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WOJCIECH BALCERZAK*, ZBIGNIEW MUCHA*

THREATS OF HIGHLY ALKALINE WASTEWATER DISCHARGE TO SURFACE WATER

ODPROWADZANIE WYSOKO ALKALICZNYCH ŚCIEKÓW DO WÓD POWIERZCHNIOWYCH

A b s t r a c t

The paper discusses the discharge of highly concentrated alkaline wastewater from an industrial reservoir and its effect on surface waters. The discharge source is the Górka industrial reservoir, located in Trzebinia. The wastewater constitutes a threat to groundwater, which serves as a source of drinking water for surrounding villages. A detailed analysis of pH confirmed a high alkalinity of the tank content; the pH values were in the range of 12.5 to 13.5. Other water quality parameters (COD, sulfates, chlorides, TSS) were also very high. Initially, wastewater was discharged to the wastewater treatment plant (WWTP) in Chrzanów. Then, since the plant had refused further treatment due to operational disturbances, wastewater had to be discharged directly to the Chechło and Ropa creeks and finally to the Vistula river. A simultaneous monitoring of the water quality in these streams was provided. Wastewater discharged from the Górka reservoir was gradually diluted on its way to the Vistula river and therefore its impact on the Vistula river has been insignificant. The method turned out to be both effective and safe for the aquatic environment.

Keywords: *alkaline wastewater*, *groundwater quality protection*, *surface water quality*

S t r e s z c z e n i e

Artykuł przedstawia zagadnienie odprowadzania silnie stężonych ścieków alkalicznych ze zbiornika poprzemysłowego Górka w Trzebini do wód powierzchniowych. Ścieki te stanowiły zagrożenie dla wód podziemnych zaopatrujących w wodę okoliczne miejscowości. Szczegółowa analiza odczynu pH wskazywała na wysoką alkaliczność cieczy w zbiorniku, wartości wahały się w granicach 12,5–13,5 pH. Pozostałe wskaźniki jakości wody (ChZT, siarczany, chlorki, zawiesina) również były bardzo wysokie. Ścieki te początkowo były odprowadzane do grupowej oczyszczalni w Chrzanowie Ze względów eksploatacyjnych oczyszczalnia odmówiła dalszego przyjmowania ścieków, co spowodowało konieczność odprowadzania ścieków ze zbiornika bezpośrednio do wód powierzchniowych potoków Ropa Chechło, a w konsekwencji do Wisły wraz z jednoczesnym monitorowaniem stanu jakości wody w tych potokach. Na każdym etapie przepływu ścieki odprowadzane z Górki ulegały rozcieńczeniu. Rozcieńczenia spowodowały znaczny spadek stężeń w odbiornikach. Wpływ odprowadzanych ścieków na rzekę Wisłę jest nieznaczny. Zastosowana metoda okazała się skuteczną i bezpieczną dla środowiska wodnego.

Słowa kluczowe: *ścieki alkaliczne*, *ochrona jakości wód podziemnych*, *jakość wód powierzchniowych*

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1. Introduction

The Górka reservoir is located in the city of Trzebinia. During industrial activities of the former Górka cement plant, marl, for cement production, was excavated from the Górka quarry. In the years 1962–1984, solid waste from the production of aluminum hydroxide from bauxite processing was stored in the existing excavation pit. The excavation pit holds about 400,000 tons of waste; the thickness of the waste layer is 16–18 m while its overall area covers approximately 2.5 acres. The excavation pit also stored other waste materials such as chamotte rubble, slag, ash and others. The total amount of stored waste (red mud type) was around 600 thousand m^3, i.e. approximately one million tons. The excavation pit and landfill areas occupy 24.3 acres in total, from which 16.8 acres is reserved for a landfill while the remaining 7.5 acres is a pond [1, 2].

A geological structure around the excavation pit is unfavorable from the perspective of environmental hazards. Jurassic and Triassic formations house the underground water reservoirs, which are very susceptible to penetration and the spread of contaminants. The landfill is located within the recharging zone of these reservoirs, which are also located in the excavation pits of the Trzebionka zinc and lead mine. Groundwater from these excavations supplies the municipal water system in Trzebinia. The uncontrolled intrusion of polluted water into the mine may cause the irreversible contamination of groundwater; additionally, it makes possible hydraulic contact with the underground reservoir GZWP no. 452. The reservoir serves as the main water supply source not only for the Trzebinia residents, but also for the whole of Chrzanów County [3].

2. Methods

By 1991 the wastewater was occasionally pumped out into the sewer and then transferred to the Chrzanów. In 1991, the discharge from the reservoir was brought to an end and a surge of water was observed. Water table elevations rose steadily and eventually a pond was created, where approximately 400,000 m^3 of contaminated water was retained. In 1992, the pumping of wastewater out of the reservoir to the Chrzanów WWTP was resumed. Such actions lasted intermittently until 1997 when it was discontinued due to the treatment plant failing to meet the water permit conditions [3].

In 2000, due to the uncontrolled discharge from the reservoir, the concept of pumping out the wastewater from the reservoir to the surface water came up. Such an approach enabled the satisfactory protection of the reservoir bed and withhold the discharge flow to the excavation pits and groundwater. In the same year, a water permit was granted for wastewater discharge. However, there was still no proper installation to perform such actions and the wastewater kept overflowing into surface waters. In 2005, a lowering of the liquid level was initiated by pumping and discharge of wastewater through a tunnel to a creek flowing out of the Balaton quarry and then to the Ropa and Chechło creeks. The liquid was pumped out with a set of floating pumps with a capacity of 8.1 dm^3/s; the pumps were resistant to alkaline compounds. The draining off of the reservoir, initiated in 2005, proceeded in several stages and eventually a liquid level in the reservoir was over 10 m lower. Wastewater drained from the tank was transported via a 160 mm pipe diameter directly to the Ropa creek.

In the years 2005–2008, the draining of the reservoir was continued and wastewater was pumped out and discharged to the Ropa creek. Currently, the reservoir holds 20,000 m^3 to 23,000 m^3 of wastewater; about 15,000 m^3 saturates a solid waste dump while the remaining 8,000 m^3 fills the cavities in the bottom of the reservoir [4].

Table 1 presents data relating to the water quality in the reservoir. The parameters show a systematic increase along the reservoir's depth (13 m); particularly hazardous for the aquatic environment are high values of pH and alkalinity. Additionally, other parameters significantly exceed the values specified in the Regulation of the Minister of Environment dated July 24, 2006 on the conditions to be met when discharging sewage into water or soil, and on substances particularly harmful to the aquatic environment [4].

Table 1

The Górka reservoir water quality [1]

Parameter	Unit	Range, min.–max.
pH	–	11.7–13.3
Alkalinity	mval/dm^3	97–304
COD	mg/dm^3	177–1,311
Specific conductivity	mS/cm	5,000–71,130
Sulfates	mg/dm^3	500–2,300
TDS	mg/dm^3	7,628–38,966
Lead	mg/dm^3	0.006–0.0166
Aluminum	mg/dm^3	25.2–206.7
TSS	mg/dm^3	10–538

3. Wastewater discharge from the Górka reservoir

Wastewater from the Górka reservoir, having passed through a tunnel and pipes laid in trenches A and A-1, are discharged to the Ropa creek and the Chechło river; their final destination is the Vistula river.

Trench A drains excess water from the Balaton reservoir ('class I' water quality, used for recreation). Here, the contaminated wastewater from the Górka reservoir becomes for the first time substantially diluted with water from the Balaton reservoir. A subsequent dilution takes place in ditch A-1, which receives both water from a storm water drainage system from the Metallurgy Plant in Trzebinia and excess water from the Balaton reservoir. Wastewater then enters the Ropa creek. The Ropa creek receives water from the areas located between the Chechło river (from the south) and the industrial premises (to the north). Its water, though diluted with water from the Balaton reservoir and pre-treated wastewater from the EkoNaft Refinery in Trzebinia, is already polluted. The pollutant load is mixed with a diluted stream from the Górka reservoir.

The Ropa creek enters the Chechło river, which is a left-bank tributary of the Vistula river. The Chechło river, which is also a recipient of the Chrzanów WWTP effluent and waters

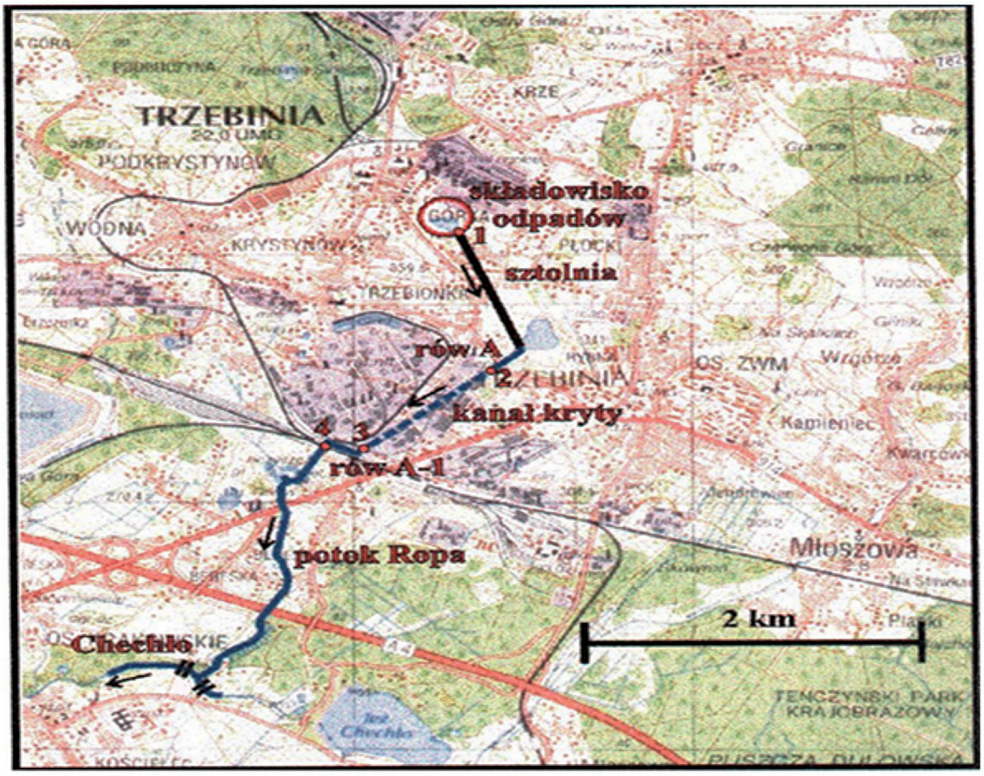

Fig. 1. Discharge of wastewater from the Górka reservoir: 1 – drill hole, 2 – manhole at the inlet to a tunnel, 3 – tunnel outlet, 4 – culvert under a railway embankment [1]

from catchment areas, once again dilutes water from the Górka reservoir by mixing it with other surface waters. The Vistula river is the final destination of all water streams; its flow is so large, compared to the Chechło river, that it dilutes the incoming waters.

4. Quality of a discharged wastewater

Water quality in creeks was monitored to determine how the reservoir discharge affects their water quality. The following parameters were analyzed: pH, alkalinity, total suspended solids (TSS), COD, sulfate and chloride [5]. The discussion was based on changes of pH shown in the graphs (Figs. 2–4). They present the pH changes taking place along the Ropa and Chechło creeks in the years 2006–2008. In 2006, the average pH level was maintained at 8–8.4 at all measurement points (Fig. 2). At the end of the year (October–December) a drop of pH down to 7.1–7.8 was observed in the Ropa creek (first measurement point, upstream from the Oil Refinery WWTP); the pH values again reached a level of 8–8.3 in mid-December.

Such a decrease may be caused by a higher water inflow from the areas adjacent to the Ropa creek due to the torrential rains that occurred at that time. Downstream from the Oil Refinery WWTP, the pH level in the Ropa creek resumes the value of 8–8.4, probably due to wastewater discharged from the Chrzanów WWTP. In 2007 (January–March), the pH level increased to over 13 at the first measuring point (Fig. 3) while at the remaining points,

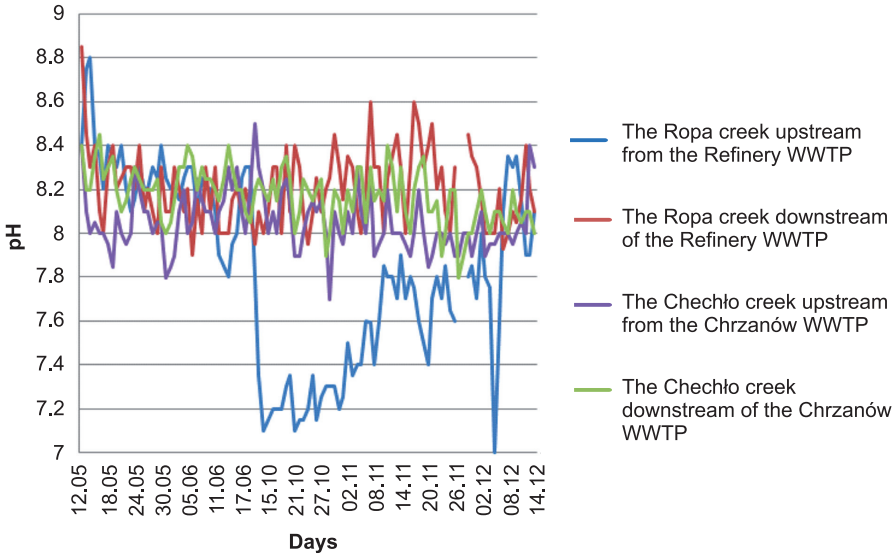

Fig. 2. pH changes along the discharge from the Górka reservoir in 2006

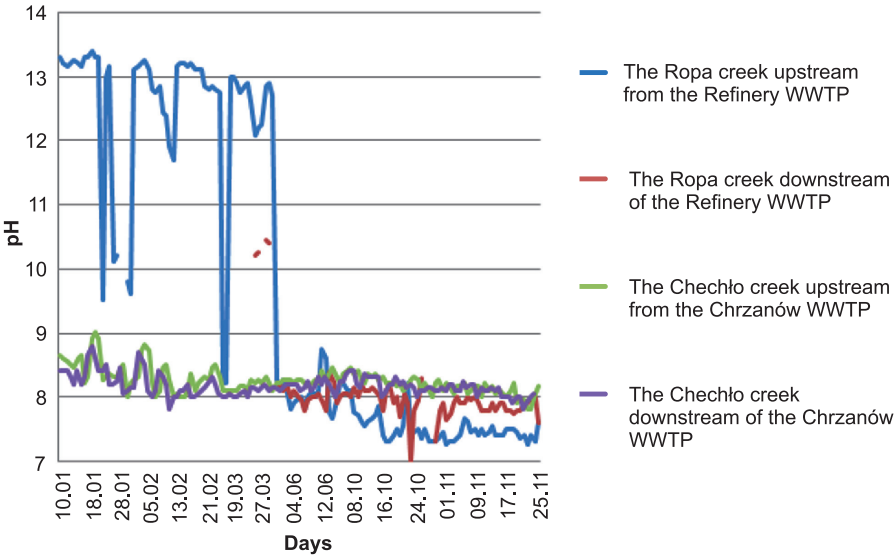

Fig. 3. pH changes along the discharge from the Górka reservoir in 2007

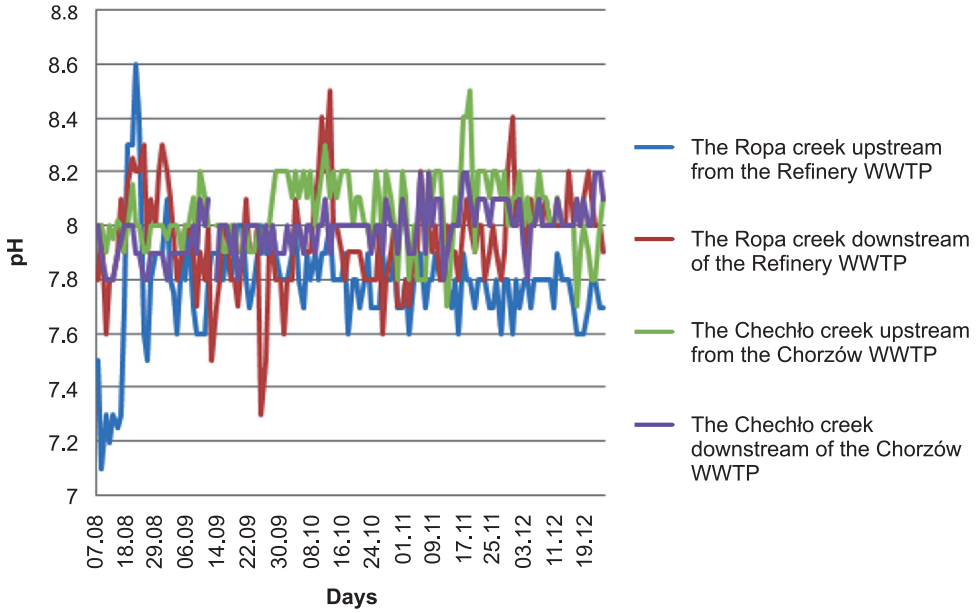

Fig. 4. pH changes along the discharge from the Górka reservoir in 2008

it maintained the level of 8–8.4. This increase may be attributed to the fact that the liquid pumped out from the Górka reservoir was discharged to the Ropa creek at that time. Once the pumping stopped, pH values returned to their previous levels. However, in 2008 (Fig. 4), the pH value remained rather stable within the range of 7.6–8.6 with no significant changes observed.

Except for an increase of pH in early 2007, pH values have remained stable and do not pose any hazards to the biological life in rivers.

5. Impact of Górka's reservoir discharge on water quality

The impact of wastewater from the Górka reservoir was determined by calculating the pollution increase Δs in the creeks' water, according to the formula:

$$\Delta s = S_e \times Q_{śc} / (SNQ + Q_{śc})$$

where:

SNQ – reliable flow in a creek [dm^3/s],
$Q_{śc}$ – wastewater flow from the reservoir, $Q_{śc}$ = 8.1 dm^3/s,
S_e – average concentration in wastewater from the Górka reservoir [mg/dm^3].

The results are shown in Tab. 2, while Tab. 3 presents the actual concentrations of pollutants in creeks' water at the time of wastewater discharge from the Górka reservoir.

Table 2

Increase of concentrations in receiving water due to the Górka reservoir discharge

Creek/river	$Q_{śc}$ /SNQ [dm³/s]	COD [mg/dm³]	Chlorides [mg/dm³]	Sulfates [mg/dm³]	TSS [mg/dm³]
Górka	8.1	707	397	1,803	57
Ropa	10.8	303	170	773	24
Chechło upstream from the WWTP	350	16	9	41.5	1.3
Chechło downstream from the WWTP	640	9	5	23	0.7

The calculated changes in concentrations in the various streams indicate a gradual dilution of the Górka discharge in following receiving water bodies.

Table 3 presents the average values of parameters in the creeks/rivers in 2006–2007 and compares them to the limits set for the water of class II (Annex 5, Regulation of the Minister of the Environment, issued on November 9, 2011) [6].

