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History of flood protection structures in Slovakia

Andrej Šoltész, Michaela Červeňanská, Dana Baroková

***Abstract* – The contribution deals with historical aspects of the protective and drainage structures in Slovakia. It describes the first attempts for an organised flood protection on our territory and claims Samuel Mikovíny to be one of the pioneers of technical progress and education. The construction of protective dams and drainage systems are emphasised as well as the development of dike structures on the Danube River. At the end the water associations against floods and internal waters (mostly in the Danubian lowland) are pointed out. They have done a great amount of useful work continuously provided complex solutions and, basically, were the longest operating organisations in the array of water management organisations in Slovakia.**

***Keywords* – dikes, flood protection in Slovakia, protective and drainage structures, Samuel Mikovíny.**

1. HISTORY

Historically, territory of Slovakia, in lowlands of the Danube, Váh, Tisa and Bodrog river basins, has been exposed to repeated ravages when these rivers flooded [1]. The floods endangered lives and properties and resulted in the water-logging of farmland, which created marshes and had other undesirable effects. All of this led the inhabitants of these areas starting in the 13th century to take unusual countermeasures, according to historical records. They built dikes – embankments to protect their homes. The development of flood protection of an area together with the construction of dikes can be illustrated by the example of the Rye Island region [2].

The Danube River stream has been freely divided into many branches, especially in the area of current river bed and the river bed of Small Danube River. There were also branches in the area that, during floods, created a broad network of streams throughout the island. Even the river beds of the larger streams were not yet

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constant; they were formed and changed by the water with the active help of erosive and sedimentary processes. Some branches were filled with fluvial deposits, and others were formed anew. The ample extent of these processes also endangered the settlements.

The oldest historical document concerning the protection of Rye Island is an open order issued by Kind Zigmund of Luxembourg to provide Rye Island, especially near the town of Šamorín, with a free workforce for construction and maintenance of protective dikes. This way of arranging for the construction of dikes was retained in future centuries, as proven by a 1569 law [5], [6].

Historians also assume unusual countermeasures had been in the 13th century previously taken. For example, a German record mentions Rye Island (in German, "Schütt Insel"), which became suitable for permanent settlement after the construction of dikes. Based on record it is assumed that the construction of the dikes had already started in the 13th century. But the agriculture on Rye Island still suffered from the consequences of flooding. The floods impaired the compactness of the dikes, overflowed them or even washed them completely away. Written records report, for example, heavy floods on Rye Island in 1568 and 1569 and the chronicles of the County of Bratislava mention 21 floods on Rye Island between 1673 and 1818. It is no wonder that in the 17th century, historians still considered the region of Rye Island as a marshy ground to the extent of two thirds of it, admitting only the western part to be drier [6].

The first attempts of organised protection started in the 17th century. The administration of the structures was within the jurisdiction of the counties which were obliged to carry out flood protection. The counties therefore had specialised technical authorities. The County of Bratislava had already elected a dike director (director aggarum) in 1616 and decreed yearly repairs always by the St. Michael's Day (i.e., September 29) in 1731.

Among the experts engaged in the protection of Danubian Lowland against floods was the noted scientist and engineer Samuel Mikovíni (1686? – 1750). He considered the main cause of the floods to be the numerous meanders and branches that should be removed by tapping. His "Mappa Comitanus Posoniensis" of the upper part of the Danube and the Small Danube from the beginning of the 18th century probably served for this purpose (**Fig. 1**).

We can unambiguously claim Samuel Mikovíni to be not only a pioneer of scientific mapping in Hungary, but also of technical progress and education. It is worth noting, that his activities in cartography became a good basis for the preparation of the regulation of water regimes not only in the case of the water reservoirs in the area of Banská Štiavnica (more detailed in [6]) but also in the cases of the Danube River below Bratislava, Budapest and other places in Hungary. As an example, records from 1727 note his participation in the regulation of the water regime in the surroundings of the town of Tata in northern Hungary on the property of the Esterházy family.

One of his accomplishments that is held in high esteem is the establishment of the Civil School of Technology as a part of the School of Mining in Banská Štiavnica (the future Academy of Mining).

Fig. 1 A historical map showing the part of the Danube River below Bratislava

The Civil School of Technology provided its graduates with a technical education not only in the basic specialisation of mining, but also in the fields of mechanics, hydraulics and hydrology as well as water management and water management structures. The Academy of Mining (1770) constituted great contribution to the training of mining engineers as well as engineers in general in that its individual departments were also aimed at the education needed for posts as civil, water management and surveying engineers.

Other schools established in Hungary contributed to the education of water management engineers, as well. For example, Empress Maria Theresa established the “Collegium Oeconomicum” in Senec in 1763, which specialised in economics and the technical sciences. An “Institutum Geometrico-Hydrotechnicum” was established in Hungary in 1782. After the further development of this Institute, the establishment of the Technological College (1846) and their merger (1850), the University of Technology in Budapest was officially established in 1857. Many of our engineers graduated from this university, e.g., the future academician Prof. Ing. Štefan Bella in 1902.

The most important role in the education of water management engineers in Slovakia since 1938 has been the Slovak Technical University, now called the Slovak University of Technology in Bratislava, especially the Faculty of Civil Engineering.

This faculty has contributed to the research, design and construction of water management structures and graduated thousands of engineers.

Unfortunately, the names of water management experts and “engineers” until the middle of the 19th century remain unknown, because the official administrators were various state plenipotentiaries, vice-heads of counties, great landowners, etc.

The development and implementation of engineering in the field of flood protection dates back to the middle of the 19th century. Progress was achieved with the establishment of various types of associations and unions of owners of endangered properties and villages that were established, e.g., in 1854 with jurisdiction over Rye Island. The activities of these associations were more intensive during high water on the rivers, cracks in or breakdowns of the dikes and flooding of the land by internal water. On the other hand, after a series of several dry years, these associations broke up.

The final legislation concerning the organisation and operation of the associations was contained in Act XXIII (1885), the so called water law, and its subsequent amendment, Act XVIII (1913). When one considers the era in which it was created, the law can be evaluated as relatively flawless, despite its complexity.
The law divides the associations into two groups according to their purpose:

a) associations to protect against floods and internal waters (flood control association),
b) associations for the utilisation of water for drainage, irrigation and other purposes (land reclamation associations).

The main goal of the first group of associations is flood protection and the draining of internal water from protected areas. The law thus devolved the management of flood protection and the drainage, maintenance and operation of constructed protective and drainage structures appliances upon the water associations, and granted them autonomy.

The contents of the 1885 law were basically retained for decades and was just amended and strengthened until 1955. On January 1, 1955, according to Law No. 11/1955 concerning water management, the water associations were dissolved, and their property and activities were taken over by the Administration of Water Bodies and Land Reclamation. After many organisational changes, the present managers of the water bodies, the structures belonging to them, protective dikes and drainage systems are companies which are divided according to the particular river basins – the River Basin of the Danube, the River Basin of the Váh, the River Basins of the Bodrog and Hornád and the River Basin of the Hron. The managing directorate of the Slovak Water Management Company in Banská Štiavnica supervises all of these companies [6].

2. CONSTRUCTION OF THE PROTECTION DIKES

We illustrate the development of the development of protective dikes again using the example of Rye Island. As stated above, until the 19th century, construction had been carried out depending on the local conditions, without any regard to the possible resources of the landowners in a particular county. As a result, the dikes were

not continuous and adjoined each other. Records from the 18th century mention connecting two separate sections of a dike as a great success.

The geodetic surveying of the Danube, which should have provided a basis for the construction of dikes, started in 1823. Based on the results, the County of Komárno proceeded to a more systematic construction of dikes on the lower part of Rye Island in 1826 and the overall length of the dikes thus grew from 34.6 km to 70 km between rkm 1830 – 1844. However, this protection was far from flawless, and many parts of the dikes were even destroyed during floods (e.g., in 1850 and 1853). It is recorded that the crest of the dike in the County of Bratislava was only 0.5 – 2.0 m wide in the first half of the 19th century. The systematic construction of protective dikes started only in the second half of the 19th century, after the establishment of two water associations to protect against floods and internal waters. To economise and to speed up the construction, older local dikes were used as a part of a continuous protection line, and additional dikes were constructed along the network of active branches of the Danube. Thus there arose an irregular protection line with curves. The width of the space between the dikes varied from 1.1 km to 4-5 km, because the definite course of the main waterbed had not yet been stabilised at that time. The regulation of the Danube from the Devin castle down to Gönyü was implemented between rkm 1886 – 1896. The object of this regulation was to concentrate the small and middle sized flows into a single channel deep enough for shipping. This channel was built by damming the adjacent branches by longitudinal and directional dikes and short regulation of the water flow. It was designed for a mean rate of 2,890 $m^3.s^{-1}$ discharge.

As mentioned above, the protective dikes on the Danube were first built relatively low and weak and were reinforced and elevated mostly only after previous disasters. This was also the case of the flood and dam breakdown in 1899, after which the dam was heightened 1.0 m over the level of culmination and reinforced [4]. It was also likewise after the dam breakdown and floods in 1954 (Ásványráró) and 1965 (Číčov), when additional raising occurred and the body of the dike was reinforced (**Fig. 2**). The raising of the water level during peak flows enforced the raising of the dikes. That was a consequence of the raising of the waterbed caused by the deposit of floating debris in a slope-breaking section of the Danube. An increase in the water level causes an increase in seepage through the subsoil and dikes and thus endangers the stability of the dikes and causes the rising of the ground water level in the protected area. These old dikes were earth dams poured from surface soils (sandy soil and soil) extracted from borrow pits on both sides of the dikes.

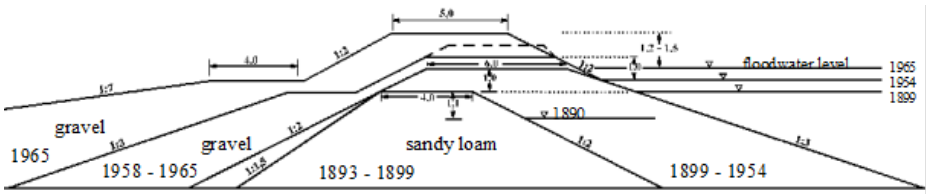

Fig. 2 Development of the dike structure on the Danube river

Countermeasures against seepage – especially seepage of the subsoil – on the Danube River were taken after 1965. As an example, we can mention the construction of counter-seepage structures on the left-hand dike below the Gabčíkovo waterworks between Sap and Číčov, which consists of an approximately 12 m deep underground sealing wall in the heel of the dike and sealing foil on the water slope (scheme, **Fig.3**), [3].

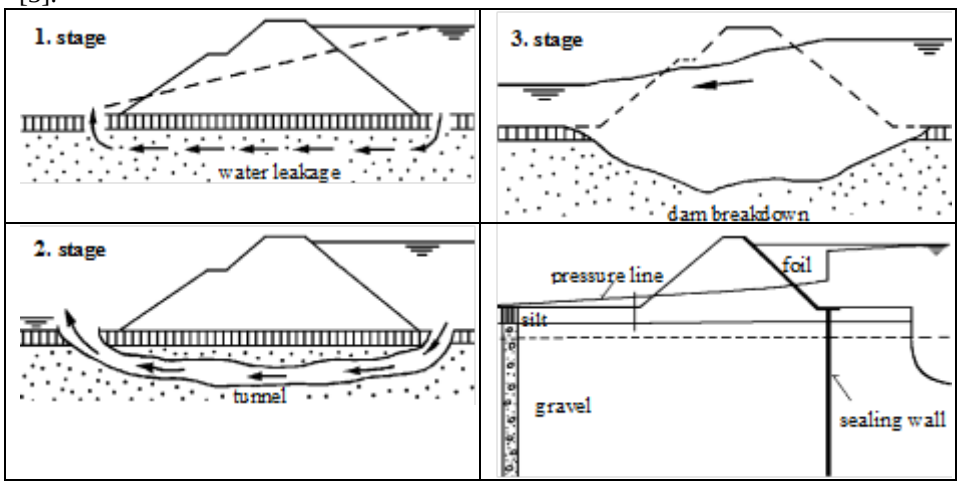

Fig. 3 Development of the dike structure on the Danube River

3. ASSOCIATIONS AGAINST FLOODS AND INTERNAL WATERS

According to the Water Law (1885), the water associations are responsible for the flood protection of their area and for drainage of internal waters. The scope of this paper does not allow us to discuss all their activities and work, which we highly appreciate, in detail. That is why we only mention briefly their achievements until their dissolution in 1955. They are related directly to the Danube River.

1) An association to protect against floods of the upper part of Rye Island with an area of interest of 50,252 hectares and 130 km of protective dikes was established in 1887 and had its seat in Šamorín. The activity of the association was, according to its location and its name, concentrated on flood protection. The recorded length of the drainage canals was 71 km and was connected to the region of Šarrét, where the Šarrét Water Association operated.

2) An association to protect against floods and internal waters on the lower part of Rye Island and Medzičilízie was established in Komárno in 1876. The area of interest covered 87,360 hectares; 116 km of protective dikes were built together with a 605 km-long canal drainage system (140 km of main and 465 km of auxiliary canals). The canal network was divided into individual drainage systems, the structure of which has been retained, more or less, until the present day. The association built 7 pumping stations with an overall capacity of 36.75 m3.s^{-1}. Similarly to other water associations, its first pumping stations were driven by steam engines (the oldest one; Číčov (1896), had a capacity of 0.2 m^3.s^{-1}), and later by diesel engines (Čergov

(1937), with a capacity of 9 $m^3.s^{-1}$; Asód (1943), with a capacity of 18 $m^3.s^{-1}$) and, after the dissolution of the association by electricity (e.g., by the association-designed Komárno pumping station (1959), with a capacity of 20 $m^3.s^{-1}$). It is interesting to note that at the Čergov pumping station, one of the pumps was also adjusted for reverse pumping from the river into the canal network for irrigation purposes.

3) An association to protect against floods and internal waters on the left-hand side of the Danube with a seat in Nové Zámky was established in 1847 (it has been active only since 1883). The area of interest was 48,000 hectares; the length of the protective dikes on the Danube, Váh, Nitra and Žitava rivers was 148 km with other dikes built along the peripheral canal and smaller tributaries at a length of 90 km. The overall length of the canals was 520 km, according to some records. The association operated 5 pumping stations with an overall capacity of 16.5 $m^3.s^{-1}$. The most powerful was the Komoča pumping station (1935), with a capacity 8 $m^3.s^{-1}$. Unfortunately, a dam breakdown near this pumping station was one of the stimuli to change the organisation of water management, which resulted in dissolution of the water associations in 1955 [6].

4) An association to protect against floods and the internal waters in Ebedfod with a seat in Karva was established in 1896. It provided protection against floods on an area bordered by the Danube and the higher hilly terrain in the north. The area of interest was 4,551 hectares, the length of the protective dikes 20,150 km and the length of the canals 41 km. The pumping was conducted by embedded pumps only (0.2 and 0.5 $m^3.s^{-1}$).

Apart from these large associations to protect against floods and internal waters, smaller ones were also established, which were orientated mostly towards the lining of smaller water flows and drainage. Some of them were categorised into the type of water associations concerned with water utilisation (land reclamation associations) before 1918.

4. CONCLUSIONS

In this paper, for the sake of briefness, we have concentrated on the construction of protective dikes on the Danube River only. Of course, similar structures were also built on other rivers in Slovakia.

The floods in the past, together with the recent ones (2002, 2006, 2010 or 2013 on the Danube) prove that flood protection is still a current topic and can never be deemed finished. Besides the existing structures, new ways of securing the retention volume (especially detention reservoirs – lateral, and valley) are sought to provide satisfactory protection especially to inhabited areas along the water flows. Data from precise hydrological forecasts of peak flows also contribute to flood protection, together with considering proper evaluations of the effect of the retention volume between dikes on flattening a flood wave.

We have tried briefly to describe the creation, work and achievements of the water associations providing flood protection in Slovakia until 1955. The organisational pattern always copied changes in the state administration as well as political changes (Hungary, Czechoslovakia, Slovakia). It is necessary to appreciate

that the water associations provided the construction, management, operation and maintenance of the protective and drainage structures in a significant part of southern Slovakia, both in the east and in the west, for decades. They have done a great amount of useful work, continuously provided complex solutions and, basically, were the longest operating organisations in the array of water management organisations in Slovakia.

5. ACKNOWLEDGMENTS

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Evaluation of Water Management in Regions Affected By Water Structure Construction and Operation

M. Červeňanská, A. Šoltész, D. Baroková, and A. Janík

***Abstract* – Consequently to the construction and operation at water power plant Gabčíkovo came to changes in water regime in adjacent area. The power canal of Gabčíkovo power plant cut the upper parts of the drainage system in Danube Lowlands. Groundwater level changes have occurred in the vicinity of the reservoir what caused changes in discharge and water level regime of the drainage channels.**

Presented contribution deals with theoretical background of the water management in agriculturally exploited regions with possibilities of solution by means of improved operation on hydraulic structures and by construction of new structures.

***Keywords* – Danube Lowlands, channel system, groundwater level regime, hydrologic and hydraulic assessment.**

1. INTRODUCTION

Hydrologic regime of the Danube River before the damming influenced directly the hydrologic regime in river branches as well as the ground water regime in the floodplain area. In pre-dam conditions the increase of water level in the Danube River bed caused a synchronous increase of water level in the branch system. At discharges higher than 4000 $m^3.s^{-1}$ in the river bed came to a direct fulfilling of the branch system with water and the branches created together with the Danube a complicated hydraulic

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surface water system. Most important for surviving of floodplain forest and for improvement of hydrologic conditions in this region is a favourable ground water regime. The only data concerning the measured ground water levels before damming were available from the time period 1987-1992 [5]. From these data is apparent that during the vegetation period in pre-dam conditions (before 1992) the ground water table was in cover layer representing the root zone of the floodplain forest and never decreased into gravel layer wherefrom no capillary rise is available [1]. The second very important fact was that the fluctuation of ground water table reached the value 2.5–3.0 m [4]. The lowest ground water levels were in winter and the highest were in summer according to the fluctuations of water levels in the Danube River. After damming the Danube River the discharges became artificial and due to the power canal scheme of Gabčíkovo power plant they became lower, as well. The consequence of this hydrologic situation was the decrease of ground water level in the adjacent area of the Danube. To improve and to control the ground water regime in the floodplain area an inlet structure situated on the upstream power canal was constructed. This structure allows an artificial water supply into the branch system and consequently into the ground water. It is dimensioned up to discharge 234 $m^3.s^{-1}$. This quantity would serve not only for water supply into the branch system and for ground water control but for securing of regulated floods in the floodplain region, as well [3].

In presented contribution the problem of affecting the water regime due to construction and operation of Gabčíkovo power plant has been assessed from viewpoint of irrigation water needs in agricultural region of Danube Lowlands. We have focused on the upper part of Rye Island which is mostly influenced by waterworks operation. Total area of the attached territory is approximately 15 000 hectares. Consequently to the construction and Gabčíkovo power plant operation came in this region to changes in groundwater regime as well as in water level regime of drainage channels. The power canal of Gabčíkovo power plant has cut some drainage channels and another solution of their existence had to be found. This solution consists of water supply structures on left hand-side seepage channel along the power canal of Gabčíkovo power plant. This seepage channel is supplied by seeped water from the Hrušov reservoir. Projected quantities of seeped water should be after reservoir clogging more than 5.0 $m^3.s^{-1}$. The actual reality is that there is less than 2 $m^3.s^{-1}$. There are several drainage channels in this region mainly flowing in the direction from the Danube River to the north-east, some of them are more than 20 km long (**Fig. 1**).

