HOW TO ACHIEVE A ZERO-ENERGY BUILDING.

ABSTRACT

A net zero building energy or zero energy building (ZEB) is an overall term applied to a residential or commercial building with zero net energy consumption and/or zero carbon emissions annually. It has greatly reduced energy needs so that the remaining energy need can be supplied with renewable technologies on- or off-site, depending on the definition. In this paper is a short sketch of the energy-use of buildings present followed by why zero energy building is a solution to the energy problem with advantages and potential disadvantages. It's divided in two bigger parts: reducing the energy needed, and generating energy from renewable resources. The later gives a short list of possible techniques to generate energy. In chapter four are some guidelines to select materials to construct the building. The focus there is on sustainable materials.

KEYWORDS: zero-energy, building, method, energy efficiency

1. INTRODUCTION

Buildings worldwide use around one-third of the world's energy, a proportion that only will continue to increase as the population grows and becomes more urban. In the United States and European Union the building sector already accounts for 40% of the primary energy use of which residential buildings are responsible for 21%. They are also responsible for approximately 15% of the water use and 15% to 40% of the waste in landfills (depending on the region).

Diminishing the energy- and water-usage in buildings is a possibility to lessen the impact on the environment. It will save money and resources while reducing pollution and CO_2 in the atmosphere and it even leverages greater savings at power plants in the form of primary energy. One of the tools to reduce the overall energy- and water-usage is zero energy buildings. [1]

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2. A SOLUTION: ZERO ENERGY BUILDINGS

A zero energy building (ZEB) or net zero energy building is an overall term applied to a building with zero net energy consumption and/or zero carbon emissions annually. This can be measured in different ways: relating to cost, energy or carbon emissions and so resulting in multiple definitions for the same concept. The definition used for a zero-energy building usually depends on the country where it's situated. But in general, a zero energy home is designed to produce as much energy as it consumes over the course of a full year using assumptions for typical occupant behaviour.

In most of the on-the-grid buildings the homes exchanges energy with the utility power grid. It delivers energy to the grid when the photovoltaic (PV) system is producing more energy than is being used in the home and draws from the grid when the PV system is producing less energy than needed in the home. An off-the-grid ZEB has no connection with any utility net and because of that it has to store energy for later use and is therefore harder to achieve.

We need to be aware that most definitions do not include the emissions generated in the construction or the embodied energy in the structure. Because of this given fact should we keep an eye out for the materials. The amount of energy used to fabricate them could dwarf the energy saved over its useful life-span.

There are numerous advantages when constructing a zero energy building. However, many others can be mentioned too, the improved insulation, for example, increases the indoor comfort due to more-uniform interior temperatures and safeguards the owner from future energy price increases. Generally a higher resale value is also present.

Potential disadvantages are the initial costs that are higher, however, when qualifying for low energy subsidies or zero energy building subsidies, which are present in for example California U.S., one can greatly diminish those costs. Sceptics also point out the lack of experience in the field of building zero energy homes, this can only be solved by time when this type of building gets more and more popular. One problem that isn't easily solved is the peak demand of electricity: while the individual house may use an average of net zero energy over a year, it may demand energy at the time when peak demand for the grid occurs. In such a case, the capacity of the grid must still provide electricity to all loads. Therefore, a ZEB may not reduce the required power plant capacity. [2] [3]

3. ACHIEVING ZERO ENERGY BUILDINGS

3.1. Evaluation

One of the answers to reducing raw material- and energy-consumption is to evaluate first whether a new building really needs to be built. Renovating an existing building can save money, time, and resources. It also enables a family or company to be located in a part of town with existing infrastructure and public transportation, enhancing convenience and reducing sprawl. Next, if a new building is required, it should be sized only as large as it really needs to be. This in contrary to the cultural assumption that we should buy or lease as much square meters as we can afford. Smaller buildings require fewer building materials, less land, and less operational energy. Furthermore smaller houses and commercial buildings allow the budget to be spent on quality, rather than what may be underused quantity.

3.2. Reducing energy use

The Energy Policy Pyramid (Fig. 1.) shows five levels of strategies to employ to reduce energy use. The course of action should start with the foundation of the pyramid and work upwards. [4]

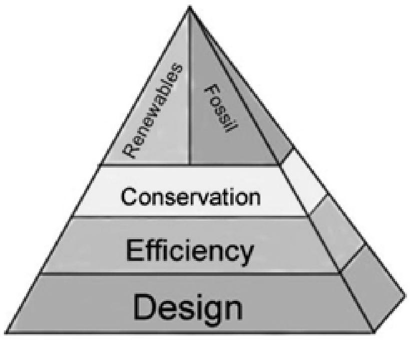

Fig. 1. The Energy Policy Pyramid

3.2.1. Design

The most cost-effective energy reduction in a building usually occurs during the design process. Great opportunities lie in the simple design solutions that intelligently respond to location and climate. It's during that design stage that the highly cost-effective strategies for making an impact are selected. Making the building envelope (exterior walls, roof, and windows) as efficient as possible, taking in consideration the climate, can dramatically reduce heat loads. For residences and other small buildings, optimal sealing, insulation, and radiant barriers, can reduce heat losses to less than half that of a building that simply meets code. Once the building envelope is efficiently designed to reduce heat flow, natural heating and cooling methods can be used to greatly downsize, or even eliminate, fossil fuel-based mechanical heating and cooling systems.

Techniques used for reducing the heat load include: solar heating, efficient and right-sized HVAC systems with heat-recovery units and utilization of waste heat. The cooling load, the excessive heat, is generated inside the building by lights, equipment, and people. Installing efficient lighting and appliances (which emit less heat) will significantly reduce the building's cooling load. Furthermore, using daylight as much as possible, will reduce cooling loads even more, because daylight contains the least amount of heat per lumen of light. Other techniques include natural ventilation and cooling, and also the efficient and right-sized HVAC systems.

These previous points can be integrated into a passive solar design, which uses the sun's energy for the heating and cooling of living spaces to maintain interior thermal comfort. It doesn't require mechanical systems and only a minimal maintenance is needed. This passive design particularly uses thermal mass, which stores heat. The thermal mass is used for cooling or heating, depending on the climate. Other common found elements in this type of design are operable windows and thermal chimneys (used for natural stack ventilation). [5]

3.2.2. Efficiency

Beyond the design stage, there are a number of efficiency measures that can be utilized. Efficiency measures provide the same or better benefit at no inconvenience to the user. For example: selecting a more energy-efficient air conditioning system that produces the same amount and quality of cooling, while using far less energy.

However keep in mind that over a building's lifetime, the equipment, windows and light fixtures may be improved and the building will perform at a higher level; but even with similar improved components, a prescriptive code building might not perform at the level of the building with the better design.

3.2.3. Conservation

Conservation initiatives that involve reducing energy use, often comes with a change in occupant behavior. Studies of identical homes in the United States have shown dramatic differences in energy use, with some homes using more than twice the energy of others. This differentiates conservation from the definition of efficiency described above where only the buildings efficiency was the factor. Although some conservation efforts are automated (e.g. motion sensors on light controls), others require continued consumer persistence for the savings to remain.

The graph below (Fig. 2.) are the results of a study done by Florida Solar Energy Center (FSEC) in the mid 1990's measuring the energy use in ten identical Habitat for Humanity all-electric homes. The houses were constructed the year before and monitored in Homestead, Florida. For each of the 10 homes the graph shows the annual energy use. Even though all homes had two or more occupants, with identical appliances and equipment, energy use varied by 2.6 to 1 from the highest to lowest consumer. Detailed measurement of the end-uses in the homes revealed that while the electrical consumption of appliances like refrigerators were remarkably similar, other uses such as air conditioning varied by 5:1 from the highest to lowest. Evaluation of interior temperature and operation showed that much of that difference was due to differing thermostat behavior. [6]

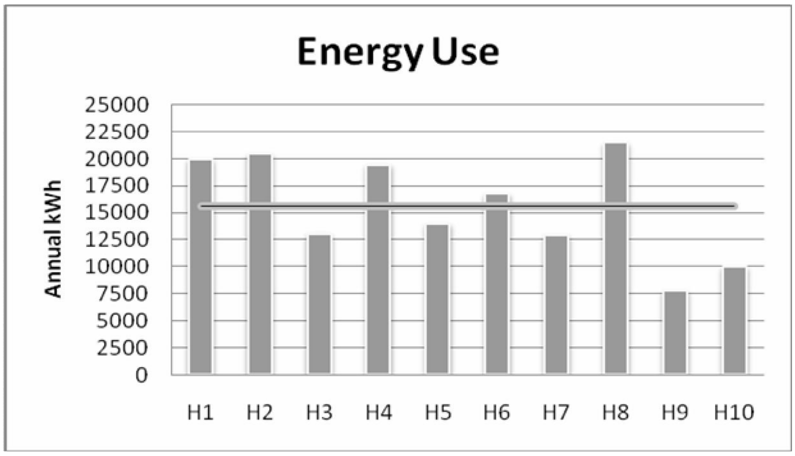

Fig. 2. Energy use in 10 identical homes

3.2.4. Renewable energy

Alternative or renewable energy technologies look at supplying energy from resources that offer some benefit (such as less CO2 production) as compared with conventional resources and, as such, should be considered prior to or in conjunction with conventional resources.

Most solar thermal systems typically convert 50 to 70% of the sun's incident radiation to useful energy (depending on the region) in compare to commercially installed silicon cell solar electric systems that only convert about 10% of the solar energy to useful electricity. Therefore we should choose the energy systems performance based. Keep in mind that using renewable energy doesn't reduce energy use, but rather reduce use of some conventional energy fuel. And because of these commodities are often costly they should be considered if all other potential is been exhausted. [7]

3.3. Generating renewable energy

Energy is obtained in the form of electricity or heat generated from renewable resources which are mayoralty powered by the sun (solar, wind, hydropower, biomass) or from heat generated deep within the earth (geothermal resources). One can generate energy in the building itself, on-site, or generate the energy off-site on a larger scale.

Biomass is a collective name for all renewable fuels by plant (or animal) origin: woodblocks, wooden pellets, wheat, rapeseed, sunflower seed. By using bio-energy, there are no additional net CO_2 emissions. The conversion (by fermentation, burning or gasification) of biomass into usable power, the same amount of CO_2 comes free as the plants and trees have stocked during their life.

Geothermal power generates energy from heat stored in the earth. The heat can also be directly used to heat a space. The large scale off-site generating of energy happens on specific locations: the edges of tectonic plates where high temperature geothermal resources are available near the surface. In a smaller scale geothermal heat pump systems are applied. This is a central heating and/or air conditioning system that actively pumps heat to or from the shallow ground depending on the climate and time of year.

Hydropower is a term collecting multiple techniques in one, but for each of them is the energy derived from the force of moving water. Hydropower generates energy on a large scale. The most important form of hydropower is hydroelectric power, it is also the most widely used form of renewable energy, delivering approximately 19% of the world's electricity. Smaller forms of hydroelectric power dams can be used for little communities or single houses. Newer technologies that use the energy from waves or tides also exist.