Table 3

Average concentrations of pollutants in creeks/rivers during wastewater discharge from the Górka reservoir

Creek/river	COD [mg/dm³]	Chlorides [mg/dm³]	Sulfates [mg/dm³]	TSS [mg/dm³]
Górka	707	397	1,803	57
Ropa	116	63	128	13
Chechło upstream from the WWTP	24	75	491	10
Chechło downstream from the WWTP	34	86	481	13.6
Limiting values as in [6]	30	300	250	50

Environmental objective of the water segment defined as "the segment of the Chechło creek measured from the Ropa creek down to the Chechło's creek mouth" assumes a good water status.

As shown in Tab. 3, the water quality standards for class II (Annex 5, Regulation of the Minister of the Environment [6]) were exceeded only for sulphates and the Górka's reservoir discharge had little effect on the increase of this parameter (see Tab. 2). Higher concentrations of sulphates observed in the Chechło creek were caused by pre-treated wastewater, discharged from the Eco-Naft WWTP. Elevated values of the other parameters in the Chechło creek may be attributed to effluent discharges from the Chrzanów WWTP.

6. Conclusions

- The Górka reservoir, filled with strongly mineralized water with a high content of organic compounds and metals, was a major threat to the environment. Permeation of the liquid into groundwater threatened contamination of water resources used for municipal purposes. It was necessary to empty the tank and clean up waste lying at its bottom as soon as possible.
- The proposed solution assumed the discharge of wastewater from the reservoir by periodic pumping and transfer to surface waters in a controlled manner; such an option seems to be acceptable with no negative impact on water quality.
- After the Górka reservoir's discharge, the limiting values for the class II of water [6] were exceeded only for sulphates (Chechło creek, upstream from the Chrzanów WWTP). The increase was not caused by a wastewater discharge from the Górka reservoir.
- Discharge of wastewater from the Górka reservoir enables further comprehensive reclamation of lands degraded by industry.

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[5] *Materials provided by the Regional Water and Sewage Works in Chrzanów*, characteristic of wastewater for 2006–2007.

[6] Regulations of the Minister of Environment of November 9, 2011 on classification of surface water bodies and environmental quality standards for priority substances (Journal of Law, 2011, No. 257, paragraph 1545).

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OCCURRENCE OF ANTI-CANCER DRUGS IN THE AQUATIC ENVIRONMENT AND EFFICIENCY OF THEIR REMOVAL – THE SELECTED ISSUES

WYSTĘPOWANIE LEKÓW PRZECIWNOWOTWOROWYCH W ŚRODOWISKU WODNYM ORAZ SKUTECHNOŚĆ ICH USUWANIA – WYBRANE ZAGADNIENIA

A b s t r a c t

The article discusses the occurrence of selected cytostatic drugs in the aquatic environment. The authors start with a preliminary introduction to the characteristics of the most commonly used cytostatic drugs. Then, based on the review of the literature they show that such drugs occur in small amounts in an aqueous medium and that there is no reliable research data on the long-term exposure of aquatic organisms to cytostatics. Until now, the studies on the stability of some cytostatics showed that these compounds were extremely stable, not only in natural waters, but also at wastewater treatment plants (WWTPs). In spite of the advanced treatment technologies used at wastewater treatment plants, cytostatics pass relatively easily through the treatment line and end up in surface waters, thus posing a threat to water quality.

Keywords: *cytostatic drugs*, *cyclophosphamide*, *ifosfamide*, *5-fluorouracil*, *wastewater*, *surface water*

S t r e s z c z e n i e

W artykule przedstawiono zagadnienia związane z występowaniem wybranych leków cytostatycznych w środowisku wodnym. Dokonano wstępnej charakterystyki najczęściej stosowanych leków cytostatycznych. W oparciu o przegląd literatury wykazano, że związki te występują w środowisku wodnym w niewielkich ilościach i nie ma wiarygodnych badań dotyczących długotrwałego narażenia organizmów wodnych na działanie cytostatyków. Wyniki dotychczasowych badań nad stabilnością wybranych cytostatyków wykazały, że związki te są wyjątkowo stabilne nie tylko w wodach naturalnych, ale także w warunkach panujących w oczyszczalniach ścieków. Mimo stosowania najnowszych technologii w oczyszczaniu ścieków, rozpatrywane cytostatyki, w mniejszym lub większym stopniu, przedostają się przez oczyszczalnie ścieków i trafiają do wód powierzchniowych, stanowiąc zagrożenie dla ich jakości,

Słowa kluczowe: *leki cytostatyczne*, *cyklofosfamid*, *ifosfamid*, *5-fluorouracyl*, *ścieki*, *wody powierzchniowe*

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1. Introduction

The presence of contaminants of pharmaceutical origin in the aquatic environment is not a new issue. In the 1970s, clofibric acid was detected in surface waters in today's Germany. This compound is a metabolite of lipid regulators, including clofibrate and etofibriate. In the 1980s, the presence of thirty-two other different drugs and their metabolites was detected. Up to the year 2007, more than ninety pharmaceuticals have been identified all over the world in the seas, lakes, rivers, groundwater, soil and lake sediments [4]. New developments in medical and pharmaceutical sciences brought along tens of thousands of pharmaceuticals used for the diagnosis, prevention and treatment of numerous diseases and disorders. In many cases, once the drug has been introduced to the human body, it undergoes biotransformation into predominantly hydrophilic forms in the liver; only in the form of metabolites are the drugs excreted by kidneys, which is the major manner of their removal from the body. However, the compounds which are not absorbed from the gastrointestinal tractor readily soluble electrolytes can pass through the body in an unchanged form [25].

Given the continuous growth of the world's population, one can expect a gradual increase in the demand and consumption of pharmaceuticals; it may be associated with the increased presence of these compounds in the aquatic environment. There have been well-known cases with a negative impact of pharmaceuticals on animals, including the impact of diclofenac (from the group of non-steroidal anti-inflammatory drugs) on kidney failure and the death of vultures in Asia [6] or the impact of the synthetic hormone ethinylestradiol (EE2) on changes of the sex of fish [8, 9, 15]. Looking at the growing number of people suffering from cancer, Cytostatic drugs maybe a potential threat to the aquatic environment. This paper attempts to describe the occurrence of some anti-cancer drugs in the environment and estimate the efficiency of their removal in wastewater treatment processes.

2. Characteristic of cytostatics

Cytostatics comprise a group of natural and synthetic compounds used widely in cancer treatment (chemotherapy) and are toxic to rapidly dividing tumor cells. Since they may also be harmful for other rapidly dividing cells (such as bone marrow, hair and mucous membranes), the drugs are highly dangerous with many undesirable side effects, such as anemia, nausea, vomiting and alopecia. Furthermore, cytostatics exhibit carcinogenic, mutagenic and teratogenic effects, and they are therefore considered potentially the most dangerous pollutants in the water environment [11, 12]. Currently, some of the most popular anti-cancer drugs are cyclophosphamide (CP) and ifosfamide (IF), which are alkylating cytostatic agents, and 5-fluorouracil (5-FU), which belongs to a group of antimetabolites.

Cytostatic drugs act differently, depending on the type of drug; but their main role is to inhibit or completely block the replication of DNA in the tumor cell. The above--mentioned cytostatics (cyclophosphamide, ifosfamide, and 5-fluorouracil) are not only used in chemotherapy of different types of cancers, such as cancer of breasts, bronchial, testes, ovaries (CP, IF) and cancer of the digestive system (5-FU) but also in the treatment of leukemia, lymphoma and autoimmune diseases. They are also used for immunosuppression after organ transplantation (CP, IF) [9].

3. Cytostatics in water environment

Pharmacologically active substances present in the environment originate from numerous different sources, as shown in Fig. 1 [16]. Pharmaceuticals, including cytostatics, are not always fully metabolized in the body. The average percentage of original compounds excreted in urine in the case of CP, IF, and 5-FU is 21%, 26% and 18%, respectively [2]. The use of cytostatics in chemotherapy means that the main source of pollution with these compounds is wastewater from hospitals or hospital patients undergoing chemotherapy.

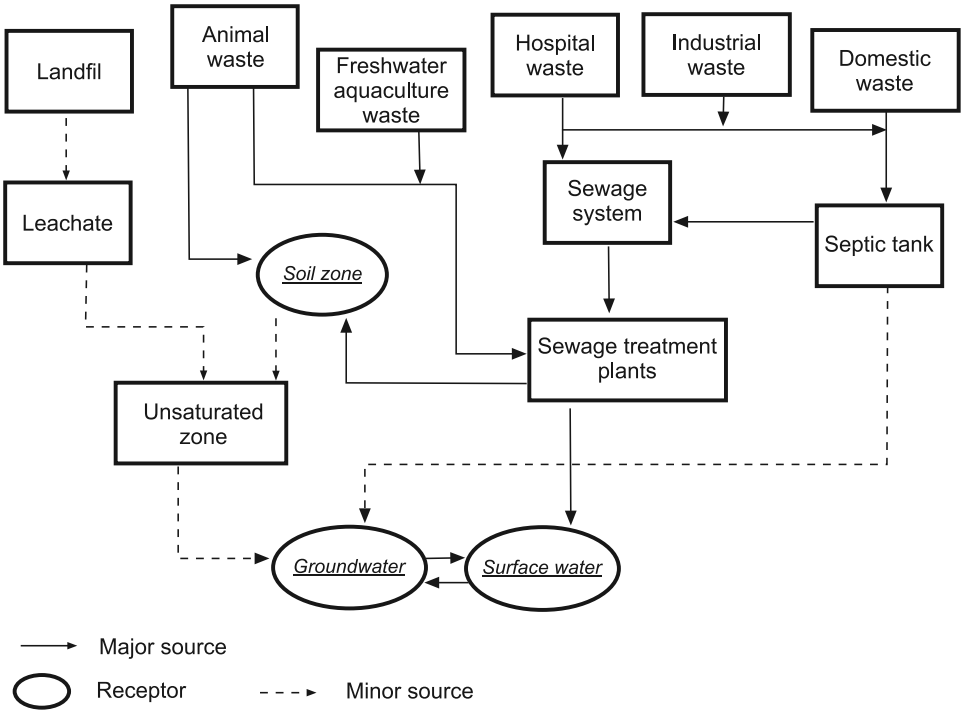

Fig. 1. Potential sources and pathways of pharmaceutical pollution in water and soil

The previous research on biodegradation of alkylating cytostatics (in the 1990s) [1, 5–7] at higher concentrations, as well as the research conducted at lower concentrations in Switzerland in 2006 [3], have demonstrated the exceptional resistance of cyclophosphamide and ifosfamide to biodegradation in activated sludge. This means that the cytostatic drugs pass to the aquatic environment despite the use of advanced treatment technologies. A study conducted in Switzerland indicated the presence of CP in the effluent from the Mannedorf WWTP (CP = 2 ng/l without chemotherapy at a local hospital, and 10 ng/l when such treatment took place) and CP and IF in the effluent from the Zurich WWTP (CP = 2.1–4 ng/l and IF = 1.7–6 ng/l) [3]. The same study showed the presence of these compounds in the river Limmat, downstream from the Zurich WWTP effluent discharge (CP = 0.15–0.17 ng/l and IF = 0.08–0.14 ng/l). Other studies have shown that cyclophosphamide concentrations

in surface water samples reached the level of 64.8 ng/l [21]. However, along with a growing number of cancer cases and a higher consumption of cytostatic drugs, concentrations of these substances in the aquatic environment may also increase. Such a trend has been confirmed by, for example, studies carried out in Spain, where ifosfamide was detected in local surface waters [26]. Cytostatics (CP, IF, 5-FU) were also detected in surface waters throughout Taiwan; water samples were taken from rivers located in the regions of Taipei and Kaohsiung-Pingtung. In the rivers of the Taipei region, the concentrations of 5-fluorouracil, cyclophosphamide and ifosfamide were 5–70 ng/l, 1.9–13 ng/l and 1.9–8.9 ng/l, respectively. However, in the rivers of the Kaohsiung-Pingtung region, the concentrations of these compounds were 35–160 ng/l, 0.9–96 ng/l and 0.1–4.7 ng/l, respectively [17]. Although there are small quantities of these compounds present in the aquatic environment, there is lack of reliable research data on the long-term exposure of aquatic organisms to the cytostatics. In such situations, the best solution seems to be reduced emissions of these compounds into the aquatic environment.

4. Methods

Though cytostatic concentrations in hospital wastewater are sometimes reported at the level of μg/l [17], their actual concentrations at the wastewater treatment plant are much lower due to dilution. Techniques used for the determination of cytostatics include gas chromatography coupled with a mass spectrometer (GC-MS – gas chromatography mass spectrometry) [3] and liquid chromatography coupled with a mass spectrometer (LC-MS – liquid chromatography mass spectrometry). However, sometimes the detection of very low concentrations (ng/l or lower [3, 17]) requires the use of more sophisticated analytical techniques with low limits of quantification like high performance liquid chromatography HPLC-MS. Another common technique is liquid chromatography coupled with tandem mass spectrometry (LC-MS/MS – liquid chromatography-tandem mass spectrometry) [3, 19, 20, 22] and HPLC-MS/MS [18].

5. Treatment efficiency

If chemotherapy were performed only in hospitals, pretreatment of hospital wastewater would sufficiently reduce the amount of anti-cancer drugs discharged to the wastewater treatment plant. However, oral chemotherapy is one of the methods of treatment. Undoubted advantages of this therapy include the comfort of the patient (treatment at home) and no need for continuous hospitalization – this assures a better sense of stability and well-being. On the other hand, a significant disadvantage of this solution is the excretion of metabolites and parent substances of cytostatics with domestic sewage; in such cases, wastewater cannot be pre-treated before its discharge to the treatment plant, this is why it is vital to provide efficient treatment and disposal of cytostatics at the municipal wastewater treatment plant.

The study on the stability of cyclophosphamide and ifosfamide showed that these compounds are extremely stable, not only in natural waters, but also under the

conditions prevailing at the wastewater treatment plants [3, 10, 13, 14, 23, 24]. In the case of 5-fluorouracil [17], the authors report the following results of its biodegradation: in activated sludge – 38% after 3 days and 65% after 4 days; aerobic biodegradation – less than 60% after 50 days. Despite the use of the latest wastewater treatment technologies, cytostatics pass through the plants (to a lesser or greater extent) and end up in surface waters. Table 1 shows cyclophosphamide and ifosfamide concentrations observed in the influents and effluents of sewage treatment plants in the canton of Zurich, Switzerland [3].

Table 1

Concentrations of CP and IF (ng/l) in influents and effluents of treatment plants in the canton of Zurich, Switzerland

WWTP	Sampling time	Cyclophosphamide		Ifosfamide	
		Influent	Effluent	Influent	Effluent
Mannedorf	20–23.09.2002	~4	~2	< 15	< 2
	24–27.09.2002	11	10	< 15	< 2
Zurich	23.03–03.04.2005	5	4	5	6
	18–24.2005	2	2.1	~1.4	1.7

Tests carried out on samples from sewage treatment plants in the region of Taipei in Taiwan [17] confirmed the poor removal of cytostatics in the local sewage treatment plant; concentrations in the plant influent and effluent were as follows: cyclophosphamide 280 ng/l and 80 ng/l; ifosfamide 12 ng/l and 15 ng/l; 5-fluorouracil 8.3 ng/l and 10 ng/l.

Table 2

Degradation of CP and 5-FU [18]

Compound	Initial concentration	Method	Residual concentration [C/C_0]	
			120 min	240 min
5-fluorouracil	200 µg/l	UV	~0.98	~0.99
		UV + ZnO (mg/l)	~0.55	~0.38
		UV + Aldrich-TiO_2 (5 mg/l)	~0.30	~0.10
		UV + Degussa P25 (5 mg/l)	~0.01	~0.00
Cyclophosphamide	27.6 mg/l	UV + Degussa P25 (20 mg/l)	~0.39	~0.05
		UV + Degussa P25 (300 mg/l)	~0.18	~0.00

Since certain cytostatics are resistant to biodegradation, alternative methods of their removal from wastewater and natural waters should be investigated. The latest studies on the photocatalytic oxidation of cyclophosphamide and 5-FU with e.g. UV/TiO2 [18] show the possibility of successful oxidation of these compounds. In this work, the authors used HPLC-MS/MS technique to determine CP and 5-FU concentrations; the results have been summarized in Tab. 2.

6. Conclusions

- Pharmaceuticals are becoming a serious problem for the well-being of organisms living in water that receives the effluent from wastewater treatment plants.
- In many countries, cytostatics have been found in the effluent from wastewater treatment plants and surface waters.
- Poor removal of some cytostatics during the treatment process and their high resistance to biodegradation suggest the need for other methods to eliminate these compounds from wastewater.
- There are no research studies that would clearly indicate the effects of the prolonged exposure of organisms to anti-cancer drugs. Therefore, it is difficult to introduce measures restricting their emissions into surface waters.
- The introduction of regulations defining the allowed and safe maximum concentrations of substances in the aquatic environment seems to be an urgent task for both the European Union and Poland.

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CHANGES IN SLUDGE QUALITY AT THE WATER TREATMENT PLANTS

JAKOŚĆ OSADÓW POWSTAJĄCYCH W PROCESACH UZDATNIANIA WODY

A b s t r a c t

This paper presents the variability of the quantity and specific composition of the sludge produced at two different surface water treatment plants – the Raba water treatment plant (WTP), collecting water from an artificial reservoir; and the Rudawa WTP, collecting running surface water. The authors analyzed the differences in the amount and characteristics of sludge and tried to relate them to specific technological systems in use at the treatment plants. The paper mainly focuses on heavy metals removed from the water and then trapped in the sludge. The aim of the work was to describe changes that involved both the quantity of sludge and its characteristics, especially with respect to its further disposal (e.g. heavy metals, etc.). Additionally, some aspects of an activated carbon dosing regime and its impact on a heavy metal content in sludge were discussed.

Keywords: *sludge processing*, *water treatment*, *sludge disposal*, *sustainable development*

S t r e s z c z e n i e

Artykuł przedstawia zagadnienia zmienności ilości i specyficznego składu osadów powstających przy uzdatnianiu wody powierzchniowej na przykładzie dwóch Zakładów Uzdatniania Wody: ZUW Raba pobierającego wodę ze zbiornika i ZUW Rudawa pobierającego wodę z rzeki. Analizowano różnice w ilości i specyfice osadów w powiązaniu z układami technologicznymi porównywanych Zakładów Uzdatniania Wody. Szczególną uwagę zwrócono na metale ciężkie usunięte z wody, a następnie zatrzymane w osadzie. Rezultatem prac było określenie zmian, które dotyczyły zarówno ilości osadu, jak i jego charakterystyki, szczególnie w zakresie czynników warunkujących jego dalsze zagospodarowanie (np. zawartość metali ciężkich). Wykazano także wpływ sposobu dawkowania węgla aktywnego na zawartość metali ciężkich w osadzie.