One of the main factors for soil fertility is the optimum soil moisture in the root zone of the soil profile. This can be improved by technical measures according to level and discharge regime control in open channels in the region. The Rye Island belongs to regions, where agriculture production in vegetation period due to irregular precipitation distribution suffers very often by droughts. The probability of ensuring at least 10 mm rainfall in one decade of the vegetation period is less than 50 % and probability of 30 mm rainfall is less than 20 %. From this follows that without irrigation it is impossible to supply the soil profile for water uptake by agricultural plants. It can be expressed by the ratio of potential evapotranspiration and precipitation at given meteorological conditions for the Gabčíkovo gauging station (**Fig. 2**).

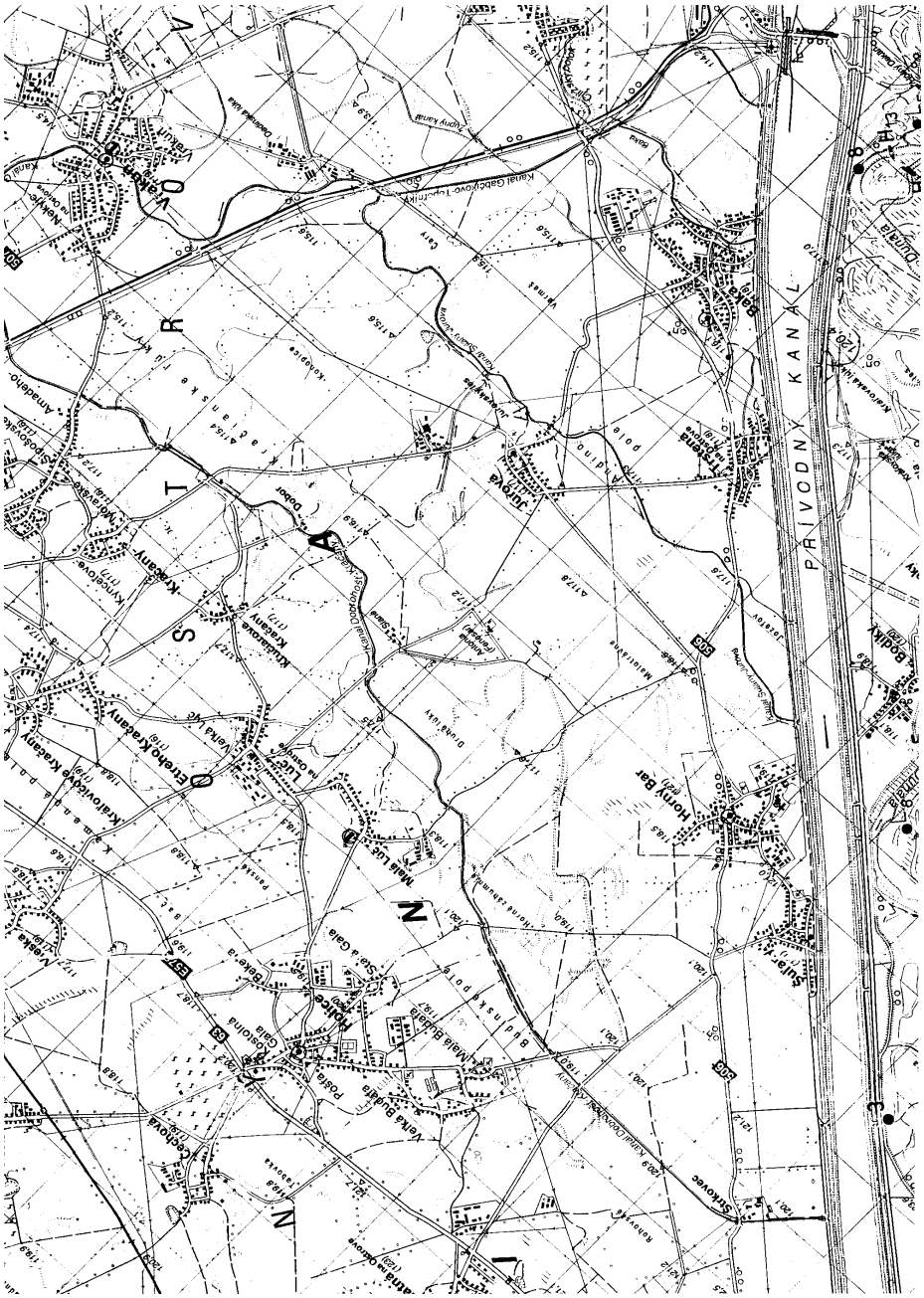

Fig. 1 The situation of the investigated region – upper Rye Island

2. GROUNDWATER REGIME

The groundwater in the Rye Island region is mostly in high permeable sediments formed by gravel and sandy gravel. These are supplied from three basic sources:

- river bank infiltration from the Danube River as well as from Hrušov reservoir,
- precipitation infiltration and
- ground water recharge from higher situated Small Carpathians.

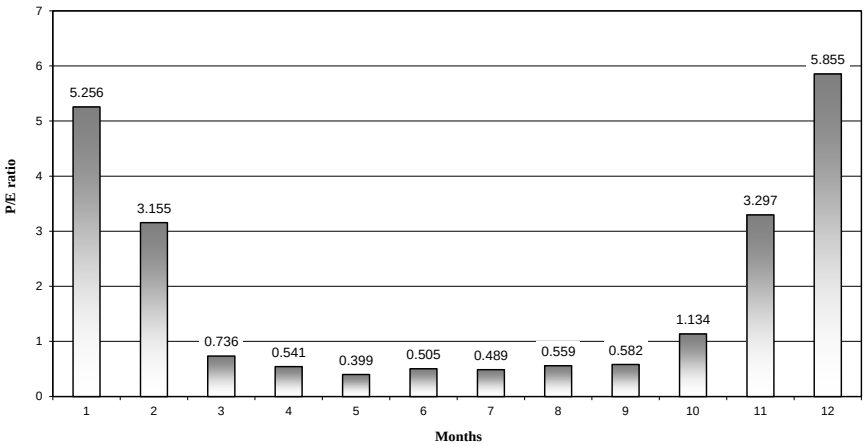

Fig. 2 Ratio of mean precipitation and evapotranspiration values in vegetation period for years 1955-2004

Each of these sources represents a certain groundwater regime. From hydraulic point of view it can be characterised as non-steady groundwater flow. An important indicator of soil water saturation is the depth of groundwater table. The most favourable groundwater levels in the upper part of the Rye Island occurred 2 – 2.5 m under the surface. It means, according to [1] that at prevailing thickness of cover layers up to 2 m the groundwater cannot reach this layer.

After construction of Gabčíkovo power plant the groundwater level regime in the investigated region has changed. From observed monitoring data (**Fig. 3**) it is apparent that in first six years of operation (1992-1998) came to groundwater increase (15-50 cm) depending on the distance from Hrušov reservoir. The graphical illustration constructed upon the measured groundwater level data has shown in this period the main direction of groundwater flow from the Hrušov reservoir. After this period the reservoir has been clogged (1998-2000) and the groundwater level regime started to get a decreasing trend (**Fig. 4**). Due to this unfavourable situation we have tried to find a solution to improve the groundwater as well as soil moisture regime in this region and have stated possible measures at several levels of decision making and with different effect. Results of research provided in this study field have shown that

there are possibilities of groundwater and soil water control by water level and discharge regime in the drainage channel system of the upper part of Rye Island.

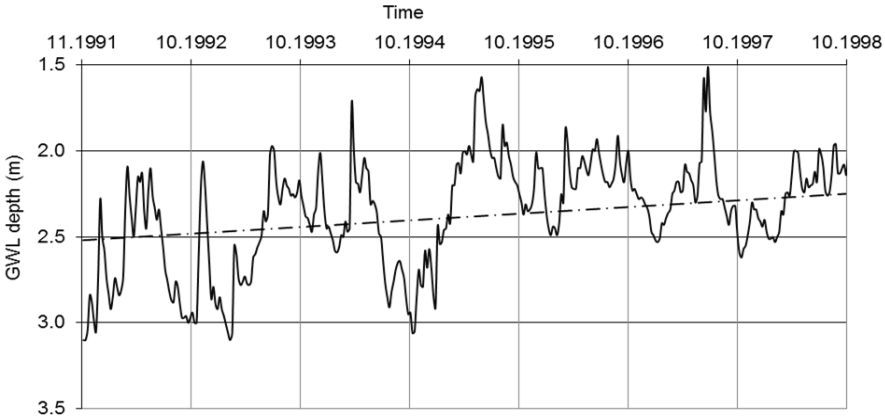

Fig. 3 Chronological development of groundwater level in Baka borehole in years 1992-1998

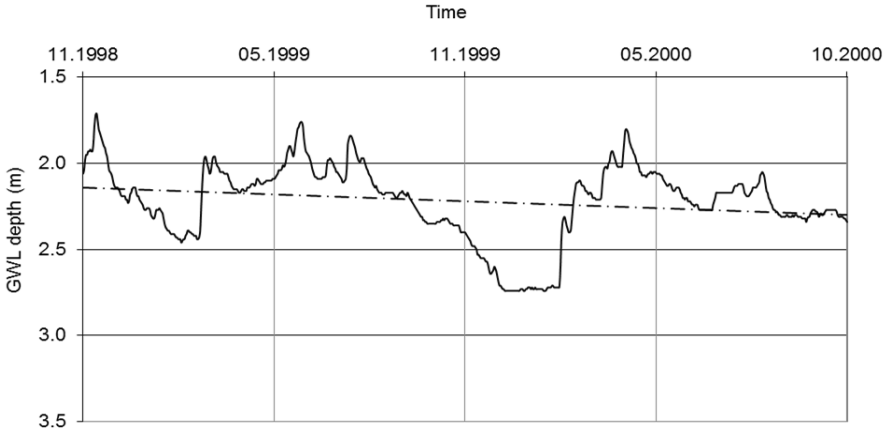

Fig. 4 Chronological development of groundwater level in Baka borehole in years 1998-2000

3. METHODOLOGICAL PROCEDURE AND POSSIBLE TECHNICAL MEASURES

As it was mentioned above, in the investigated region there is lot of drainage channels which can be artificially supplied with seepage water from seepage channel of upstream canal of Gabčíkovo power plant. The drainage channel system in Danube Lowlands is multipurpose. Its components are channels, gates and pumping stations

with their accessories. Each of these components can serve just one purpose in limited area or more purposes in cooperating areas. The structure of all units has optimally to contribute to overall water management and to agricultural requirements of the region. The conception of drainage is adequately followed by conception of irrigation. It is given by the position of the warmest agriculturally exploited region of Slovakia with the longest vegetation period and with richest groundwater sources.

Principally, there are four possibilities for groundwater regime improvement:

- to decrease the drainage effect of the Danube River by means of construction of underwater structures,
- to increase water uptakes (mainly before and in vegetation period) artificially into the river branch system,
- to increase seepage from the Hrušov reservoir and
- to decrease the drainage effect of the channel system in the region.

These solutions, as it is apparent for everybody, have consequences in influencing the groundwater regime in different extent. The largest influence would be obtained by the first measure. The problem is that for these measures needs the Slovak part agreement from the Hungarian side (state border river). The second possibility would very strongly influence water levels in the seepage channels as well as the groundwater regime. The smallest effect on groundwater regime will occur providing the last mentioned possibility. But it is the easiest way to control the groundwater regime and it depends only on well organised water management in the region [6], [7]. We have focused our research in this region on this problem and have suggested some operation rules.

Our first suggestion was to fulfil drainage channels in this region as soon as possible after the winter period so that they can be pre-filled for irrigation purposes. In a very detailed “in situ” investigation we have tried to measure at different discharges and water levels in the drainage channel the extent of its influence on groundwater regime [5]. Unfortunately, the effect of operation on controlling gates (increase or decrease of water level) have shown not more than 50 m distance from the channel which is consequently effected. But measuring discharges downstream the channel it was shown that the drainage effect can be decreased by means of good operation up to 45 %. It means in digits, that in beginning of the drainage channels there are usually about 25 $l.s^{-1}$ flowing from seepage channel and without any operation on gates and the discharge flowing out from the drainage channel is about 250 $l.s^{-1}$ leaving the channel. After handling the gates before and during the vegetation period (March-October) the amount at the end of the channel decreased to 150 $l.s^{-1}$. The longitudinal profile of one of drainage channels with constructed gates is illustrated in **Fig. 5**.

Next question which had to be answered was when the gates should be closed after the winter period. This problem was solved with respect to meteorological conditions, especially temperature. Taking into account the average and minimum daily temperature we considered a thirty-year series for evaluating the starting of operation. The criterion for it was a 5-day series with mean daily temperature more than 5 °C. Calculating this assumption we have suggested the first or second decade of March. Next calculations have shown that g this operation will succeed to fulfil the

drainage channels in the second half of May what is favourable time for starting the irrigation season.

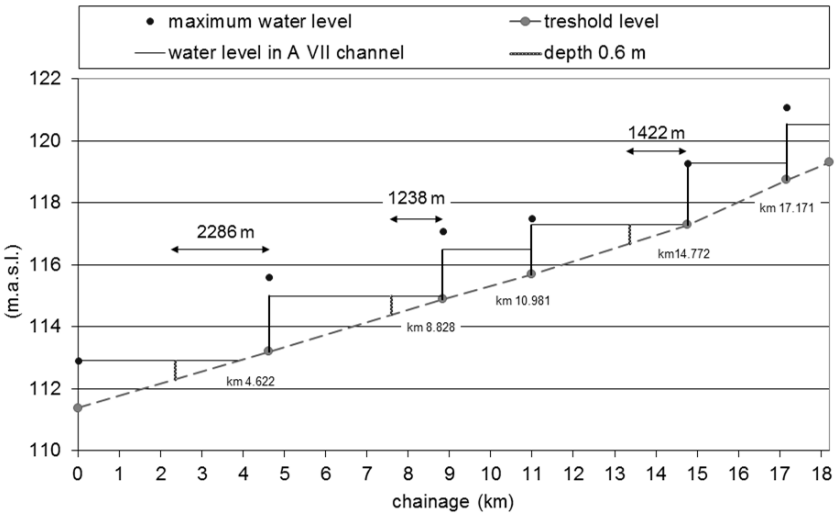

Fig. 5 Longitudinal profile of A VII drainage channel with constructed gates

4. CONCLUSIONS

Gabčíkovo power plant is one of the largest water management projects in Europe. Its construction and operation have changed the groundwater regime in the adjacent area, as well. Presented paper has tried to show very briefly the water management in the agriculturally exploited region of the Danube Lowlands – Rye Island, especially the upper part of it. In last fifty years a dense system of drainage channels has been constructed in this region. Nowadays, in period of climate changes, the second – irrigation - function of these channels is appearing more and more. It gives a possibility of subsurface groundwater supply as well as the opportunity for a surface irrigation from the directly from the channel.

Other possibilities of influencing the groundwater regime have been investigated in the past. All of them are very important, maybe more important that an artificial supply of drainage channels, but there are much more expensive and their realisation take a lot of time.

5. ACKNOWLEDGMENTS

The contribution was developed in frame and on the basis of the financial support of the APVV-14-0735 project „New possibilities of use of drainage channels taking the biodiversity in a landscape into account”.

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Mitigation of coastal erosion by beach nourishment of Romanian Black Sea Shore

Mari-Isabella Stan, Dragoş-Florian Vintilă

***Abstract* – Coastal erosion affects natural habitats and human settlements, destroys economic activities and threatens human life. For the reduction of coastal erosion in the southern part of the Romanian Black Sea shoreline, engineering solutions for construction and rehabilitation of coastal erosion protection system through specific hydro-technical works were used: emersed detached breakwaters, submerged breakwaters and groynes. However, the most effective form of marine protection is beach nourishment. This contributes to the increasing of human activities in the Romanian Black Sea shoreline due to the benefits brought (tourism, economic and social activities, properties, recreation) to achieve the ecological, economic, and social objectives of the area.**

***Keywords* – erosion, coastal zone, beach nourishment, marine spatial planning, Black Sea.**

1. INTRODUCTION

Beaches consist of accumulated, unconsolidated sediments from offshore reefs and shoals transported to shore by wave-generated motion. Beaches are not stable entities but are rather dynamic landforms constantly subjected to forces that promote accretion and/or erosion [4].

The coastal erosion can be defined as the removal of material from the coast due to wave action, tidal currents and/or human activities, usually causing the landward retreat of the coastline [5]. The largest erosion along the coastal zone is caused by waves which moves towards the shore, especially during the storms.

Coastal erosion affects natural habitats and human settlements, destroys economic activities and threatens human life, so it has a negative impact on the environment [6].

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Romania has a coastal zone of about 240 km along the north-western part of the Black Sea. In recent decades, the Romanian Black Sea coast has been affected by serious erosion problems which is why according to the strategy for coastal zone management - The Master Plan "Protection and rehabilitation of the coastal zone"- the "Coastal zone protection and rehabilitation" project was implemented [8], [10]. Within this project for the rehabilitation of the southern part of the Romanian Black Sea shoreline in addition to protection engineering solutions for "hard" works, "soft" works such as planning and consolidating by sanding beaches have been completed [6].

Beach nourishment is a soft coastal protection solution that involves adding artificial sediment of a quality which is suitable for a beach area, which has a deficit of sediment. Sediments brought from the outside of the sedimentation cell, either offshore or from quarries or riverbeds are placed on the beach. It is very important to select the appropriate sediment mineralogy and grain size for the location of the project, which should normally be coarser or similar to natural material on site [3], [6], [9].

This solution has the advantage of restoring "natural" beaches which is the most effective form of marine protection whereas it has the natural ability to adapt to changing of waves conditions and dissipating wave energy [11].

2. THE DESIGN AND THE EXECUTION TECHNOLOGY OF THE BEACH

In *designing the beach* located in the southern part of the Romanian coastline it was taken into account the positive effects of the construction and rehabilitation of coastal erosion protection system by the following specific hydro-technical works: emersed detached breakwaters, submerged breakwaters and groynes.

The orientation of the Constanta beach area and the beaches adjacent to Constanta city was established as a result of numerical simulations. **Figure 1** presents the plan view of the beach orientation where the red line is represented by future contour of the beach and the black line is the ground contours [13].

Fig. 1 Plan view of the beach orientation

Comparing the two lines it is observed that the width of the beach is the most critical in the northern and the central parts of the beach where its width is 100 m. More to the south, along the beach, the width of the beach gradually increases.

A cross-sectional view along the black line perpendicular to the beach from **Fig.1** is presented in **Fig. 2** by the red line.