Solar power can be applied in many ways. It can be used to generate electricity or for its radiant heat. By using photovoltaic solar cells, concentrated solar power, or heating trapped air which rotates turbines in a solar tower, one can generate electricity. One can also heat water, used for domestic appliances, by the sun in solar-thermal panels. Heating and cooling air can be achieved by using solar chimneys or a passive solar building design.

Wind power is the conversion of wind energy into electricity by using wind turbines. Large scale wind farms are present in a lot of regions and the sector is growing rapidly. Smaller turbines can also be applied to buildings providing its needed energy.

The above stated forms of renewable energy are effective alternatives to fossil fuels and will help to meet a building's energy requirements and reduce its carbon dioxide emissions. Selections between these possibilities, according to the climate, have to be made to achieve an optimal performance. [8] [9]

4. MATERIALS

The extraction, manufacturing, use and disposal of building materials have many resource, and environmental impacts. Each of these phases uses significant quantities of raw materials, water and energy. The embodied energy for building materials is estimated to account for 10%-60% of a conventional building's total energy use over a 50-year life cycle. Thus, as buildings become more energy efficient, the impact materials have on the total life cycle energy consumption of a building will become more significant. The resources used to produce or recycle a material could minimize the energy savings over its useful life-span.

Therefore are the embodied energy and the ecological footprint of materials characteristics that needs to be taken in consideration while designing or renovating a building. The usage of green- or sustainable materials is an answer to this problem.

In practice there are materials that significantly reduce the negative impacts on people and the environment. Rapidly renewable plant material is typically considered a green material. But also other products that are renewable, reusable or recyclable can get the 'green' status. Those materials can be found on online databases of green materials such as: 'http://www.greenbuilder.com'

Selecting materials isn't a simple task given that a lot of different factors are playing a part. One should consider the climate where the building is going to be constructed hence the characteristics the materials need to have. But also the sources of the raw materials and the amount of recycled content and later recyclability of materials; the total life-cycle cost (the costs of production, and maintenance); the embodied energy; the transport; …

However, in spite of the complexity, some rough guidelines can be stated: use durable materials; make use of low-maintenance materials; minimize packaging waste; use low embodied energy building materials preferably made from sustainable resources; locally produced building materials also have advantage (transporting distance); use recycled building products. [10] [11]

5. CONCLUSION

To realize a new Zero Energy Building, the goal is to achieve two thirds energy reduction with (passive solar) design, efficiency and conservation and the other third of energy from renewable sources. Different types of installations can be used to generate the required energy on- or off-site.

When renovation, one should find an equilibrium between costs and energy reduction. When the costs of reaching zero energy standard are too high, one can opt to realize a Near-Zero Energy Building (stricter then a passive house) or a passive building.

The embodied energy of materials, either when renovating or building, is a factor that cannot be forgotten. When taking the time to weigh the advantages against the disadvantages of different materials, one can select materials with a minimized environmental impact while still living up to the wanted properties.

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BUILDING ENVIRONMENTAL ASSESSMENT - SITE SELECTION AND PROJECT MANAGEMENT

ABSTRACT

Building environmental assessment is a specific process oriented to systematic, comprehensive and integrated evaluation of building performance. These processes lead to design, construction and operation of buildings with respect to criteria of sustainable development. Building environmental assessment is not only tool of control, but also tool of sustainable building design. The building environmental assessment systems deal with site selection criteria, the efficient use of energy and water resources during building operations, waste management during construction and operations, indoor environmental quality, demands for transportation services, and the selection of environmentally preferable materials. The Slovak building environmental assessment system was processed. The fields and indicators are proposed on the base of available experiences database analysis from environmental performance of buildings. The field site selection and project management will be introduced in the paper. Also the proposal and verify of this field will be presented for selected office building.

KEYWORDS: site selection, project management, building environmental assessment

1. INTRODUCTION

Since 1990s building environmental assessment systems, methods and tools are developed for evaluation of building performance in many countries. The determination of real buildings state is performed for the purposes of building assessment from environmental aspects, safety and reliability. The result of assessment is to design sustainable buildings with great environmental potential and following this ranking of buildings. The most significant building environmental assessment systems used over the world are BREEAM, Green Globes, LEED, SBTool, CASBEE, HK-BEAM, NABERS, LEnSE, etc.

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2. BUILDING ENVIRONMENTAL ASSESSMENT SYSTEM PROPOSAL

The process of building environmental assessment has been developed in Slovakia as well through the last years. This topic is highly discussed in professional civil engineering practice. The fields and indicators of building environmental assessments are proposed on the base of available information analysis from particular fields of building environmental assessment and also on the base of our experimental experiences. The background for developing this system in Slovakia was mainly SBTool, The proposed indicators respect national standards and rules. The proposed fields are presented in the Table 1 [1, 2, 3, 4].

Table 1. Site selection, project management

Fields, sub-fields and indicators			Weights [%]
A	Site Selection, Project Planning and Development		14
A1	Site selection	Selection of ecologically valuable or sensitive land, Land vulnerable to flooding, Land close to water endangered contamination, Brownfield lands; Distance to commercial and cultural facilities, to public green space, to engineering networks, to road-traffic infrastructure	
A2	Project Planning	Assessment of renewable feasibility, Preparation of impact assessment report, Applicable orientation to maximize passive solar potential	
A3	Urban Design and Site Development	Development density; Possibility change building purpose; Relationship of design with existing streetscapes; Policies governing use of private vehicles; Use of trees for solar shading and sequestration of CO_2; Maintenance or development of wildlife corridors	
B	Building Construction		12
B1	Materials	Certified building products; Use of cement substitutes in concrete, materials that are locally produced, recycled materials; Non-renewable primary energy embodied in construction materials; Radioactivity building materials; Creation hazardous substances during production building materials; Selection low - emission building materials; Constructions limiting migration pollutions between occupations rooms, Eco-labeling	
B2	LCA	Dismountable, reuse and recycling; LCA impact on cost; LCA; Renewable	
B3	Energy flows	Factor of the building shape; U (exterior wall construction, roofing construction, the heating space floor area situated on ground, window, door); Insulation of exterior wall construction; Extension humidity in construction; Passive solar design; Period illumination indoor space of daylight; Relation between daylight and artificial light; Shadow; Ratio transparent constructions to non-transparent constructions exterior envelope ; Window glasses	
C	Indoor Environment	Thermal comfort in heating season, in cooling season; Ventilation; Air quality; Noise attenuation through the exterior envelope; Noise isolation between primary occupancy areas; Daylight; Shading and blind; Artificial light; Interior materials; Particular matters; Pollutant migration between occupancies	19
D	Energy		28
D1	Operation Energy	Energy for heating, domestic hot water, mechanic ventilation and cooling, lighting and energy for appliances	
D2	Active systems on using renewable energy sources	Solar system; Heat pump for heating and domestic hot water and cooling; Photovoltaic technology; Heat recuperation	
D2	Maintains Energy	Energy management; Operation and maintains	
E	Water	Reduction and regulation water flow; Surface water run-off; Drinking water supply; Using filtration “grey water”	12
F	Waste		14
F1	Solid waste	Solid waste; Measures to minimize solid waste resulting from building construction and operations; Composting	
F3	Emission	Measures to minimize gas waste from building construction, operation	

3. SITE SELECTION AND PROJECT MANAGEMENT

Site planning is an integral part of the land-use planning process; it determines the detailed layout of an area of land so that it functions effectively in relation to a given range of land uses on the site and others around it. It occurs directly before or is part of the detailed design process, depending on the complexity and scale of the site. In the overall planning process site planning occurs after the strategic planning has taken space and after the land use has been decided in relation to social, economic and environmental needs. Site planning is about working out the detail of what should happen on a given area of land, how it should happen and what it will cost to implement and manage the project on that area of land. Site plan are required for all developments involving the construction of buildings or other engineering structures: housing developments, industrial developments, commercial developments, recreation development, communications development. The site planning process aims to be more consistent in its cover of the environmental issues witch should determine the detailed layout and design of a site, than the relatively random approach so often taken by landowners and developers [5]. Site selection issues include transportation and travel distances for building occupants, impacts on wildlife corridors and hydrology, energy supply and distribution limitations. Decisions made during site selection and planning impact on the surrounding natural habitat, architectural design integration, building energy consumption, occupant comfort and occupant productivity. Maximize sustainability opportunities in the site selection and site planning process it is need to integrate site issues into the pre-design process. Some opportunities continue through design development and to a more limited extent, through facility and landscape construction [6]. Integrate the building with the site in a manner that minimizes the impact on natural resources, while maximizing human comfort and social connections. The development footprint should enhance the existing biodiversity and ecology of the site by strengthening the existing natural site patterns and making connections to the surrounding site context [7].

Table 2. Site selection and project management

A	Site selection and project management
A1	Site selection
A1.1	Selection of ecologically valuable or sensitive land
A1.2	Selection of land vulnerable to flooding
A1.3	Selection of land close to water
A1.4	Selection of Brownfield land
A1.5	Proximity of site to public transportation
A1.6	Proximity to commercial and cultural facilities
A1.7	Proximity to public green space
A1.8	Proximity to engineering networks
A2	Project planning
A2.1	Assessment of renewable feasibility
A2.2	Preparation of Environmental Impact Assessment report
A2.3	Lot orientation to maximize passive solar potential
A3	Site development
A3.1	Planned development density
A3.2	Plan for mixed used within the project
A3.3	Compatibility of urban design with local cultural values
A3.4	Planned policies governing use of private vehicles
A3.5	Provision of public green spaces
A3.6	Planned use of threes for solar shading and sequestration of carbon dioxide
A3.7	Maintenance or development of wildlife corridors

Above presented table (Table 2) summarizes the proposed field of site selection and project management. The way of proposed indicators evaluation is further presented in below table (Table 3).