Słowa kluczowe: *przeróbka osadów*, *uzdatnianie wody*, *zagospodarowanie osadów*, *zrównoważony rozwój*

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1. Introduction

Changes in Polish legislation after accession to the European Union stipulated the reorientation of views on the operation of water treatment plants, particularly on issues related to the quality of the sludge produced at the water treatment plants and methods for its processing [3–5]. The higher efficiency of pollutant removal, accompanied by the more stringent requirements placed on methods of disposal of the final sludge, forced the majority of the existing water treatment plants to upgrade sludge processing lines. The demand for sustainable development in municipalities, which has been launched on a global scale, includes great interest in the proper processing of sludge [10, 11]. The main goal of this study was to check whether sludge origin has an impact on its final handling at the water treatment plant, with a special emphasis on heavy metal content in the sludge. The water treatment plants (WTPs) described in the paper serve a community of approx. 1 million inhabitants. The article presents the problem of variability in the composition of the sludge produced by two surface water treatment plants – the Raba WTP, withdrawing water from the water reservoir; and the Rudawa WTP, collecting water from the river. The aim of the work was to determine seasonal changes in both sludge quantity and its characteristics, especially in terms of the factors determining its further disposal, i.e. a heavy metal content. Additionally, the authors determined an impact of the way activated carbon is dosed on the heavy metal content in sludge [8, 12].

The production of drinking water is usually carried out through the coagulation of water with hydrolyzing metal salts such as aluminum sulfate (‘alum’) or ferric chloride (‘ferric’). This process is effective at removing turbidity, color, and micro-organisms, but it also results in a waste by-product, such as the coagulant precipitates and particles aggregated together in the form of ‘flocs’. They settle in a form of sludge that can be thickened, centrifuged or filtered prior to its ultimate disposal; these dewatering procedures reduce the final volume of the waste stream. Since the mid-1950s, activated carbon has been widely introduced into water treatment technology to improve water taste and remove odour.

Removal of solid aggregates from water is influenced by numerous factors, including structural configuration, a density difference between solid and liquid phases, as well as the use of dewatering mechanisms. However, from an operational perspective, it is useful to understand how such practical parameters as a coagulant dose and the process pH affect dewatering performance, or sludge ‘dewaterability’. A phenomenological theory developed by Landman, White et al. provides a rigorous approach to modeling the dewatering behaviour of compressible materials, and has been adopted to model various dewatering unit operations [2, 6, 7]. Selection of the right sludge treatment process has now become one of the most important operational problems, this has an adverse impact on the financial situation of water utilities. The continuous demand for higher water quality results in the production of larger amounts of sludge (both by volume and dry weight) as a waste product of the treatment process. As the sludge mass is the difference between the mass of solid particles in ‘raw’ and ‘tap’ water, the better removal of finest suspensions of microorganisms from water resulted in a higher content (concentration) of the organic substances in sludge and worsened its susceptibility to conventional dewatering on plots, forcing new technologies such as mechanical dewatering on belt filter presses [1, 8, 9].

2. Sludge processing at the two reference water treatment plants

The methods of sludge treatment and final disposal at WTPs usually depend on the chemical and biological composition of treated water, the technical and technological potential of WTPs, as well as economic (e.g. capital and operating costs) and field (land area designated for the facility) conditions. In the case of the reference WTPs, both the water quality and the capacity of the plants were decisive factors. The Raba WTP is the larger of the two reference plants and its nominal capacity is 186,000 m^3/d. The plant collects water from the water reservoir – the Dobczyckie lake. There are two different types of sludge produced in two main treatment lines [1]; the slight differences between them are described in detail below.

Sludge from the older treatment train (in operation since the early 1970s, including basic physical and chemical processes such as coagulation, sedimentation + rapid anthracite/sand filters '+') was collected:

- periodically, from the bottom of the rectangular clarifiers (every few days),
- daily, after the rapid backwashing of filters.

Water treated in the second train (in operation from late 1980s, with coagulation/sedimentation in suspended sludge flocculators-clarifiers i.e. accelators, ozonation and filtration using dual media filters) has the sludge discharged [1, 12]:

- periodically (every few hours), from 'accelators',
- incidentally, after ozonation chambers,
- daily, after rapid dual media filters backwashing.

The sludge passes to one of six gravity thickeners and is then pumped to the sludge drying beds (covered). Sludge thickeners are oversized, as for the actual needs. This is a typical situation found at the many Polish water treatment plants built before 1990, planned and designed for the large water consumption expected in future. The thickeners, although designed as continuous flow reactors, are operated in sequence. The plant is specific since it draws water from the water reservoir, which results in a relative reduction in the mass of pollutants (due to sedimentation in the reservoir). On the other hand, the plant has to dose powdered activated carbon to maintain the required odour and water taste.

The other unit, the Rudawa WTP, with a capacity of 55,000 m^3/d) produces three types of sludge:

- solids after sand filter backwashing, discharged daily,
- solids after carbon filters backwashing, discharged once per fortnight,
- solids from rinsing clarifiers, rapid mixing chambers and flocculation chambers, discharged about twice per year.

All types of sludge are pretreated in sludge sedimentation tanks (thickeners) after being mixed with rainwater and then the supernatant is discharged directly into the Rudawa river (as specified in the water permit). The common issue for both WTPs is a very heavy traffic in the catchments above the water intakes, which, in spite of the safety devices, always poses a risk of water contamination by heavy metals. However, as was demonstrated in previous research, the concentrations of the metals in both raw and treated water did not exceed the limiting values [1, 8].

3. Changes in sludge quality at the Raba WTP

The Raba WTP produces water at a relatively constant rate; its monthly average flow rate is about 2.9 million (about 95,000 m^3/day), with the extreme monthly values ranging from 2.5 to 3.4 million m^3. The research focused on the sludge quality and the presence of heavy metals, which is the most important issue regarding its future handling and disposal. The results of these studies are shown in Tab. 1. The values exceeding the limits for the use of sludge for both agricultural and non-agricultural purposes, as specified in the Regulation of the Minister of the Environment of 1 August, 2002 on municipal sewage sludge, were bolded [13].

Table 1

Sludge produced at the Raba Water Treatment Plant from years 2010 to 2013

Metal	Unit	Agriculture	Non-agriculture	Results						
				20.08. 2010	02.11. 2010	11.04. 2011	11.10. 2011	12.04. 2012	08.10. 2012	04.03. 2013
Chromium	mg/kg DS	500	100	90	**113**	**110**	58	**199**	**120**	20
Cadmium	mg/kg DS	10	25	4.0	< 3	< 3	< 2.2	< 2.4	6	2
Copper	mg/kg DS	800	1,200	65	85	82	89	128	75	161
Nickel	mg/kg DS	100	200	83	99	86	67	**212**	68	**151**
Lead	mg/kg DS	500	1,000	36	56	27	31	27	104	27
Mercure	mg/kg DS	5	10	< 0.5	< 0.5	0.5	< 0.4	< 0.5	4.2	< 0.4
Zinc	mg/kg DS	2,500	3,500	158	179	182	146	354	182	211

Table 2

Sludge from the Raba Water Treatment Plant and the possibility of its final disposal

Metal	Percentage of tests meeting the requirements to use		The ratio of the metal content in the most contaminated sample to the least contaminated sample
	Agriculture	Non-agriculture	
Chromium	100%	43%	10 : 1
Cadmium	100%	100%	3 : 1
Copper	100%	100%	2.5 : 1
Nickel	100%	71%	3.9 : 1
Lead	100%	100%	3.9 : 1
Mercure	100%	100%	10 : 1
Zinc	100%	100%	2.2 : 1

Table 2 shows results of tests on heavy metals content in sludge, as an important factor influencing potential sludge disposal methods. The last column of this table underlines the issue of relative variability of heavy metals in sludge; the lowest concentrations were compared with the highest values found in the samples. The authors also tried to determine whether, and to what extent, the sludge meets the requirements of the final natural application (by specifying the percentage of allowable metal content found in the worst sample). It should be noted that due to the high content of chromium and nickel, some samples did not meet the requirements of the regulations of the Minister of Environment of 1 August, 2002 on municipal sewage sludge [13].

4. Changes in sludge quality at the Rudawa WTP

The amount of sludge produced in the Rudawa WTP was clearly smaller than in the other treatment plant due to both the lower water production of the plant and because of its higher quality of raw water. The study on heavy metals in sludge collected in clarifiers was conducted once or twice a year. The results of these tests are included in Tab. 3. It was observed that all samples met the requirements for sludge applied to the land application and/or for agricultural purposes. The quality of the sludge was clearly better than at the Raba WTP.

Table 3

Sludge from the water clarifiers at the Rudawa Water Treatment Plant for the years 2008–2012

Metal	Unit	Year				
		2008	2009	2010	2011	2012
Chromium	mg/kg DS	17	7.4	10.4	23	21
Cadmium	mg/kg DS	4.5	3.7	3.7	68	6
Copper	mg/kg DS	28	40	22	32	27
Nickel	mg/kg DS	14	18	19	28	21
Lead	mg/kg DS	78	64	64	69	89
Mercure	mg/kg DS	0	0.11	0.13	0.5	0.5
Zinc	mg/kg DS	668	554	570	560	456

In Fig. 1, the average and the maximum values observed in the analyzed period in sludge from both plants were compared. One can find the clear impact of a type of activated carbon used on heavy metal content in sludge. The metals that are effectively adsorbed on activated carbon are captured within a sludge mass; in the case of granular carbon, they are retained in the carbon bed volume [4] and do not become a component of the process sludge reported in Fig. 1 and Tab. 3.

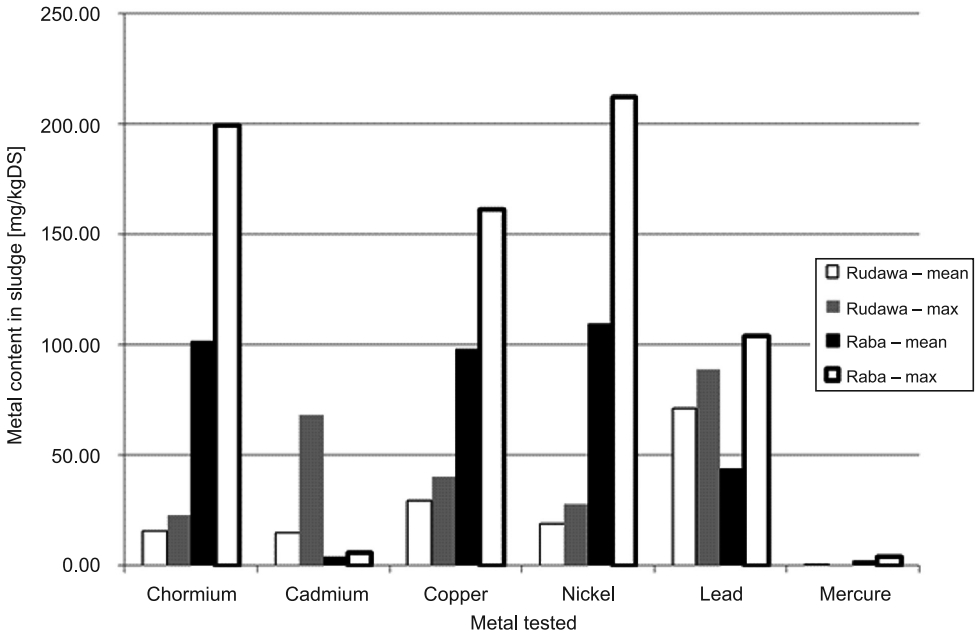

Fig. 1. Average and maximum concentrations of heavy metals in sludge from both water treatment plants during the study period

Metals with a low activated carbon removal efficiency (e.g., cadmium) are mainly removed by coagulation/sedimentation and thus, they are present in the post-coagulation sludge (the high content in the sludge from the Rudawa WTP). This phenomena requires further investigations.

5. Conclusions

– The paper analyzed sludge produced at two surface water treatment plants, treating water of similar quality but with different treatment technologies.
– In terms of heavy metals, none of the water samples tested in the years 2008–2013 exceeded the limiting values. Also, no increase in the concentration of these specific contaminants was observed in sludge during the treatment process, although in one WTP, the heavy metal content in sludge was too high to permit its use for land reclamation.
– The observed differences in the heavy metal content in sludge can be explained by the different forms of activated carbon applied; in the case of powdered activated carbon, heavy metals become components of the sludge, worsening its composition.

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KRZYSZTOF KSIĄŻYŃSKI*, IWONA PODRAZA**

GROUNDWATER OUTFLOW FROM THE BIAŁUCHA RIVERBED IN THE KRAKÓW VALLEY

ODPŁYW PODZIEMNY Z KORYTA BIAŁUCHY W KOTLINIE KRAKOWSKIEJ

A b s t r a c t

A decrease in the flow of the downstream stretch of the Białucha river (also known as Prądnik) has been determined based on the calculations of groundwater inflow to the Białucha riverbed completed for the Ojców–Giebułtów–Olsza stretch using the Wundt and Kille methods. Hydrogeological analysis of this phenomenon revealed that the water is lost at the point where the Prądnik leaves the Proszowice Plateau and enters the Vistula Plain as the Białucha. The reason for the water outflow from the riverbed is the relatively low groundwater level in the alluvia of the Vistula ice-marginal valley. Upstream from this point, groundwater is supplied to the river in a narrow valley eroded in rocky ground, while downstream water flows out into a thick aquifer drained by the Vistula river that is situated much lower in this area. A similar phenomenon is observed at the mouths of most tributaries of the upper Vistula. This problem is discussed in detail in the master's thesis by Iwona Podraza [6].

Keywords: *underground base outflow*, *hydraulic flownet*, *riverbed drainage*

S t r e s z c z e n i e

Na podstawie obliczeń dopływu podziemnego do koryta Białuchy (Prądnika) na odcinku Ojców–Giebułtów–Olsza, przeprowadzonych metodami Wundta i Kilego, stwierdzono spadek przepływu bazowego w końcowym odcinku rzeki. Przeprowadzona analiza hydrogeologiczna tego zjawiska wykazała, że straty wody występują w miejscu, w którym Prądnik opuszcza Płaskowyż Proszowicki i już jako Białucha wkracza na Równinę Nadwiślańską. Przyczyną odpływu wody z koryta jest relatywnie niższy poziom wód gruntowych w aluwiach pradoliny Wisły. Zasilanie podziemne cieku, które dotąd miało miejsce w wąskiej, wyżłobionej w skalistym podłożu dolinie, ustępuje w tym miejscu odpływowi w głąb warstwy wodonośnej o znacznej miąższości drenowanej przez znacznie niżej położoną Wisłę. Podobne zjawisko zachodzi przy ujściu większości dopływów górnej Wisły. Problem powyższy został szczegółowo opisany w pracy dyplomowej autorki [6].

Słowa kluczowe: *odpływ bazowy*, *siatka hydrodynamiczna*, *drenaż cieku*

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1. The problem

Many rivers near their mouth are characterised by a flow regime which is unusual for a humid climate. As a rule, riverbeds are fed by groundwater, but a natural outflow of water into a water-bearing stratum is observed at such points [8]. This phenomenon plays a significant role during low flows when the groundwater supply is the deciding factor of flow volume. This paper discusses a method used to calculate the value of the described anomaly, based on a case study of the Białucha, a tributary of the Vistula river.

The Wundt method was used to calculate the total underground inflow to the river. The lateral underground inflow used to determine the spatial distribution of inflow was calculated using a hydraulic flownet. The soil parameters were calibrated as required in the latter calculation to ensure consistency of both methods. The rainfall infiltration factor was also calculated using the approach described. This factor enables one to calculate the value of the actual recharge of groundwater without complicated calculations of net precipitation and losses in the runoff process. Despite the relatively general assumptions, the values obtained using this approach are more reliable than those obtained using the reverse seepage model, due to the indeterminacy of the latter approach resulting from the need to simultaneously calibrate the hydraulic conductivity and the water supply volume.

2. A description of the Białucha river drainage area

The Białucha river is a left-bank tributary of the Vistula. It is 33.4 km long and its basin covers an area of 195.8 km^2. There are three water-level gauge stations in the riverbed: Ojców (km 21.6, area 75.5 km^2); Giebułtów (km 12.8, area 111.1 km^2); Olsza (km 2.2, area 178.6 km^2). The river valley is eroded in a thick Upper Jurassic limestone complex and has a depth reaching tens meters, and in its middle stretch, even one hundred metres. As the Ojców Valley, it constitutes the major part of the Ojców National Park. The watercourse which is the hydrographical axis of the river basin is known as the Prądnik Brook in the valley, and as the Białucha in its lower course along the 8.7 km stretch flowing through the city of Kraków.

Only shallow alluvia on limestone beds occur in the areas adjacent to the river in the narrow Ojców Valley. The river valley in the city area is filled principally with alluvial soils, and the underlying aquifer is from 5–10 m thick. The main water-bearing horizon is situated in quaternary formations, present as sand of varying particle sizes, with additions of gravel towards the floor and as limestone pebbles with diameters of up to 10 cm, with additions of clay locally. The impermeable cap rock consists of sandy clays similar to loess (frequently with an addition of sand). The total thickness of quaternary sediments, depending on the ground morphology, reaches 20–22 m. The impermeable floor of the horizon consists of Miocene clays. The waters of quaternary horizons are fed with water from Jurassic limestone, principally in locations where the quaternary formations remain in direct contact with the limestone. A large alluvial fan, located principally west of the riverbed, formed as a result of the accumulation of brash carried by the river; limestone sands and gravels covered with fine brown sands are the main components of the water-bearing horizon. A system of small valleys, dating back mainly to the Holocene epoch, has also been formed here. The riverbed is characterised by a small slope – from 2 to 4.7‰.

3. Separating the underground outflow

The calculations of the balance for the Białucha are based on water level measurements taken at the water-level gauge stations indicated above. To ensure the homogeneity of our data set, we selected data from one period when measurements were simultaneously taken in the three water-level gauge stations. The only set of this type is available for the 6-year period from 1953 to 1958. The values of daily flows were calculated on the basis of daily water levels in that period for individual river cross-sections at the gauging stations, using rating curves developed as a result of hydrometric measurements. Finally, the Wundt method was used to calculate mean underground outflows.

The Wundt method [4] is one of the methods used to analyse discharge hydrographs and to separate the discharge portion corresponding to the groundwater outflow. In this method, the mean long-term groundwater outflow from European rivers in a moderate climate zone is assessed on the basis of the mean value of monthly minimum flow rates. The Kille method is a variety of the Wundt method, and consists of the assessment of mean daily underground runoff over a period of several years, based on the median of minimum monthly flow rates. Both methods can be used to determine, simply and rapidly, the mean groundwater outflow – that is, the value necessary to calculate the riverbed balance. In practice, the results obtained using the Wundt method are more consistent with the results of a detailed hydrograph analysis, and therefore, this method is applied in this study.