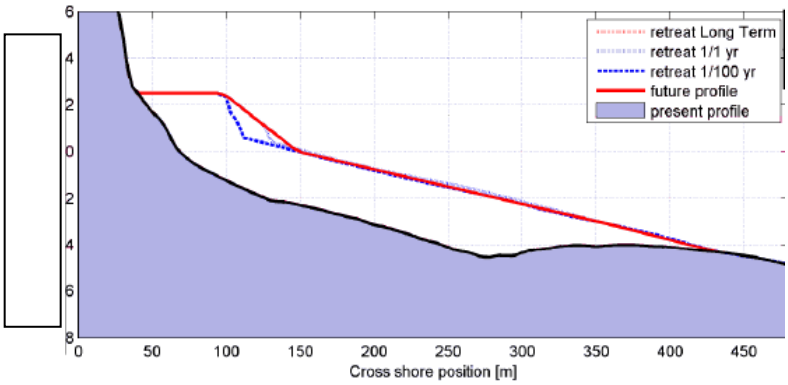

Fig. 2 Cross section to the existing configuration and design of the beach

Figure 2 presents the existing configuration and withdrawal of the beach, the beach ridge located at the maximum designed MN75+0 m in order to respect the design requirements regarding the maximum withdrawal allowed under severe conditions. The existing upper slope of the beach is 1/20 m linking the beach ridge with the MN75+0 m, the slope of the avant beach was determined to 1/60 m. The beach profile will move by 4.2 m to shore in 50 years. The beach profile is constructed as close as possible to the equilibrium profile but it undergoes natural variations.

After designing the profile beach, the beach nourishment involving the addition of sediments along the eroded shores to restore and widen beaches follows. The sand used for sanding must have dimensions, colour and texture comparable to the existing material on the beach.

Therefore, the project of rehabilitation the Constanta beach area and the beaches adjacent to Constanta city which had to be sanding, a search campaign was conducted to find the best sand. The beach nourishment was accomplished with sand brought from the Black Sea, which was found in two areas of extraction in the vicinity of Constanta city. Perimeter of the loan to dredging the deposits of sand for relocation is located in Romanian territorial waters on the continental shelf, in two areas (**Fig. 3**): north zone with an area of 2.26 km^2 and the south zone with an area of 2.84 km^2.

The water depth in the area chosen for the loan is approximately 27 m [13].

Fig. 3 Location of areas of the dredging in the Black Sea [13]

The execution technology consists in the drainage of sand. Before the start of dredging a pipeline will be installed which will be used to pumping the sand in the sanding area. This will have a floating section and one which will be ballasted during the period of execution. 12 m pipe sections are welded thus forming the pipe section future ballasted. The pipeline will be closed with valves, pulled into the water and transported into place. The ballasted pipeline will be connected to the floating pipeline using a ship and installed into place. The landward end of the ballasted pipeline will be connected by pipelines to shore which can be positioned in various locations on the beach.

The works of dredging of the material from the loan area is executed with *TSHD* technology - *Trailing Suction Hopper Dredging* (absorbent repressed self-propelled dredging). *TSHD* loaded with the corresponding material will sail to the outer end of the pipeline floating where it will be connected to the coupling point on *TSHD* and start pumping the material through floating pipeline, the ballasted pipeline and pipeline from the shore. Thus, the mixture of water and sand is pumped on the beach in established locations. When the area in front onshore pipeline will be filled with material, this will be pushed and leveled with bulldozers (**Fig. 4**). After downloading, *TSHD* will sail back to the loan area from where it will resume the charging-download cycle [13].

Fig. 4 Beach nourishment [8]

3. CASE STUDY: BEACH NOURISHMENT OF THE SOUTHERN PART OF THE ROMANIAN BLACK SEA SHORELINE

Currently Romania in partnership with Bulgaria is developing the project of the cross-border for maritime spatial planning for the Black Sea. One of the specific indicators for analysis and evaluation is coastal erosion/accumulation as well as the trends of beaches and cliffs. For the development of economic and social activities in the marine and coastal space of the Black Sea it is necessary to know the manner in which the Romanian shoreline evolves.

The project "Coastal zone protection and rehabilitation" by refurbishments and sanding works of the coastal cells from the southern part of the Romanian Black Sea shoreline restores to needed protection degree of the area and the certainty of the social stability for at least 50 years, so [8]:

- **South Mamaia Zone**

For this area the JICA study (2007) highlighted, on the basis of samples taken at depths between 1 and 3 m, that sand of beaches has an average grain size of 0.17 mm [12].

Beach nourishment was performed on an alongshore distance of 1.2 km and an amount of 353,529.47 mc of sand (**Fig. 5**).

Fig. 5 Beach nourishment – South Mamaia Zone [8]

- **Tomis North Zone**

Beach nourishment was performed on an alongshore distance of 1.05 km and an amount of 696,480 mc of sand (**Fig. 6**).

Fig. 6 Beach nourishment – Tomis North Zone [8]

- **Tomis Centre Zone**

Beach nourishment was performed on an alongshore distance of 0.85 km and an amount of 605,798 mc of sand (**Fig. 7**).

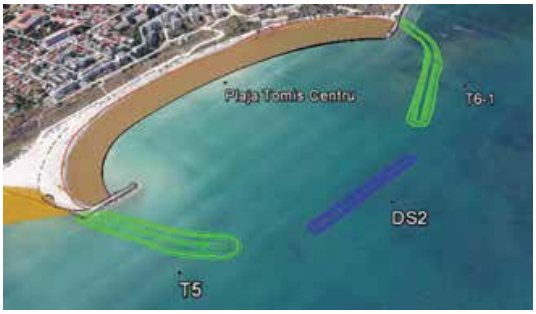

Fig. 7 Beach nourishment – Tomis Centre Zone [8]

- **Tomis South Zone**

Beach nourishment was performed on an alongshore distance of 1.05 km and an amount of 677,127 mc of sand (**Fig. 8**).

Fig. 8 Beach nourishment – Tomis South Zone [8]

- **Eforie Nord Zone**

Beach nourishment was performed on an alongshore distance of 1.2 km and an amount of 1,144,508 mc of sand (**Fig. 9**).

Fig. 9 Beach nourishment – Eforie Nord Zone [8]

The benefits brought by implementation of the project include [8]:
- reducing the risk of flooding the properties in the vicinity of cliffs enjoyed by approximately 278,000 inhabitants;
- the advantages of an extensive beaches for an estimated 32,000 domestic and international tourists but also for city residents who may prefer to choose according to closeness and accessibility beach;
- enlarging beaches with 330,000 m^2, widening beaches with 100 m to the existing one in 2013;
- stability of socio-economic activities in the area for the 122 operators identified that will generate 250 temporary and 10 permanent jobs;
- an effective measure for preserving the shore and protecting the functions of the coastline such as recreation and nature.

So, the benefits of refurbishments and sanding beaches of the southern part of the Romanian Black Sea coastline which is situated in the Constanta Metropolitan Area contributes to the complex development, in an integrated concept, with socio-economic efficiency of the zone [7].

4. CONCLUSIONS

The development of the Romanian coastal zone should be seen in terms of sustainability which has three traditional pillars – economic, social and environmental [2].

From the analysis of the advantages and disadvantages of coastal protection engineering solutions (beach nourishment) for the southern part of the Romanian coastline it was concluded that is needed a strategic management of the shoreline in the coastal zone of the Black Sea [6]. This requires the development of the area in a systematic manner in which all the components to be integrated in development projects.

One such project is the cross-border for maritime spatial planning for the Black Sea. Marine spatial planning represents "a public process of analyzing and allocating the spatial and temporal distribution of human activities in marine areas to achieve ecological, economic, and social objectives" [1].

In this context, it can be concluded that the reduction of coastal erosion by beach nourishment contributes to increasing human activities in the Romanian Black Sea shoreline due to the benefits brought (tourism, economic and social activities, properties, recreation) to achieve ecological, economic, and social objectives of the area.

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Research on the Stability of the Defense Works to Eroded River Beds

Mihail Luca, Fabian Tamaşanu, Alexandru Lucian Luca, Gelu Ilie, Anca Balan

***Abstract* – The paper presents the research results on the behavior of shore defences located in riverbeds erodible. The research was carried out in the field, a sector of Moldova River over a period of 12 years. The time period was characterized by high frequency of floods. This shows the vulnerability works hard type (pear concrete and stone) placed in eroded riverbed. Researches and studies have been conducted in an area of economic, characterized by the presence in the riverbed of the adduction pipes Timişeşti –Iași. The researches and analysis of morphological changes of the riverbed, given by flash floods during the period of study have indicated the establishment of a special type of elastic bank protection. The construction is made of elastic geo-containers (mattresses) filled with ballast. The elastic type bank protection was designed and tested for site conditions on the Moldova River.**

***Keywords* – bank protection, erodible bed stream, geo-textile bags, stability, resistance.**

1. INTRODUCTION

Rivers eroded river beds require special conditions for stability of the defense works from the shore. The erosion of the riverbed is influenced by hydrological regime of the river. In the last period of time (about 15 ... 25 years) there was a high frequency of floods on the rivers in Romania. The action has caused floods in the niche rapid degradation of the bank protection construction. High frequency of floods forced the development of research on reaction of the bank protection in conditions of our site. The last time there was a change of design concepts to the regularization of

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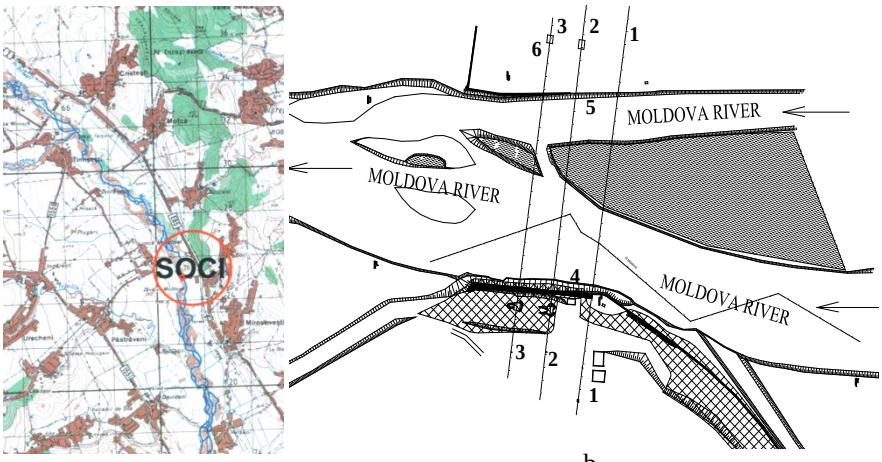
the river. New concept aims at collaboration of nature with human activity in modifying the hydrodynamic balance of the river. Tracking over time the collaboration with river regulation works allowed us to obtain useful results for design and execution. Also, new theories have been developed to regulate rivers [7].

Morphological modification of riverbed has determined the new shares stability hydrotechnical construction of river bank. Bank protection constructions of shore are made of various materials, natural (wood, stone), (plain concrete, reinforced concrete, plastics) and composites. The bank protection works may be rigid (concrete slabs, jointed pitching) and elastic (gabions with stone) [2]. The bank protection is not behaving efficiently in erodible beds. These are partially or totally degraded in a time less than the standardized.

2. GEOPHYSICAL PARAMETERS OF THE EXPERIMENTAL AREA

Research has been done on a sector of the Moldova River in Soci, County of Iaşi [3]. The hydrographic basin of river Moldova is located in the north-eastern Carpathians and north-western Moldova Plateau. Land Code of river is XII.1.040.00.00.00.0. In the undercrossing main features of the river Moldova are: length of the watercourse, 160 km; catchment area, 3,567 km2; analysed the average slope of 1.30‰.



a b

Fig. 1 Map of the undercrossing Moldova River in Soci zone:
a – geographic map; b – scheme of the protection works
1, 2 and 3 – adduction pipes; 4 - protection works of the left bank; 5 – protection works of the right bank; 6 - inspection chambers

Moldova river has crossed the riverbed in 2005 a flood flow Q = 1168 m3/s. In 2010 there were two floods: first summer with Q = 660 m3/s; second autumn with Q = 965 m3/s [5]. Flow and level of probability using calculation Soci section of the

river Moldova (Fig. 1) are given in Table 1.The effects of floods have resulted in the degradation of shore defense [3], [4], [6].

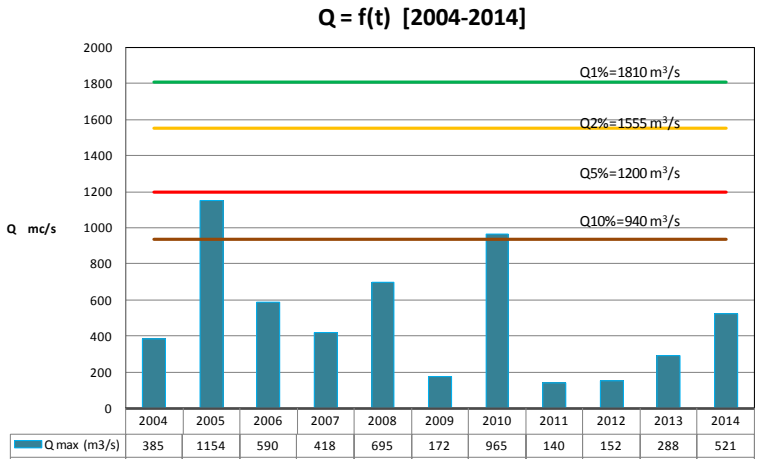


Fig. 2 The frequency of annual maximum flows during the analysed period

Table 1. Calculus discharges and levels

Calculus probability, p %	1%	2%	5%	10%	50%
$Q_{p\%}$ (m^3/s)	1810	1555	1200	940	28.8
H (m LBS)	257.15	256.95	256.50	255.90	253.10

Research area includes hydrotechnical constructions of the undercrossing Timişeşti-Iaşi adduction pipes of Moldova River. In the undercrossing zone are three adduction pipes: the first adduction pipe (Ad I), put into operation 1911; two adduction pipes (Ad II), put into operation in 1973.

On river Moldova is performed a hydrotechnical construction of undercrossing transmission pipes. Settlers in the area of research consist of layers of ballast with thickness of 18...15 m [5]. The section of river regulation work has a bed and shore protection. Shore protection works were carried out in the 1970...1973. Defending the shore is formed by massive stone upon which were placed concrete pears. Moldova River over a length of 325 m presents the current stage a channel linear calibrated. The river bed was calibrated to determine high flow speeds of floods.

3. RESEARCHES AND EXPERIMENTAL RESULTS

Theoretical and experimental research watched the following issues:

A. Research on hydrological and hydraulic parameters of the river Moldova in the area of study.

B. Research of the bank protection constructions in the study area.

C. Design and research of new constructions of bank protection that interoperate with erodible riverbed.

D. Research the parameters of hydraulic resistance and new type of bank protection construction

Along its course the river investigated were carried out topographic measurements at regular intervals. Topographical plan were accomplished through cross-sectional and longitudinal profiles anyway. On the longitudinal and transverse profiles are analysed the evolution over time of the construction of bank protection [3], [5].

Research of construction of shore defense was conducted on three main areas:

Area I: The research of the old constructions of the bank protection on the sector river studied.

Area II: Analysis of new type construction of the bank protection.

Area III: Theoretical researches of reaction-new construction from hydraulic and mechanical actions from the location (riverbed Moldova).

Bank protection of rigid type was achieved in 1971. It consists of two overlapping construction structures (Fig. 3.a, Fig. 3.b.). The first structure consists of a massively stone set on a mattress of fascia. The stone has a width at base of 2.90 m and height of 1.80 m. The second consists of concrete slabs supported on a concrete simple beam [5], [6].

Concrete slab are poured onto a layer of ballast in the thickness of 10 cm. Slabs of concrete have dimensions 50x50x10 cm. concrete simple Beam with 40x40 section cm shall be fixed as part of a rubble stone. Pitching of concrete slabs is mounted on a height of 1.70 m tilting results from the stone and tiles are 1:1.5. Research in the field looked at how the behavior of the construction factors from location.

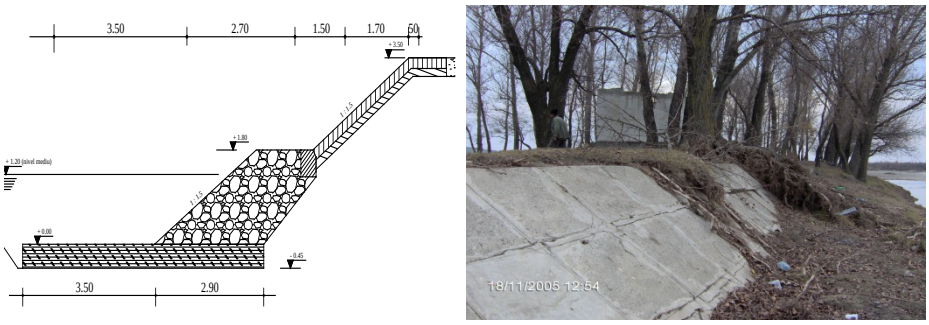


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Fig. 3 Images of the protection works on the Moldova River:
a – detail of the execution; b – bank protection in the year 2005

The research was realized in 2004...2015. Experimental data processing showed a sequential decay of construction over the past 10 years. In the first years of the operation was achieved with clogging with sediment a construction of the bank protection. In 2005 ... 2015 has changed the hydrological regime of the Moldova River. Modification is represented by the appearance of 2 ... 3 major floods per year [5].

Degradation of the bank protection was realized through the following processes destructive [5], [9], [11]:

- Silt washing- of the bank protection construction (Fig. 3, Fig. 4).

- Removal from washing lines and ballast layer beneath the slabs cement mortar (fig. 5).
- Cracking concrete tiles. The pieces of tiles have been trained during the floods of water (Fig. 4).
- Training supportive of stone and the collapse of the construction (Fig. 5a).
- Breaking support beams and crumbling concrete tiles (Fig. 5 b). Involve pieces of concrete in the riverbed until the unveiling of the bank.

a b

Fig. 4 State of decay in time of the bank protection:
a – clogged construction in the 2005; b – silt washing in the 2008 year.

The degradation of bank protection contributed layer ballast which is formed the river bank. Positive forces have acted vigorously on the tiles when the water level after the flood. The riverbed share went down due to hydrodynamic erosion phenomenon. This has contributed to the degradation and caused the collapse of the bank. High frequency of floods (2 to 4 during a year) was an important factor in degradation of the bank protection [5], [7].

a b

Fig. 5 State of decay in time of the bank protection:
a – degradation of tiles, 2010; b – training tiles and collapse into bed in the 2012 year.

For the protection bank of the coastline has been selected a building made up of elastic elements. Construction of researched mattresses is made of “geo-textile bags” filled with ballast. Geo-textile bags are a bag made of "geotextile" material padded

fibers of polypropylene mechanical consolidation through woven and sewing on two sides. Length of geo-textile bags were studied by 2.38 meters and width of 1.45 m and thickness is 0.45 ... 0.50 m after filling the ballast. Geo-textile bags were fitted with the long side parallel to the river bank. Geo-textile bags are mounted on rows and delayed on the riverfront. Geocontainers mounted in the contact zone with water ballast treated with cement [5].

a b

Fig. 6 State of decay in time of the bank protection: a – degradation of the massif stone, in 2010 year; b – breaking the beam and collapse into bed in the 2010 year.