Table 3. Site selection and project management

A	**Site selection and project management**			%
A1	**Site selection**			%
A1.1	**Selection of ecologically valuable or sensitive land**			%
Purpose	*To discourage the selection of land for building, where there is a high quality environment.*		score	weight
Indicator	*Class of environmental quality.*			
Negative	Site for construction, where is class of quality environment:	Strongly disturbed / disturbed environment		-1
Acceptable		Slightly disturbed environment		0
Good		Compliant environment		3
Best		High levels environment		5
A1.2	**Selection of land vulnerable to flooding**			%
Purpose	*To discourage the selection of land for building where there is a substantial risk that the site may be flooded.*		score	weight
Indicator	*Height above 100-year flood plain as defined in official documentation or assessment by competent authorities.*			
Negative	The height of the minimum elevation of the site above the elevation of the 100-year flood plain is:		1,0 m	-1
Acceptable			1,3 m	0
Good			2,0 m	3
Best			2,5 m	5
A1.3	**Selection of land close to water**			%
Purpose	*To discourage the selection of land for building where the risk of recipient pollution is low.*		score	weight
Indicator	*Distance of the building from water body or wetland as defined in official documentation or assessment by competent authorities.*			
Negative	To distance of the nearest body of water, including wetlands from the closest part of the site, is:		15 m	-1
Acceptable			25 m	0
Good			55 m	3
Best			75 m	5
A1.4	**Selection of Brownfield land**			%
Purpose	*To encourage the use of previously contaminated land for building.*		score	weight
Indicator	*Brownfield revitalization.*			
Negative	Land for building is located on Greenfield sites.			-1
Acceptable	Land for building is in the area without the previous construction.			0
Good	Land is located on the administrative, residential and commercial Brownfield.			3
Best	Land is located in the industrial, transport, military, agricultural and mining Brownfield.			5
A1.5	**Proximity of site to public transportation**			%
Purpose	*To encourage the selection of sites that is within a short distance of a public transport stop.*		score	weight
Indicator	*Distance to public transport stop from a main building entry or exit door.*			
Negative	Distance in meter to public transport stop from a main building entry or exit door is:		> 500 m	-1
Acceptable			500 m	0
Good			250 m	3
Best			100 m	5
A1.6	**Proximity to commercial and cultural facilities**			%
Purpose	*To encourage the selection of sites with access to at least*		score	weight

	a basic range of commercial and cultural facilities within walking distance.		
Indicator	*Distance to a wide range of food, and a basic range of other retail and cultural needs.*		
Negative	Distance in meter to a wide range of food, and a basic range of other retail and cultural needs is:	> 1000 m	-1
Acceptable		1000 m	0
Good		700 m	3
Best		500 m	5
A1.7	**Proximity to public green space**		%
Purpose	*To encourage the selection of sites that are located within distance of public green spaces, suited for recreation or sport use.*	score	weight
Indicator	*Distance to public open green space suitable for recreation and sports.*		
Negative	Distance in meter to public open green space suitable for recreation and sports is:	> 1000 m	-1
Acceptable		1000 m	0
Good		700 m	3
Best		500 m	5
A1.8	**Proximity to engineering networks**		%
Purpose	*To encourage the selection of sites with access to at least within distance of engineering networks.*	score	weight
Indicator	*The possibility of engineering networks connection.*		
Negative	There is not the possibility of engineering networks connection.		-1
Acceptable	There is possibility of connection to some network engineering.		0
Good	There is possibility of engineering network connection.		3
Best			5
A2	**Project planning**		%
A2.1	**Assessment of renewable feasibility**		%
Purpose	*To encourage the consideration of the technical and economic feasibility of renewable energy at the pre-design stage.*	score	weight
Indicator	*Result from analysis of feasibility using software.*		
Negative	The software has not been used to carry out a study of the feasibility of using renewable energy systems for the project.		-1
Acceptable	The software has been used to carry out a study of the feasibility of using one renewable energy technology for the project.		0
Good	The software has been used to carry out a study of the using three renewable energy technologies for the project.		3
Best	The software has been used to carry out a study of the feasibility of using more than three renewable energy technologies for the project.		5
A2.2	**Preparation of Environmental Impact Assessment report**		%
Purpose	*To ensure that an Environmental Impact Assessment study is carried out (Law 24/2006).*	score	weight
Indicator	*Intention* (office building from 5 000 m^2 floor areas).		
Negative	A credible detailed plan exists for the preparation of an Environmental Impact Assessment (24/2006).		-1
Acceptable	A credible detailed plan exists for the preparation of a high-quality Environmental Impact Assessment (24/2006).		5
Good			
Best			
A2.3	**Orientation to maximize passive solar potential**		%
Purpose	*To ensure that the project site plan provides for the location and orientation of building than will maximize passive solar potential.*	score	weight

Indicator	*The percentage of building areas ratios oriented to East-West.*		
Negative	The percentage ratio of building oriented to East-West with deviation 15%, is:	40%	-1
Acceptable		50%	0
Good		80%	3
Best		100%	5
A3	**Site development**		%
A3.1	**Planned development density**		%
Purpose	*To encourage the efficient use of urban land, within the context of an urban development plan.*	score	weight
Indicator	Development density of the project, expressed as the ratio of overall floor area above grade of the design relative to the maximum allowed overall floor area on the site.		
Negative	The ratio of overall floor area above grade of the design relative to the maximum allowed overall floor area on the site, as a percent, is:	40%	-1
Acceptable		50%	0
Good		80%	3
Best		100%	5
A3.2	**Plan for mixed used within the project**		%
Purpose	*To encourage a diversity of major uses within the project, to support an active streetscape on a continuous basis and to reduce the need for commuting transport.*	score	weight
Indicator	Number of major uses within the project, related to a threshold area.		
Negative	The project is large than the threshold area and contain one occupancy type.		-1
Acceptable	The project is large than the threshold area, and more than 90% of the total net building area consist of one occupancy type.		0
Good	The project is large than the threshold area and contains two major occupancy types.		3
Best	The project is large than the threshold area and contains three or more major occupancy types.		5
A3.3	**Compatibility of urban design with local cultural values**		%
Purpose	*To ensure that urban design and architecture of building is compatible with local values.*	score	weight
Indicator	Expert assessment of the degree to which new features, systems and materials are consistent with local cultural values related to urban design and architecture, including both functional and aesthetic aspects.		
Negative	Architectural features of the design are incompatible with existing cultural values related to urban design and architecture, including both functional and aesthetic aspects.		-1
Acceptable	Architectural features of the design are marginally compatible with existing cultural values related to urban design and architecture, including both functional and aesthetic aspects.		0
Good	Architectural features of the design are fully compatible with existing cultural values related to urban design and architecture, including both functional and aesthetic aspects.		3
Best	Architectural features of the design are an outstanding example compatible with existing cultural values related to urban design and architecture, including both functional and aesthetic aspects.		5
A3.4	**Planned policies governing use of private vehicles**		%
Purpose	*To discourage the use of private vehicles through fees or reduce number of parking spaces.*	score	weight
Indicator	Number of parking spaces provided relative to local parking regulations, measures taken to discourage the use of private vehicles by occupants, and incentive provided for the use of public transport.		

Negative	More parking spaces are provided than is required by local regulations, and there is no plan for public transport.		-1
Acceptable	Parking spaces are provided in conformance with local regulations, and there is a general plan for public transport.		0
Good	Parking spaces are provided in conformance with local regulations, and fees are planned to control their use, and there is a detailed plan for public transport.		3
Best	Parking spaces are provided in conformance with local regulations, fees are planned to control their use, incentive are planned for users of public transportation systems, and there is a detailed plan for public transport.		5
A3.5	**Provision of public green spaces**		%
Purpose	*To provide public green spaces for informal recreation of the project population.*	score	weight
Indicator	*The provision of land within the site suitable as public green spaces because of its area, soil, access to water or other characteristic.*		
Negative	Land with suitable soil, access to water and other characteristic that is allocated as public green space, has an area, expressed as a percent of the total site area, of:	40%	-1
Acceptable		50%	0
Good		80%	3
Best		100%	5
A3.6	**Planned use of threes for solar shading and sequestration of carbon dioxide**		%
Purpose	*To encourage the use of threes for decrease of carbon dioxide concentrations, especially in location where they may also provide shading of the building during the hot season.*	score	weight
Indicator	*Native threes retained or planted, according to landscaping plans and specifications; measured as percent of building frontage facing the equator, at a height of 5 m that will be covered by leaves during the hot season within 4 years.*		
Negative	According to landscaping plans and specifications, native threes will provide shade at a height of 5 m on the building facing the equator, equal to:	10%	-1
Acceptable		25%	0
Good		70%	3
Best		100%	5
A3.7	**Maintenance or development of wildlife corridors**		%
Purpose	*To encourage the maintenance or development of continuous areas of vegetation that can serve as corridors for the passage of wildlife.*	score	weight
Indicator	*The minimum width of a continuous zone of vegetation, including threes and native planting that has connections with similar vegetated areas off the site, according to landscaping plans.*		
Negative	A wildlife corridor is provided as part of the landscaping plan, and is linked to similar areas adjacent to the site. According to the design, the minimum width will be:	< 50 m	-1
Acceptable		50 m	0
Good		300 m	3
Best		500 m	5

4. ASSESSMENT OF OFFICE BUILDING – VERIFICATION OF FIELD "SITE SELECTION AND PROJECT MANAGEMENT"

The assessed office building was evaluated for the purpose of the developed system verification. The evaluated office building was assessed according to available documentations, mainly drawings. The assessment was realized by software tool prepared in MS Excel.

The office building is located in the city of Košice, in Slovak republic. City Košice belongs to the areas with deteriorated environment. Košice is one of the major economic and cultural centers of eastern Slovakia. It is a reason for increased proportion of urban population. The evaluated office building is situated in built-up areas, partially on Brownfield and Greenfield.

The Figure 1 illustrates the results from building evaluation. The results from the comprehensive environmental assessment of selected office it can assert, that it is necessary to propose measures to improve the environmental suitability, safety and reliability of the evaluated office in all assessed fields.

Fig 1. Site selection and project management assessment of office building

5. CONCLUSIONS

In this paper is introduced the proposal of one significant field "Site selection and project management". There are presented relevant indicators from mentioned field and method of its evaluation with respects to standards and acts valid in Slovakia. In addition to this field, the comprehensive building environmental assessment system constitutes further five main fields: building constructions; indoor environment; energy performance; water management and waste management. That means the proposed system consists of six main fields and 53 relevant indicators. The theoretical level of present knowledge of building environmental assessment is completely. It is necessary to implement this knowledge to construction practice. The following research work will be aimed at determination of significance weights of indicators. For the purpose of assessment system verification, it is needed to evaluate a statistically significant set of buildings. The results from further system verification will allow modify the significant weights of indicators.

The proposal of building environmental assessment system requires a complex multidisciplinary and multicriterion approach. The aim of building environmental assessment is a sustainable building design, which demands the cooperation among civil engineers, architects, environmentalists and other experts from different areas of building environmental assessment.

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METHODS FOR THE DETERMINATION OF THE MODULUS OF ELASTICITY OF COMPOSITE MATERIALS USED IN A PAVEMENT STRUCTURE

ABSTRACT

Road building materials are exposed to forces caused by traffic load, climatic influences, but above all to air temperature changes and moisture. These effects manifest themselves in the cyclic changes in a pavement structure and its sub-grade. Force effects induce complicated stress and strain condition in individual pavement layers. The strain of various materials caused by the same load is different and depends on their deformation characteristics. The deformation characteristics of road building materials are important inputs in the project and design of a road pavement structure.

More demanding requirements on materials as well as ecological considerations regarding the utilization of materials in construction, and other issues, require more research and the creation of new materials with outstanding and more suitable physical properties than those offered by the current materials. One of the major research results is the discovery of modern materials called composites. The geometrical and physical structure of the composite material influences its overall characteristics. The analytical modelling of the particle components of composites, the so-called quasi-homogenous and quasi-isotropic models of compact materials, enables to assume and define these characteristics.

KEYWORDS: road building materials, composite materials, pavement, modulus of deformation,

1. INTRODUCTION

Nowadays, progressive technologies, such as the re-processing or recycling of bituminous materials are more and more commonly used in road construction. Recycling is one of possible solutions for saving material and energy and protecting the environment. In order to maximize such savings, it is crucial to know what an impact the recycled material has on the final material properties of the reused mixed material and how the mixture consisting of recycled material will behave in a road pavement. The deformation characteristics of road

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building materials are important inputs in the project and design of the road pavement structure.

The moduli of elasticity, deformation, the complex modulus, and Poisson's ratio are considered the most important deformation characteristics the knowledge of which is very important to the design and assessment of mixtures containing R-materials (recycled materials) classified as composite materials.