Table 1

Calculation of MLQ_m for the Ojców water-level gauge

1953	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	**Mean**
level	1.25	1.27	1.3	1.28	1.28	1.27	1.26	1.26	1.26	1.26	1.26	1.24	
discharge	0.206	0.237	0.286	0.253	0.253	0.237	0.221	0.221	0.221	0.221	0.221	0.191	0.215
1954	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
level	1.24	1.22	1.22	1.26	1.24	1.24	1.22	1.26	1.26	1.24	1.22	1.22	
discharge	0.191	0.163	0.163	0.221	0.191	0.191	0.163	0.221	0.221	0.191	0.163	0.163	0.187
1955	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
level	1.2	1.2	1.2	1.26	1.27	1.27	1.27	1.26	1.3	1.3	1.29	1.29	
discharge	0.137	0.137	0.137	0.221	0.237	0.237	0.237	0.221	0.286	0.286	0.270	0.270	0.223
1956	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
level	1.26	1.26	1.26	1.27	1.26	1.26	1.28	1.3	1.28	1.26	1.29	1.28	
discharge	0.221	0.221	0.221	0.237	0.221	0.221	0.253	0.286	0.253	0.221	0.270	0.253	0.240
1957	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
level	1.26	1.27	1.27	1.28	1.28	1.28	1.28	1.3	1.28	1.3	1.26	1.26	
discharge	0.221	0.237	0.237	0.253	0.253	0.253	0.253	0.286	0.253	0.286	0.221	0.221	0.248
1958	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
level	1.4	1.3	1.29	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.32	1.4	
discharge	0.477	0.286	0.270	0.286	0.286	0.286	0.286	0.286	0.286	0.286	0.321	0.477	0.320
													0.241

The lowest measured monthly levels (LL_m) for the years 1953–1958 [7] were set in tables and used to calculate the lowest monthly discharge values (LQ_m). The mean lowest monthly discharge values (MLQ_m) were eventually calculated for individual years and for the entire measurement period. Examples of calculations for the Ojców water-level gauge are given above. The mean of the lowest monthly discharge values amounts to 0.241 m^3/s in this case. These values for the remaining two water-level gauges amounted to 0.606 m^3/s in Giebułtów, and 0.298 m^3/s in Olsza.

4. Balance of the watercourse stretch

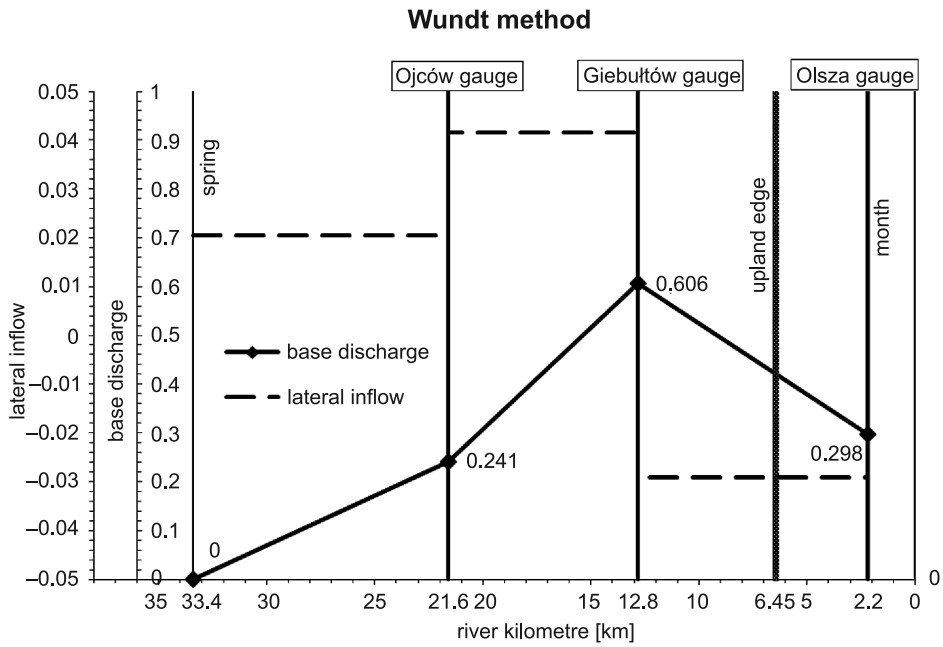

Fig. 1. Changes in base discharge along the river

The mean of the lowest monthly discharge values over a period of several years corresponds to the base discharge value in a gauging cross-section, that is part of the total flow in the riverbed coming from the inflow of groundwater for the entire river basin closed by limiting cross-section. A balance of groundwater inflow can be calculated for a river stretch based on discharge values in a cross-section. The difference between discharge values in subsequent cross-sections ($Q_2 - Q_1$) corresponds to the lateral inflow (Q_b) along the surveyed river stretch. This value can be used to calculate the mean rate of lateral inflow to the riverbed (the unitary lateral inflow, q) along the river stretch l:

$$q = \frac{Q_2 - Q_1}{l} = \frac{Q_b}{l} \, [\mathrm{m}^2/\mathrm{s}]$$

The value of q = 0.0204 m^3/s/km has been thus obtained for the Ojców water-level gauge, q = 0.0415 for the Giebułtów water-level gauge and q = –0.0291 m^3/s/km for the Olsza water-level gauge. The distribution of base discharge (MLQ_m) has been plotted based on mean underground outflows calculated for each water-level gauge along the surveyed watercourse stretch (Fig. 1). Changes in the groundwater inflow value along individual segments of the watercourse are also marked in the graph.

As shown on the graph, the Białucha in its stretch adjacent to the edge of the Proszowice Plateau is characterised by a distinct change in its feeding regime. An expected increase in the flow rate is visible along the Ojców–Giebułtów stretch, but the indications from the Olsza gauging station show a significant reduction in this value. A comparison of unitary lateral inflow values can emphasize the qualitative difference. The mean inflow to the river segment upstream from the Ojców water-level gauge amounts to 0.0204 m^3/s/km, then increases to 0.0415 m^3/s/km along the Ojców–Giebułtów river stretch, however changes its sign along the Giebułtów–Olsza stretch reaching a value of –0.0291 m^3/s/km. Hence, despite the river basin area being twice as large as the preceding river stretch, the last segment is characterised by an outflow of water from the riverbed. This reduction takes place even though the nature of land development and the associated coefficient of permeability do not change until the last three-kilometre long stretch of the watercourse. The loss in surface discharge occurs near the edge of the upland; therefore, a change in the geological structure of the base may be considered to be one of the principal reasons. A drop in water supply caused by changes in morphology or land cover could reduce the value of the lateral inflow, but could not change its direction to outflow.

5. Flow field around the watercourse

A hydroisohypse map from 1961 of the area where water outflows could occur was used to determine in detail the variations of riverbed feeding. The map covers the area surrounding the river following its entry in the Vistula ice-marginal valley where the water--bearing horizon is very thick. The hydroisohypses constituting a basis for the flownet have been plotted using measured data from the piezometers installed near the Białucha riverbed (for the upper terrace of the Vistula valley) [1, 2] and from the piezometers installed in the barrier protecting Kraków against water dammed up at Dąbie (for the area adjacent to the river). The longitudinal profile of the Prądnik along the surveyed stretch [5] and profiles of its tributaries were used to determine relationships with surface water tables. The hydroisohypses were used to plot streamlines, and the obtained flownet has been superimposed on the map (Fig. 2).

The area shown on the map includes, in addition to the Białucha, its tributaries; the right--bank Sudół and four left-bank streams, one unnamed in the Ojców Valley, the Garliczanka, the Bibiczanka and the Sudół Dominikański. The mouth of the last stream is located several metres upstream from the Olsza water-level gauge.

Using the net, we are able to precisely identify the locations where losses in the groundwater inflow occur whereas the application of the Wundt method could only indicate the river stretch where such losses took place. The map shows two segments

of the riverbed where runoffs from the river to groundwater occur. Both are situated between the mouths of the Sudół streams; in addition, water is lost in the lower segment of the (right-bank) Sudół stream bed. The value of water supply is small at the northern banks of both streams, and the segments of the Białucha running perpendicular to the hydroisohypses represent a zero balance.

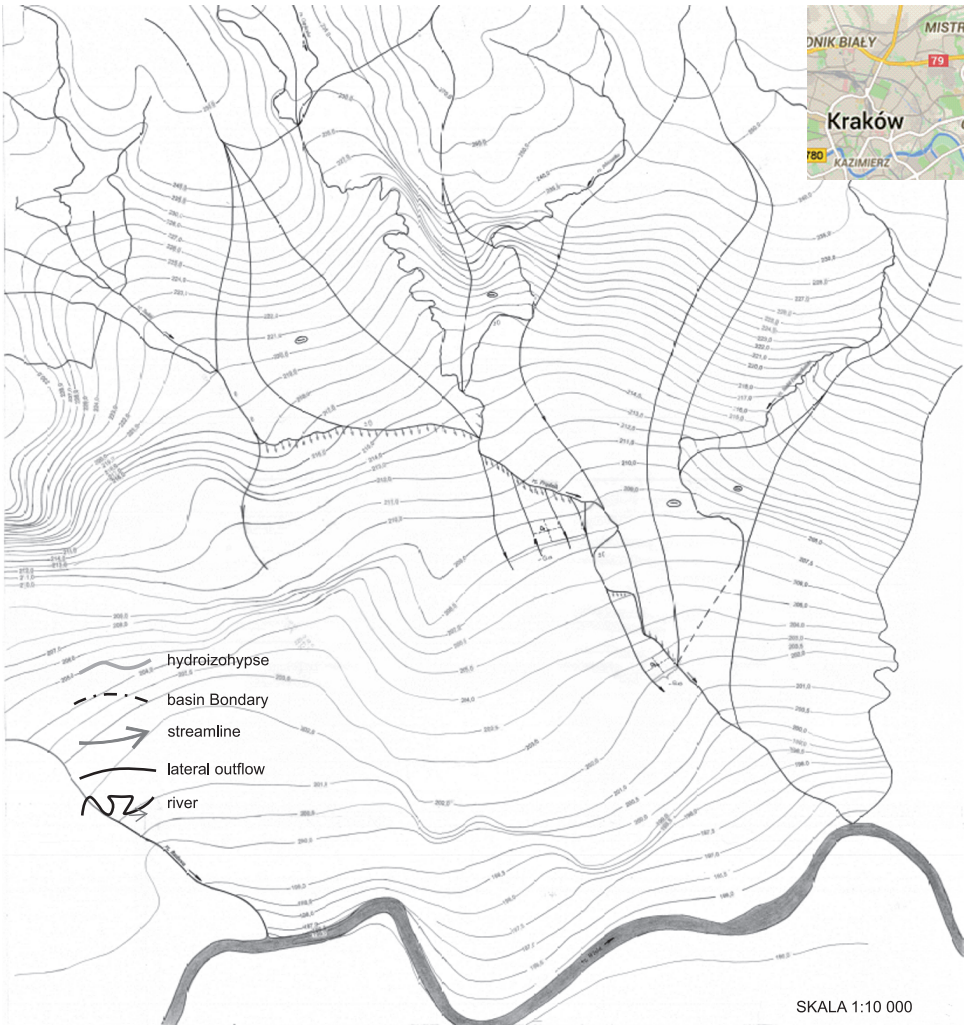

Fig. 2. A map of groundwater hydroisohypses in the area of lower Białucha river's stretch

The flownet was used to calculate values of inflow to the riverbed along its entire length between the Giebułtów and Olsza water-level gauges (Fig. 3). The value of point inflow was calculated based on the infiltration factor determined by the characteristics of the individual areas drained by the river (morphology, cover, soil types). The estimated total

volume of water that is lost by the river along the described stretch amounts to 0.569 m^3/s, of which 0.192 is along the first segment and 0.377 m^3/s is along the second segment. The northern banks of both segments are fed with about 0.001 m^3/s, and consequently, the balance of bilateral inflow is negative. A detailed curve of changes in base discharge along the described river stretch is shown on Fig. 3.

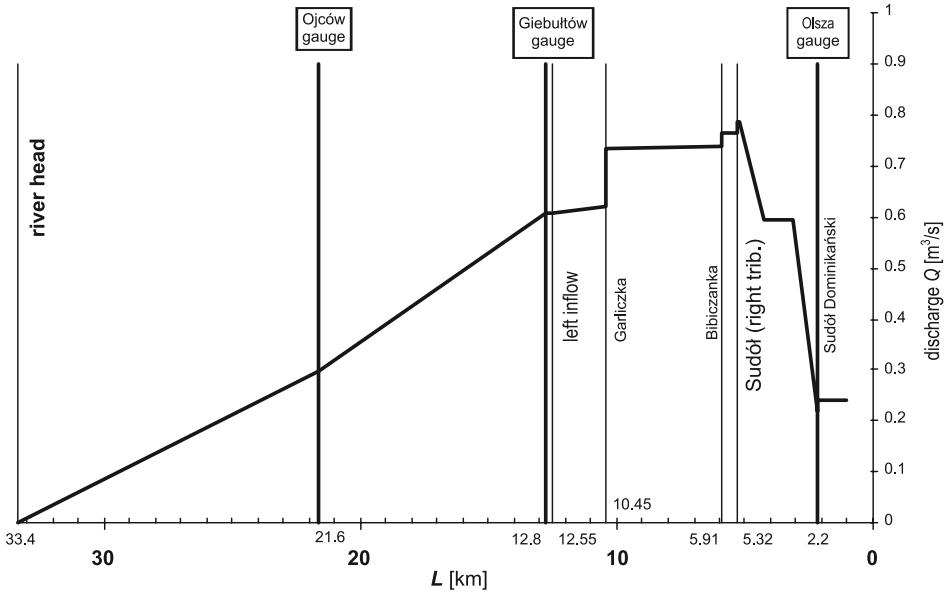

Fig. 3. A detailed graph of changes in base discharge along the drained river stretch

The graph demonstrates that the Białucha is characterised by a loss in surface discharge along the Giebułtów–Olsza stretch although it is fed from 5 tributaries. The riverbed stretch that is losing water is situated between the mouths of the Sudół and Sudół Dominikański, i.e. at the foot of the Vistula valley slope where the terrain height exceeds 212 m above sea level while the table of the Vistula, the recipient of groundwater, is situated at a height of about 196 m above sea level. This difference determines the significant velocity of outflow of water through the river bottom to the water-bearing horizon.

6. Conclusions

One of the principal reasons for water loss in streams in Polish conditions is the entry of a tributary riverbed in the main river valley (usually, the area of the alluvial fan), where the thickness of the water-bearing horizon dramatically grows while the underground water level (related to the recipient) is relatively low. The method described in this paper is suitable for identifying the volume and location of losses, but cannot be used to identify the root cause of the described feeding regime. This cause must be identified by an appropriate hydrogeological survey.

Significant losses in the base discharge of the Białucha river have been demonstrated along the river stretch between Zielonki (located north of Kraków, 6.5 km from its centre) and the city centre. The situation of the groundwater table in the area, as compared to the water table levels in the beds of the river and its tributaries (the Sudół and Sudół Dominikański), indicates that either no exchange of water takes place or that a runoff of water into the ground occurs. This water indirectly feeds the Vistula riverbed. The value of losses reaches 70% of the base discharge. An analysis of hydrogeological conditions indicates the existence of materials characteristic of an alluvial fan under the river bottom, which is a prerequisite of such high water losses. A lower surface infiltration index, resulting from dense land development in a portion of the surveyed area, may also have an effect on the value of losses.

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DAILY CHANGES OF WATER DEMAND IN THE SINGLE WATER SYSTEM ZONE IN KRAKOW

ANALIZA PORÓWNAWCZA ZMIENNOŚCI POBORU WODY W WYDZIELONEJ STREFIE WODOCIĄGOWEJ NA TERENIE KRAKOWA

A b s t r a c t

The paper focuses on changes in water demand within the water supply zone in Krakow. The analyzed time periods include the 90s of the last century and the years 2007–2012. Based on measurements of the size and variability of water demand, changes in daily water consumption within the week were analyzed and compared with the reference distribution. Both the results and research methodology can be used to update the guidelines for design and simulation of the water supply systems operation.

Keywords: *water system*, *water supply system*, *water demand*

S t r e s z c z e n i e

Artykuł zawiera analizy zmian poboru wody na terenie wydzielonej strefy wodociągowej w Krakowie. Bazując na pomiarach wielkości i zmienności poboru wody z lat 90. ubiegłego wieku oraz z lat 2007–2012, dokonano porównania zmian o charakterze ilościowym oraz nierównomierności poboru w ciągu poszczególnych dni tygodnia i porównano zmienność z rozkładem referencyjnym. Przedstawione wyniki badań oraz metodyka działań mogą posłużyć do aktualizacji wytycznych w zakresie projektowania i symulacji funkcjonowania systemów wodociągowych.

Słowa kluczowe: *system wodociągowy*, *wodociąg*, *zapotrzebowanie na wodę*

The authors are responsible for the language.

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1. Scope

Over last two decades people's attitude to the general commodity such as drinking water has changed due to political, social and economic transformations. Consumers changed their approach from purely consumerist, lacking concern for the amount of used and often wasted water, to more efficient and conscious management of this increasingly expensive good. Apart from well-known and widely described changes (e.g. [1, 2]) related to water savings and lower consumption, other changes followed related to the daily distribution pattern. They result from the new style of functioning of a modern society, where people start work at 5 a.m. or about 11 a.m. or some time they do not leave the house at all, working home. The study investigates the above-mentioned changes and looks into water demand fluctuations by observing a specific group of customers, classified as multi-family housing. Then the results are compared with the histogram of water demand for this group proposed in [3]. Similar pilot study was previously carried out in years 1993–1994 for a group of recipients living in the same region; the final results have been summarized and presented in [4] and [5].

2. Region characteristic

All information about water demand magnitude and variability relate to a single zone of the Krakow water system, the Mistrzejowice District, located in the north-eastern part of the city. From the water supply perspective. Mistrzejowice is a separate, isolated and detached zone, supplied only by a single pressure tank, Silver Eagles, located on the outskirts of the Golden Age Quarter. This zone has no water supply reservoir, which could buffer a water intake and flatten a distribution of water demand variability. This means that the pressure tank capacity matches the total demand for water in this area. Water recipients mostly occupy multi-storey buildings (10–12 floors), developed as housing estates, typical for the Nowa Huta District. They can be classified as multi-family housing. Other water consumers fall into the category of "services" and their number as well as water demand

Fig. 1. Water supply system-black contour line marks pipes served by the Silver Eagle pressure tank: a) in 1993, b) in 2010

fluctuate around 1% of the total demand in the zone. There are no other water consumers in the analyzed zone (i.e. the industry). Therefore, it can be assumed that the zone is a fairly homogeneous one and can be considered entirely as a multi-family housing zone.

Data from the 90s described water demand within the network shown in Fig. 1a), while the recent data covered the expanded zone shown in Fig. 1b). Over last 20 years the zone significantly expanded furnished with new distribution pipes supplying water to newly constructed multi-family units. Despite the significant development of the network the actual structure of water recipients in the zone remains unchanged. Hence, it still can be considered as a homogenous and classified as a multi-family housing unit.

3. Data analysis

The measurement data from the years 2007–2012 have been acquired by the authors courtesy of the Krakow Water and Sewage Works (MPWiK). The data, for various reasons, does not cover the whole period, unfortunately. Some measurements had to be excluded due to failures of measuring devices (lack of reading or a constant reading over several hours or days). Others were rejected after visual analysis of the daily distribution curve. Such cases involved the days when the flow fluctuations strongly deviated from the average values and significantly altered the daily distribution pattern. It can be assumed that these were periods of major system failures observed within the analyzed zone. Since the paper focuses on analysis of variability of water demand for a separate group of customers, such failures may interfere with the analyzed variability. For this reason, both extreme daily distribution patterns and the ones too distant from the mean were rejected from the test sample.