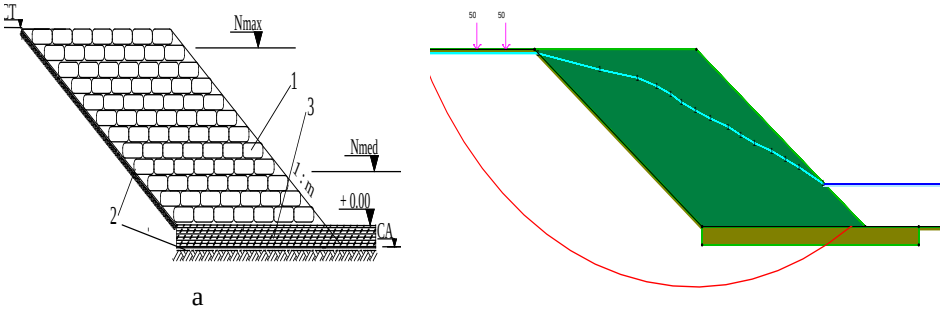
a b

Fig. 7 State of decay in time of the bank protection 2014 year:
a –degradation plating of tiles; b – training tiles in the area.

New work of bank protection was designed and checked the condition of stability in location (Fig. 8). Verification was done by using a spreadsheet program. Analysis of results in determination of stability pursued different moments of exploitation of bank protection. Stability of the bank protection (solid Earth + stone + concrete massive slabs) is estimated by comparison of two coefficients. The first is the safety coefficient and the second coefficient of restriction. The condition must be satisfied is:

$$Cs_{efectiv} < Cs_{admisibil} \tag{1}$$

Calculation of custom flow is made for situations existing in the riverbed. The program is built to be able to perform calculations of stability by using one of the following three methods: Bishop, Wire or Spencer Wright. The program can take into account the seismic action from location. Into account the diagrams were used, specifying explicitly Jambu position the centre of the circle of failure results in parameters: height, cohesion, friction angle results, the bulk weight and the slope of the results [5].



a b

Fig. 8 Stability researches of the new bank protection: a – detail for the work of geo-textile bags, 1-geo-textile bags, 2- geogrid, 3-fascia mattress; b- stability simulation of the hypotheses 2

The calculation was performed for the five hypotheses of the bank protection. Checking assumptions were: 1 - normal operating process; 2 - sizing with flood probability; 3 - Flash flood probability; 4 - implementation process-specific situation; 5 - time situation of rapid descent to level after the flood. Investigations have shown that the values of the coefficients of safety within the limits imposed by the design codes (Table 2). The works which have not fulfilled the conditions of stability have been designed in detail and used to bank protection (Fig. 8) [5].

Table 2. Safety coefficients (C_s) in the calculation of stability

Hypothesis	**1**	**2**	**3**	**4**	**5**
C_s	1.74-1.86	1.63-1.78	1.51-1.69	1.19-1.27	1.29-1.34
Conclusion	Admitted	Admitted	Admitted	Admitted	Admitted

Studied on the River was made a bank protection with geo-containers on a length of approx. 350 m new work was done in collaboration with the old damaged. The bank protection is made of 3...5 rows of -geo-textile bags depending on the height and intensity of hydrodynamic force bank. Filling the ballast has been treated with a cement volume at geo-textile bags in contact with water. The treatment had the role of increased resistance to mechanical work from site.

4. CONCLUSIONS

1. Hydrological regime of rivers in Romania is characterized lately by the high frequency of floods. During the year there was two or three flood of high value of the flow.

2. High frequency of floods caused rapid degradation of the regularization and shore protection works. Some sectors of the river were produced significant material damage in small time intervals.

3. The high degree of hydrodynamic erosion of flood in whites with erodible riverbed has contributed to the degradation of the sequential and further to pitching from concrete slabs.

4. Geotechnical and hydrological conditions existing in the erodible beds condition design and construction of shore protection works. Works most suitable in this case are elastic type (gabions with stone, geo-textile bags with ballast).

5. Research conducted in the bed of the river eroding Moldova confirms the rapid degradation of shore protection works and the need to adopt special elastic type constructions.

6. The work of mal type elastic work together effectively with the gravel beds of the river erodible and does not harm the environment.

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Earthquake Vulnerability of Water Distribution Systems from Topology Perspective

Abdullah C. Koc, Umut S. Demir, Selcuk Toprak

***Abstract* – Past earthquakes have demonstrated that water distribution systems (WDSs) suffered significant damage like buildings. System service ratio (serviceability) after earthquake is presented herein and calculated for hypothetical water distribution systems. GIRAFFE computer program was used to perform Monte-Carlo probabilistic simulations of the systems. Four different repair rates 0.2, 0.5, 2.0 and 4.0 were used in the analysis. Robustness and vulnerability of water distribution systems were evaluated with serviceability and system topology.**

***Keywords* – Water Distribution System, Earthquake, Serviceability, Vulnerability, Topology**

1. INTRODUCTION

Earthquakes are destructive for all structures. Ravage of earthquake on buildings or above ground structures is blatant but obscure for infrastructures. Piped infrastructures are one of the important components of infrastructure systems that's why they called as lifelines. Water distribution systems (WDSs) are one of the critical lifeline systems in urban areas. Past earthquakes have demonstrated that WDSs are vulnerable to earthquakes. The failure of a WDS not only impairs firefighting capacity, but also disrupts residential, commercial, and industrial activities, resulting huge economic losses. Hence, it is important to assess the seismic performance of a WDS [1]. Damage to lifelines not only results in physical impairment and cost of repair at specific locations, but also the losses of connectivity and potential for more widespread and serious losses of functionality throughout the network [2].

Lifelines are configured as networks. A water distribution network consist of hydraulic apparatus (control valves, pumps, tanks, reservoirs etc.) as the nodes (or

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vertices) and pipes as the edges (or links) of the network. Post-earthquake serviceability of WDSs of some cities and hypothetic systems were studied in the literature. Javanbarg and Takada assess the seismic reliability of Osaka City after the 1995 Kobe earthquake using availability and serviceability indices [3]. Chou and others examined the serviceability of the Lan-Yan area in Taiwan after soil liquefaction by means of GIRAFFE software [4].

In this study, damage concepts of a WDS were investigated, pipe damage modeling and negative pressure treatment were described. Then Monte Carlo (MC) simulation technique in the pipe damage process was explained. As a case study, earthquake performance of nine hypothetical water distribution systems was examined.

Earthquake damage to buried pipelines can be attributed to transient ground deformation (TGD) or to permanent ground deformation (PGD) or both. TGD occurs as a result of wave propagation or ground shaking effects while PGD occurs as a result of surface faulting, liquefaction, landslides and differential settlement from consolidation of cohesionless soils. The relative magnitudes of TGD and PGD determine which will have the predominant influence on pipeline response [5]. The effects of TGD and PGD are evaluated for the components of above ground and underground facilities of WDS. The underground facilities performance under seismic loading is assessed by models for soil-structure interaction, including empirical models based on the observations from the past earthquakes, closed form analytical methods and numerical simulations such as finite element analysis. To account for uncertainty with respect to component or facility response, seismic behavior is frequently characterized by fragility curves that provide the probability of failure as a function of demand (Peak Ground Acceleration, Peak Ground Velocity etc.). Fragility curves can be derived from either the observations of past earthquakes or Monte Carlo techniques that have special capability in quantifying uncertainty [2]. PGD hazards are usually limited to small regions with high damage rates due to the large deformation imposed on pipelines. In contrast the TGD hazards typically affect the whole pipeline network, but lower damage rates [6]. In this study WDS damage is represented by only with pipe damages, earthquake effects on the other components of the WDS like pumping and storage facilities were not taken into consideration. Pipe damages were represented with repair rate (RR) (repairs/km).

2. MATERIAL AND METHODS

In a real WDS, water distributed along pipe but in the mathematical model pipe junctions are accepted as consumption points. A hydraulic network is a mathematical model of a WDS in which the physical components are represented as nodes and links. Pipes are links and junctions of the pipelines are nodes in the hydraulic network model. In the event of an earthquake a WDS may sustain various kinds of damage, previous research shows that buried pipelines in a WDS are the most vulnerable components [7].

In this study Graphical Iterative Response analysis for Flow Following Earthquakes (GIRAFFE) software and its methodology of pipe damage simulation

and negative pressure elimination will be used. GIRAFFE is developed at Cornell University dedicating for the hydraulic analysis of the damaged water supply systems [8]. GIRAFFE works iteratively with the EPANET which is a computer program that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks [9]. GIRAFFE groups pipe damages into two as break and leak.

A break is defined as the complete separation of a pipeline, such that no flow will pass between the two adjacent sections of the broken pipe. In case of the break, water flows from the two broken ends into the surrounding soil. In GIRAFFE the broken pipe is modeled by using the EPANET elements as a fictitious pipe and reservoir are attached at each broken end of the pipe, check valves ensure that water only flows from the broken pipe into the reservoirs which are fixed at atmospheric pressure to simulate the broken pipe being open to the atmosphere [8].

A leak is defined as a gap in pipe, such that water will continue to flow through the pipeline, while some loss of pressure and flow through the leak. In GIRAFFE leakage is simulated by a fictitious pipe open to the atmosphere, simulated as an empty reservoir with the same elevation as the leak location. A check valve constrains flow from the leaking pipe in one direction. The roughness and minor loss coefficients of the fictitious pipe are taken as infinite and 1, respectively, such that all energy loss from the leak is related to the minor loss [8]. Since a leak is modeled as a fictitious pipeline in hydraulic network analysis in GIRAFFE, the area of this pipeline should be calculated from the equivalent orifice diameter. GIRAFFE classifies leaks into five scenarios as annular disengagement, round crack, longitudinal crack, local loss of pipe wall and local tear of pipe wall which are frequent leak types for metallic pipes. Equivalent orifice area equations for each leak type can be found in the GIRAFFE manual [8]. Probabilities of the leak scenarios are dependent to the pipe material. In this study pipe material is accepted as ductile iron and default leak type probability values for this material in GIRAFFE are annular disengagement 80%, longitudinal crack 10% and local loss of pipe wall 10%; round crack and local tear of pipe wall type cracks are not expected for ductile iron pipes.

Hydraulic network analysis solves incompressible water flow in a pressurized pipeline network based on two principle laws: conservation of mass and conservation of energy. The conventional hydraulic network analysis algorithm does not differentiated positive and negative pressures and only uses the total head difference to drive water flow to satisfy demands. Commercial hydraulic network analysis software packages are designed for undamaged systems. The forced satisfaction of all demands may lead to the prediction of unrealistically high negative pressures at some nodes. Water distribution systems are not air tight so that their ability to support negative pressures is very limited [6].

In GIRAFFE, an isolation approach is applied to treat the negative pressures. This isolation approach works with EPANET hydraulic network engine iteratively. After hydraulic network analysis of the damaged system using EPANET engine, nodes with negative pressures are identified and isolated step by step, starting with the node of highest negative pressure. After each elimination, network connectivity is checked. If part of the system is isolated from the main system without water sources,

it is taken out of the system. Flow analysis and the elimination process continue until no negative pressure nodes exist in the system [8].

To simulate the earthquake performance of a WDS, pipe damage, including breaks and leaks, needs to be added in the network. Hydraulic simulation is then performed on the damaged network to predict the flow and pressure distributions. In GIRAFFE the pipeline break and leak models can be implemented into a hydraulic network both deterministically and probabilistically. The deterministic implementation specifies the number and location of leaks and breaks, and leak types, occurring in a pipeline network. This implementation can be used to simulate the performance of a WDS under a specific damage scenario. Whereas damage type and place has a probabilistic character even the RR of the pipes are known. The probabilistic implementation generates randomly distributed pipe damage in the system according to pipeline repair rate. Three normally distributed random numbers are used to determine the place of the damage, state of the damage (break or leak) and the leak type if it is a leak. In HAZUS [10] it is recommended that 80% and 20% of earthquake damage occurs as leaks and breaks, respectively, under seismic wave effects or TGD (percentages swap in case of PGD). With GIRAFFE, users have the ability to set the percentages of leaks and breaks to values other than the default settings of 80% and 20%, respectively [8].

The probabilistic implementation applied many times by using MC simulation to show the random character of damage. In GIRAFFE, number of MC simulations can be specified by user (MC Fixed Number) or automatically determined according to the user-specified convergence criteria (MC Flexible Number).

Reliability assessment of water networks comprises a complex, yet essential process. The seismic reliability of water networks is possible to be measured using different indices of physical nature or not, like vulnerability, connectivity, serviceability, maximum flow, redundancy and economic loss [7].

GIRAFFE uses the Seismic Serviceability as the performance indicator of the earthquake damaged water distribution network which is given in Eq. (1).

$$S_s = Q_T / Q^*_T \tag{1}$$

Where, S_s is the seismic serviceability, Q_T is the available demand and Q^*_T is the required demand. The serviceability can be calculated for each demand node and for the entire system. For deterministic simulation, the serviceability for each demand node is either 1 if this demand can be satisfied, or 0 if this demand node is isolated due to the negative pressure or connectivity problems. The serviceability for the entire system is the sum of the demands that can be satisfied over the sum of the total required demands. For probabilistic simulation, the system serviceability is reported in a matrix format. For each Monte Carlo simulation run, the serviceability is reported for each demand node and for the entire system [6].

Performance indicators of a damaged WDS will be examined by means of 9 hypothetical WDSs in this study. The basic hypothetical system consists of 51 pipes (edges), 36 pipe junctions (nodes, vertices) and a reservoir. Elevations of all junctions are 100 meters and lengths of all pipes are 400 meters. Demand from each node is uniform and 2 liters per second. Water surface elevation of the reservoir is 160 meters

and all pipes are ductile iron with the Williams Hazen roughness coefficient of 130. Pipe diameters are varying from 80 mm to 500 mm as shown in the Fig. 1. The basic hypothetical water distribution system will be called as H1.

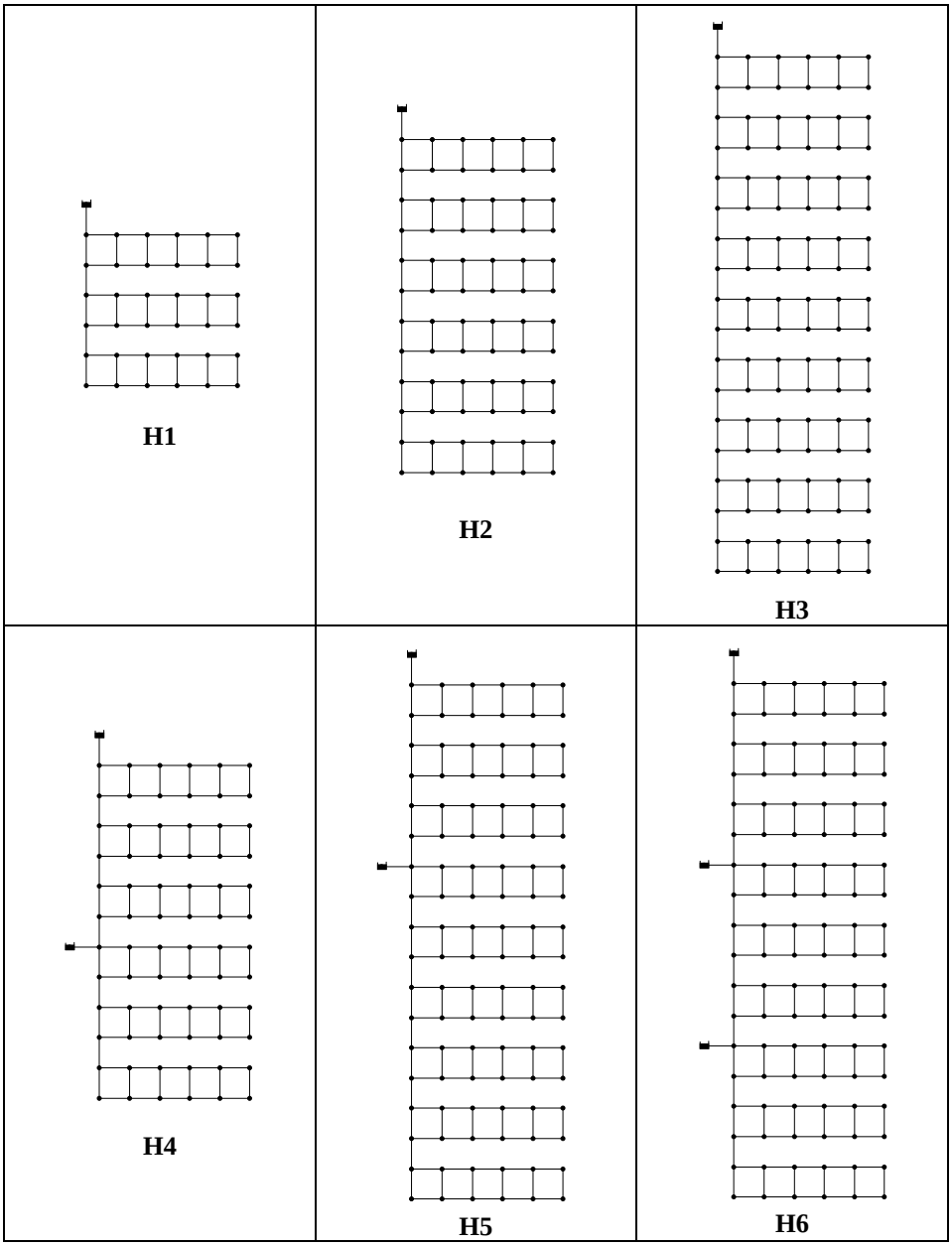

Fig. 1 Schematic drawings of the hypothetical water distribution systems (H1 – H6)

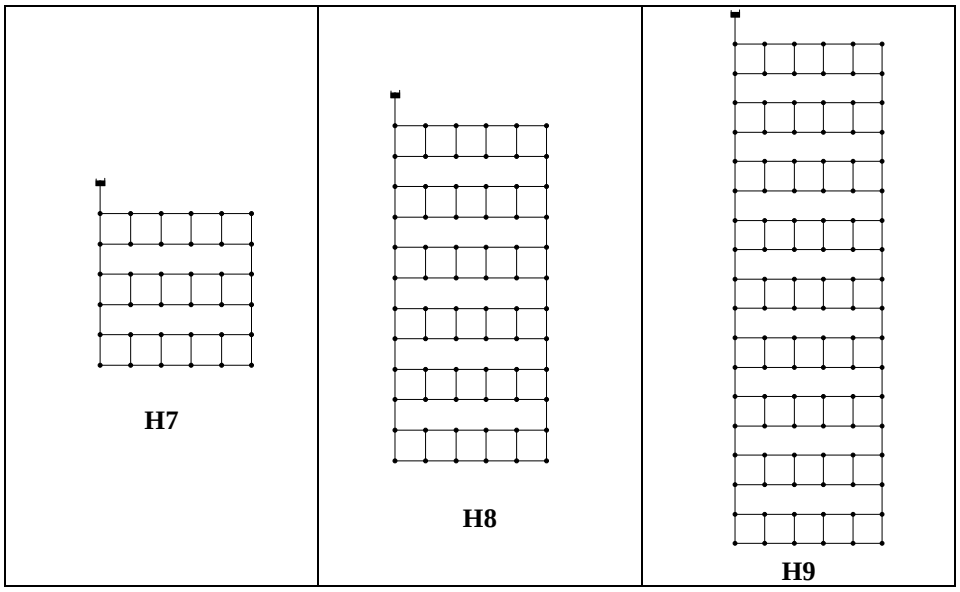

Fig. 1 (Cont.). Schematic drawings of the hypothetical water distribution systems (H7 – H9)

Based on the system H1, 8 more hypothetical systems were derived. Schematic drawings of these systems are given in Fig. 1. Other hypothetical water distribution systems were formed as follows; H2 system was obtained by doubling the H1 distribution system. H3 system is the triple of the H1 distribution system. An extra reservoir added in the mid of the system H2 and then the system H4 formed. One and two reservoirs added to the system H3 and then the system H5 and H6 developed, respectively. Edge of the eaves of systems H1, H2 and H3 were connected with pipes then the systems H7, H8 and H9 were obtained. Pipe lengths (100 meters), pipe materials (ductile iron) and Williams-Hazen roughness coefficient values (130), elevations of the nodes (100 meters) and reservoirs (160 meters), node demands (2 liters per second) are same for all other derived hypothetical water distribution systems. Only the pipe diameters of the main distribution line were changed to ensure reasonable velocities in the pipes and pressures in the nodes when the system enlarged. An extra node and 1 meter long pipe added just at the exit of the reservoirs in all systems and these nodes have no water demand.