2. MECHANICS OF COMPOSITE MATERIALS

From the main parameters, on which the characteristics of composites depend, we define in particular:

- Characteristics of the phases, i.e. mechanical characteristics and their proportion (firmness, modulus of elasticity, Poisson coefficient, pressure-volume diagram, marginal deformation) and anisotropies of the characteristics of the individual phases, of which the composite is composed of.
- capacity representation of the phases, their geometric shape and their geometric organization in the system i.e. quantity, orientation and average distances of the inserted phases.
- Interaction of the individual phases and character of the contact i.e. transferability of the load from the matrix in reinforcing sections and on the contrary. Cohesion at contact of the phases substantially depends on the proportion of the thermal expansion coefficient, on visco-elastic characteristics of the contact and on anisotropy of the characteristics of inserted elements.
- Interaction with surrounding settings, which to large extent depends on the first three parameters.
- History of the material and its phases from their creation, which involves in particular time factors, production technology, transformation of elasticity modulus with temperature, external strain, influence of the settings etc.

It follows from the abovementioned that the composites are related to a range of science discipline, each of which has specific approaches, special aspects of research and realization level. Since strain systems are concerned, in which characteristics of materials are bound to their structure, a simplified view can be provided by the mechanics of composite materials [4], [5].

Many times the composite materials can be represented by equivalent idealized homogeneous material, in case of which we assume that its response is identical as the average response of the real material in the same border conditions.

- First access, so-called fictive phase homogenization, is successful in case of determination of elastic constants and of other physical characteristics under fulfilment of the condition of full phasing geometry description.
- Second approach, so-called stress approximation is based on the well - known phasing geometry and uses approximation about character of the stress field. The materials are represented by various combination of simple elements in series or in parallel. These methods can be only exceptionally used successfully.
- Third access utilizes variation principles for obtaining of the sought physical characteristics. The most used one at present is the finite element method.

The following empirical relations under assumption of insertion of solid elements into a more flexible matrix and their perfect cohesion can be used for calculation of the elasticity modulus and other physical characteristics for composite materials with the granulate elements in matrix:

$$E = E_m\left(1 + \alpha V_f\right) \tag{1}$$

where E_m is elasticity modulus of the matrix, V_f is volume proportion of the dispersed granulate, $\alpha = 2{,}5$.

Interaction of the particular elements can be included by introduction of so-called factor of organization and this way we obtain:

$$\lg\frac{E}{E_m} = \frac{2{,}5V_f}{1 - \beta V_f} \tag{2}$$

where β depend on organization of the particles and filling of the space.

According to experimental results further modifications of the equation (1) have been proposed, e.g.:

$$E = E_m\left(1 + \frac{1{,}25V_f}{1 - 1{,}28V_f}\right)^2 \tag{3}$$

For lower volume share of the filling (approximately up to 30% for round particles and up to 10% for others) the empirical equation can be used:

$$E = E_m\left(1 + 2{,}5V_f + 14{,}1V_f^2\right) \tag{4}$$

In the effort to include geometry into the calculation, influence of the dimensional coefficient *f* has been included into the calculation:

$$E = E_m\left(1 + 0{,}67 fV_f + 14{,}1V_f^2 f^2\right) \tag{5}$$

The experimental results has showed that in practical applications other influences take effect in a decisive way, and therefore in case of larger filling the material characteristics can be according to the fuel type also smaller and the stated relations will not provide a satisfactory forecast of the composite's behaviour.

3. RECYCLING OF MIXED BITUMINOUS MATERIAL

Technologies for the recycling of bituminous mixtures belong to very progressive reprocessing technologies. Besides their economic benefits, they have a fair number of other positive effects on saving natural resources and protecting the environment. According to the type of material used for road surfacing and then its recycling, these technologies can be divided into:

- the recycling technologies used for bituminous pavements,
- the recycling technologies used for concrete pavements [1].

The recycling of bituminous road surfacing material may be applied in road reconstruction, repair or renovation of pavements, whether by hot recycling (in-situ or off-site) or cold recycling (in-situ or off-site).

In-situ recycling technologies enable the 100 per cent reuse of the original material removed from a bituminous pavement. Moreover, the in-situ recycling takes place continually without the necessity for traffic to be diverted.

Cold in-situ recycling is preferred in the renovation of flexible bituminous road pavements in the other half of their intended service life that need serious excessive repair (not only changing the wearing course of the road but also repairing the layers in the road foundation). Using this technology, the original structure of the pavement is scarified by a recycler to the desired depth and by adding a binder it is worked into a load-bearing road foundation on which a new road surfacing is laid. Various substances, such as bituminous emulsion, blown bitumen or cement can be used as a binder.

Cold off-site recycling can be carried out in a stationary mixer or in a travel mixer. Mixing in a travel mixer is more efficient as the travel mixer moves along the line of the road being renovated and thus helps to save the cost of transfer of stabilized mixed material. What is more, the scarified bituminous material from the original road pavement can be effectively used as the substitution for aggregate. There is a wide range of possibilities for combining binders, which is identical to the cold in-situ recycling [1].

In the design of the composition of a recycled mixture, it is essential to apply such testing procedures that simulate the strain of the recycled layer in a pavement structure. In practice, it is necessary to detect how the layer containing the recycled material will resist the loads of traffic and to know its deformation characteristics. Besides the observation techniques, these characteristics can be determined by laboratory tests.

4. EXPERIMENTS – THE DESIGN OF COLD RECYCLED MIXTURES

Within the frames of a research programme currently in progress at our workplace, some cold recycled mixtures bound with bituminous emulsion and cement suitable for the use in the upper foundation layers of a road pavement structure were designed and tested.

The design was based on the assumption that the recycled mixed material will be used as the upper foundation layer in the renovated pavement and will serve as an alterative to bituminous CMGM road-mix (coated medium-grained macadam). In mixing the recycled material the following materials were used:

- the scarified material from the bituminous pavement layers,
- natural crushed quarry stone from Trebejov (limestone), grading 0-2,
- limestone dust from Margecany,
- cement graded as II/B S 32,5 R,
- slowly-breaking cationic bituminous emulsion containing 60 per cent of bitumen.

For further research studies, two series of material mixtures were designed, while in the determination of the mutual ratio between the two kinds of material in the fundamental mixture, a grading curve for the aggregate containing the extracted R-material as well as the grain-size distribution of the R-material were taken into account. The grading of the mixed material was designed so that the resulting grading curve fell within the grading range for coated medium-grained macadam given in STN 73 6121 [3].

The mixture marked A contained 80 per cent of the R-material and the one marked B 60 per cent of the R-material. The grading curves for the fundamental mixtures are shown in Fig. 1.

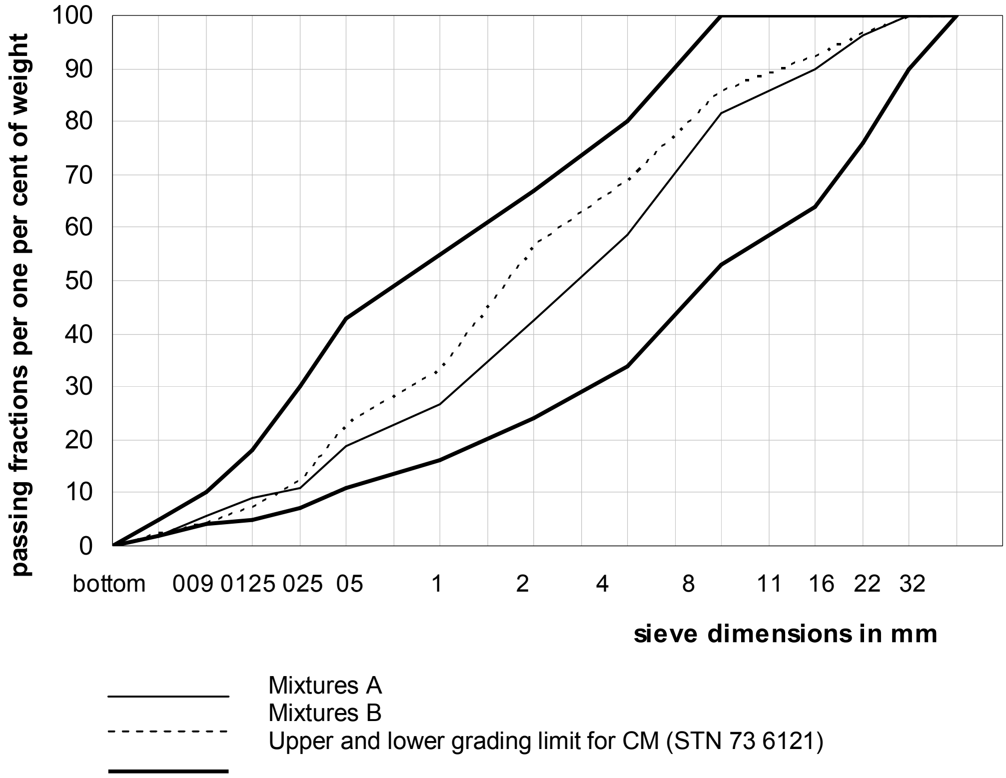

Fig. 1. Grading curves for the designed mixtures A and B

For both types of mixture, four combinations of the amounts of bituminous emulsion and cement were designed. The composition of stabilized mixtures is shown in Tab. 1.

Table 1. Composition of mixtures stabilized with bituminous emulsion and cement

Designation of stabilized mixtures	The amount of material per one per cent of weight of the fundamental mixture					
	R-material	Grading 0 - 2	Limestone dust	Bituminous emulsion	Cement	Water
A4	80	14	6	2.0	3	4.9
A6	80	14	6	3.5	3	3.8
B4	60	40	–	3.0	3	3.1
B6	60	40	–	4.5	3	2.0

The optimum moisture content in the designed mixtures was determined by Proctor compaction test, method B, in compliance with STN 72 1015 [4].

As part of the research study, the deformation characteristics of the mixtures (the modulus of elasticity and Poisson's ratio) necessary for the design and evaluation of the designed pavement structure were determined.

5. THE DETERMINATION OF POISSON´S RATIO AND THE MODULUS OF ELASTICITY FROM A TENSILE TEST

Poisson’s ratio and the modulus of elasticity were extracted from a tensile test according to STN EN 13286-43 [5]. During the test the force *F*, the changes in the length of a horizontal diameter AB and a diameter CD shifted by 60˚ (Fig. 2) were recorded.

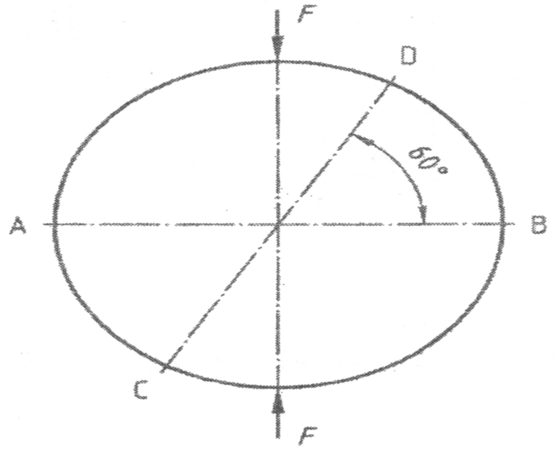

Fig. 2. The data necessary for the determination of Poisson’s ratio and the modulus of elasticity

A Poisson’s ratio v can be specified as follows:

$$v = \frac{(1 + 0.4\Xi)}{(1.73 - 1.07\Xi)} \tag{6}$$

where: Ξ - the ratio $\Delta\O_{60} / \Delta\O_0$,
$\Delta\O_{60}$ - the change in the length of a diameter CD shifted by 60˚ when $F=0.3\ F_r$,
$\Delta\O_0$ - the change in the length of a horizontal diameter AB of the object when $F=0.3\ F_r$,
F - the force applied to the object (N),
F_r - the maximum force which the object can withstand (N).