The measurement data, as digital readings of instantaneous flow and pressure on the pressure line, were collected in the local computer. Flow measurements were carried out with the POWOGAZ 250 MW water meter while pressure was measured with a pressure transducer Aplisens PC-28; the readings were carried out every 15 minutes. The readings used as a reference material [5], were carried out every 10 minutes. In order to compare the two sets of measurements it was decided to interpolate the actual 15-minute values and extract the values with the 10 minute intervals. The data from the 90s and the recent data have been analyzed with the same dedicated application HSO, which was created in previous studies. This procedure allows for a closer look at the comparable factors by analyzing the same test environment.

4. Water demand characteristics

The entire dataset was segregated to identify the groups describing demand variability in the consecutive days of the week. For such groups of data quantitative calculations were carried out to describe characteristic values of water demand. Additionally, matching indicators were determined for the whole period. The results are summarized in Tab. 1 while Tab. 2 shows corresponding results for years 1993–1994, as comparison. Values Q_r^* ask for some additional explanations; they present the annual demand for water, assuming that the whole year consists only of the one particular weekday, for example Monday.

Table 1

Water demand throughout the week, March 2007–May 2012

	Number of days	Q_r^* [m³/year]	$Q_{d\max}$ [m³/d]	$Q_{d\acute{s}r}$ [m³/d]	N_d	$Q_{h\max}$ [m³/h]	$Q_{h\acute{s}r}$ [m³/h]	N_h
Monday	167	783,397	2,713	2,146	1.264	200	113	1.769
Tuesday	165	776,793	3,147	2,128	1.479	172	131	1.312
Wednesday	161	775,325	2,604	2,124	1.226	190	108	1.751
Thursday	164	776,359	2,574	2,127	1.210	168	107	1.566
Friday	162	793,836	3,261	2,174	1.499	183	136	1.347
Saturday	174	866,371	3,283	2,373	1.383	193	136	1.411
Sunday	176	792,402	3,163	2,170	1.457	184	131	1.396
Total	1,169	809,750	3,283	2,218	1.480	193	137	1.411

Table 2

Water demand throughout the week, February 1993–December 1994

	Number of days	Q_r^* [m³/year]	$Q_{d\max}$ [m³/d]	$Q_{d\acute{s}r}$ [m³/d]	N_d	$Q_{h\max}$ [m³/h]	$Q_{h\acute{s}r}$ [m³/h]	N_h
Monday	69	1,384,164	5,608	3,792	1.479	328	233	1.404
Tuesday	70	1,390,592	5,619	3,810	1.475	325	234	1.388
Wednesday	71	1,431,046	6,829	3,921	1.742	471	284	1.655
Thursday	70	1,414,197	5,634	3,874	1.454	337	234	1.436
Friday	73	1,417,930	6,109	3,885	1.573	376	254	1.477
Saturday	73	1,605,705	6,536	4,399	1.486	367	272	1.348
Sunday	72	1,381,241	5,491	3,784	1.451	321	228	1.403
Total	496	1,433,407	6,829	3,927	1.739	471	284	1.655

Comparing the average daily and annual values it can be seen that that over 20 years water demand has decreased by about 40%, despite of expansion of the existing network and a noticeable number of new connections (new multi-family housings). Such observation complies with a general trend that manifests itself in a decrease of water demand per capita [2]. Unfortunately, the lack of detailed information on the number of residents in the studied zone makes it impossible to determine an interesting indicator – a water demand per capita for housing developments. However, some approximate data were used to estimate its value in both periods.

Scarce data from the year 1993 shows that population in the region was about 8,300 people at that time. Therefore:

$$W_j = \frac{Q_{d\acute{s}r}}{LM} = \frac{3,927\,[\mathrm{m}^3/\mathrm{d}]}{8,300\,[\mathrm{people}]} = 437\left[\frac{\mathrm{dm}^3}{\mathrm{person}\cdot\mathrm{d}}\right] \tag{1}$$

The value is substantially higher than the values assumed during that time (300 dm^3/person/d). Any attempts to estimate the value for the current data have also been very inaccurate. No information was available on a population density in this region; only the average population density for the Mistrzejowice District was available. In order to somehow use this information the authors determined the surface area of the zone and estimated a probable number of inhabitants:

$$LM = D \cdot F = 9,781 \left[\frac{\text{pop. density}}{\text{km}^2} \right] \cdot 1.24\,[\text{km}^2] = 12,128\,[\text{people}] \tag{2}$$

$$W_j = \frac{Q_{d\acute{s}r}}{LM} = \frac{2,218\,[\text{m}^3/\text{d}]}{12,128\,[\text{people}]} = 183 \left[\frac{\text{dm}^3}{\text{person} \cdot \text{d}} \right] \tag{3}$$

W_j is still very large, here. Current guidelines regarding water usage [6] assume the indicator value within the range of 80–160 [dm^3/person/d]). Such a large value (a rough estimate) may result from large water losses in this area, or poor estimation of the indicator, by assuming a lower number of inhabitants.

Comparing both analyzed periods with respect to N_d and N_h it may be noted that the coefficients decreased slightly. The differences are so small that they may be attributed to some in accuracies in data sampling. It is difficult to determine a well-defined trend line of changes based on the obtained results.

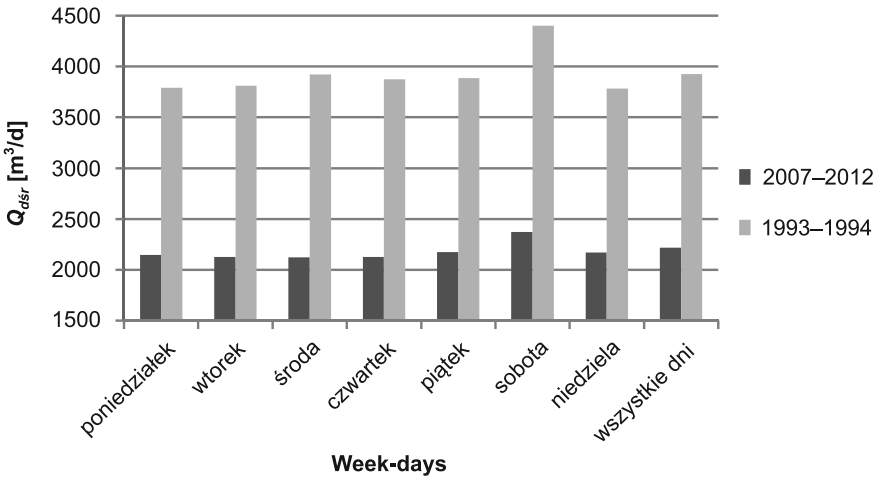

Fig. 2. Changes of $Q_{d\acute{s}r}$ during week-days for two data sets

Figure 2 compares $Q_{d\acute{s}r}$ in both analyzed periods, for each week-day. A daily trend line is maintained, however a significant drop in water demand in the area is observed, recently.

5. Fluctuation of water demand in specific time intervals

Two data sets (1993–1994 and 2007–2012) of fluctuations of water demand during each week day were compared with a distribution for a multi-family housing proposed by [3]. A series of graphs have been developed (Fig. 3.1–8), which illustrate the changes. Each graph includes the histogram of water demand proposed by [3] and the histograms developed from the data collected in years 1993–1994 and 2007–2012, including a standard deviation band for each hour of the day for all days (years 2007–2012 marked in black). Horizontal marks above and below the graph show the standard deviation for each hour. The gray dashed line marks histograms for years 1993–1994; solid dark-gray line shows a reference histogram.

When comparing samples from the years 1993–1994 and 2007–2012 some changes can be observed. A flattening of the evening peak as well as delay and spread of the afternoon saddle was observed on Monday to Friday, while at the same time higher flows occurred in night hours. Changes in the time interval of 6:00–23:00, involving reduction of graph dynamics, may result from greater diversification of the local community in terms of their activity and social functions. Popular human behaviors such as e.g. departure for work at 6:00–7:00 a.m., popular in 70s/80s of the last century, has evolved and got shifted to later hours. An increasing share of night flows may be associated with more accurate measuring devices that have been installed in recent years. Readings of night flows from the years 1993–1994 were incorrect due to a relatively high threshold value of water meters; they did not recorded flows lower than 12 m^3/h, what must be the cause of so low histogram value during this period.

Saturday and Sunday differ from the rest of the week; for both data sets the graphs look similar and do not follow the reference distribution.

Comparison of the average distributions for each day with the distribution proposed by [3], and considered as a reference, leads to the following conclusions:

- From 23:00 to 6:00 graphs looks very similar and overlap partially.
- On working days, the graphs differ from 6:00 to 23:00; morning rush hours are more stretched and flattened, if compared with a reference distribution. Also the middle day saddle is lifted and shifted by about two hours. The evening peak remains within the same hours, however it is more flattened.
- A reference graph, throughout its range, stays within the standard deviation boundaries determined for graphs for individual working days.
- Saturday and Sunday distributions are significantly different from the reference distribution and should be considered as functions describing a completely different human behavior.

The discrepancies between the reference distribution and the empirical distributions, however slight, may reflect the changes in the way the water system is used. It can be argued that over the past few years there have been significant changes in the functioning of urban communities. Growing social diversity in relation to activities outside home (different work hours and working part-time at home) is shown in the way the sanitation facilities are used at home. The symptoms of this change include flattening and delay of morning rush hours, as well as a delay and flattening of an afternoon saddle, if compared with the reference distribution. It is difficult to predict what might be the cause of a lower water demand during

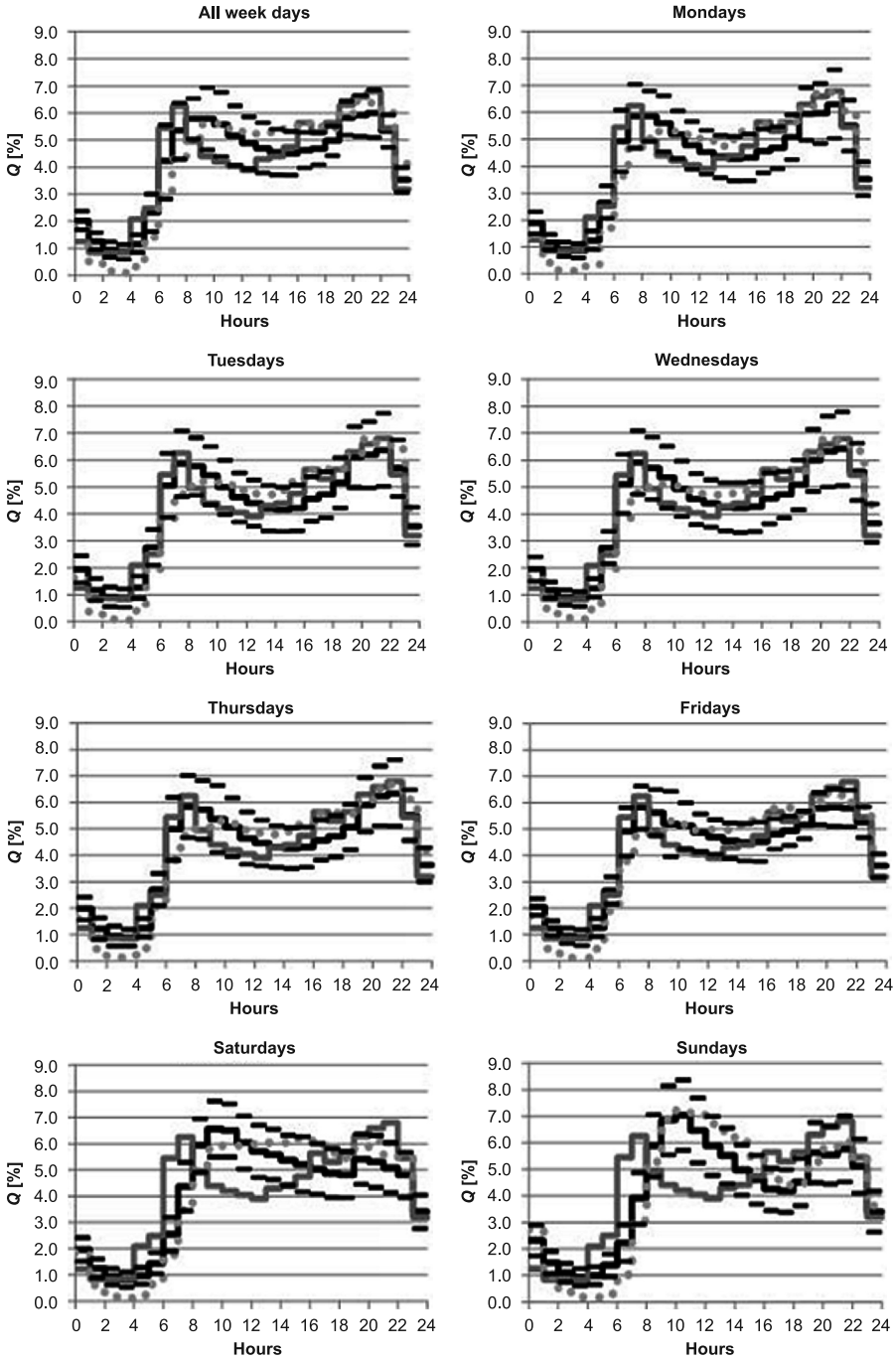

Fig. 3. Water demand histograms of for each week day and the follow-on histograms

the evening peak, though this phenomenon is clearly visible during each working day. Weekend distributions differ significantly from working days distributions with the highest water demand on Saturday and the lowest on Sunday. This observation is confirmed in both data sets. This may be related to the nature of the two days – Saturday is usually spent on chores while Sunday is a day of rest, also from the activities involving water use.

The empirical resultant distribution, describing the average weighted changes in the region does not differ significantly from distributions observed in working days. As such, it confirms the trend set by a reference distribution with all previously reported discrepancies. The empirical resultant distribution can be considered as standardization of human behavior in the analyzed water.

6. Seasonal variations of water demand

The measurement data set was used to estimate the seasonal changes in water demand. Hence, the annual measurement periods were divided into 3-months' intervals, which overlapped strongly with the seasons. Within these ranges summation of demands during available days were performed and then the average daily flow ($Q_{dśr}$) was determined for each season. The results are shown in Fig. 4. It can be seen that the greatest daily water demand was observed in winter, while the lowest in summer. The differences between the seasons are not large; they deviate from the average value by no more than 2%. A similar analysis was done for the data from years 1993–1994 and the results are shown in Fig. 5. During this period, an observed seasonal variability remained within a range of ±15% of the average value. It is difficult to explain and interpret this rather large quantitative difference between the two measurement periods, though the same trend in changes in water demand in different seasons was observed, for both cases. Strangely, the demand for water is lowest during the summer. It can be explained by the fact, that the analyzed water supply zone is located in a large city, which are usually deserted during summer vacations and total water demand in this period is the smallest.

The above relationships merely attempt to grasp a cycle of seasonal changes of the water demand in the zone with multi-family housing. Information obtained in this way, can be

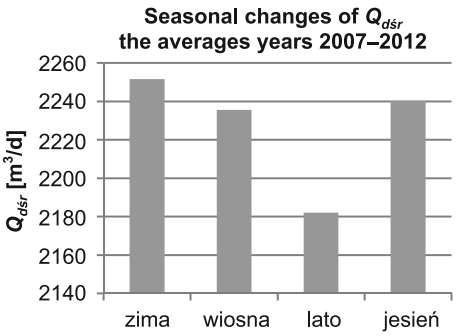

Fig. 4. Seasonal changes of $Q_{dśr}$; averages for years 2007–2012

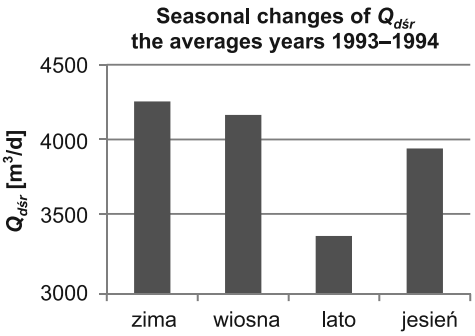

Fig. 5. Seasonal changes of $Q_{dśr}$; averages for years 1993–1994

further used only after more detailed studies. The studies would enable to determine seasonal peak and minimum flows, which are used to estimate water demand fluctuations, as well as in design or modeling of water supply systems.

7. Summary and conclusions

The obtained results can be used to identify some relationships in water demand within the water zone. The histograms of hourly variation generated for each week day as well as for the entire measurement period may be helpful when updating already old but valid guidelines for water demand programming within the multi-family housing zone.

Comparison of distributions of water demand variation over the past few years points out also at some changes, such as: a delay of the morning rush hours and the flattened southern saddle. They can be associated with major transformations that took place in our country over the past decades.

Analysis of seasonal variation showed different water demands at different seasons but the differences are rather small and difficult to interpret in a definitive way. Additional research on a larger test sample is necessary to get a more conclusive picture of these changes.

The research zone can be considered as homogeneous (in terms of land use) and classified as multi-family housing. The final results obtained by the authors do not allow for their propagation to other city zones, with a different land use. However, they may provide a good basis for further more detailed analysis; the methodology for the analysis has been developed during this project.

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METHODOLOGY OF STREAM HYDROCHEMICAL ANALYSIS IN SPATIOTEMPORAL TERMS

METODOLOGIA ANALIZY HYDROCHEMICZNEJ CIEKU W UJĘCIU CZASOPRZESTRZENNYM

A b s t r a c t

Hydrochemical profiles were used to create a spatiotemporal hydrochemical analysis of a watercourse. This method is based on the correlation between the concentration of pollutants and the intensity of water flow in the watercourse. It is a method that allows for an analysis of qualitative and quantitative changes of pollutants occurring in the watercourse. The paper describes the steps to be followed to obtain hydrochemical profiles of the watercourse. The function of time has been introduced in order to illustrate the analysis in a three-dimensional way. All of the profiles are shown in one graph, where qualitative and quantitative changes are visible within water levels at a specified time interval. The results are presented as graphs allowing for the classification of the course due to water quality, showing the hydrochemical shell in the watercourse during the year.

Keywords: *spatiotemporal hydrochemical analysis*, *surface water quality*

S t r e s z c z e n i e

Do stworzenia czasoprzestrzennej analizy hydrochemicznej cieku wykorzystano metodę profili hydrochemicznych. Metoda ta opiera się na zależności korelacyjnej pomiędzy stężeniem zanieczyszczeń a natężeniem przepływy wód w cieku. Umożliwia ona analizę zmian jakościowych i ilościowych zanieczyszczeń zachodzących na długości cieku. Opisane zostały kolejne kroki, jakie należy wykonać w celu uzyskania profili hydrochemicznych cieku. Aby przedstawić analizę w ujęciu trójwymiarowym, wprowadzono funkcję czasu. Wszystkie profile przedstawione zostały na jednym wykresie, gdzie widoczne są zmiany jakościowe i ilościowe wód w cieku na przestrzeni zadanego przedziału czasowego. Wynikiem były wykresy pozwalające dokonać klasyfikacji cieku ze względu na jakość wody przedstawiające powłokę hydrochemiczną na cieku w ciągu roku.