Seismic performances of the hypothetical WDSs are evaluated using Monte Carlo simulations in conjunction with GIRAFFE. Up to 2000 MC runs were performed for each system and for each repair rate. There are 36 different system and repair rate combinations. Repair rates were selected as 0.2, 0.5, 2.0 and 4.0 for all pipes and pipe damage probability for ductile iron pipes was partitioned as 20% breaks and 80% leaks. Minimum pressure to elimination of a node is accepted as -5 psi (-0.34 Atm.). Duration of the simulation is taken as 0 hours because water source of the system is a reservoir not a tank, the amount of water will always be the same for the other simulation times. Reservoir connected pipes were not let damaged so their repair rates were taken as 0.

3. RESULTS AND CONCLUSIONS

Serviceabilities were calculated for each node and each simulation with the GIRAFFE. Serviceability of the system was calculated for each simulation by dividing the number of nodes whose request was met to the number of total nodes. Serviceability of the nodes was calculated by dividing the number of the simulations in which node demand was met to the total number of simulations. The means of the both serviceabilities should be the same and this is the general serviceability of the system [11]. Mean serviceability values after Monte Carlo simulations of all hypothetical systems with repair rates were given in the Fig. 2.

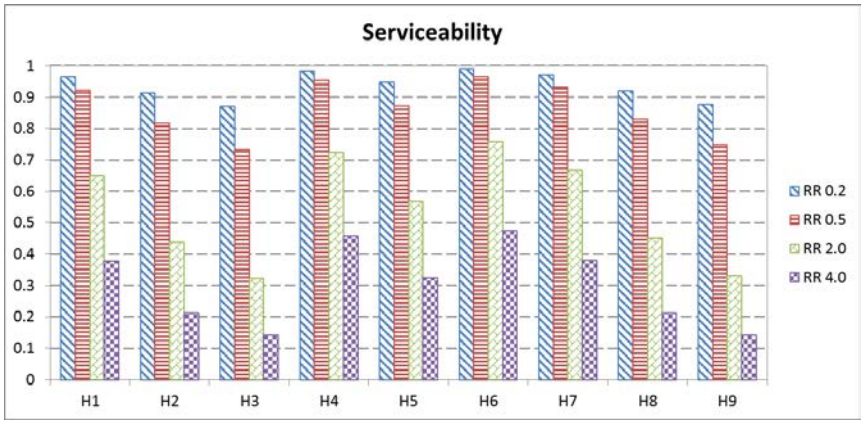

Fig. 2 Mean serviceabilities of all hypothetical systems for different repair rates

According to the Fig. 2 Serviceability decreases when repair rate (RR) increases for all hypothetical systems. When the system grows (H1-H2-H3) serviceability decreases for all RRs. Because the hypothetical system was enlarged along a main line and when this line breaks the other side was isolated from the system. Reservoirs addition (H2-H4 and H3-H5-H6) increases the serviceability for all repair rates. One of the reasons may be the system can meet the demand from the other reservoir when the main line coming from the first reservoir breaks. Connecting the edge of the eaves of system (H1-H7, H2-H8, H3-H9) increases the serviceability for all repair rates. This is because there is an alternative route for the transmission of water. Addition of a reservoir to the system increases the serviceability much more than a pipe addition. The simulation results show that to decrease the earthquake vulnerability of water distribution systems they should not be expanded along a line and alternative routes should be available in the system. Also the system should be fed from many reservoirs.

4. ACKNOWLEDGMENTS

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The Groundwater Level Variation Due to Seepage Losses from Sinoe Irrigation Systems

Mădălina Stănescu, Constantin Buta, Geanina Mihai, Lucica Roşu

***Abstract* – The ensemble of natural conditions characteristic to agricultural areas determines a real particular production potential of soils, usually, lower than the maximum potential. This difference between the real potential and the maximum potential is essential due to the mismatch between growth factors and physiological requirements of the crop plants. The link between the manifestation level of natural factors and plant needs can be achieved through land improvement works, which controls the main factor - water. The irrigation arrangements belong to the group of land improvement works and constitute the main hydraulic works to combat the effects of drought. Due to their magnitude, water loss from the great network of irrigation canals are factors of environmental pollution and can have major implications for the economic and social situation. Combating the loss of water from canals is the main issue related to the development of efficient works and reveal their difficulty degree of solving.**

***Keywords* – land improvement, hydro technical works, seepage, impact study**

1. INTRODUCTION

Land improvement works, which controls the main factor water, consist of linking the manifestation level of natural factors of with plant needs.

The purposes of the land improvement works are:

- Enhancing the productive capacity of the agricultural lands;
- Raising the soil fertility of poor productive lands;
- Commissioning of the unproductive lands;
- Preventing and combating natural phenomena that negatively influence the productive potential of agricultural lands.

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Intensification of irrigation is the main measure to reduce the drought effects on climate and for the irrigation arrangements and for the watering technologies used is required a more efficient use of water applied without losses.

Like any intervention into a complex of elements where there are well-defined relationships, the irrigation arrangements, as part of hydro-technical facilities, have multiple effects to the environment.

The assessment of the size of these effects produced as a whole and, sequentially, on each natural factor or social factor in part, can be determined by the environmental impact studies on the environment.

Possibilities of reducing or eliminating environmental impacts are:

-works to control water discharges and storages from irrigation systems;

- waterproofing works for the irrigation channels;

- drainage works to drain the water excess appeared following of defectively applying irrigation water;

- works and measures against landslides and erosion;

- works to combat salinization of soils;

- reclamation works of hydrological observations by drilling;

- reclamation works of degraded land with waste derived from petroleum industry, power plant industry steel industry, chemicals, etc.

If an irrigation system is analysed over a period of time, T (cycle of watering, month, season), regarding the volumes of water required, starting from the plant and reaching to the gross outlet volume (base pump station), three sections (levels) of calculation can be highlighted (Fig. 1):

- Section (1), which represents the cultivated area, where, as a matter of fact, the irrigation of crops is produced;
- Section (2), consists of all sections through which the water access into the irrigation system is ensured;
- Section (3), consists of all sections through which the transport of water into the irrigation system is achieved (water supply).

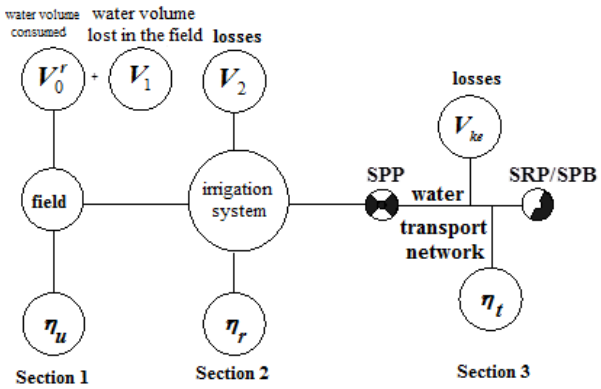

Fig. 1 General scheme of water movement in an irrigation system

As it results from Fig. 1, some water losses are occurring, starting from the outlet point to the plant, generated by a complex of phenomena and status processes.

Several types of volumes, analysed from the hydraulic transport point of view, which are lost during the operating system time (T), can be distinguished:

- $\mathbf{V_o^r}$- the whole water volume consumed by the plant evapotranspiration process (irrigation norm), in the section (1);
- $\boldsymbol{V_1}$ - the water volume lost in the field during the application of the irrigations, in the section (1);
- $\boldsymbol{V_2}$ - the water volume lost into the irrigation system, on the water transport network, in the section (2).
- $\boldsymbol{V_{ke}}$ - the water volume lost throughout the water transport network by infiltration (k) and evaporation (s), in section (3);
- $\boldsymbol{V_T}$ - the total of water volume lost in the irrigation system from the outlet point to the plant.

$$\boldsymbol{V_T = V_0^r + (V_1 + V_2 + V_{ke})} \quad (1)$$

Research conducted regarding water loss through network channels seepage led to the establishment of some coefficients of the variation functions in relation to the size of wetted section and type of lining adopted for waterproofing.

Worth noting that the initial water losses are on average with 50% higher for higher-order distribution channels (CD) and with 60% higher for the supply channels (CA), compared to the normal operation regime (Fig. 2).

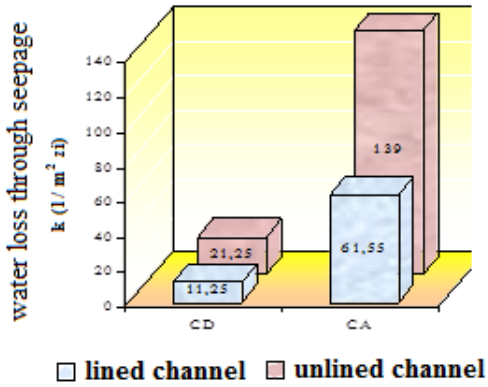

Fig. 2 The amount of water loss through seepage from irrigation channels, p_k[l/m^{2}zi]

2. EXPERIMENT DESCRIPTION. WATER LOSSES ASSESSMENT IN IRRIGATION CHANNELS

2.1. Solutions to combat the water losses in open channels

Combating the water losses from the channels and, generally, controlling the water's exchange between the channel and the field, as well as combating the changes which the water produces on physical and mechanical characteristics of the soil are the main issues related to the establishment of effective works and which highlight the difficulty of the solving.

The main method for conservation of the hydraulic equilibrium existed before the execution of a channel is to prevent water seepage, and this goal is achieved by sealing or lining its perimeter.

In most cases, the water losses reduction is the objective being pursued by the lining works for channels waterproofing.

The action to reduce water losses in channels requires prior knowledge of the complex hydraulic phenomenon that occurs, and therefore, it will be studied for each situation of the site in two scenarios: unlined channel and lined channel in a particular constructive solution for comparing data to showing the effectiveness of this measure in terms of reducing the water losses.

For the effective measurement of the water losses, the following indices, whose meaning is clear from Fig. 3, are using:

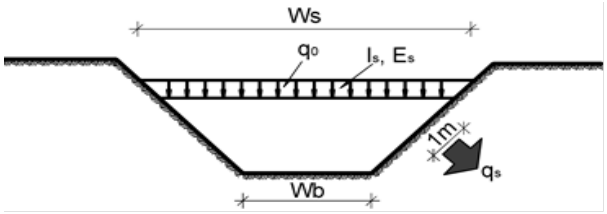

Fig. 3 Indices for the characterization of the size of water losses in channel
W_s –water surface width; W_b –bottom width of the channel; q_0 –unit flow infiltrated at the water surface level; q_s –flow infiltrated at the channel perimeter; I_s –flow infiltrated; E_s –seepage flow

I_s, E_s (cm/zi) - ***statistical infiltration or statistical seepage,*** or water loss measured in the steady state, from a water column given, without complete the water loss, determined by the ponding test method;

q_o (m^3/zi) –***unit flow infiltrated*** at the water surface level

$$q_0 = 0.01 \ \ pt\, E_s \ \ si\ W_s \tag{2}$$

$$q_0 = q_s \frac{W_b - 2H_w\sqrt{1-m^2}}{1000} \tag{3}$$

q_s (l/zi/m^2)–***the average specific seepage flow*** per unit area of lining or waterproofing;

$$q_s = \frac{10E_s \cdot W_s}{W_b + 2H_w\sqrt{1+m^2}} \Rightarrow q_s = 10E_s \frac{W_b + 2m \cdot H_w}{W_b + 2H_w\sqrt{1+m^2}} \tag{4}$$

2.2. Assessment of water losses in unlined channels

Bouwer has developed one of the methods for calculating the water loss and to capitalize data easier, for which he has processed and then he has synthesized the results using working graphs with the following basic requirements:

Condition A. The channel is provided in the contents of a uniform soil with isotropic permeability, succeeded by a more permeable base layer (k = ∞) (Fig. 4 a).

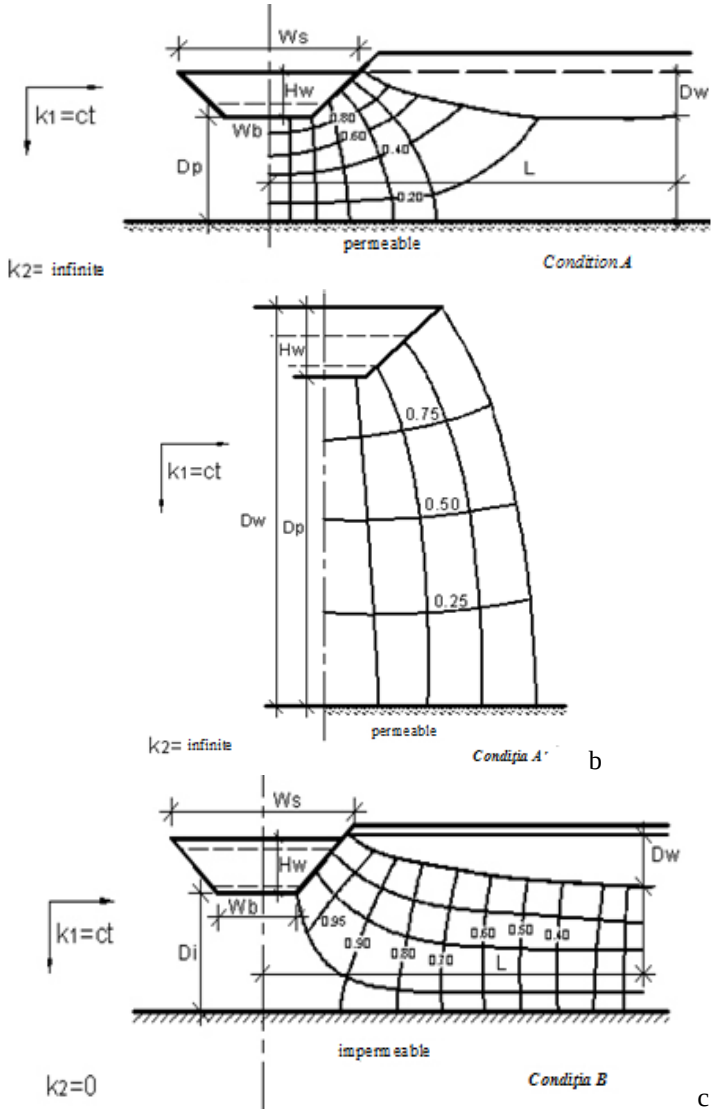

Fig. 4 Basic conditions schematization, after Bouwer, when seepage is occurring from the channel

k = permeability coefficient; $\boldsymbol{H_w}$ = water depth in the channel; $\boldsymbol{W_s}$ = apei water surface width; $\boldsymbol{W_b}$ = bottom width of the channel; $\boldsymbol{D_p}$ = permeable soil layer depth; $\boldsymbol{D_i}$ = impermeable soil layer depth; $\boldsymbol{D_w}$ = groundwater depth; ***L*** = the distance where D_w is measured

Under this condition infiltration and seepage flooded off may occur depending on the permeable soil layer position and the groundwater level.

If the groundwater level is provided in the drained base soil layer, we deal with free seepage and thus *condition A* is defined as the *condition A'* (Fig. 4 b).

Condition B. The channel is provided in the contents of a fine soil layer followed at its base by a less impermeable layer (k = 0). Obviously, under this condition infiltration shall be free or flooded depending on the impermeable soil layer position and the groundwater level. (Fig. 4 c).

Results are shown in diagrams like those in the Figure 5 and represents families of curves for a given value of the ratio of water depth from the canal, H_w, and depth of groundwater, D_w, both measured by to the same reference axle.

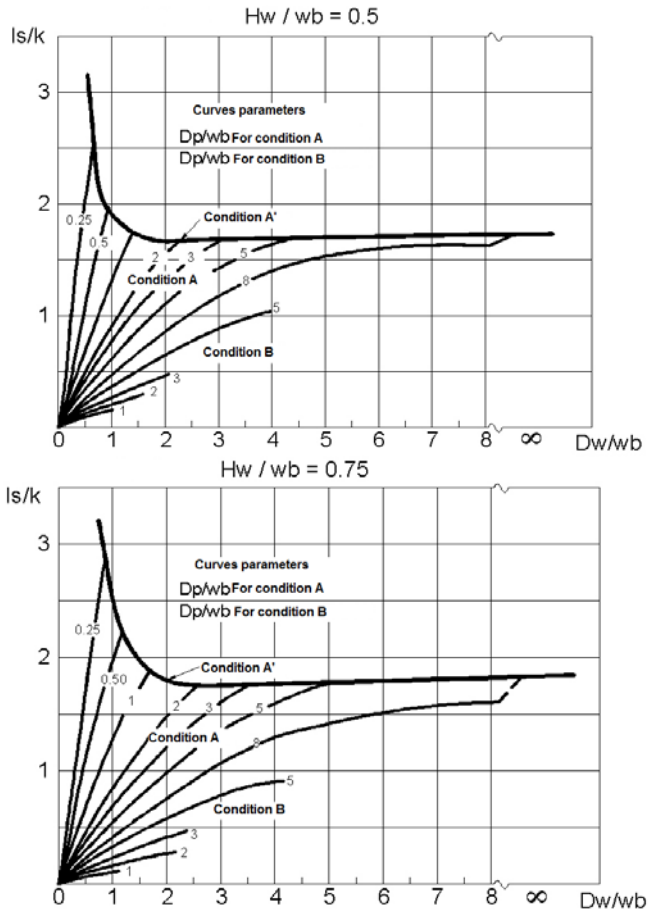

Fig. 5 Bouwer diagrames ***Is/k = f (Dw/ Wb)***:
k = permeability coefficient; ***Hw*** = water depth in the channel; ***Wb*** = bottom width of the channel; ***Dp*** = permeable soil layer depth; ***Is*** = flow infiltrated; ***Dw*** = groundwater depth

Therefore, relations like I_s/k and D_w/W_b are presented, relations which have the advantage of being dimensionless and allows more synthetic representation of the results.