The tensile test can be used also for the determination of the modulus of elasticity E (MPa) using the following formula:

$$E = \frac{0.5\overline{P}(0.274 + \mu)}{h\Delta_{05}} \tag{7}$$

where: P - the average value of the breaking force (N),
h - the height of the object tested (mm),
Δ_{05} - the value of transverse strain corresponding to 0.5P (mm),
μ - Poisson’s ratio.

The Poisson’s ratio values and the modulus of elasticity present an important input in the calculation of strain in a pavement structure. It is necessary to specify these values at three temperatures typical of the temperature regime of a pavement. This regime is specified separately for three year’s seasons in a form of average equivalent temperatures, such as:

- 0 °C - for the winter season that covers 20 per cent of the year,
- 11 °C – for the spring and autumn seasons that last for 50 per cent of the year, and
- 27 °C – for the summer that covers 30 per cent of the year.

The Poisson's ratio values and the modulus of elasticity were tested and determined at temperatures of 0, 11 and 27°C. The values of the deformation characteristics of the designed cold recycled mixtures (Poisson's ratio and the modulus of elasticity in tension) were compared to the values of the deformation characteristics of the comparative/reference road-mix of medium-grained coated macadam (CM).

a) The modulus of elasticity in tension

The resulting values of the modulus of elasticity in tension determined at temperatures 0, 11 and 27 °C are given in Tab. 2.

Table 2. Modulus of elasticity in tension at temperatures of 0, 11 and 27 °C

Mixture/Mixed material	E^{0} (MPa)	E^{11} (MPa)	E^{27} (MPa)
A4	5233.53	4810.92	4107.05
A6	6038.36	4896.01	4580.52
B4	5337.49	4307.61	4330.67
B6	5585.54	4695.57	4426.81
CM (the comparative/reference road-mix)	4500	3050	1250

For greater clarity, the resulting values of the modulus of elasticity in tension are shown in a graph (Fig. 3).

It can be seen from the comparison of the values of the modulus of elasticity of the designed cold recycled mixtures with the modulus of elasticity of the comparative/reference road-mix of coated macadam (CM) that all observed mixtures reached higher values of the modulus of elasticity than the comparative mix at various temperatures.

The highest values of the modulus of elasticity were observed in the mixtures at a temperature of 0 °C and the lowest values were determined in the mixtures at a temperature of 27 °C.

The higher values of the modulus of elasticity were observed in both series A and B in the mixtures with a higher content of bituminous emulsion at all three temperatures. In mixtures A, it was A6, and in mixtures B, it was B6.

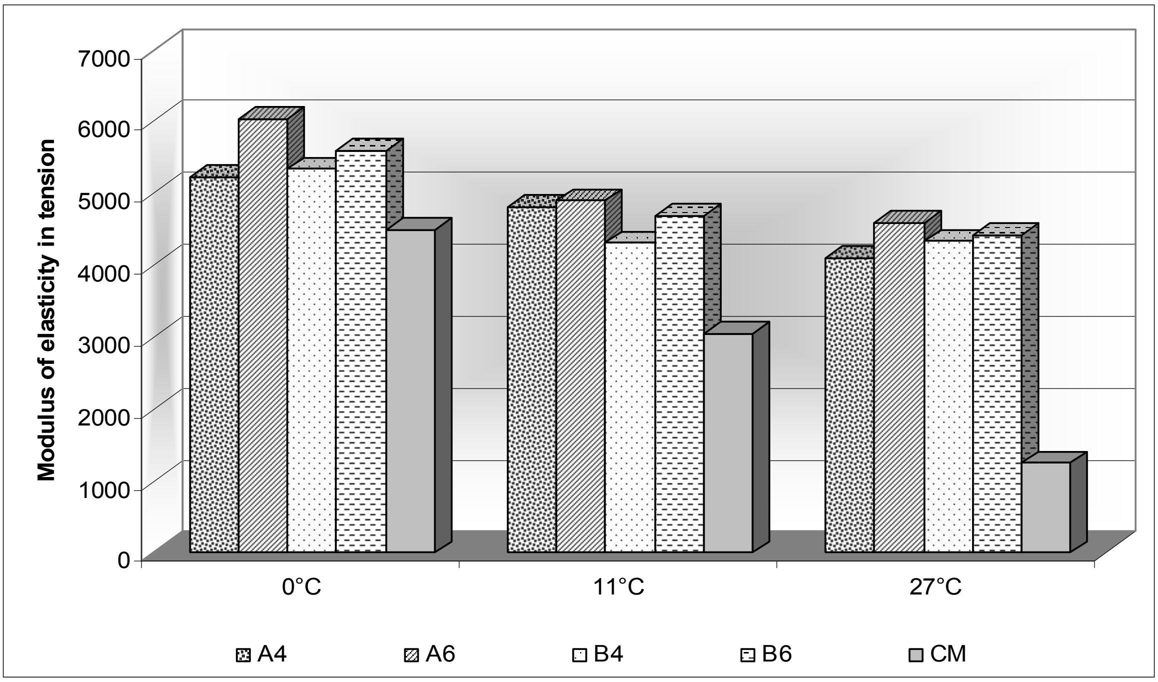

Fig. 3. Modulus of elasticity in tension for the designed mixtures at temperatures 0, 11 and 27 °C

The greatest value of the modulus of elasticity of all mixtures was recorded for mixture A6 with an 80 per cent content of the R-material at all temperatures (3.5 per cent of bituminous emulsion + 3 per cent of cement).

b) Poisson's ratio

The Poisson's ratio values determined for the designed cold recycled mixtures at temperatures of 0, 11 and 27 °C are shown in Tab. 3 and, for greater clarity, in a graph in Fig. 4. Identically, the values determined were compared to the values for the comparative/reference road-mix of coated macadam (CM).

Table 3. The values of Poisson's ratio determined at temperatures 0, 11 and 27 °C

Mixture/Mixed material	Poisson's ratio		
	0 °C	11 °C	27 °C
A4	0.36691	0.38235	0.40186
A6	0.36694	0.38684	0.41238
B4	0.36293	0.38734	0.40457
B6	0.35302	0.38049	0.39354
CM (the comparative/reference road-mix)	0.21	0.33	0.44

The Poisson's ratio values for the designed cold recycled mixtures determined at temperatures of 0 and 11 °C are greater than those for the comparative/reference road-mix. The lower values of Poisson's ratio were recorded only at a temperature of 27 °C.

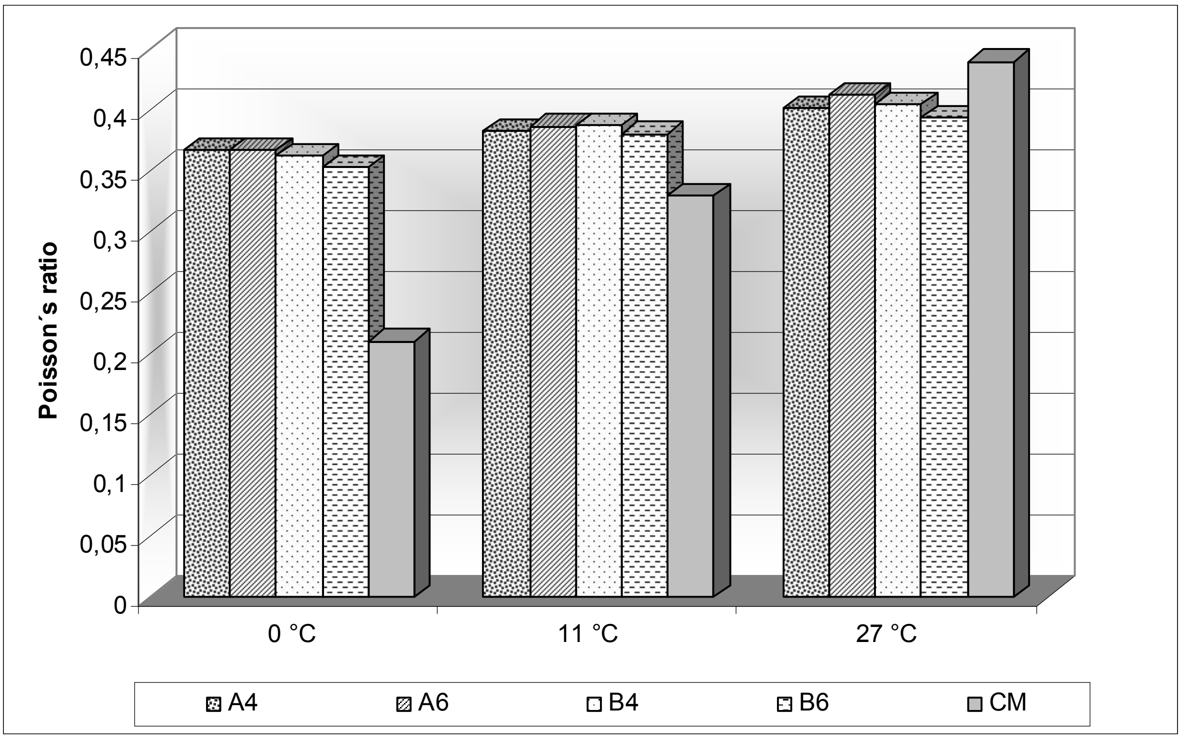

Fig. 4. Poisson's ratio values at temperatures 0, 11 and 27 °C

As can be seen from the results presented, Poisson's ratio increases with an increasing temperature. There is no significant difference between the individual values of Poisson's ratio for the tested recycled mixtures. An increase in Poisson's ratio related to an increasing temperature is caused by the content of bituminous emulsion. Once again, a higher content of bituminous emulsion proved to be more appropriate (particularly at a temperature of 27 °C for mixture A6).

6. CONCLUSION

A great amount of high quality material, aggregate and bitumen in particular, can be found in the road pavement structures that no longer satisfy the requirements for operational reliability of roads and need repair. This material can be recycled and effectively reused in the construction of roads. Cold in-situ recycling processes combine a number of social, economic and ecological aspects. An obvious and major advantage of this technology is the 100 per cent reuse of all original material (contained in the bituminous mixture and the road foundation), which means a 100 per cent of savings on the investment costs. The cost of transport of the scarified/used material and the optional cost of its storage are saved as well. Therefore, the use of new material in road construction is restricted only to the amount of aggregate improving the grading of the recycled material and a binder.

Based on the results obtained, it can be concluded that the designed cold recycled mixtures are suitable for the use in the foundation layers of a road pavement. The best results were achieved for the cold recycled mixtures containing a great amount of binding bituminous emulsion. Moreover, the achieved results have shown that there is still a possibility of using the recycled material that meets the standard criteria for road pavement construction.