Słowa kluczowe: *czasoprzestrzenna analiza hydrochemiczna*, *jakość wód powierzchniowych*

Translated by Małgorzata Gąsińska.

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1. Introduction

Consumers of water take in water from a source, use it and then return it to circulation. As a result of use, water quality changes due to changes in the physical, chemical and biological properties. Some of this water flows into rivers in the form of sewage, contributing to the deterioration of the quality of the existing water resources. The consumers include the population, industry, public utilities, agriculture and forestry. The water may be used by them for drinking, as a basic production raw material or an essential element of production processes. The discharges of post-consumer water and sewage generated in the production process greatly affect the quality of water flowing in courses. Solving the problem of wastewater treatment to allow for its reuse is a necessary condition of a well--functioning economic water cycle [2].

Surface waters have a certain self-purification potential associated with the biological life developing in the aquatic environment. An adequate dissolved oxygen content in the water is a factor determining aquatic life development, and thus the continuity of the process of self-purification. Dissolved oxygen is consumed in the mineralization of organic compounds entering the water with sewage. It is therefore necessary to have a balance between the self-purification potential of water and the amount of oxygen required for the mineralization of pollutants which naturally enter the watercourse and originate from municipal and industrial wastewater [3].

The level of water pollution in rivers depends not only on the amounts introduced with sewage, but also on the flow in the river, which affects the dilution of pollutants [2]. Water volume in streams is not constant over time. It is connected with variable rainfall during the year. The geographical location and the geological catchment area also affect the water supply. Controlling the flow rate of watercourses is also related to the specific needs of consumers. Even short-term water shortages could cause major damage in many areas of economic activity. Further intakes are determined so as not to impede the existing ones.

The necessity for the continuous monitoring of the quality and quantity of water pollution in watercourses is due to the specific requirements of water for use. This is an important part of water management, the task of which is to supply or make water available in the necessary quantity and quality sufficient both for the population, and for individual sectors of the economy [2].

2. Methods

The method of hydrochemical profiles is one of the methods used to perform analyses of watercourses. It was created as a result of attempts to harmonize ways of testing watercourses, it was proposed by H. Mańczak in 1963 [2]. Hydrochemical profiling is a statistical method of assessing the degree of water pollution in rivers based on the results of periodic testing of water in measuring and control sections. The research allowed for establishing the correlation between the concentration of the pollutant and the water volume flow rate (***Q***) in individual cross-sections. The existence of this correlation allowed for the reference of average pollutant concentrations to the low flow (*SNQ*) which was formally

recognized as a concentration relevant for the assessment of the level of pollution in river water. Concentrations related to *SNQ* are applied onto the longitudinal profile of the course. In this way, changes in the level of water pollution in the river along its length are illustrated. The resulting hydrochemical profile includes abrupt changes due to point-discharges of pollutants or resulting from the increase in water flow in the river due to its tributaries. Hydrochemical profiling has been used in the assessment of water quality in rivers in Poland.

Taking into account the function of time in carrying out the hydrochemical analysis, we obtain a three-dimensional hydrochemical shell showing the changes in the watercourse quality, as they occur between the profiles within a year.

The following information is needed to perform such a hydrochemical analysis:

- location of balance cross-sections along the length of the watercourse,
- water gauges ***W***,
- water quality at monitoring points ***M***,
- cross-sections of the estuaries of side tributaries ***d***,
- water intake points **p** and the discharges of sewage ***z***,
- partial catchment areas closed with balanced cross-sections [km^2],
- the volume of the average intensity of low flow $\boldsymbol{SNQ_R}$ in water gauge cross-sections,
- the volume of the relevant $\boldsymbol{BZT_5}$ concentrations at monitoring points [mg/dm^3],
- the volume of water intakes $\boldsymbol{Q_p}$ [m^3/s] and the volume of wastewater discharges $\boldsymbol{Q_z}$ [m^3/s] and the BZT_5 content in the discharges $\boldsymbol{S_z}$ [mg/dm^3] [1].

If it is impossible to measure gauges on tributaries, it is necessary to add the catchment area of each tributary to the catchment area of the main course. This results in increases in the catchment area of the main watercourse in the cross-sections of estuary tributaries. Knowing the actual *SNQ* values [m^3/s] measured in gauge-points and the volume of water intakes Q_p [m^3/s] and discharges Q_z [m^3/s], natural SNQ_N values can be determined in gauge cross-sections. Then, using the principle that the SNQ_N increase growth is directly proportional to the increase in the catchment area, SNQ_N values across all balance cross-sections can be determined by means of interpolation and extrapolation. Knowing the value of SNQ_N across all balance cross-sections, and the volumes of intakes and discharges, SNQ_R [m^3/s] can be determined across all balance cross-sections. The next step is to determine the size of pollution loads in all cross-sections. This can be done based on the values measured at monitoring points. In connection with the occurrence of water intakes along the course, it is necessary to set the actual loads at water intakes. The pollution load from the watercourse along with water is the product of the concentration of pollutants at the intake S_p and the volume of the intake Q_p. Both of these values (S_z and Q_z) are known for sewage discharges. To calculate the concentrations of pollutants in intakes, it is necessary to calculate the size of a unit load l_j, i.e. the load increase associated with the SNQ_N increase at the watercourse section between the monitoring points. Following the calculations of the SNQ_N concentrations in cross-sections where water is taken, pollutant loads collected from the course along with the water can be calculated. Knowing the values of the collected loads and the values of the loads discharged into the course measured at monitoring points, the natural values of loads at individual monitoring points can be calculated, followed by interpolation and extrapolation to calculate natural load values for all balanced cross-sections. Using the known volumes

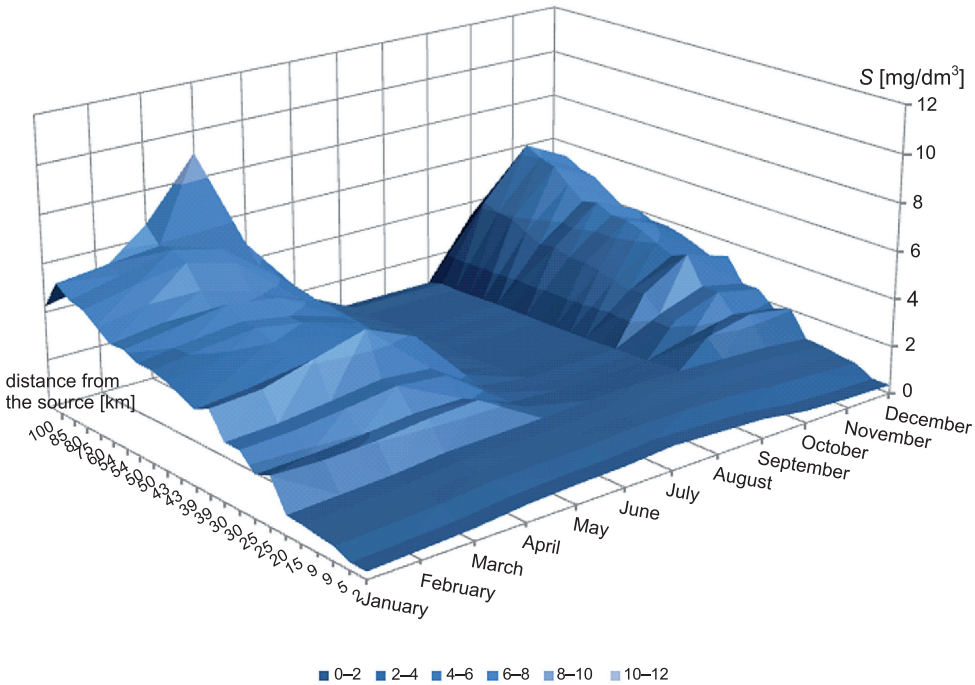

Fig. 1. Spatiotemporal graph of pollutant concentrations

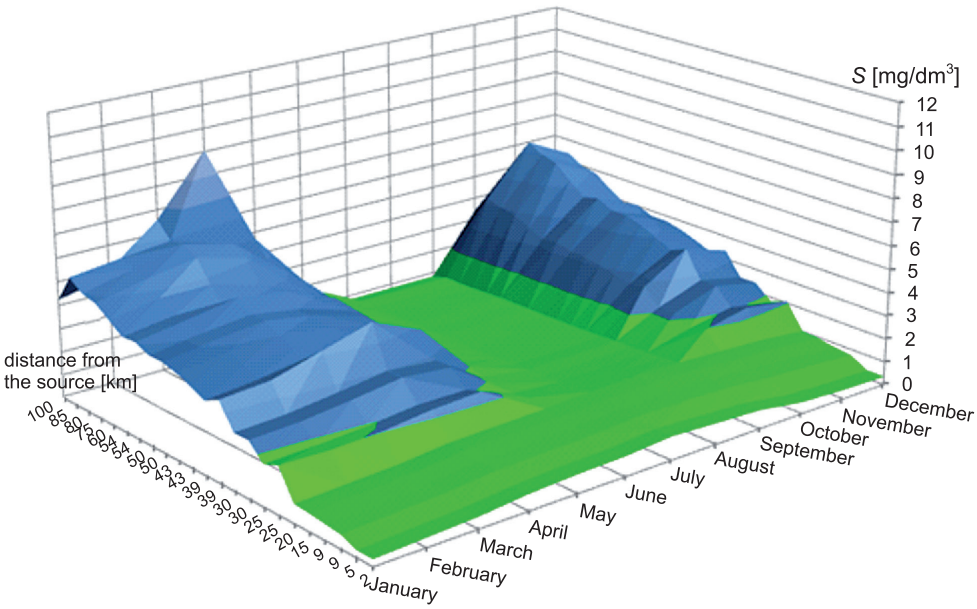

Fig. 2. Spatiotemporal graph of pollutant concentrations with the selected threshold concentration

and the volumes obtained by calculations, we can determine the volumes of the actual loads $Ł_R$ in all of the balanced sections.

In order to determine BZT_5 profiles, it is necessary to calculate concentrations in cross--sections along the entire course. The concentrations are calculated as the ratio of the actual load in the section and the actual value of the average low flow. Performing analogous calculations for a number of time intervals and plotting the results on a graph, we obtain the spatiotemporal image of the concentrations of pollutants in the course during a year.

Knowing the concentrations of all of the analyzed sections in the course, absorbency can be calculated. In the case of the assumed value of the threshold concentration, the load for each balance cross-section is calculated. Using the information on the volume of the threshold load, one can determine the ability of the course to adopt a sewage load in the given balanced cross-section so as not to exceed the threshold load. This value is absorbency, calculated as the difference between the threshold load at the cross-section and the actual load present at the same cross-section. Continuing the spatiotemporal assumptions of the analysis, absorbency can also be represented in its spatial form.

3. Results

Calculations were carried out in a tabular form, as presented. All of the calculations and graphs were prepared in MS Office Excel, 2010. This tool works well in the case of the hydrochemical method by Mańczak, however, a disadvantage appears in the case

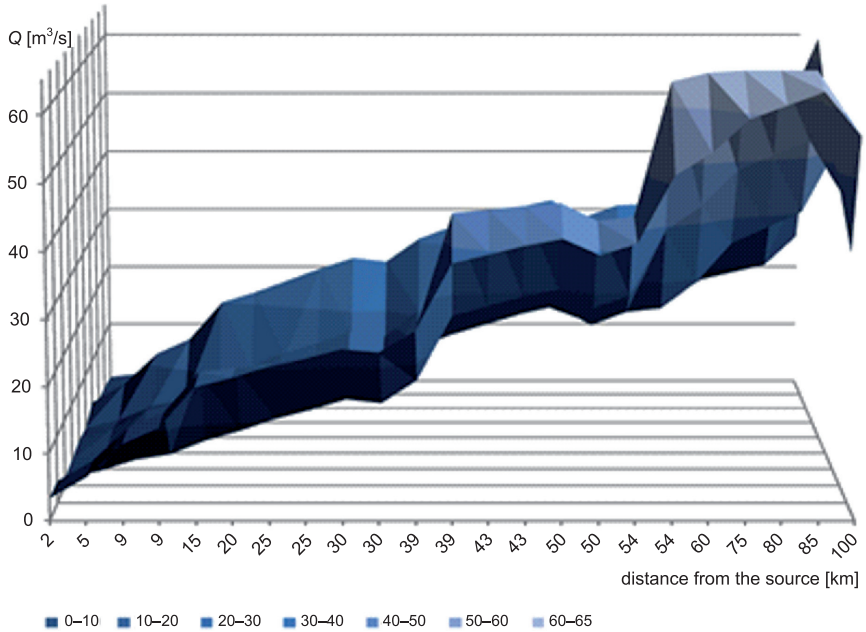

Fig. 3. Average low flow in the course in a particular month, presented as a panel graph cross-section

of the creation of three-dimensional diagrams. The panel graph, on an axis representing the course longitudinal profile, at the points of the increase of flows (tributaries, discharges, intakes), values are taken separately. Below, the two forms of graphs illustrate the course flows. Figure 3 shows *SNQ* as a cross-sectional panel graph in a selected month, while Fig. 4 shows *SNQ* of the same month as a flow value profile along the course.

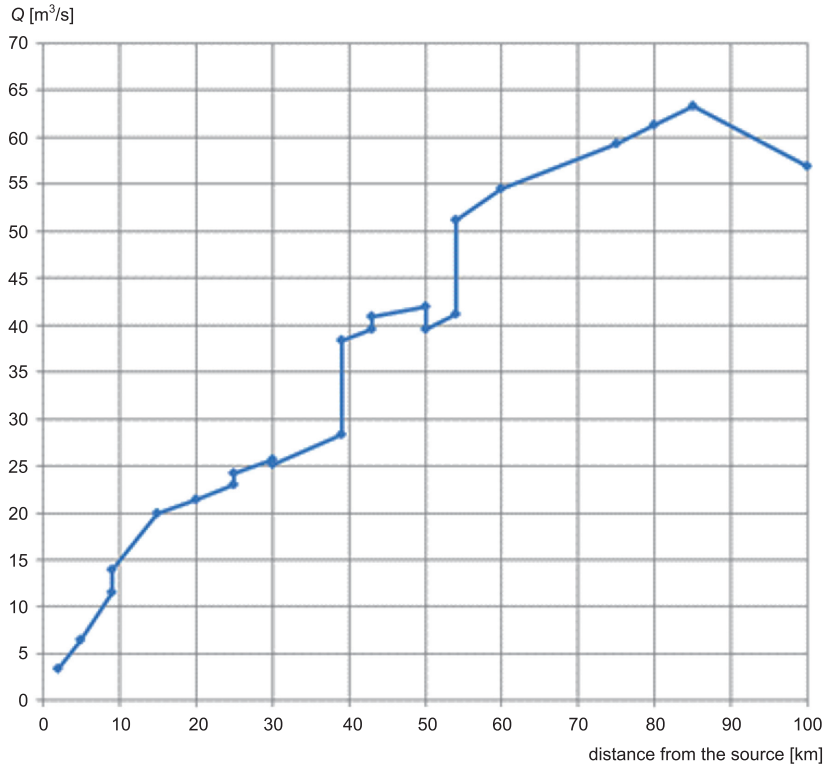

Fig. 4. Average low flow in the course in a particular month, presented as a watercourse profile

A three-dimensional image of the status of water in the watercourse was obtained as a result of the simulation. Three-dimensional graphs can easily provide profiles of pollutant loads in various research cross-sections along a watercourse during a year. An illustration of the qualitative and quantitative status of watercourses in a particular selected point over time can be easily obtained. Following the calculation of the concentrations of pollutants, it is also feasible to prepare a spatiotemporal diagram classifying waters into grades according to the criteria listed below.

Due to assuming proper formatting criteria, the graph shows when, and at what points of the watercourse, given purity grades occur. The above watercourse graph shows that the waters of the course did not reach the levels of pollutants that would classify them in the fifth grade.

Table 1

Classification of water quality due to BZT_5 concentration

Water grade	Quality requirements	
First	$0 < S \leq 2$	[mg/dm^3]
Second	$2 < S \leq 3$	[mg/dm^3]
Third	$3 < S \leq 6$	[mg/dm^3]
Fourth	$6 < S \leq 12$	[mg/dm^3]
Fifth	$S > 12$	[mg/dm^3]

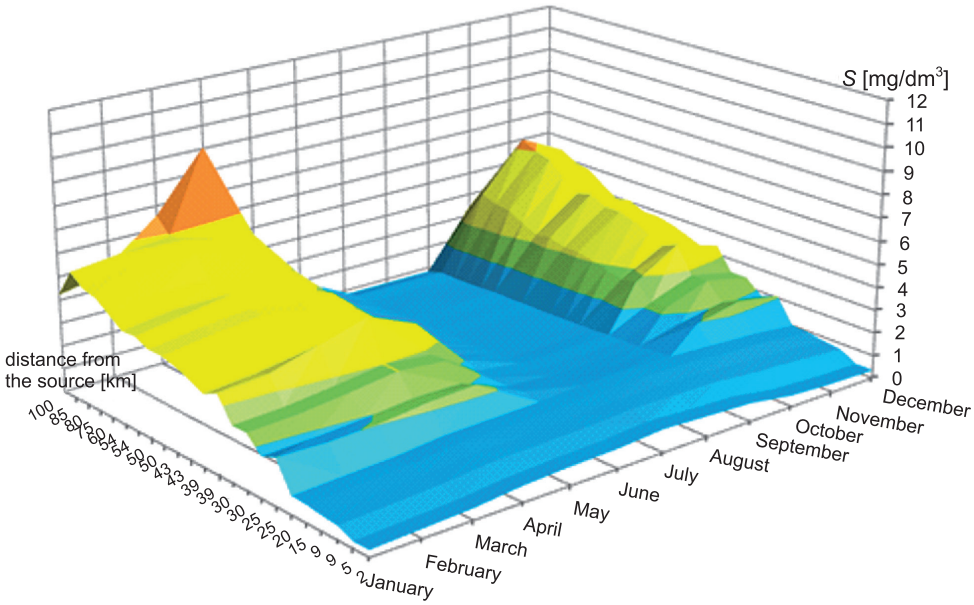

Fig. 5. Concentrations of pollutants with grades determined

4. Conclusions

The aim of the study was to present a three-dimensional hydrochemical analysis. It is based on the method of hydrochemical profiles that has been functioning for half a century. The methodology of the analysis has not been changed, however, an additional function of time has been introduced and the way of presenting the results has been changed. This was possible due to the available computer tools. The hydrochemical analysis allows for classifying rivers according to their purity. The three-dimensional representation of the hydrochemical analysis of water quality allows for referencing to the entire length of the course at a selected point in time. Hydrochemical profiling allows for the consideration of qualitative and quantitative needs in terms of water use for all water

consumers. The three-dimensional graphs greatly facilitate the interpretation of results. They also provide additional aspects of the exploration of information on the status of the watercourse. They allow for modeling how will the changes in the method of watercourse exploitation affect the quantity and quality along the course in a broader perspective. They also provide the possibility of predicting how the changes in the environment of the watercourse during a year affect the selected points along the watercourse.