By processing this mode ***of representation of the correlation between water losses, channel elements and the terrain*** and using a coordinate transformation it can make a representation of ***the correlation between water*** **losses and the groundwater level depth** (considering a terrain with *K* determined and the channel geometry set), presented in Fig.6.

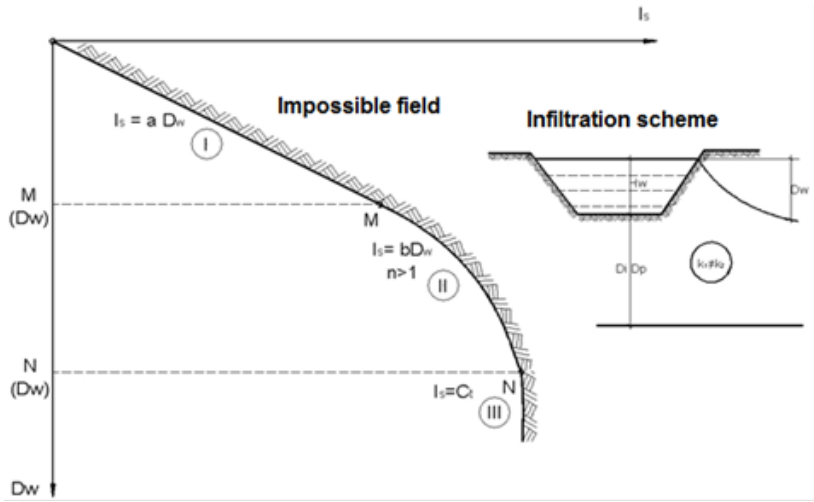

Fig.6 Correlation curve ***Is = f (Dw)*** obtained by processing diagrams developed by Bouwer:

a, ***b***, ***n*** = coefficients; D_w = groundwater depth; I_s = infiltrated flow; ***k*** = soil permeability coefficient; H_w = water level in the channel; D_i = impermeable layer depth; D_p = = permeable layer depth.

On this correlation curve ***Is= f (Dw)*** in this chart are three segments that characterize the regime of infiltration:

- ***First Segment (I) -*** for *lower depths of groundwater level*, the seepage amount varies practically linear with $\mathbf{D_w}$, being in *drowned infiltration domain* and the relationship is of the form:

$$I_s = a \cdot D_w \tag{6}$$

The phenomenon can occur ***under Bouwer conditions (A or B).***

- ***Second Segment (II***) which provides the connection between the two lines, which is a curve defined by a relationship of the type:

$$I_s = b \cdot D_w^n \text{ , } n>1 \tag{7}$$

The phenomenon can occur under conditions A or B defined by Bouwer.

- ***Third Segment (III***)- for *higher depths of groundwater level, a free infiltration* ocurrs and its size is virtually unchanged expressed by the relationship:

$$Is = constant \quad (8)$$

This relationship is valid to ***condition A'*** defined by Bouwer

This method presented allows an expeditious determination of the I_s value and $I_s = f(Dw)$, particularly important to allow an estimate of water losses to the changes in groundwater level position.

It is obvious that this method, besides schematic imposed by analogy used, does not take account only of the channel elements and the terrain, without taking into account the fact, like most calculation methods used, the fluid elements and the operating mode of the channel.

3. RESULTS AND SIGNIFICANCES. STUDY CASE – CA SINOE IRRIGATION CHANNEL FROM THE SINOE IRRIGATION SYSTEM

3.1. Location. General characteristics of the CA Sinoe channel

Sinoe channel was executed in 1971-1975 period and the extension works were performed in 1981-1987 period 1981-1987. Sinoe channel is located in the eastern part of the Sinoe irrigation system and is the main channel of the Sinoe irrigation system, it has a length of 37,56 km and serves an area of 64597 ha.

The water supply is pumped from Goloviţa Lake through Sinoe pumping station (SPA), which discharges at 51.00 elevation (with a pumping height of 55.25 m) from where water is distributed to different beneficiaries.

Sinoe channel is divided into five successive functional sections gravity supplied of each other. the flow transported is 46 m^3/s upstream and reach 12 m^3/s downstream at repumping station SRP1-6.

The maximum depth of the channel is 4.20 m upstream and decreases to 2.50 m downstream.

From the geotechnical point of view the soil found in the area is loess, with silty-loam texture and macroporous structure, sensitive to wetting under its own weight which exceed 40 cm in some areas on the field. Loess is dry, hydrostatic level and is quartered in silty clay and silty clay deposits in depth.

Under the loess horizons were intercepted intercalations of clay loams and silty clay loams, yellow or reddish-brown and at soil based were intercepted intercalations of silty clays and reddish clays with calcareous concretions.

Given that the channel is in an area which, in geological terms, is characterized by deposits of yellow loess susceptible to collapse it anticipated that the operation of the channel in first stage to be done without waterproofing, so that the bottom channel and slopes to stabilized.

By 2008, the CA Sinoe channel functioned without waterproof, with a longitudinal section and cross section with the following characteristics:

- bottom width b = 4.00 m;
- interior slope 1: m = 1: 1.5;
- water level(in the hydrodynamic regime) ha = 3.00 m;
- safety height hs = 0.36 m;
- channel bottom slope = 0.15 ‰.

Although during channel operation the channel section without waterproof was maintained by earthworks (additions embankments and profiling cuttings for clogging areas), however on the section III between km 9 + 409 and km 11 + 042 on the embankments were produced a series of local degradation as a result of land subsidence due to loess soil and because of surface runoff of rainwater that eroded unconsolidated channel slopes.

3.2. Water losses assessing in CA Sinoe channel (without waterproof)

We use the notations in paragraph 3.1, with schematization of natural elements of the land represented in Fig 4 a.

$\boldsymbol{H_w}$ = 3,00 m - water depth in the channel

$\boldsymbol{W_b}$ = 4,00 m - bottom width of the channel

1: m = 1:1,5

$\boldsymbol{k}$ = 5x10^{-3} cm/s –soil permeability coefficient

$\boldsymbol{D_p}$ = 20 m - permeable layer thickness (between the bottom of the channel and impermeable base layer)

$\boldsymbol{D_w}$ = 24 m - groundwater level (between the water level in the channel and impermeable baselayer)

$$\frac{H_w}{W_b} = \frac{3,00}{4,00} = 0,75$$

$$\frac{D_p}{W_b} = \frac{20}{4} = 5$$

$$\frac{D_w}{W_b} = \frac{16}{4} = 4$$

We are in case of ***Condition A*** where we use the Bouwer diagram $\frac{I_s}{k} = f\left(\frac{D_w}{W_b}\right)$ from Fig. 5 for $\frac{H_w}{W_b} = 0,75$.

Because $\frac{D_p}{W_b} = 5$, We consider *the curve (index 5)*for the ***Condition A***.

With value $\frac{D_w}{W_b} = 4$ in the curve index 5 we find the value $\frac{I_s}{k} = 1{,}7$

Resulting the value of static seepage:

$$I_s = 1{,}7 \cdot k \text{ cm/s} \Rightarrow I_s = 1{,}7 \cdot 5 \cdot 10^{-3} \text{ cm/s} \Rightarrow I_s = 8{,}5 \cdot 10^{-3} \text{ cm/s} \Rightarrow$$

$$I_s = \frac{8{,}5 \cdot 10^{-3}}{\frac{1}{86400}} cm/zi \Rightarrow \boxed{I_s = 734{,}4 \text{ cm/day}} \quad (9)$$

Using relation (4) gives the amount of qs (l / day / m²) Average specific infiltrated flow per unit area:

$$\Rightarrow q_s = 12553 \text{ cm/zi} \cdot m^2 \Leftrightarrow \boxed{q_s = 1255{,}3 \text{ l/day} \cdot m^2} \quad (10)$$

The value obtained is close in order of magnitude values for soil with sandy loam texture, with groundwater level > 5 m.The water loss through seepage in unlined channels have values 700-900 l/m^2 /day [6].

If we consider the total area of wetted perimeter of the section III of the channel CA Sinoe, S = 13624 m2, results a flow lost through seepage,

$Q_i = 171021203$ ***l /zi***, adică $Q_i = 171021$ ***m^3/day,*** (11)

In a irrigation season with duration Ti = 6 months (180 days) the apparent volume of water lost

$$V_i = Q_i \cdot T_i = 171021 \text{ m}^3\text{/zi} \cdot 180 \text{ zile} \Rightarrow V_i = 30783780 \text{ m}^3 \Rightarrow V_i = 30{,}8 \text{ mil. m}^3$$

3.4. Water losses assessing in CA Sinoe channel (with waterproof)

Determination of seepage through lining joints with large tiles, commonly used in our country can be addressed theoretically, considering the plate perfectly tight, hypothesis tested in the field and in the laboratory. In Fig. 7 are the diagrams for determining seepage through the joint considered to be open to different dimensions of the prefabricated large tiles (a x b): a = the width of the slab; b = length of the slab.

For the CA Sinoe channel, with slabs of 2m^2 and height of water in the channel noted in diagram by $h_0 = 3m$ from the chart above results water loss value about q_s = ***30 l/s km***.

Knowing the value of seepage flow, and applying the relation (4) for calculating this seepage flow is obtained ***the seepage value, Es:***

For the channel dimensions the term:

$$\frac{W_b + 2m \cdot H_w}{W_b + 2H_w\sqrt{1+m^2}} = 1{,}71.$$

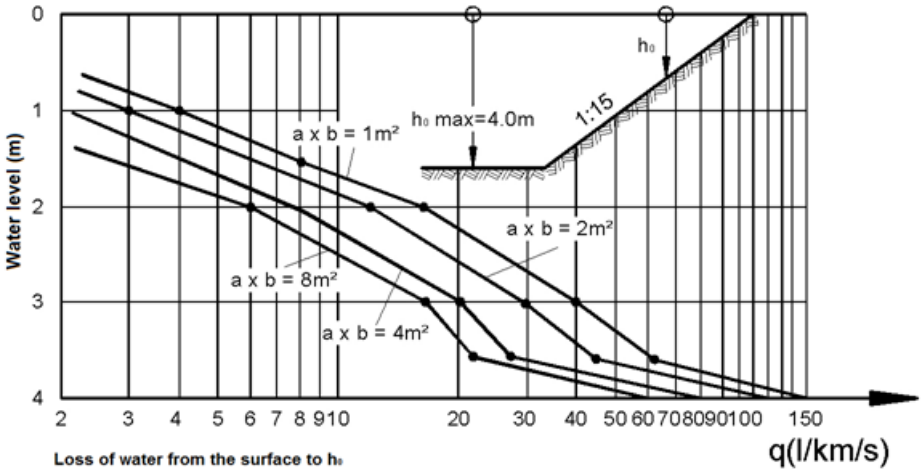

Fig .7 Diagrams for the determination of infiltration through the joint considered to be open to different sizes of large precast slab (a x b): a = the width of the slab; b = length of tile

Resulting, after transformations the value of $\boldsymbol{E_s = 94.40}$ cm/day (12)

Assessing the hydraulic effectiveness of linings with large slabs, compared to the initial situation of a channel made in the natural terrain without any protection, can be achieved based on hydraulic efficiency index of the linings and the waterproofing, index defined as yield in relation (3.24):

$$\eta = \frac{I_s - E_s}{I_s} \cdot 100 \text{ în \%} \qquad (13)$$

With the values previously obtained,

- for the infiltration of the unlined channel, from the relation (9) $\boldsymbol{I_s = 734.4}$ ***cm/day*** and,

- for the seepage of lined channel with slabs system of $2m^2$, from the relation (4.4) $\boldsymbol{E_s = 94.40}$ ***cm/day*** *results:*

$$\eta = \frac{734{,}4 - 94{,}4}{734{,}4} \cdot 100 \Rightarrow \eta = 77{,}8\ \%$$

From engineering practice, it result that the large slabs are hydraulically effective (η> 90%) in following domain:

- for the complete lining of the channel section, where the surface of the slab is more than 4 m²;

- for the complete lining of the channel section, where the soil hydraulic conductivity is k> 5x10-4 cm/s and where the water level in the channel is H_w <3 m.

4. CONCLUSIONS

Due to their magnitude, water loss from the great network of irrigation canals ***are factors of environmental pollution*** and can have major implications for the economic and social situation.

Combating the water loss from canals and, generally, controlling the exchange of water between the channel and the land and controlling the changes which water produce on physical and mechanical characteristics of the soil are the main issues related to the completion of works effective and highlighting the and the difficulty of solving.

The main method for conservation of the hydraulic equilibrium present before the execution of a channel is to prevent water seepage, the goal that is achieved by lining or waterproofing its perimeter:

- channel lining is a series of works that realize one or more watertight rigid or elastic layers on the channel perimeter. In most cases, channel lining is a definitive work.

In each case of the location, hydraulic phenomenon occurs differently, according to:

- the operating conditions of the channel, respectively unlined channel or lined channel;
- the desing solution for achievement.

By comparing data under different conditions, is clear the efficiency of this measure in terms of reducing water losses.

The size of water loss into unlined channels is determined by the following elements:

- the soil characteristics: the diameter of particles and their specific surface, porosity, arrangement of layers in the soil profile, level of groundwater;

- geometry of the channel: water depth, width at the bottom, slope;

- delivered liquid characteristics: the suspension content, the liquid viscosity, temperature, flow rate;

- flow regime: unsteady, steady etc.

Size of water losses in lined canals is influenced, besides these four elements by another element, namely, the hydraulic resistance opposed to the infiltration of water. Water loss in lined canals have special importance because they define precisely the hydraulic efficiency action of the lining process and therefore must be well known and correlated with other elements, particularly with grouwater table position.

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Mechanisms and Risk of Embankment Dam Failure

B. Ruedisser

Embankment dam constructions involve a high risk potential. The risk of failure may never be eliminated despite careful design and monitoring measures. In the past, embankment dam failure caused the most severe catastrophic events in the history of mankind. Due to intensifying colonization combined with the effects of climate change, the risk of failure and the potential of damage will both increase. In this article, the main causes of embankment dam failure will be displayed as well as the occurrence. Furthermore, two mayor events of embankment dam failure in the past will be presented.

***Keywords* – Embankment dam failure, failure mechanism, risk of failure**

1. INTRODUCTION

The US- American *National Oceanic and Atmospheric Administration (NOAA)* define dam failure as a catastrophic event caused by the sudden, rapid and uncontrolled release of impounded water.

The causes and the mechanisms which lead to the failure of an embankment dam are different and can be both natural and anthropogenic origin. In many cases the combination of multiple issues leads to failure. To reduce the hazard potential of an embankment dam construction it is necessary to understand the possible failure-mechanisms and the risk potential. For each single construction, the potential failure mechanism may vary and accordingly the design- measures to prevent the construction from failure will be different.

2. CAUSES AND MECHANISM OF EMBANKMENT DAM FAILURE

As a first subdivision the matter of embankment dam failure can be divided into natural and anthropogenic causes. Natural issues can for example be extreme rainfall events, subsoil- failure, landslides, rockfall, avalanches, sediment deposition, earthquakes, falling trees or bioturbation of burrowing animals. Anthropogenic caused

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failure can be related to construction –and design faults, operating errors, imperfect monitoring or maintenance as well as sabotage or war related impacts.

The mechanisms are displayed in **Fig. 1** below.

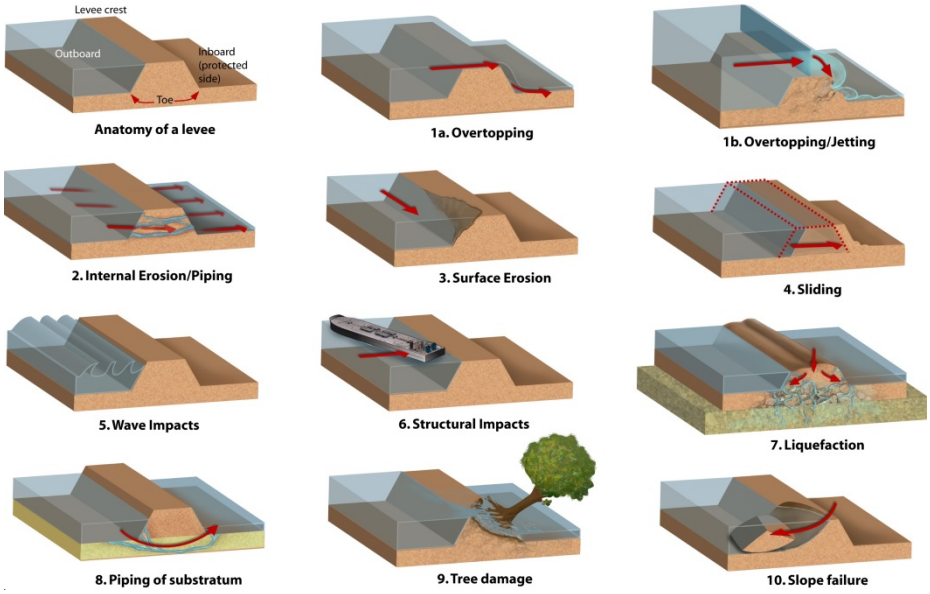

Fig. 1 Mechanisms of embankment dam failure, illustration from Zina Deretsky, National Science Foundation

Whereas the five most numerous failure mechanisms are: [1]

1. Overtopping due to extreme flood events
2. Failure of the dam body due to internal erosion
3. Entire or partly sliding of the dam body
4. Failure of the subsoil
5. Seismic related failure

Number 1-2 can be categorized as hydraulic mechanisms whereas 3-5 are related to geotechnical issues. Overtopping can occur naturally (e.g. extreme flood events, rockfall) or man- made like operating errors or a defective spillway. An initial breach, which leads to overtopping- failure can also result through sudden subsidence, structural impacts or sabotage.

Internal erosion is based on the uncontrolled seepage of water through the dam body with related erosion of the core- material. Therefor the reasons can be defective facing elements, imperfect connection of the facing element with the solid underground, insufficient construction of the filter- elements or biological factors like roots or borrowing animals.

Failure mechanisms with geotechnical relation (3-5) occur, if the acting forces (e.g. hydraulic pressure, ice- load, wave- impact, seismic impacts, traffic load, weight

of trees etc.) exceed the resistibility of the dam body. The origin shape cannot be obtained and sliding failure occurs.

3. RISK AND STATISTICS OF EMBANKMENT DAM FAILURE

Statistically, the risk of failure for an embankment dam with at least 5 years of operation, can be considered as 0,8 x 10^{-4} per year, or even higher. [2]

A distribution of the failure mechanism resulting from an analysis of 160 historical events (Failure of rock-filled- and earth-filled embankment dams) is displayed in **Fig. 2**.

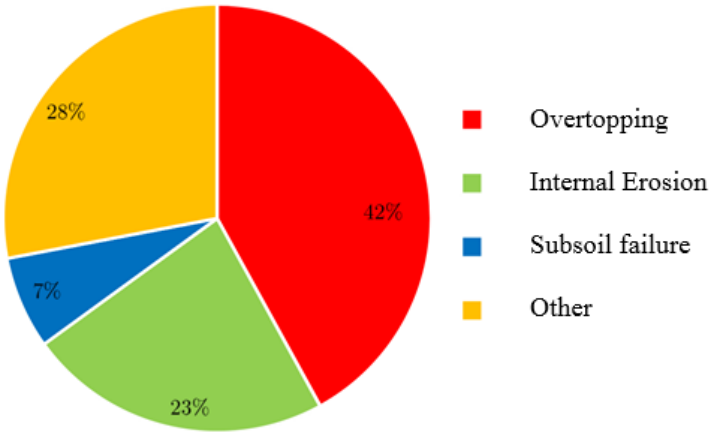

Fig. 2 Distribution of failure mechanisms [3]

Whereas the field „Other" differs further in cracks and construction faults (3 %), external erosion, material failure, mechanical failure (each 2 %).