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Szczepan WOLIŃSKI[1]

CONFORMITY CONTROL OF CONCRETE STRENGTH BASED ON THE RISK ASSESSMENT

ABSTRACT

In the conformity control of concrete, different criteria are in use. The current design codes are based on the semi-probabilistic design format. This supposes that a sufficient degree of control exists to assure that a distribution of concrete strength is in accordance with the assumption on which the design procedure is based. In the paper the following aspects of conformity control of concrete strength are covered:

- general concept and types of conformity criteria,
- hazard and risk in conformity control,
- risk-based conformity criteria,
- concept of risk balance between producer and client.

KEYWORDS: conformity control, concrete strength, OC-curves, risk assessment.

1. INTRODUCTION

Conformity control of concrete has been defined as verification of the produced concrete compliance with the specified properties. The most important specified property of concrete is compressive strength and most control plans are related to this property. However, other properties, especially these related to durability requirements are also specified in different standards and recommendations, e.g. in EN 206-1 [1]. Some examples of commonly specified properties of concrete include: W/C-ratio, consistence, cement content, air content, density. Generally, conformity control is based on various criteria but always the sample size is limited. Hence, there is always a possibility of taking the wrong decision which means that in some cases a lot of good concrete will be rejected or a lot of bad concrete will be accepted. The associated possibilities are commonly called the producer's "risk" and the client's "risk". However, in this context the term "risk" means in fact the probability of rejection or acceptance of a concrete lot and conformity criteria proposed in contemporary standards and design guidelines are based on balancing probabilities of rejection and acceptance of concrete lot between the producer and the client. Contemporary, the risk is defined as a measure of a danger or hazard and is often expressed as the combination (usually the product) of probability and consequences of hazard. An approach based on the weighted risk balance is presented and discussed in this paper.

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2. GENERAL CONCEPT OF CONFORMITY CRITERIA

The current design standards are based on the partial factors, semi-probabilistic design format. This implicitly supposes that a sufficient degree of control exists to assure that a distribution of concrete strength is in accordance with the assumption on which the design procedure is based. In compliance control the applied statistical conformity criteria are of the following type:

$$g(x) \geq \alpha \tag{1}$$

where $g(x)$ is the compliance function consists of test statistics combination, α is an expression which is linear with characteristic strength of concrete strength f_{ck}.

The evident choice of $g(x)$ is an estimate of the fractile of a distribution, that is a tolerance limit. This leads to the basic form of the statistical criterion:

$$g(x) = \bar{x}_n - \lambda\, s_n \geq f_{ck} \tag{2}$$

where $\bar{x}_n$ and s_n are the sample mean and standard deviation, λ is the parameter which depends on the confidence level of the tolerance limit, related to the acceptance probability P_a. The specified characteristic strength f_{ck} corresponds to the 0,05-fractile of the theoretical distribution of concrete strength. In practice, the fraction of sample units which strength is below the specified f_{ck} is smaller or higher than 0,05. The fraction of samples with the strength below f_{ck} in a plot offered by producer is called the fraction defective θ:

$$P(f_c \leq f_{ck}) = \theta \tag{3}$$

where f_c is the compressive strength, which is a random variable.

The probability that a concrete lot, characterized by a random fraction defective θ, is accepted is called the probability of acceptance P_a. The operation characteristic of the conformity criterion is the function $P_a(\theta)$, called the OC-curve or OC-line (Fig. 1).

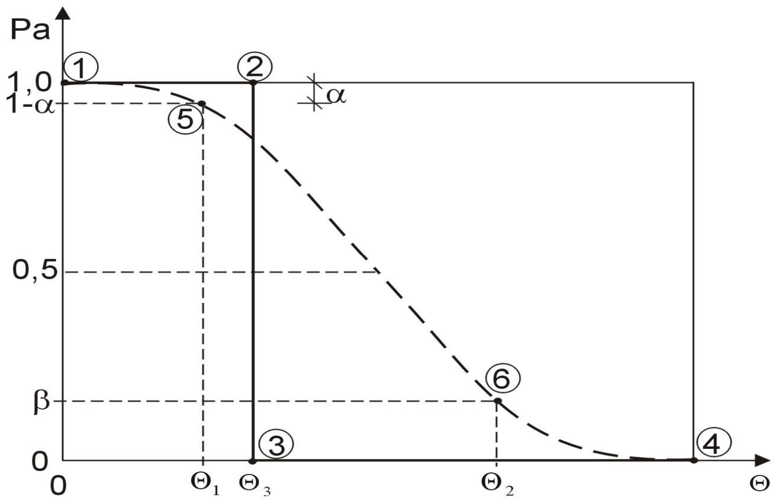

Fig. 1. Illustration of an OC-line

For a hypothetical sample of infinite size, the ideal OC-line has a shape 1-2-3-4 (Fig.1) and allows to make a perfect distinction between good and bad production. In this case $P_a = 1$ for $\theta < \theta_3$ and $P_a = 0$ for $\theta > \theta_3$. The real OC-line for a sample of finite size, e.g. 1-5-6-4 (Fig.1), allows to make a distinction between good and bad production with continuously changing probability. In this case the rejection probability of a good quality lot $\alpha = 1 - P_a$, and the acceptance probability of a bad quality lot β, should be balances between the producer and the client. However, the equal probability of rejection and acceptance does not assure the same producer's and the client's risk.

3. TYPES OF CONFORMITY CRITERIA

A representative types of common conformity criteria were included in CEP-FIP Model Code 1978 and today are specified in the European Standard EN 206-1 [1]:

$$\bar{x}_n \geq f_{ck} + \lambda\,\sigma \quad \text{and} \quad x_{\min} \geq f_{ck} - k_2 \tag{4}$$

$$\bar{x}_n \geq f_{ck} + \lambda\, s_n \tag{5}$$

$$\bar{x}_n \geq f_{ck} + k_1 \quad \text{and} \quad x_{\min} \geq f_{ck} - k_2 \tag{6}$$

where: $\bar{x}_n$ is the sample mean value, σ and s_n are the known and the sample standard deviation, $x_{\min}$ is the smallest result value in the sample and λ, k_1, k_2 are parameters.

In case of continuous production of concrete with a sample size $n \geq 15$, $\lambda = 1{,}48$ and $k_2 = 4$ the compound criterion (4) or the criterion (5) are recommended and in case of initial production with $n = 3$, $k_1 = k_2 = 4$ the criterion (6) is recommended in EN 206-1 [1].

Generally, conformity criteria should fulfil three major requirements:

a) with bigger sample size *n* lower probability of acceptance of bad quality lot and higher probability of acceptance of good quality lot should correspond;
b) among lots of good quality the conformity criteria should privilege those with lower variability;
c) the probability of acceptance of bad quality lot should not be higher than a predefined value (for the concrete strength $\beta \leq 0{,}05$).

Systematic analyses of conformity criteria of types (4), (5) and (6), using the Monte Carlo simulation method, have shown that in many cases they do not fulfil these requirements and can have prejudicial effects to producers and clients [2, 3, 4]. They may lead to wrong production strategy with higher costs, because:

a) for small sample sizes the probability of acceptance increases with standard deviation of the concrete production, and it may lead to the wrong production strategy with high mean values and standard deviation of concrete strength,
b) for sample sizes $n \geq 6$ standard deviation does not influence significantly the probability of acceptance;
c) the requirement of minimum values is critical only for very high values of standard deviation,
d) they may lead to production with too high values of the client's risk.

Generally, the usefulness of statistical criteria for small sample sizes, expressed by the probability that a lot of concrete with known fraction defective can be accepted, is questioned. In case of the non-destructive control of concrete strength when rejected lots are screened the Average Outgoing Quality Limit (AOQL) concept can be used [5]. The λ - value in the criterion (5) is determined for assumed value of $AOQL = (\theta \times P_a)_{\max}$, e.g. for concrete strength the AOQL value should be fixed at 0,05 corresponding to the definition of f_{ck}. In this case probabilities α and β are balanced between the producer and the client, but the equal probability of rejection and acceptance of a concrete lot does not assure the same producer's and the client risk.

4. CONFORMITY CRITERIA BASED ON THE RISK ANALYSIS

4.1. Hazard and risk

Hazard is defined as an attribute of activities which may cause harm to persons and assets or as a set of circumstance with the potential for causing events with undesirable consequences. Risk may be referred to as a measure of the danger or hazard and usually is defined as combination of the probability of occurrence and the consequence of a specified hazardous or undesired event [6, 7, 8]. In the case of conformity control, hazard is an attribute of wrong decision, namely rejection of a good quality lot or acceptance of a bad quality lot of concrete. For a set of hazardous design situation H_i the total risk *R* can be calculated as follows:

$$R = \sum_{i=1}^{n_H} p(H_i) \sum_{j=1}^{n_D} \sum_{k=1}^{n_S} p(D_j | H_i) p(S_k | D_j) C(S_k) \tag{7}$$

where: the system (process of conformity control) is subjected to n_H different hazards that may cause harm to the producer's and the client's assets in n_D different ways and the performance of the two parties involved in control can be discretised into n_S adverse states S_k with corresponding consequences $C(S_k)$, and $p(H_i)$ is the probability of occurrence of the *i-th* hazard H_i, $p(D_j | H_i)$ is the conditional probability of the *j-th* undesired state of the system given in the *i-th* hazard and $p(S_k | D_j)$ is the conditional probability of the *k-th* adverse overall system performance *S* given in the *i-th* damage state.

An exposure is considered as any circumstance or set of circumstances with the potential to cause undesirable consequences. A description of a series of circumstances in time, and inter-relationship among the circumstances provided the occurrence of a hazard, is called a scenario of an exposure.

In the conformity control of concrete, only two basic hazardous situations may be considered; rejection of a good quality lot and acceptance of a bad quality lot. Therefore, using the formula (7) the producer's risk R_{pr} and the client's risk R_{cl} can be expressed as follows:

$$R_{pr} = [p(H_1)]^2 \sum_{k1=1}^{n_{S1}} p(S_{k1} | H_1) C(S_{k1}) \tag{8}$$

$$R_{cl} = [p(H_2)]^2 \sum_{k2=1}^{n_{S2}} p(S_{k2} | H_2) C(S_{k2}) \tag{9}$$

where $p(H_1) = \alpha$, $p(H_2) = \beta$ are the rejection probability of a good quality lot and the acceptance probability of a bad quality lot, respectively.

4.2. Risk-based conformity criteria

Generally, risk treatment involves identifying the range of options for treating risks, assessing these options and implementation of treatment plans [6, 7, 9]. Risk criteria are expressions of decision-maker's values and they must be consistent with the risk management goals. There are four general approaches to risk treatment: avoidance, reduction, transfer (usually insurance) and retention.

Risk in conformity control of concrete strength should be balanced between two sides involved; the producer and the client. The balanced risk does not necessarily mean the equal risk. A risk partition into the producer and client should be the matter of a trade contract. A risk balance index i_R expresses how the risk is divided into two sides involved:

$$i_R = \frac{R_{cl}}{R_{pr} + R_{cl}} \tag{10}$$

The risk balance index takes values $0 \le i_R \le 1$. If all risk is transferred to the producer $i_R = 0$. If all risk is transferred to the client $i_R = 1$. If a risk is transferred fifty-fifty to both sides $i_R = 0{,}5$.