The spatiotemporal graphs allow for locating the areas where actions need to be taken to improve the quality of the water flowing along the watercourse during a year. In order to effectively improve the quality of the flowing water, it is necessary to fully identify and locate the factors which adversely affect the status of the watercourse. Perspective actions should be taken, not only causing a one-time effect on the improvement of water quality, but also bringing long-term effects, reducing the risk of pollution of watercourses.

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IMPORTANCE OF GEODETIC SURVEYS IN TERMS OF ASSEMBLY AND OPERATION SAFETY ON EXTERNAL TENDON – STRESSED TANKS

ZNACZENIE POMIARÓW GEODEZYJNYCH DLA BEZPIECZEŃSTWA MONTAŻU I EKSPLOATACJI ZBIORNIKÓW SPRĘŻANYCH ZEWNĘTRZNYMI CIĘGNAMI

A b s t r a c t

This article describes methods for control over geometry of tanks made of precast panels tensioned circumferentially with external tendons. Certain advantages and disadvantages of applied engineering methods for measurement, along with reasons for necessity to apply more accurate geodetic survey methods are discussed. An example of optimal, according to the author of this article, geodetic technology for control over panel assembly and monitoring of panels tensioning, including relevant study, have been detailed.

Keywords: *prestressed tanks*, *geodetic monitoring of structures*

S t r e s z c z e n i e

W artykule przedstawiono metody kontroli geometrii zbiorników z prefabrykowanych płyt sprężanych obwodowo kablami zewnętrznymi. Scharakteryzowano wady i zalety stosowanych metod inżynierskich pomiaru i przedstawiono argumenty przemawiające za koniecznością stosowania bardziej dokładnych metod geodezyjnych. Przedstawiono przykład optymalnej, według autora, geodezyjnej technologii kontroli montażu płyt oraz monitorowania procesu ich sprężania wraz z opracowaniem.

Słowa kluczowe: *zbiorniki sprężane*, *geodezyjny monitoring budowli*

The author is responsible for the language.

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1. Introduction

Recently, circular tanks made of precast ferroconcrete components and tensioned by external circumferential tendons have gained significant popularity. All systems of that type are noted for short-term construction, winter-time completion option, relatively low cost, as well as durability, and leak tightness of assembled structure (Fig. 1). These tanks have been widely used in waste-water treatment plants, farms, etc.

Individual systems for construction of precast tanks that are tensioned by external tendons are characterised not only by different structure of the panels, method of tightness or tension but by the procedures for ensuring appropriate structure geometry as well. Not always do these procedures require application of geodetic measurements. Tank erection and operation safety depends mainly on provision of accurate geometric parameters to the structures [1]. Insufficient erection accuracy may be one of the reasons for construction disaster [2].

Fig. 1. Circular tank made of precast components

Basing on professional experience [3] and bearing in mind the results of construction disaster reasons, the author proves that the technology for tank construction shall consider application of geodetic measurement methods in the course of assembly and tensioning.

Various recommendations related to geodetic projects, necessary to be carried out on site, along with accuracy requirements and technology for tension monitoring were developed.

2. Technology for tank construction

The foot of the tank constitutes a monolithic, reinforced ferroconcrete bottom plate. For assembly of tank walls, 1,295–2,295 mm wide and 3–7 m high precast panels are used (Fig. 2). Multiple capacity tanks are erected, both small, e.g. 260 m^3 volume consisting of 10 panels, and huge ones with volume of 6,460 m^3 consisting of 60 panels [4]. The structure is tensioned into circumferential direction through the application of external steel tends.

Fig. 2. Tank erection

Regardless of the system or the size of tanks, technology applied for construction remains almost unchanged. Bottom plate is reinforced and poured out on the site. On the plate circumference, approximate layout of wall components is set out through geodetic or engineering method and then some washers are arranged. These washers are placed at identical level with the application of levelling instrument. Precast wall panels are transported to the site in succession. Each time a couple of those panels are brought in by truck-tractor platforms. Directly from a vehicle, wall panels are moved to the bottom plate by a crane and then arranged onto the washers around its circumference. Consecutive panels are pushed close to each other. Every one plate is stabilised with a spacer fixed to the bottom of the tank.

This spacer holds the plate but still allows it to slide and incline within a limited range. Consequently, correction of the wall plate orientation against bottom plate and its verticality determination are ensured. Upon arrangement of all elements, control over tank geometry is carried out with application of various methods. One of those consists in controlling angle between panels with 1 meter staff. The staff is applied to the panels in a manner its endings are aligned within identical distances from their joints. Height of a triangle that is created by the staff and the panels is translated into the angle between those elements. In case of other systems, control over slits on joints between panels is not required. Panels are plumbed with the level line.

Upon pulling in and fabricating strands, initial tension is commenced and continued up to obtaining usually 20% of the final tension. A foundation ring is placed around new-built wall. Wall plate vertical joints are filled in with expansion concrete if in the course of arrangement adhesive sealing or mortars have not been applied. Then, final tension is commenced in double-stage process. Upon completion of each tension stage, control of tank geometry is carried out.

3. Tank geometry impact on failure hazard

The technology that is described above does not ensure implementation of tank geometry parameters with required accuracy. Plate inaccurate placement does present problems during erection, especially in the course of matching the last plate. The shape and size of the tank has a crucial impact onto stress pattern within the structure both throughout tensioning and, later, at some stage in operation as well [2].

Fig. 3. Construction disaster

Failure to maintain geometric conditions might have been one of the reasons for the construction disaster where some wall components of earlier tensioned finished tank collapsed (Fig. 3). Vertical plate joints were damaged since internal corners within concave parts of the joints were truncated. Component of force that causes truncation is all the bigger as the angle between the panels increases. In this case 3,990 m^3 tank, with R_w = 14.655 m internal radius, contained a wall consisting of 61 fabricated elements. Therefore the angle, 193.44^g, between those components was exaggerated from a basic premise (Fig. 4).

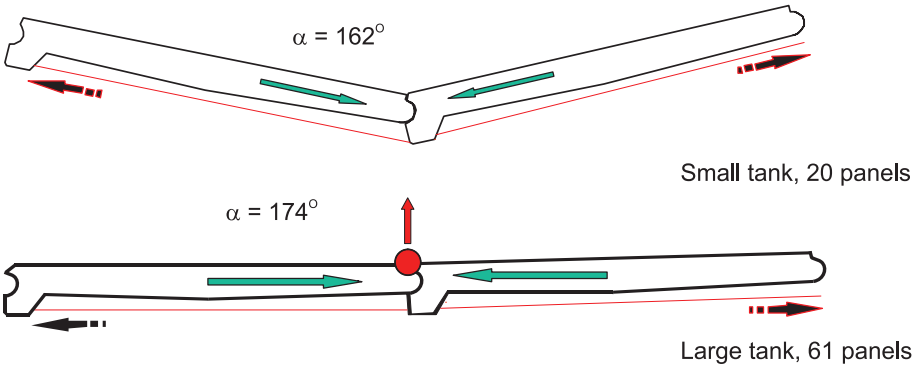

Fig. 4. Influence of tank size on angle between panels

Assembly manual for this tank allowed, among others, the following tolerances:

– provision of a tank radius Ra ±10 mm in comparison with design value,
– provision of an angle between panels ±1,5° in comparison with design value.

Recommended by the manual, geometry control methods that constitute a part of engineering methods included following measurements:

– measurement of radius with a tape from the centre of circle determined at the beginning of assembly works,
– measurement of angle between panels through measurement of distance (x) from panel joint to 1 meter staff applied symmetrically to tank walls (Fig. 6),
– it does not provide control of the size of the slit on panels joints.

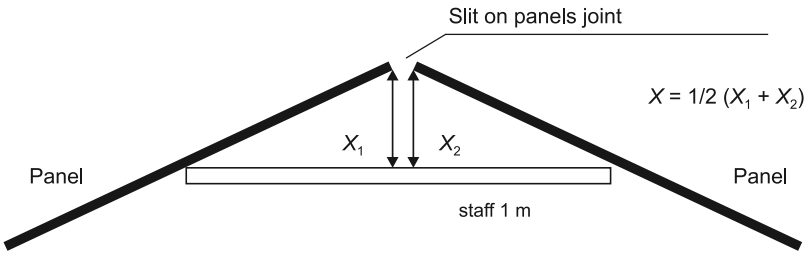

Fig. 5. Manner for determination of angle between panels through measurement of distance (X) from 1 m staff

It is hard to meet those requirements through the application of recommended measurement methods, as:

Measurement of radius is only theoretically easy to be carried out. In practice, accuracy of this measurement is limited on account of difficulties in identification of appropriate points on a plate. The second factor that deteriorates accuracy of measurements is the fact that in the course of tension panels relocate. For that reason the centre of the tank, which was determined at the beginning of erection, becomes outdated (the differences may achieve a few cm).

Angle measurement by recommended method is not accurate as well. The reason is too short staff and local irregularities of surface that may interfere the results of the measurements. Actual accuracy of the measurement was to be estimated only on the basis of records included in logbooks developed upon inspection that was carried out prior to disaster. On that basis, sum of internal angles for polygon created by the panels was determined. Angular deviation determined on the basis of polygon closure was fk = 7.7°. Assumption related to absence of mistakes brings following measure error for 61 angles:

$$m_\alpha = \sqrt{61} \cdot 7.7^o = 0.9^o$$

Similar analysis that was carried out for other tanks, proved that measure error for application of this method fluctuates from 0.3° up to 1.1°.

Obtained results prove that measuring methods applied for control over tank geometry are characterised by insufficient accuracy precluding verification of assembly tolerances that are required for that type of tanks.

4. Geodetic tasks to be carried out in the course of tank assembly

Recommendations related to geodetic observations that are to be carried out in the course of completion of tanks made of fabricated tension panels were developed. This method was confirmed in practice [3].

In the corners of each plate, 4 testing marks are installed. Optimally, these marks should be installed in the course of plate manufacture. In case it cannot be provided, this action may be carried out when panels are still placed on platform, prior to installation. These marks should be spaced within identical distances, e.g. 5 cm from plate edge with ±5 mm accuracy. The marks at the bottom of panels should be installed above foundation ring.

Control over geometry of spaced panels is carried out through geodetic observations of marks' placement. Topcon GPT-9003 scanning tachometer was applied for the observation. The measurement was completed through polarity method with application of the workstation placed nearby the centre of the tank. A distance to marks, measured with laser rangefinder, was carried out with 2 mm accuracy.

Differences between the distance to the lower mark and the distance to the mark placed above it constitute plate deflection from verticality. Distances between neighboring marks may indicate size of slits on panel joints.

Coordinates of the lower marks are determined. Basing on these coordinates, parameters of the circle approximating panel lower points are calculated. The difference between the distance of the point from the centre of the circle and the length of the circle radius constitutes radial stand-off distance of the panel from optimal location. These calculations may be carried out directly on the site. Duration of such measuring cycle depends on the number of the panels from which the tank wall is constructed of. Even in case of large tanks, the measurement duration including calculations does not exceed 1 hour. Typically, three observations are to be carried out. Firstly, upon panels' arrangement when on the basis of observation results the location of those panels is adjusted. On the strength of the results of the second observation, which is implemented upon initial tension, it is measured whether existing relocation of the entire structure does not impact geometric conditions and whether final tension may be carried out. Upon final tension, it is necessary to check the tank final geometric condition.

5. Example of measurements

Practical results of measurements and the manner for the data handling constitute an example of observations carried out in the course of the tank erection at waste water plant in Gdów [3]. It is 400 m^3 tank consisting of 2.275 m wide and 6 m high 12 wall components (Fig. 6).

Table 1 contains the results of tank measurements that were carried out prior to tension. Differences between measured distances to plate lower mark and to the mark placed above it (col. 10) constitute a degree of plate deflection from verticality. 'Plus' sign means deflection to outside, 'minus' sign means deflection to the centre. Maximum 40 mm difference means deflection from verticality by 0.4°. On account of obtained results, deflection of panels from verticality was corrected continuously.

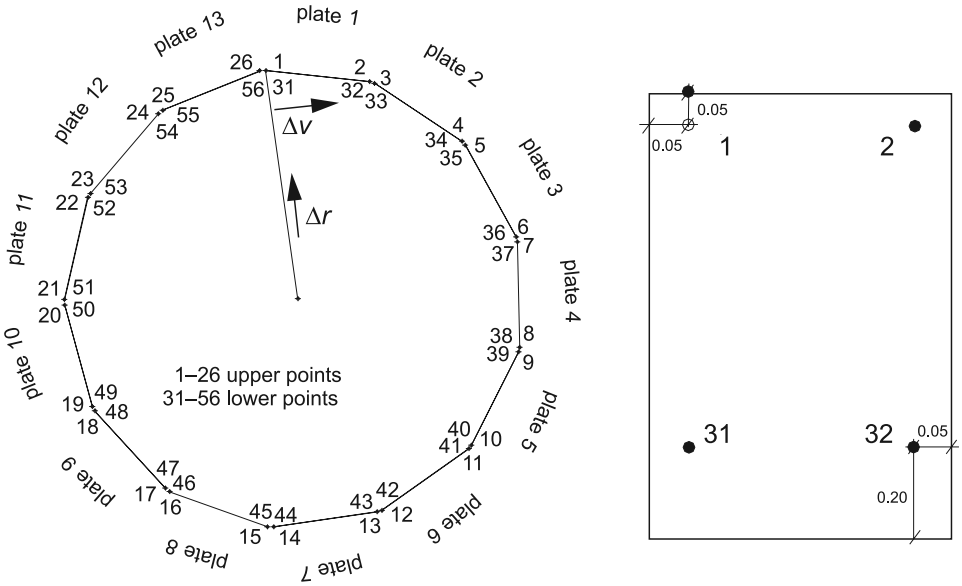

Fig. 6. Sketch of testing points arrangement

Table 1

Measurement results for control marks

Plate number	No. of upper point	*x*	*y*	R_g	No. of lower point	*x*	*y*	R_d	$\Delta R = R_g - R_d$ [mm]
1	2	3	4	5	6	7	8	9	10
1	1	4.685	–0.661	4.732	31	–0.661	0.239	4.733	–1
	2	4.461	1.492	4.704	32	1.496	0.239	4.703	1
2	3	4.424	1.594	4.703	33	1.597	0.240	4.704	–1
	4	3.235	3.401	4.694	34	3.407	0.239	4.694	0
3	5	3.148	3.477	4.691	35	3.478	0.240	4.695	–4
	6	1.253	4.530	4.696	36	4.528	0.240	4.698	–2
4	7	1.151	4.558	4.701	37	4.558	0.238	4.701	0
	8	–1.017	4.606	4.715	38	4.603	0.239	4.714	1
5	9	–1.126	4.576	4.712	39	4.577	0.239	4.713	–1
	10	–3.057	3.599	4.722	40	3.598	0.241	4.722	0
6	11	–3.142	3.522	4.720	41	3.527	0.243	4.727	–7
	12	–4.394	1.753	4.730	42	1.754	0.244	4.737	–7
7	13	-4.431	1.652	4.730	43	1.654	0.244	4.770	–40
	14	–4.745	–0.489	4.734	44	–0.493	0.244	4.773	–39

8	15	–4.732	–0.598	4.771	45	–0.603	0.243	4.804	–33
	16	–4.013	–2.641	4.777	46	–2.648	0.245	4.808	–31
9	17	–3.942	–2.732	4.798	47	–2.731	0.242	4.808	–10
	18	–2.345	-4.197	4.802	48	-4.198	0.244	4.811	–9
10	19	–2.248	-4.249	4.810	49	-4.247	0.244	4.825	–15
	20	–0.158	–4.823	4.807	50	-4.816	0.242	4.819	–12
11	21	–0.051	–4.823	4.825	51	-4.816	0.237	4.816	9
	22	2.064	-4.342	4.806	52	-4.330	0.240	4.795	11
12	23	2.164	-4.283	4.801	53	-4.279	0.243	4.779	22
	24	3.813	–2.881	4.791	54	–2.871	0.240	4.766	25
13	25	3.874	–2.790	4.774	55	–2.786	0.234	4.746	28
	26	4.682	–0.782	4.770	56	–0.770	0.240	4.740	30

In order to determine an optimum orientation of wall panels on the foundation plate, the points stabilised at the lower part of the plate were approximated with the circle. Basing on x, y coordinates (col. 7, 8), through parametric method, the elements of the circle equation were determined.

$$(x-A)^2+(y-B)^2=R^2$$

where:

A, B – coordinates of the circle centre,
R – the circle radius.

Determined equations for correction are as follows:

$$2(A_0-2x)\,dA+2(B_0-2y)\,dB-2R\,dR+x^2+y^2-R_0^2=0$$

Calculation results upon two iterations bring parameters for the circle equation:

$$A=-0.021;\quad B=-0.056;\quad R=4.752$$

It must be noted that A and B parameters represent displacement rate of the circle centre from its initial placement at the stage of panel assembly. In this case, measurement of the radius with application of traditional method would bring incorrect results.

Table 2 includes calculated distances of the points from the centre of the circle and radial deflection from the optimal placement $\Delta r = D - R$. 'Plus' sign means that the plate shall be slipped to the outside, whereas 'minus' sign means that the plate shall be slipped to the inside [3].

Graphic display of displacements is shown in Fig. 7. Basing on measurement results, correction of wall panels' placement was carried out continuously. Measurement & calculation process, described above, was repeated twice, i.e. upon initial tension and upon final tension.

At the same time, testing on plate deformation during tension was carried out.

Table 2

Radial deflections for control points

Plate	Point	*D*	*R*	Δ*r* = *D* – *R*
1	31	4.746	4.752	–0.006
	32	4.740		–0.012
2	33	4.743		–0.009
	34	4.749		–0.003
3	35	4.750		–0.002
	36	4.758		0.006
4	37	4.761		0.009
	38	4.764		0.012
5	39	4.762		0.010
	40	4.751		–0.001
6	41	4.755		0.003
	42	4.739		–0.013
7	43	4.735		–0.017
	44	4.746		–0.006
8	45	4.750		–0.002
	46	4.760		0.008
9	47	4.753		0.001
	48	4.752		0.000
10	49	4.750		–0.002
	50	4.762		0.010
11	51	4.760		0.008
	52	4.754		0.002
12	53	4.750		–0.002
	54	4.750		–0.002
13	55	4.755		0.003
	56	4.752		0.000

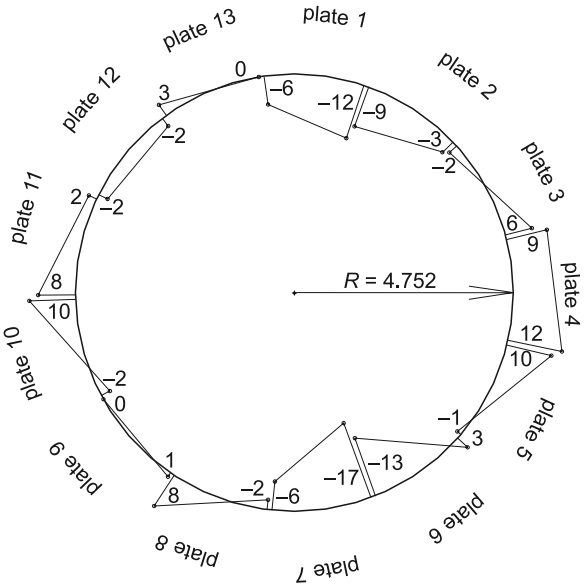

Fig. 7. Plate deflections from optimum location [3]

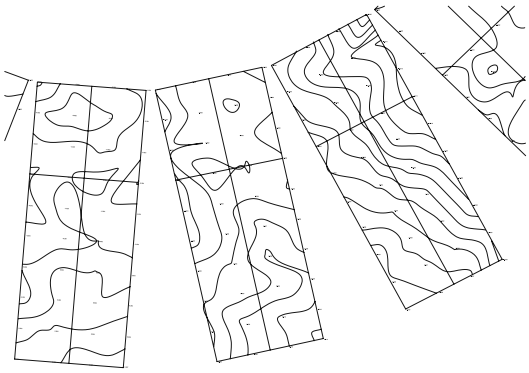

Fig. 8. Plate deformation isolines upon tensioning [4]

The panels were scanned prior to tension and upon final tension with Topcon GPT-9003A scanning tacheometer. Points network was obtained at approximately 60 cm distances, i.e., approximately 40 points in 10 rows meaning 4 points in each row. Comparison of the results of the scanning that was carried out prior to and upon tension enabled determining panel deformation. Deformation contour lines, which occur within panels as a result of tension (Fig. 8), were generated for individual panels. Within described example, panel deformations were minimal as these were not exceeding ±2 mm measurement accuracy [5].