4. HISTORICAL EVENTS

In this section two examples of historical embankment dam failure events are displayed with the circumstances which led to the disaster. First example is the US-American South Fork Dam which failed in 1889, followed by the Chinese Banqiao Dam- failure of 1975.

The South Fork Dam was an earth- filled embankment dam with an impermeable blanket. After reconstruction in 1962 the original design criteria were elided, the crest was lowered and the width of the spillway was reduced almost by half. [2]

After heavy rainfall with intensities up to 250 mm in 24 hours, the dam was overtopped in the afternoon of the 31th of May 1889. The following flood wave was estimated 12 m high and after only 45 minutes the entire volume of about 20 Million m³ was gone. There were no warnings or evacuations. Due to the catastrophe, more

than 2200 people lost their lives. The main cause for the disaster was the reducing of the spillway capacity.[4] **Figure 3** contains the impounded dam with the spillway during operation (left) and a photograph of the final breach after the failure.

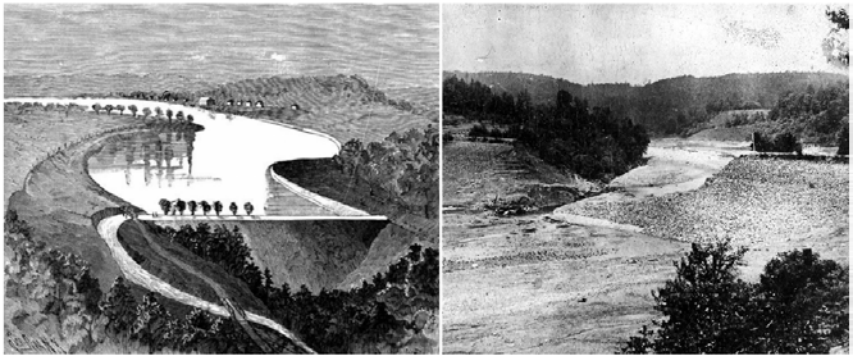

Fig. 3 South Fork Dam before and after the disaster
[source: http://damfailures.org]

The impacts of the flood wave on the city of Johnstown, located 22 km downstream the dam, can be seen in **Fig. 4**.

Fig. 4 Consequences of the flood wave [5]

The Banqiao Dam in Henan province, China, failed on the 8th of August 1975. It was the most severe disaster, which ever occurred due to the failure of a man- made structure. The 24.5 m earth-filled dam with a central clay core was finished in 1952 as part of a flood- protection- and irrigation project at the Huai- River. The impounded volume was about 492 million m³ with additional flood- reserve of 375 million m³. The spillway capacity was originally designed for a 1000- year flood with 1742 m³/s and 12 sluice gates. During construction phase this was seen as too conservative and the number of gates had been reduced to 5. In August 1975 a typhoon caused heavy rainfall with intensities up to 1000 mm in three days. Technical problems and communication difficulties hindered an effective release of the reservoir. [6]

After overtopping the crest for about 30 cm, the dam body started eroding and 600 million m³ discharged in about five hours. The peak of the flood wave is estimated with 78.000 m³/s. [7]

In the aftermath, 61 other dams failed in the region. Only one village was evacuated successfully. [6]

As direct consequences due to the flood wave about 26.000 people lost their lives. Many others died subsequently from famine and epidemics. The total number of victims is estimated with 171.000. Multiple causes are seen as responsible for the disaster. The original design was changed and the spillway capacity was far too low. The available hydrological data was also insufficient for modelling an accurate flood-prediction. Additionally, the natural flood retention areas had been cut off from the river during the „Great leap forward" in the 1960s. [8]

Figure 5 shows the breach of the embankment dam and the flooded downstream area.

Fig. 5 Failure of the Banqiao Dam
[Source: Henan Provincial Water Resources Bureau]

5. CONCLUSIONS

Embankment dams involve a relative high risk of failure as well as comparatively high hazard potential. Failure events in the past display the possible disastrous effects on civilization. Due to intensifying colonization and climate change, the design criteria for flood protection systems may be insufficient in many cases. This may lead to an increasing risk potential. To reduce the risk of failure, it is necessary to understand all possible failure mechanisms for each individual structure. Enhanced design criteria combined with an adequate monitoring system and eventually the modification of existing structures may be a sustainable increase of the current safety status.

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Practical application of wastewater treatment plant modeling and simulation

M. Koleva, A. Hammrich, W. Chen

Along with the software development in the field of Sanitary Engineering, wastewater treatment plant modelling is becoming a powerful tool for tracking and predicting the development of biochemical, physical and chemical processes in wastewater treatment plants. One practical application of wastewater treatment plant modelling and simulation is illustrated in this article by using WEST software. Main subject of this paper is model development and simulation of the wastewater treatment plant in town Schwerte, Germany. All results derived from simulations under dynamic conditions are discussed here. The project aims to develop a model for efficient use of water resources through energy optimization in wastewater treatment plants. The computer simulations of these processes has to be performed using adequate values of the relevant kinetic and technological parameters, associated with certain wastewater characteristics.

***Keywords* – wastewater treatment plant, modeling, simulation, WEST**

1. INTRODUCTION

This paper outlines an overview of building, calibration and use of WEST software developed by Danish Hydraulic Institute (DHI) for simulating the complicate nature of wastewater treatment processes which occurs in sewage treatment plants. The simulation model which was developed and discussed in this paper is created for the wastewater treatment plants (WWTPs) in town Schwerte, Germany.

The research project under the name “Towards the energy-optimized wastewater treatment plant of the future - Development and model-based integration of innovative treatment technologies for transformation processes” (E-Klär), which is discussed here, aims the development of municipal „waste water treatment plant of the future". These plants will not only cater to today’s requirements for nutrient

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removal. In many cases, further processing steps regarding the elimination of micropollutants and/or pathogens will be included. At the same time the WWTP of the future will recover the energy contained in the wastewater as well as material resources as far as possible. Therefore it is necessary to critically analyse best practices and use new technologies.

Municipal wastewater is a potential source of energy and valuable materials. The energy of organic compounds in municipal wastewater is approximately 155 kWh/(PE•a) at a load of 110 gCOD/(P•d). Depending on effluent quality requirements, applied process technology and equipment, the recoverable electrical energy by anaerobic sludge digestion is currently in the order of 7.7 to 18.9 kWh/(PE•a). However, the average power consumption of wastewater treatment plants is 34.0 kWh/(PE•a) on average in Germany. Besides effluent quality requirements, other requirements and claims increasingly affect everyday plant operation and make the use of new technologies attractive.

The research project E-Klär aims to develop strategies in order to use energy and other resources present in wastewater most efficiently and to reduce energy consumption of next generation wastewater treatment plants. A methodology will be developed that will help to transform existing plants into energy-optimised, resource-protecting and economically feasible plants through a plant specific step-by-step process. The strategies to be developed focus on an extensive nutrient removal alongside with a reduction of micropollutants and pathogens.

The project will result in innovative process chains using new methods which are conceived and tested in their interactions with each other and with best practices processes. In addition, innovative approaches to integrated material and energy flow modeling will be developed for the whole wastewater treatment plant, so that after calibration on representative facilities it is possible to compare various wastewater treatment plant concepts in terms of energy, resources and costs. Based on this, practical recommendations for the transformation of current wastewater treatment plants in more energy-efficient future concepts are developed in an interdisciplinary transferable methodology [1].

2. PROJECT KEY ACTIVITIES

«E-Klär» project has three subproject. Within the first (SP I: “Design“) research will focus on technical questions regarding innovative processes and their interdependencies among each other as well as with established technologies. Thereby all relevant process steps of the next generation WWTP will be considered.

The objective of subproject II "Operation" is the development of a modular simulation tool that allows the modelling of the entire WWTP by using a holistic approach. This approach involves the connection of various process steps of the WWTP, the integration of material and energy flows as well as the description of annual costs as time series and as a forecasting tool. By using a holistic and systematic approach to model the entire WWTP including its numerous individual process units and the complex interactions between energy and material flows - it is possible to identify optimisation potentials derived from the interaction of production, distribution and utilisation of energy and other resources. On the one hand, particular

emphasis is given to the modelling of time series of material, energy and cost characteristics of different WWTP concepts over the period of the transformation process (static). On the other hand, emphasis is also given to the dynamic modelling of relevant process variables (forecast) to support the WWTP operation in terms of an energy-optimised plant operation. All in all, subproject II "Operation" forms a link between subproject I and subproject III. The innovative treatment processes of subproject I “Design” will be integrated as individual process modules into an enhanced plant-wide modelling approach of the entire WWTP.

The objective of subproject III (“Transformation”) is to develop and test a sequential approach for deriving and evaluating long-term transition strategies for WWTP in terms of energy efficiency, economic viability, flexibility and lock-in situations. Eventually, conclusion will be drawn to improve strategic investment decisions of municipalities that are willing to improve on WWTP’s energy-efficiency [1].

3. RESULTS AND SIGNIFICANCES

The above described project for optimization of waste water treatment plants includes research and optimization of Schwerte WWTP in Germany. This article describes some of the obtained results so far in the subcategory II "Operation." The project is still ongoing and therefore all results obtained from WEST simulation procedures discussed in this article are intermediary and are to be further developed.

Schwerte WWTP has a design capacity of 50,000 inhabitants and is dimensioned for a maximum inflow of 640 L·s-1. The dry weather flow is 220 L·s-1. A special feature of the system is the two-line operation of the biological treatment with separate sludge cycle. The total hydraulic capacity of the plant is 640 l/s, which flowrate passes sequentially first through mechanical treatment processes - screens, grit chamber and two primary horizontal settling tanks with in service only one. After primary sedimentation, water enters in biological treatment instantaneous aeration tanks located in two main lines. The water enters first in anoxic reactor together with both nitrate internal recirculation and external activated sludge flows. This pre-anoxic zone create complete anaerobic conditions in the second anaerobic reactor. Following three oxygen/oxygen-free areas depending on the oxygen supply in these sections and two aerobic nitrification sections. The primary sludge flowrate in the range of Qrs = 90 m3/d enters the gravity thickener where the concentration of dry solids reaches 2-2.5% and the thickened sludge flowrate - 52.2 m3/d. Activated sludge at a rate 300 m3/d enters a mechanical thickener, where together with thickened primary sludge flows to an anaerobic digester. Dewatering of the stabilized sludge is performed with centrifuges.

To feed the model and simulate Schwerte WWTP all data has been used from different measurement campaign regarding the quantity, temperature and composition of the waste water after each technological step on the plant. All registered sludge quantitative and qualitative parameters - flow, humidity, organic matter content, etc. has been used to simulate the sludge line operation. All technical parameters related to each facilities was calibrated.

First, technological scheme of the plant has been implemented on WEST platform with all the necessary technology and communication parameters (**Fig.1**). All plant equipment and every facility can be design in WEST by selecting the

relevant icons and their model associations and to be placed on the program layout. This allows easily and affordable to construct a flow chart by selecting the appropriate icons and interconnecting them with the appropriate process connections. Biological stage in Schwerte WWTP includes two activated sludge tank lines, each divided into 7 sections, which provide relevant technological conditions of biological treatment - anaerobic, aerobic and oxygen-free. Schwerte WWTP provides also a biological phosphorous removal. On WEST platform has been implemented each of these aeration tanks and has been designed with seven independent icons (**Fig.1**). This allows very accurate monitoring of all ongoing biochemical processes which occur in the activated sludge tanks and very fast location of all problematic issues that could emerged during operation time.

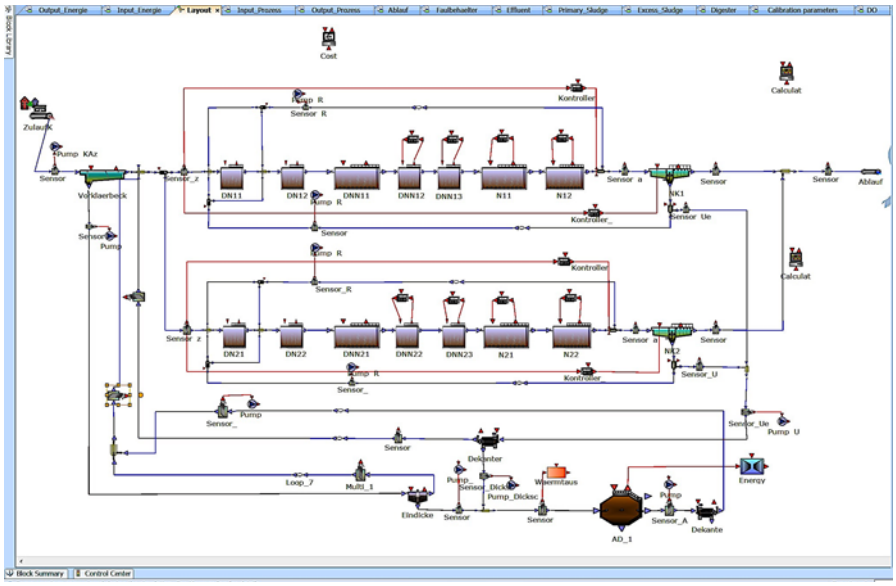

Fig.1 Basic layout which represent the wastewater treatment plant flow-chart in Schwerte, Germany, implemeted in *WEST*

After entering all technological parameters and building up all plot windows to visualize the obtained results in every step of the WWTP, a simulations can be run. In WEST software all simulations can be performed in a certain sequence. First steady state simulation mode has to be start, which practically describe the behavior of the plant immediately after its first start up in operation. In this mode, the input parameters (water quantities and concentrations of major pollutants) do not change and accept permanent, unchanging values. Dynamic simulations are conducted after reaching a steady state operation of the plant. Here simulation processes correspond to the behavior of the plant in real terms. In dynamic mode every step of wastewater treatment processes can be traced at any point at any time [2].

After calibration plant simulation model is ready to be tested under different operating conditions to optimize the cost with providing an efficient and high level treatment at same time. Furthermore, the model can be used to test the effect of any future

plant extensions, or to follow the development of wastewater treatment plant processes at various initial conditions, such as hydraulic load changes and different toxic contaminants that could be finding out in WWTP influent. This allows an adequate operational decision to be taken under extreme operating conditions at lower costs.

At this stage of the project conducted simulations of Schwerte WWTP are for a period of one year. On **Fig.2** and **Fig.3** the results obtained from the simulations in terms of some basic effluent parameters can be followed and compared. The simulations data are indicated with a solid line and the actual measurements data observed on the WWTP effluent has been marked with gray squares. Figure 3 shows the suspended solids data obtained from the simulation which are slightly higher than the actual measured results on the plant. This is partly due to the fact that at this stage of the project a ferric chloride installation for chemical phosphorus removal is still not implemented in WEST.

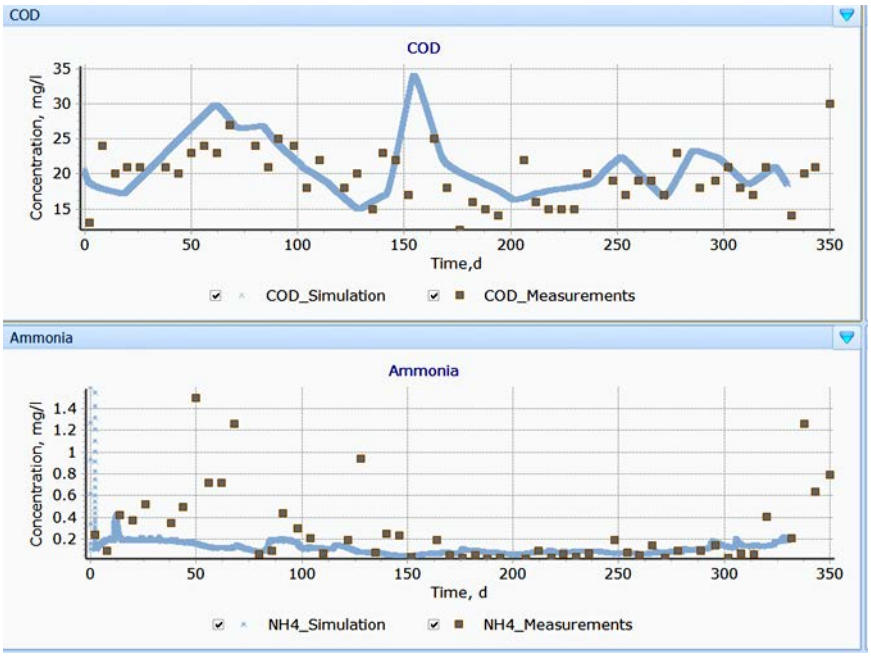

Fig. 2 Monitoring data from Schwerte WWTP (gray squares) and data obtained from dynamic simulations (solid line) in the effluent in terms of COD and Ammonium nitrogen (mg/l)

On **Fig.4** can be seen and compared actual data measurements for the activated sludge growth and suspended solids concentration in it and analogous data obtained by dynamic simulation for one year. Again, the model is fully calibrated and the results obtained from simulations overlaps with the real data measurements received from the treatment plant.