The reliability-based codes specify design and detailing requirements that are intended to achieve a minimum safety level. The use of characteristic material properties which are defined as the 5%-fractile of the theoretical distribution of material property, implicitly supposes that the basic form of the conformity criteria should be of the type (4), (5) or (6). For the known in advance probabilities and costs of hazard's consequences which must be accepted by the producer and the client, and for an assumed i_R-value, the probabilities $p(H_1) = \alpha$ and $p(H_2) = \beta$ can be calculated from equations (8), (9) and (10) as follows:

$$\alpha = \beta \sqrt{\frac{1 - i_R}{i_R}} \sqrt{\frac{\sum_{2k=1}^{n_{S2}} p(S_{k2} | H_2) C(S_{k2})}{\sum_{k1=1}^{n_{S1}} p(S_{k1} | H_1) C(S_{k1})}} \tag{11}$$

If statistical conformity criteria are used, the β-value should be fixed at 0,05 (5%), corresponding to the definition of characteristic strength of concrete f_{ck}. In this case, for $i_R = 0{,}5$, the α-value can be calculated using:

$$\alpha = 0{,}05 \sqrt{\frac{\sum_{2k=1}^{n_{S2}} p(S_{k2} | H_2) C(S_{k2})}{\sum_{k1=1}^{n_{S1}} p(S_{k1} | H_1) C(S_{k1})}} \tag{12}$$

5. ILLUSTRATIVE EXAMPLE

Calculate the rejection probability of a good quality concrete lot α in the case when probability of a bad quality concrete lot is $\beta = 0{,}05$ and the risk balance index $i_R = 0{,}5$ (a risk is transferred fifty-fifty to both sides; the producer and the client).

There are $n_{S1} = 3$ states adverse to the producer: S_{11} - repeated control, $p(S_{11}|H_1) = 0{,}75$ and $C(S_{11}) = 0{,}1$; S_{12} - reduction in price, $p(S_{12}|H_1) = 0{,}20$ and $C(S_{12}) = 0{,}30$; S_{13} - rejection of a plot, $p(S_{13}|H_1) = 0{,}05$ and $C(S_{13}) = 1{,}0$ (the relative cost of a concrete plot rejection is 1,0).

Similarly, three states $n_{S2} = 3$ adverse to the client are defined: S_{21} - reduction in price, $p(S_{21}|H_2) = 0{,}95$ and $C(S_{21}) = 0{,}5$; S_{22} - repair or strengthening, $p(S_{22}|H_2) = 0{,}0499$ and $C(S_{22}) = 10$; S_{23} - catastrophic event, $p(S_{32}|H_2) = 0{,}0001$ and $C(S_{32}) = 10^4$.

Using equation (12):

$$\alpha = 0{,}05\sqrt{\frac{0{,}95 \times 0{,}5 + 0{,}0499 \times 10 + 0{,}0001 \times 10000}{0{,}75 \times 0{,}1 + 0{,}2 \times 0{,}3 + 0{,}05 \times 1}} = 0{,}163\,,$$

and the probability of acceptance of a good quality lot is $P_a = 1 - \alpha = 0{,}837$.

In practice, an equal probabilities of rejection and acceptance are usually assumed $\alpha = \beta = 0{,}05$. The risk balance index calculated using equation (11) is equal i_R = 0.914, and significantly bigger risk is transferred to the client in comparison with a risk transferred to the producer: $R_{cl} = 10{,}67 R_{pr}$.

6. CONCLUSIONS

The general concept of statistical conformity criteria consistent with the semi-probabilistic design method of concrete structures are explain. Some anomalies have been mentioned that can have prejudicial effects to both sides involved in conformity control process, i.e. the producer and client.

The conformity criteria for concrete strength recommended in EN 206-1 under the assumption that the rejection probability of a good plot and the acceptance probability of a wrong plot of concrete are equal, e.g. $\alpha = \beta = 0{,}05$, lead to unbalanced producer's and client's risk. Generally, the risk should be referred to a combination of the probability and consequences of hazards connected with the conformity control.

From the foregoing analysis it follows that in the case of conformity control all major types of adverse states (e.g. repeated control, reduction in price, rejection of a plot, and repairs, strengthening, catastrophic events, etc.), with the corresponding probabilities of occurrence and consequence should be identified and considered in risk analysis.

A risk balance index which expresses how the risk is divided into two sides involved is defined. A risk partition into the producer and the client may be a matter of a trade contract. For a negotiated risk balance index, the α and β - values can be calculated and for different sample sizes the OC-curves corresponding to an assumed type of conformity criteria can be calculated.

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CARE OF THE TECHNICAL STATE AND SAFETY LEVEL OF A BUILDING OBJECT EXEMPLIFIED BY A MULTI - STOREY APARTMENT BUILDING IN RZESZÓW

ABSTRACT

The article presents the problems of maintaining the usability of the structures erected in the late 80s and early 90s using OWT- 75 large panel technology. Its example is a ten-storey department building in one of the housing estates of the Rzeszów Building Cooperative. For a few years the building has been checked by the authors. During that time many defects have successively been found. Their extent and origin have been described, proper remedies suggested and then applied. They ensured a good technical state of the building and enhanced the residents' feeling of safety.

KEYWORDS: large panel building, technical state, repairing, strengthening, safety level, durability

1. INTRODUCTION

The conditions of putting up large panel buildings at the turn of the 80s and the 90s (during the decline of prefabrication) left a lot to be desired regarding the quality of materials, finished products as well as precisions and building methods. Comprehensive solutions also have designing defects. These factors undoubtedly impair the safety level of such structures during their use. They cause trouble and generate high maintenance cost. They also determine the building's durability. All the defects of the building manifesting themselves in cracks, deformations and interdisplacements of elements create a feeling of danger and instability at the residents. They are structures that age fast technically and take special care and attention. It is the administrators that are responsible for the technical state of the buildings. The defects that occur must be successively removed based on the identification of the cause and previously prepared expert opinions and design solutions.

For the last few years the authors have been observing the technical state a lot of a prefabrication system based department buildings evaluating their technical state and designing anticipated repair.

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2. DESCRIPION OF THE OBJECT

The department building discussed in the paper is an L-shaped object with ten storeys, a cellar and four staircases. Its layout is shown in Fig.1. The building is divided in three parts with transverse dilatations. The number of storeys is fixed, differences in the foundation levels were caused by the need to adjust it to the natural terrain configuration.

The building rests on a 60 cm thick foundation plate divided with dilatations as above. The cellar walls are made of reinforced concrete; the outer and inner walls are 30 and 25 cm thick, respectively.

The overground part of the building was made in OWT-75 and OWT-75 NS technologies. The load bearing structure consists of: 15 cm reinforced concrete, precast longitudinal and transverse inner walls; 40×40cm, U-1 reinforced concrete columns; three – layer elements of the longitudinal external walls; three-layer gable BTZ walls; ceilings – 16 cm precast reinforced concrete slabs. The roofing of the house consists of reinforced concrete precast roof slabs. Balconies - the balcony plate supported on the longitudinal wall of the building by means of two outer cantilevers and additionally it is propped by a supporting structure consisting of a cantilever beam and a column. The measurements of the horizontal projection of the balcony plates:

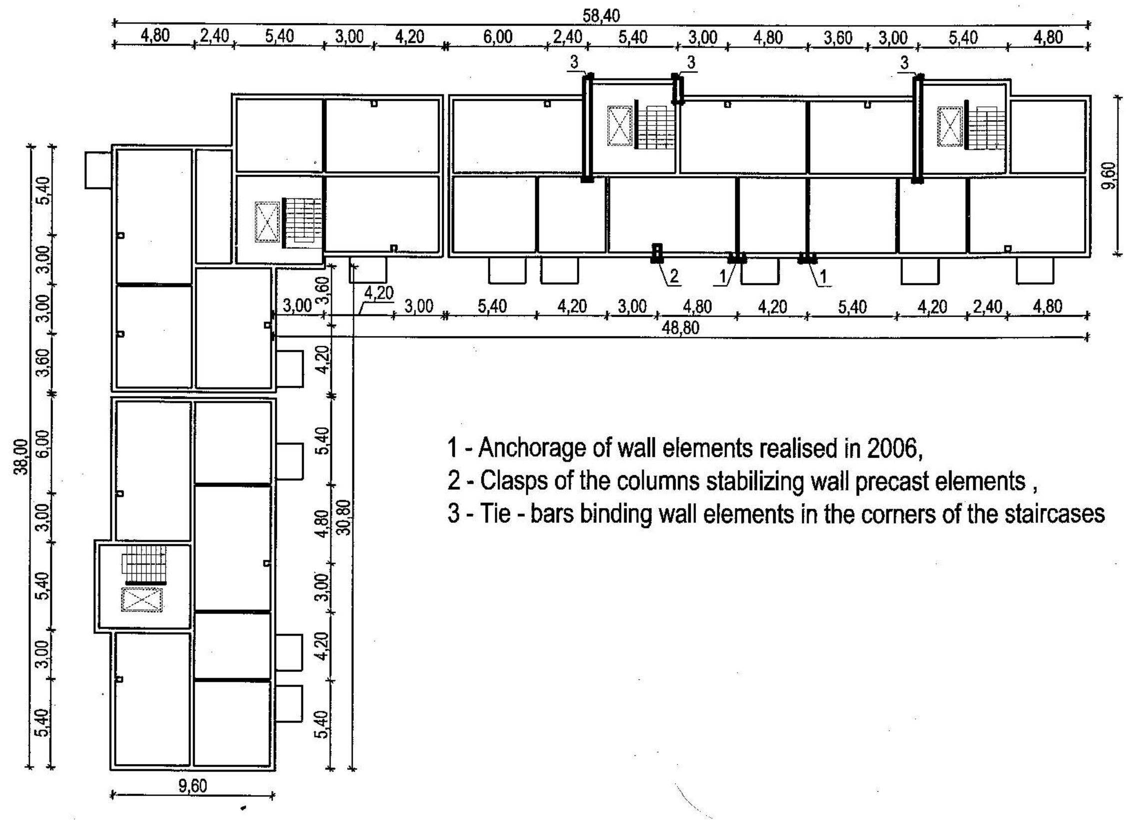

Fig. 1. Layout of building

- length (along the wall of the building) – 2.40m,
- width – 1.8m,
- changing balcony plate thickness – from 11cm at the wall of the building to 8cm along the opposite longitudinal edge.

In January 1991 the building was put to use.

3. DEFECTS AND DAMAGE

The building has been monitored by the authors since 2002 [1, 2, 3, 4]. Its detailed technical state assessment has been done twice. The following kinds of damage (defects) have been found:

- loss of stability of the reinforced concrete columns supporting the balcony plates (Figs 2 and 3), the columns of the balconies of two ground floor flats in staircase II were in the worst state of repair,
- advanced corrosion of the concrete and the steel of the balcony slabs as well as the column footings (Fig.6),
- outward displacement of the facade sandwich plates of the longitudinal walls of the highest storeys of the building (Figs 8 and 9),
- numerous, very narrow and also wider cracks in the walls and at the connections (Fig.10),
- partial decay or/and lack of some connectors, so called "suspension rods" holding up the texture layer of the facade plate.
- air blows-through from outside and rainwater leaking into the building (Fig.11).