6. Conclusion

The technology of construction of tanks made of fabricated panels tensioned circumferentially by tendons is unusually efficient. Numerous examples of failure indicate the necessity for distinctive care in the course of erection. Applied methods for control in some systems are limited to a certain degree, especially in case of large tank construction. In such cases, it seems to be necessary to apply geodetic methods of measurement.

The technology that has been applied by the author turned out to be efficient for two major parameters, i.e. in respect of the measurement duration and regarding obtained accuracy. This method enables current controlling over accuracy of plate assembly, as well as monitoring of plate tension. Monitoring refers to both displacement of individual fabricated components and determination of panel surface deformation under the influence of tensioning.

Major and crucial advantage of this technology is duration of measurements. Considering the fact that the panels are assembled directly from vehicles and it is necessary to carry out immediate control measurements on the site, it seems that this method may be optimal.

Significant acceleration of measurements including enhancement of accuracy might have taken place in case of assembly of control marks on panels in the course of their manufacture.

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JAN WRONA*

AN INTRODUCTION TO AN INSTALLATION FOR THE PRODUCTION OF ALTERNATIVE FUELS FROM WASTE POLYOLEFIN

INSTALACJA DO PRODUKCJI WYSOKOENERGETYCZNEGO ALTERNATYWNEGO PALIWA Z ODPADOWYCH POLIOLEFIN

A b s t r a c t

Plastic waste is material that can not only be repeatedly processed into other products, but also the energy contained within it can be recovered through burning.
This article presents an installation for the thermal utilization of waste plastics. The installation allows for the production of high energy fuel from waste polyolefin.

Keywords: *plastic waste*, *energy*

S t r e s z c z e n i e

Odpady z tworzyw sztucznych to materiał, który może być nie tylko wielokrotnie przetwarzany na inne produkty, ale również poprzez spalanie można odzyskać zawartą w nim energię.
W artykule przedstawiono instalację do termicznej utylizacji odpadów z tworzyw sztucznych (poliolefin). Umożliwia ona wytworzenie dwóch wysokoenergetycznych produktów – karbonizatu oraz regeneratu.

Słowa kluczowe: *odpady z tworzyw sztucznych*, *energia*

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1. Introduction

Global plastic production in 2011 was 280 million tons, Europe alone produced 58 million tons. Europe's demand was at the 47 million ton level (Russia excluded). This number represents a 1.1% increase in comparison to the previous year. Particular usage segments remain the same and packaging remains number one at 39% of total orders. The construction industry is number two followed by the auto industry, electrical and electronics industries [9, 11, 13]. The table below shows the demands for plastics in some European countries with the highest demand.

Table 1

The demand for plastics in Europe by country (kt/year) (Source: Plastics Europe Market Research Group (PEMRG))

Country	Germany	Italy	France	Great Britain	Spain	Poland
Demand around (kt/year)	11,800	7,110	4,600	3,700	3,600	2,800

As is shown, Germany has the highest requirements followed by Italy, France, Great Britain, Spain, and Poland in sixth position with a demand of 2,800 kt.

From among many different types of plastics, about 80% of the plastic market in Europe belongs to the following:

- Polyethylene (PE),
- Polypropylene (PP),
- Polyvinyl chloride (PCW),
- Polystyrene (solid-PS, foam-ESP),
- Polyethylene terephthalate (PET),
- Polyurethane (PUR).

It can be concluded from the data shown above that the needs for plastics are systematically growing and therefore, it is imperative that ways are found to reuse polymer plastics at the end of their life cycle.

As the packaging industry remains the highest user of such plastics, most polymer plastic waste that goes through the process of recycling originates in the packaging industry.

The growing awareness in regards to the utilization of any waste product decreases the amount of plastics being simply thrown out at dump sites. In Europe in 2011, more than 25.1 million tons of post-consumer plastics were collected. This represents an increase of 2% compared to the previous year, 2010. From this amount, a little more than 10 million tons ended up in the dump sites, but almost 15 million tons were reprocessed or used as a source of fuel. Plastics in general are perceived negatively, having a negative effect on the environment. The recycling of plastics limits greenhouse gas emissions and decreases the energy need to make new plastics. The methods used play a major role in achieving these goals [3, 13].

2. Plastics Recovery Methods

Proper natural resource management and maintaining adequate energy usage and production methods play a vital role in utilizing discarded plastics precisely because of their particular properties. From among many different substances, plastics can be reused in two different ways. Firstly, they can be remolded into new products and secondly, the energy they contain can be retrieved as an alternative source of fuel.

There are three possibilities of plastics recovery, as shown in Fig. 1. These are: mechanical recycling; stock recycling; energy recovery.

MECHANICAL RECYCLING involves sorting and shredding leaving the chemical structure unaltered in order to achieve a granulated form that can be reprocessed for remodeling. This kind of recycling is possible when the plastic is 'clean' and chemically homogenous. Proper sorting selection processes must be maintained in order to be able to collect desired plastics which meet the strict criteria.

STOCK RECYCLING is a process where plastics are down-cycled into three basic components (gases, hydrocarbons and monomers) by chemical reaction or thermal treatment. The basic components can in turn be used to create new products. This kind of recycling allows for the reuse of mixed or contaminated plastics. Technologies most often used are gasification, pyrolysis, and depolymerization. One of the new stock recycling methods is the blast furnace smelting reduction process. During the smelting reduction process, a synthesis gas is released comprised of carbon monoxide and hydrogen. The synthesis gas released in the process additionally increases the efficiency of the iron reduction process through bringing the cost down by limiting the amount of natural resources needed.

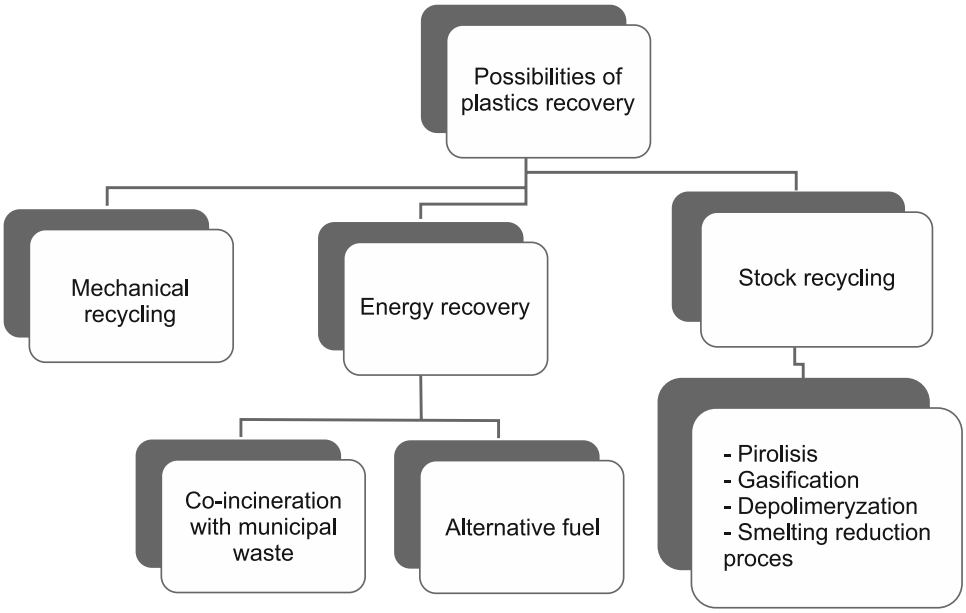

Fig. 1. The possibilities of plastics recovery

Technologically speaking, the above methods are well proven. In reality, only the smelting reduction in blast furnaces is economically profitable on an industrial scale. One of the methods to recover the energy from plastics is their co-incineration with municipal waste. This method is recommended whenever we are dealing with mixed or contaminated plastics and the intention is heat and/or electricity production. Stock recycling also represents an alternative high power energy source. Waste polyolefin can be successfully implemented in the production of high-grade ingredients needed to make diesel fuel, gasoline, and fuel or heating oil. The processes mentioned above are highly advanced, but because of catalyst implementation significantly increasing cost, this renders the process economically ineffective. Without any subsidies, this process is unprofitable (eg. in Poland, after the removal of excise tax incentives in 2007) [1, 2, 5–7, 10, 12].

3. Polyolefin – alternative fuel production system

Some of the components in stock recycling are being used for the production of diesel fuel, gasoline, and heating oil. The existing technologies don't allow for the production of high-energy alternative fuel in solid or semi-fluid state from so called 'polluted' polyolefin scrap material [6].

The innovation developed at Cracow University of Technology, is based on utilizing contaminated waste plastics by retrieving two components char and the reclaim which in turn may be employed in two ways; either as an alternative source of fuel or as a paraffin fraction for further processing [8].

The depolymerization block represents the main part of the installation and comprises of: the heater (4); melter (5); cracking furnace (6); set of burners (7). The remaining elements of the block are: segregation appliance (1); prefabricator (2); buffer (3); boiler (10); char container (9); reclaim reservoir (8). The depolymerization process starts with the segregation of the waste plastics. Municipal and non-municipal waste must be separated first. Next, the segregator enables the separation of the polyolefin fraction from any other undesired solid state waste. From the remaining material, we must isolate products that do not require any more treatment, among them are: rubber; polyvinyl chloride (PVC); polyurethanes (PU); polyamides (PA); polyimides (PI); ABS; others like nitrogen in polymer molecules or ferrous and non-ferrous metals.

The batch may consist of single packaging plastics or wrapping plastics made out of polyethylene, polypropylene, polystyrene or ethylene polytherephtalate (PET, PETE, PETP). As pointed out before, the above mentioned plastics have the largest share of around 80% of the European waste plastic market. Volume wise, they are of great importance but too often they end up being sent to landfills. Having the potential of being an alternative fuel source, they lie dormant in the dump. The useful polyolefin fraction must be prefabricated because of undesired physicalities like size or shape and also must be stored in the form of a buffer to be used as a batch ingredient in the depolymerizer. By heating and gluing, the material can be shaped into desired size and form by the prefabricator for ease of storage and depolymerizer insertion. Depolymerization and prefabrication modules are therefore independent of each other. Any polyolefin surplus buffer allows for the equalization and/or compensation of the not always equal efficiency of either the prefabricator or the depolymerizer devices.

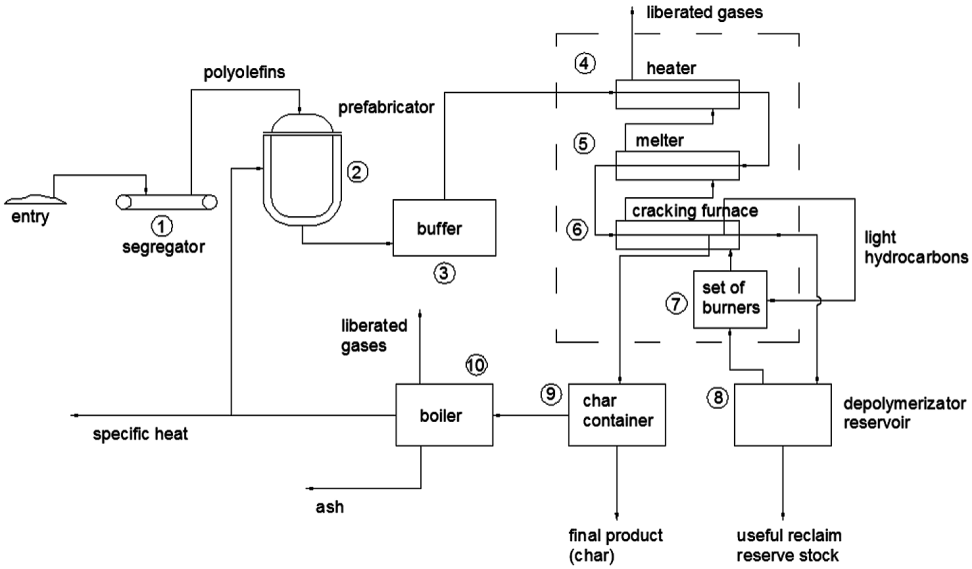

Fig. 2. Installation diagram for the production of high energy fuel from waste polyolefins

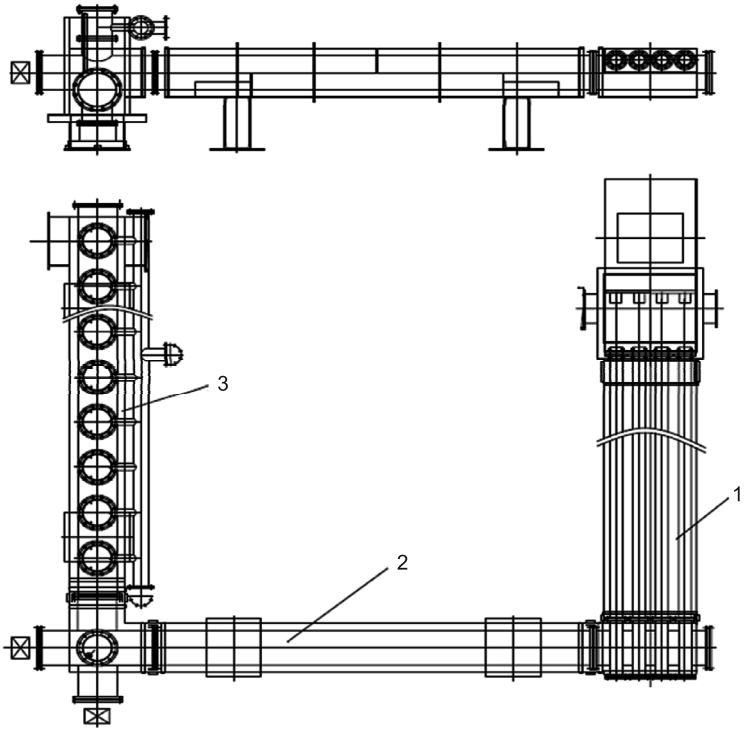

Fig. 3. Depolymerization block (1 – heater, 2 – melter, 3 – cracking furnace)

As was said before, the heater, melting and cracking furnace and set of burners represent the main depolymerization block. The heater brings the temperature up to the melting point. The gravity forces the semi-fluid substance into the melter where the matter assumes a fully liquid state. At this stage, all mineral and metal pollutants that could not be cleared out before settle at the bottom of the apparatus and the process of thermal disintegration starts. Some of the sediments on their way to the cracking furnace will be extracted at this point. The actual process of depolymerization occurs in the cracking furnace. A small amount of liberated gases are being used in the counter-current as a heating fuel in the heater, melter and cracking furnace. At the bottom of the cracking furnace, the char will accumulate together with some mineral matter. The primary product, the reclaim, will accumulate in the reservoir next to the depolymerizator. Part of the reclaim (gas) will be used by the set of burners in the heater, melter and the cracking furnace.

The continuously heated depolymerizator reservoir plays the role of a buffer to be used later when the need arises. Any surplus of the buffer stock can be stored in a solid state (as an alternative fuel) or in the form of a liquid paraffin.

Installation is designed for continuous operation and will have an efficiency of around 200 kg/h. The char will be accounted for approximately one third of total final production. The remaining portion will be considered as a regenerate.

4. Products

The primary product, either in a solid form or as a liquid paraffin, may be used in two ways. The solid state serves as an alternative fuel and has superior characteristics if compared to conventional heating fuels such as heating oil.

The basic characteristics of the reclaimed fuel are:

- high calorific value ca. 42–47 MJ/kg,
- self-ignition elimination,
- mechanical stability,
- dimensional stability,
- diminished humidity absorptivity.

The reclaimed fuel can be burned in all functioning fluid boilers. High energy output (above 90%), and the possibility of cinder reimplementation [4], both play a positive role in environmental conservation efforts. High-heat output plants and high-power generating plants use the reclaimed fuels. Paraffin as a by-product represents a highly desired commodity that, after some further processing, is widely used in the cosmetic, medicine and pharmaceutical industries.

The boiling and self-ignition temperature of paraffin is above 300°C. Liquid paraffin ignition temperature equals 175°C and the solid state paraffin ignites above 220°C. Paraffin doesn't need any specially marked transportation equipment as it is not the subject of any traffic regulations.

The second by-product, char, maintains a solid state with optimum parameters when technological and environmental conservation aspects are taken into consideration. Most important is the high caloric output ca. 20 MJ/kg and an additional positive characteristic is the lack of NOx, SO_2 and HCI.

5. Conclusions

The proposed installation enables the production of high-energy alternative fuel. The installation makes it possible to utilize polluted scrap plastics by the regeneration of two by-products – the char and the reclaim.

The reclaimed fuel is used in two ways; as an alternative fuel and as a paraffin fraction for further processing. The burning process of the reclaimed paraffin is a clean burning process as the paraffins are comprised of oxygen, hydrogen and carbon.

Polyolefin, being an end product of the depolymerization, is free from any nitrogen and sulfur, therefore, during the incineration process, a minimal emission of NOx and SO_2 occurs. Except for natural gas, (needing purification from any remnants of the crude oil or any solid pollutants) there aren't any cleaner hydrocarbon fuels on the market. The paraffin must be melted before it can be burned. Unlike heating oil, paraffin may be available locally from the waste segregation facility. On a small scale, it represents a good alternative to the costly importation of any liquid fuels.

After further processing, the paraffin fraction can be used in the production of paraffin, candles, torches, waxes etc.

It is imperative that we underline an important fact that the reclaimed fuels, either as an alternative source of fuel or as paraffin, are not classified as dangerous goods. There are no danger codes: NDS (chemical compound, highest permitted toxic concentration); NDSCH (chemical compound, highest momentarily permitted toxic concentration level); NDSP (chemical compound, highest permitted toxic threshold concentration) established for the above chemicals.

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