Fig.2. Ground floor column after stability loss

Fig. 3. Detail „A”

Fig.4. View of corrosion damage of structural elements of balconies

Fig.5. Corrosion damage of cantilever beam supporting balcony plate

Fig.6. Corrosion damage of column base

Fig.7. Filling between balcony slab and column base

Fig.8. View of displacement (~ 20 mm) of facade plate of the highest storey

Fig.9. View of façade wall displaced from inside

Fig.10. Cracks at wall connections

Fig.11. Damage to facade plates

4. ANALYSIS OF CAUSES OF DAMAGE AND CONCLUSION

- The stability loss by the columns was the result of negligence and carelessness of work. The columns are too short in relation to the level of the top balcony surface. The space between the wearing-ironed column bases and the balcony plates of most balconies was filled in with steel washers made of sheets arranged in a pile (Fig.7). In some flats the space was filled in with foamed polystyrene trimmings, mortar waste and other building site rubbish. Loading these columns with effects from higher storeys caused deformation of the soft cover plates which are the only support of the columns, and consequently stability loss.
- Excessive (regarding the age of the building) corrosion advancement of the concrete and steel of the balcony plates and the columns was due to bad support, in view of durability, of the balcony plates (by means of a through pole) and poor concrete quality. Improper sealing of the area around the column (or no sealing) combined with improperly shaped floor slope caused frequent rainwater leaks in the area.

- Cracks in the walls and their connections can be divided into:
  - Cracks along the connections of the precast units and along precast unit - partition brock wall connections. Those are cracks that cannot be got rid of. They are characteristic of that kind of objects and do not pose any threat.
  - Cracks due to unprofessional making of the plaster of walls and columns and almost no adherence of the plaster to the base. That can be improved by putting new, firmly sticking plaster, following the sound principles of good building (after removing the old plaster).
  - Cracks in the connections of precast units, joints and, consequently, blows-through of outside air are alarming signs meaning a need of immediate repair (see the description below).
- Blows-through of outside air and rainwater leaking into the building (into the flats and staircases) result from the local faults in making joints. They involve filling the joint space with concrete waste and careless damp proofing. The blows-through are also due to leaks in precast unit connections and in precast units that have been faultily produced or damaged during delivery or assembly. Imperfectly shaped slope of the sheet metal work at precast unit junctions is yet another cause of rainwater leaking into the building.
- The outward displacement of the sandwich facade plates of the longitudinal walls of the top storeys occurs due to unprofessionally made (from the structural point of view) joints at plate connections.
- Partial lack of connectors, so called "suspension rods" holding up the texture layer of the facade plates is the effect of making them of ordinary steel (instead of stainless steel as predicted in the design) which is prone to corrosion and mounting them, in the prefabrication stage, in a number much smaller than that assumed in the system solutions.

5. SUGGESTED REPAIR SOLUTIONS

The solutions are divided in two groups:

Group I – works already completed that include:

- Repair of the reinforced concrete columns supporting the balcony plates which involves restoring their stability through wedging up the columns with steel profiles and plumbing them (Fig.12 and 13), finally cleaning the reinforcement and filling all the cavities in the concrete (the SIKA system of reconstruction and surface protection was applied).
- Repair of the balcony structures involving: the repair of the balcony plates and the reinforced concrete beams [5], filling appropriately the space between the column bases and the heads of the columns of the lower storeys, putting a new surface of milled rock tiles, careful sealing of the plate-column (design solution shown in Fig.14) and plate-longitudinal wall connections and finally replacing the old balustrades with new ones with a bottom clearance enabling rainwater to flow off the balcony slabs in three directions.

Fig.12. Column of supporting structure of balcony immediately after plumbing

Fig.13. View of column after being wedged up with steel profiles

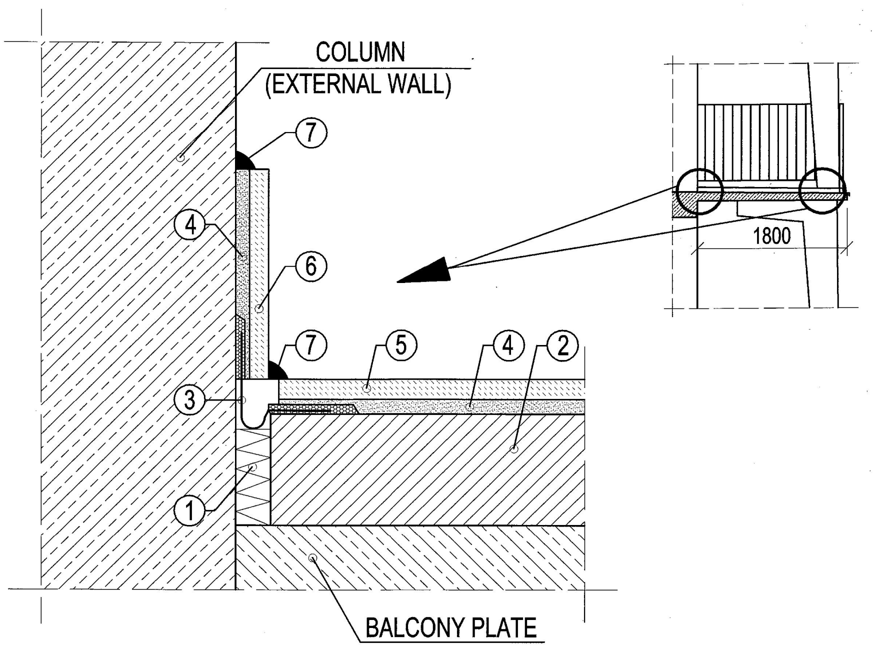

Fig.14. Sealing spots crucial for building's lifetime (balcony plate-column and balcony plate-wall connections)

1– polystyrene, 2- cement mortar modified with polymer, 3- elastic sealing up tape, 4– waterproof, elastic glue, 5- surface of balcony from frost-proof tiles, 6- wall partly tiled by frost-proof tiles, 7– polyurethane seal

- Anchoring three external wall elements of the longitudinal south wall in the transverse load-bearing walls following the solution shown in Fig.15.

Group II – work to be completed in the nearest future:

- monitoring which involves a geodesic check of the building's displacements and settlements for a year, checking the width of the cracks in selected flats, carrying out annual overhaul of the technical state and the selected flats in particular.

The works aimed at improving the stability of the structural elements of the building:

- Making steel structures anchoring the longitudinal walls (external wall elements) in the transverse walls.

- Making clamping rings around the columns which enable pulling as well as stabilizing external wall elements.
- Making steel stays clamping wall elements converging at staircase corners (between floors VII-VIII, VIII-IX and in the attic).
- Repair of protruding corner joints of the building and the staircases and stabilizing them with supporting steel structures.

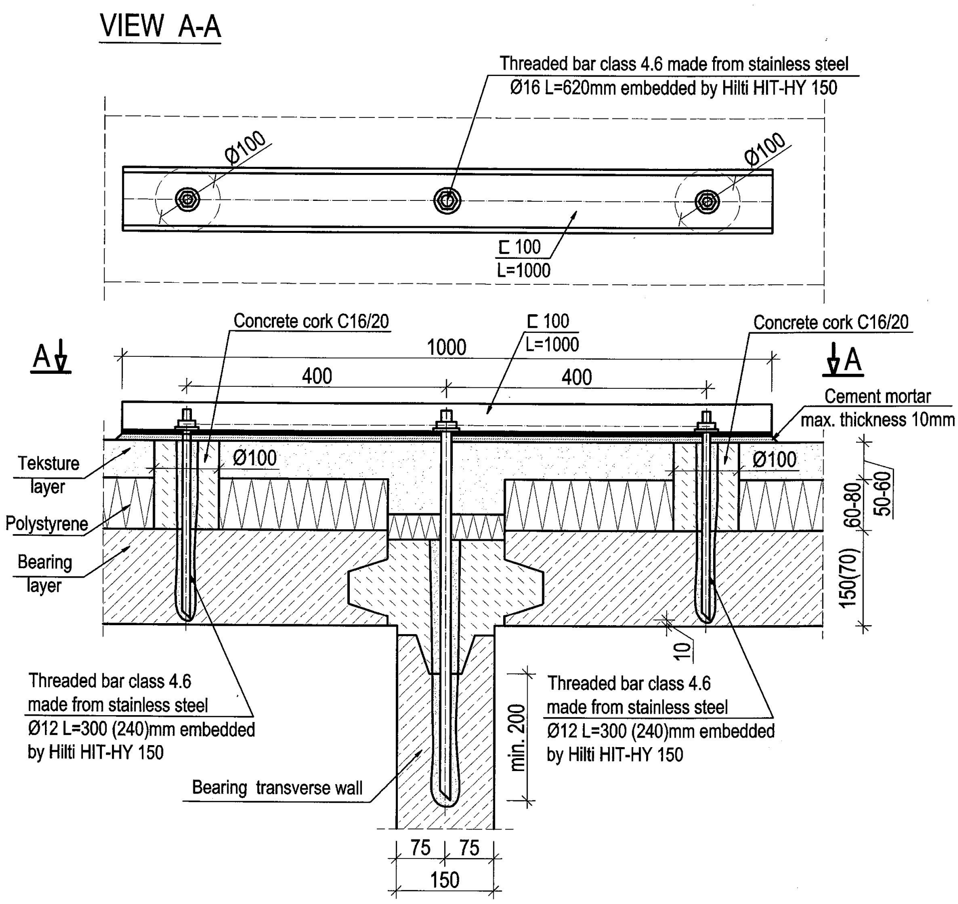

Fig.15. Anchoring external wall elements in transverse load-bearing walls [6]

The location of the above structures is marked in Fig.1.

The other works:

- sealing precast element junctions and precast element-sheet work ones with a permanently flexible material to stop rainwater leaks,
- thermorenovation of the object proceeded with the repair of the texture layer of the facade plates and anchoring the texture layer in the load – bearing layer of the sandwich plates. The number and the kind of the anchors must be adjusted to the size of the plates and the level of corrosion of the steel connectors.

6. SUMMARY

The combination of thermorenovation work of the external walls with that improving the stability of the structural elements of the building is well-founded and desirable with a view to ensuring a much better access to the elements to be repaired and a possibility of carrying out a more precise assembly of anchoring elements from stationary scaffolding

(rather than from any other units). The quality of the assembly has a considerable impact on the effectiveness of repair work of constructional nature [7, 8, 9].
The combination of these works is also sensible from the economy point of view. This way some works do not overlap and changes in the previously applied solutions can be avoided.

All the repair work must be carried on based on the previously prepared design.

After many years of monitoring the technical state of objects built with the use of the large panel technology it is possible to state explicitly that the youngest buildings made of precast elements (erected in the turn of the 80s and the 90s of the last century) are the worst considering their work quality. This is decisive for their lifetime and generates maintenance cost that is too high with relation to their age.

The object which is the subject of our paper is a positive example of the activity taken by the administrator that ensures its undisturbed usability.

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