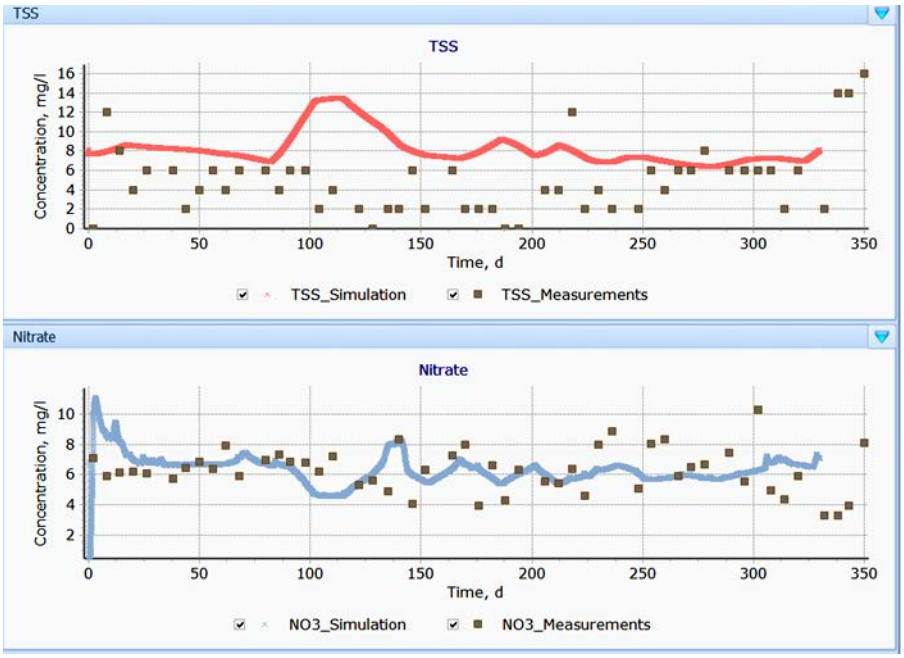

Fig. 3 Monitoring data from Schwerte WWTP (gray squares) and data obtained from dynamic simulations (blue line) in the effluent in terms of TSS and Nitrates (mg/l)

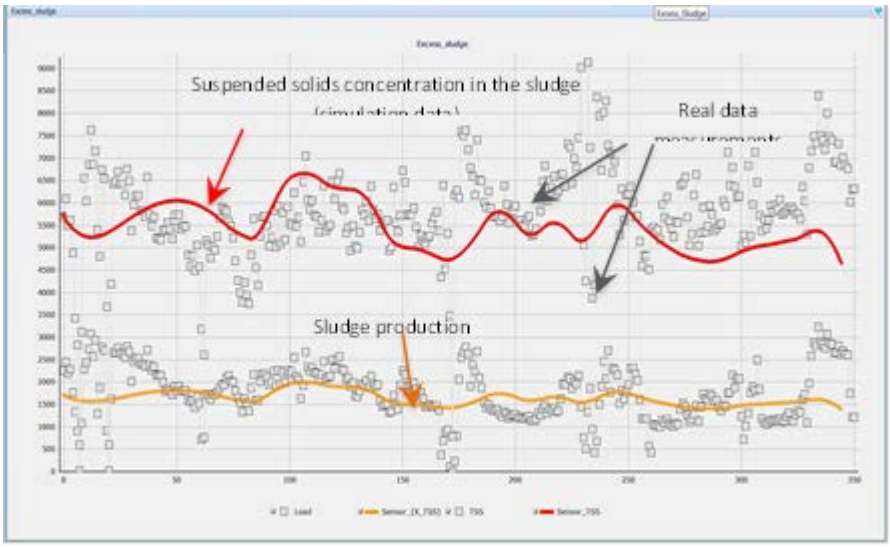

Fig.4 Monitoring data from Schwerte WWTP (gray squares) and data obtained from dynamic simulations (in red and orange) for the TSS (mg/l) in the activated sludge and its production in the WWTP (kg/d)

As it is well known, proper design and adequate operation of the digesters are crucial to achieve the necessary degree of sludge stabilization, obtaining a sufficient amount of biogas and effective use of these facilities in general. For this purpose a number of technological, chemical, biochemical, physical and microbiological parameters specific to each phase of biochemical processes occurring in the digester, should be kept in range. At high doses of sludge supplied in the digester large deviations in pH values from the optimum for the process are observed, due to the increase of fatty carboxylic acids in the acid phase. In such a situation the operation of these facilities may be impaired fatal and they go out of operational control. Therefore it is essential to monitor continuously all ongoing biochemical processes and the parameters of the final products obtained by the decomposition of organic matter in sediments. With proper calibration of the model, these processes can be continuously monitored, and to simulate different situations on which to seek the optimal operation of these facilities. In WEST software an IWA model for processes simulation of anaerobic sludge stabilization (Anaerobic Digestion Model – ADM), has been used.

The digesters behavior in dynamic mode through visualization of the values of the main technological parameters characterizing the processes at the facility, can be seen in **Fig.5.** The graphics shown in **Fig.5** represents the content of methane gas, pH and individual products derived from alkaline decay in various stages of sludge decomposition. Such processes visualization which take place in the digesters at any point of time, allows for extremely improving the operational control of these facilities in case of break down or diversions from their normal operation. Moreover, simulating the processes of sludge decay for any particular time, allows the WWTP operator to anticipate and optimize all occurring biochemical processes, which in turn lead to the most energy - efficient operational control of these facilities.

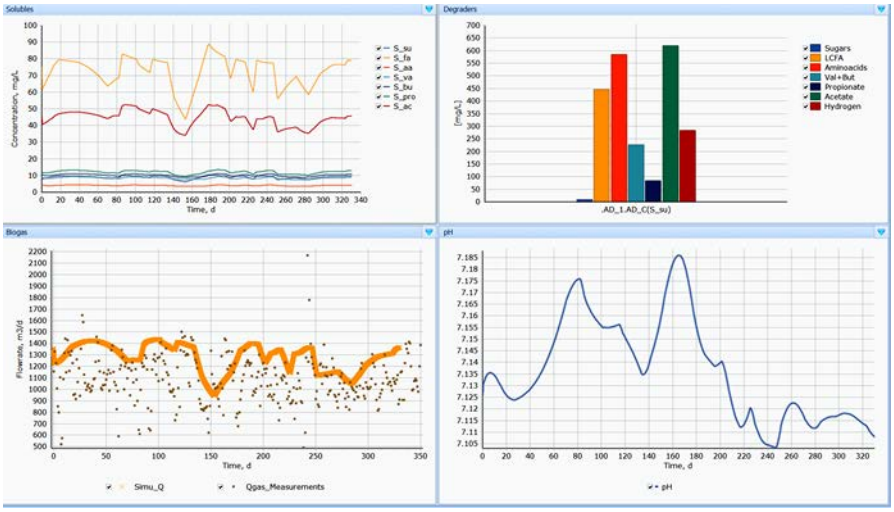

Fig.5 Dynamic simulation data in the anaerobic digester fro a period of one year
Up on the left – solubles concentration variations in the Digester; Up on the right – Variations of the different compounds in the digested sludge; Down on the left – Biogas production, m^3/d; Down on the right – pH variations in the Digester

4. CONCLUSIONS

Modern commercial software for simulating processes in wastewater treatment plants involve complex mathematical models that describe most of the biochemical, physical and chemical processes in the plants. Using computer models, under certain conditions, all processes can be described adequately, allowing prediction of their development under different operating conditions. The user does not require programming knowledge but a good understanding of the wastewater treatment technologies for correct interpretation of the data obtained through computer simulations results and taking appropriate operating decisions.

This article presents a simulation platform on Schwerte WWTP using WEST software. As shown, computer modeling makes it possible to achieve processes optimization on the treatment plant, without the need to do so by the well-known approach "trial and error" in operating conditions. In the existing wastewater treatment plants, by conducting various simulation scenarios can be established causes of improper conduct of the relevant processes and to carry out the operational adjustments.

5. ACKNOWLEDGMENTS

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Urban Storm Water Runoff Pollution – An Overview and Recent Trends

Dan Rădulescu, Gabriel Racovițeanu, and Adriana Pienaru

***Abstract* – The rise in population moving to urban centers and the land development to support this growth creates additional stress on natural resources, including the aquatic ones. In many countries that have made significant strides in addressing the point source pollution through massive investments in sewer collection systems and wastewater treatment plants, the quality of the receiving waters still shows signs of impairment due to diffuse pollution. In order to address these remaining challenges, especially in controlling the quality of urban storm water runoff, new approaches, closer to natural systems, are implemented. These new approaches and techniques include Low Impact Development, Green Infrastructure, Sustainable Urban Drainage Systems and Smart Growth, among others.**

***Keywords* – Diffuse pollution, green infrastructure, low impact development, urbanization.**

1. INTRODUCTION

In the late part of the 20th and early part of the 21st century, despite significant success in controlling water pollution from various sources, including domestic and industrial wastewater, in many industrialized countries (e.g. in United States, Europe, Australia) the health of the receiving waters showed signs of continued impairments [1]-[2]. While significant investments have been made in building a better infrastructure to control the point sources through better conveyance systems, more efficient wastewater treatment plants, the non-point sources and the diffuse pollution [3] proved to be a bigger challenge and a far more difficult task to address [4]. This paper will attempt to identify the challenges posed by the urban storm water runoff pollution, the efforts made throughout the world to address it, and the approaches,

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methods implemented to provide for a real change that will result in the adequate protection of the receiving water resources.

2. OVERVIEW OF THE CHALLENGES

2.1 Early attempts to quantify and characterize the pollution impacts

In its 2000 National Water Quality Inventory report, United States Environmental Protection Agency states that the "leading causes of impairment reported by the states in 2000 include bacteria, siltation, nutrients, and metals (primarily mercury). Sources of impairment include agricultural activities, hydrologic modifications (such as channelization, dredging, or flow regulation), municipal sources, and urban runoff/storm sewers." In particular, the report identified that out of the total number of river and stream miles assessed, about 39% showed impairments for one or more beneficial uses. Similarly, out of the total acres of lakes, reservoirs and ponds assessed, about 45% showed impairments for one or more uses. In case of the coastal resources evaluated, out of the total surface of estuaries assessed, 51% showed impairments.

Fig.1 Urban pollution impacts on water resources

Historically, a number of studies, such as the Nationwide Urban Runoff Program (NURP), conducted in United States in the 1980s, showed that polluted urban runoff was a significant contributor to water quality impairments [5]. Water quality impairments are defined as waters too polluted or degraded to meet the water quality standards set by the states. Under the framework established by the Clean Water Act in United States, water quality standards represent the legal framework to control discharges of pollution into surface waters and consist of designated uses, such as recreation or drinking water supply, water quality criteria to protect designated uses, antidegradation requirements and other policies. Water quality criteria can be numeric, as in the highest level of contamination

concentration that is allowed in a water body for a particular material, or it can be narrative, as in “no oily sheen” visible on the water. Antidegradation requirements have the goal to maintain the gains obtained in improving the quality of receiving waters, resetting the bar to the highest level of protection achieved over time.

In addition, the NURP study identified the constituents that contributed mostly to the pollution, such as heavy metals, polycyclic aromatic hydrocarbons, pesticides, plasticizers, coliform bacteria. Other authors also identified urban storm water runoff as a significant source of contamination to separate storm sewer systems and receiving waters, finding that 41% of the storm water sources assessed showed moderate to high toxicity [6]. Other studies demonstrated that the phenomenon of wet weather pollution is not limited to discharges to separate storm drain systems, the same problems are encountered in case of the discharges of urban storm water runoff through combined sewer systems [7]. In case the capacity of the combined sewer is exceeded the resulting sewer overflows are similarly impacting the quality of the receiving waters and the contribution of pollution from urban runoff is significant [8]. Recent studies show that the phenomenon of urban runoff pollution is not characteristic only to industrialized countries but to countries undergoing currently significant economic growth, such as China [9], Korea [10], and others, such as Iran [11].

2.2. Characterization of major sources of pollution

The changes can be quite dramatic and they may lead to increases of flows discharged and frequency of floods. In some instances, the increase in peak flow discharge may be as high as five times larger between pre and post urbanization [16]. However, these hydrological changes are combined with biological and geomorphological impacts, together with a decrease in the volume of water that resupplies the groundwater aquifer [17]. This has a major influence on the availability of clean drinking water supply in many urbanized areas of the world with significant consequences in providing healthy water to the inhabitants and results in significant costs to arrange for alternative sources.

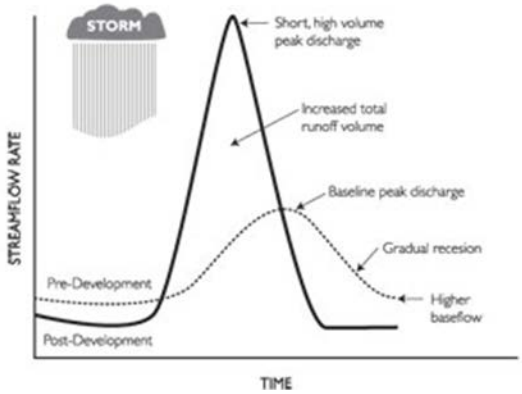

Fig.2 Impacts of urbanization on streamflow (Schueler, 1987)

The impervious urban surfaces, such as streets and highways, sites of industrial or construction activities, are sources for a number of pollutants and stressors that can be discharged into the receiving waters either directly or indirectly. The 2000 National Water Quality report identified bacteria as a leading source of impairments with urban runoff/storm sewers as a major contributor. Metals carried off from urban impervious areas by the storm water runoff have been also found as a major contributor to the impairments. Furthermore, due to high vehicular traffic, in addition to heavy metals, polycyclic aromatic hydrocarbon compounds have been identified as a significant source of pollution. In addition, even residential areas, that in general are considered “cleaner”, have proven significant sources of nutrients, pesticides and pathogens.

2.3. Conveyance: a tale of two systems

Increased imperviousness creates preferred pathways to wash off contaminants accumulated on urban surfaces rapidly into the municipal conveying systems.

While the wastewater from residential and industrial/commercial sources is relatively easy to collect, convey and treat, the storm water runoff remains a significant challenge not necessarily due to collection and conveying problems to address flooding issues but also due to few treatment options, especially at peak events. The conventional method of providing the collection, conveyance and treatment of wastewater and storm water runoff through a combined sewer system has its advantages; however, it also shows its limitations during peak storm events that may overwhelm both the capacity of the conveyance system and of the treatment works [18]. At peak events and in absence of adequate retention capacity, the combined sewer systems allow significant amounts of partially treated or untreated wastewater of poor quality to be discharged in the receiving water [19]. In addition to raw wastewater released in the case of combined sewer overflows, the same group of contaminants and stressors, such as pathogens and toxic pollutants, found in the releases from the separate storm sewer systems, contribute to the impairments of the receiving waters.

On the other hand, where separate sewer systems are available to convey the storm water, while very efficient to transport the runoff and discharge it rapidly to the receiving water, most of the time the polluted flow is treated moderately or, in most cases, not treated at all. Studies showed that despite the existence of an infrastructure consisting of one or the other sewer systems or a combination of both [20, 21], the challenges presented by the polluted urban runoff are real and still difficult to tackle.

It is interesting to note that the researchers recommend a thorough analysis of the performance of both options, from the technical, environmental protection and economic point of view, before making a decision which system is the better choice for implementation under the particular urban conditions.

3. METHODS TO ADDRESS THE CHALLENGES POSED BY THE DIFFUSE POLLUTION AND TRENDS IN IMPLEMENTATION

Based on the demonstrated continuous degradation of the water resources due to wet weather discharges and in response to the challenges posed by the diffuse pollution [22], the European and American legislations have been updated to address these continuing threats posed by the non-point source pollutants. Water quality directives from the European Union include stringent limits for various constituents that contribute to the diffuse pollution and in United States the 1987 reauthorization of the Clean Water Act provided an impetus for addressing the quality of discharges from the separate storm sewer systems. These laws also outlined a framework for addressing other sources of non-point pollution.

Following in the footsteps of the Clean Water Act, Section 438 of the U.S. Energy Independence and Security Act of 2007 (EISA) instructs federal agencies to "use site planning, design, construction, and maintenance strategies for the property to maintain or restore, to the maximum extent technically feasible, the predevelopment hydrology of the property with regard to the temperature, rate, volume, and duration of flow" for any federal project with a footprint that exceeds 5,000 square feet (465 m2). Australia also issued a new Water Quality Policy in 2003, and it was updated periodically, that addresses, among other topics, the quality of storm water runoff discharges and other non-point source activities that may have deleterious impacts on the receiving waters.

Fig.3 EcoRoof Portland, Oregon

In the meantime, there were a lot of efforts made to address the technical, jurisdictional and policy challenges put by the difficult problem of non-point pollution. A significant amount of effort has been spent in obtaining background data, research, modelling, and trying to come up with adequate answers to this complex problem. All these efforts converged toward a paradigm shift in addressing the storm water runoff problem in an urban environment. From a conceptual point of view, there is agreement that storm water must be addressed in the larger

framework of Integrated Urban Water Management (IUWM) since the pressures of a growing population migrating toward urban areas, depletion of water resources, higher energy demands, climate change, etc., are only increasing with time [23]. The IUWM provides a platform to address all aspects of planning, designing, and managing urban water systems integrating fresh water, wastewater, storm water, and solid waste, and enables better management of water quantity and quality.

More specifically, when addressing the urban storm water management issues in a comprehensive and integrated manner, a number of techniques and new methodologies have been developed. These methodologies take in consideration other aspects of urban development such as growth, redevelopment, flood protection, pollution prevention, groundwater resources conservation and augmentation, overall water quality protection of the receiving water against degradation. However, they all lead to a new trend, to a change in the philosophy of how to address the challenges of storm water management in an urban environment and beyond. Storm water must be seen as a valuable resource and treated as such. Many of the concepts contained in these new techniques focus on a holistic approach when designing the storm drain systems, to address adequately both the flood protection and the pollution control and treatment necessities [24] and to preserve the post development hydrology at the pre development levels. It also points to the need to start at the source and at the initial phases of the planning process [25]. Many of the concepts, such as Sustainable Urban Development [26], Green Infrastructure [27, 28], Smart Growth [29] point out to the role and the need to address and integrate adequate storm water management techniques in the planning and development of the future urban areas. These new approaches also offer guidance on how to retrofit the existing urban infrastructure to a more environmentally friendly condition.

As early as 1998, the European Union initiated a framework of action that promoted “more holistic, integrated and environmentally sustainable approaches to the management of urban areas” [26]. In the European Union “Green Infrastructure is specifically identified as one of the investment priorities [and] … is recognized as contributing to regional policy and sustainable growth in Europe and facilitating smart and sustainable growth through smart specialization. GI solutions are particularly important in urban environments in which more than 60% of the EU population lives.”[28]

Techniques such as Water Sensitive Urban Design [30], Low Impact Development [31], Sustainable Urban Drainage Systems [32], Low Impact Urban Design and Development [33], go to the more specific aspects of storm water management and storm infrastructure design and demonstrate how they can be integrated in a holistic urban water management. Many of these approaches share common features and put an emphasis on natural features such as avoiding or disconnecting as much as possible the impervious areas to allow for storm water to infiltrate close to the source using vegetation, permeable pavements, rain gardens, filter strips, green roofs and similar techniques. In order to lower the volume of storm water released, rain barrels or rain tanks are a preferred alternative, together with preserving and expanding the urban green areas and vegetative canopy. These methods move away from traditional end-of-pipe, single site treatment approach to

a distributed, closer to the source treatment train mode that relies greatly in preserving and using the existing natural features.

4. CONCLUSIONS

It can be concluded that despite the recent progress made in controlling and treating urban point source wastewater, it became clear that the non-point, diffuse pollution from the urban areas is still impacting significantly the quality of the receiving waters. No matter if the urban runoff is discharged through a separate storm drain system or as an overflow from a combined sewer system, the challenges are real and the need to address them is pressing. Continued urbanization, the negative aspects of climate change, the dwindling of adequate water resources, all lead to the conclusion that there is a need for a change in the paradigm of urban water management. This change involves different approaches in how water is managed, and especially how storm water runoff is viewed and treated as a resource instead of a nuisance. The choice between using a separate conveying system or a combined one must be done after careful analysis, however, no matter which system is used for conveying and treating urban storm water runoff, there is a need to consider it as an asset, while assuring proper protection against floods. The conventional approach of transporting expeditiously the storm water runoff away from the urban areas via separate storm drain systems, while still valid from the flood protection aspect, still requires significant investments and maintenance costs and typically does not provide any type of treatment, in the system or at the “end of pipe.” On the other hand, a combined sewer system which accepts certain levels of storm water runoff flows that are treated at the end of the system, has its own challenges when the capacity is reached and exceeded. No matter what system is used, the problems and challenges remain, significant investments are required, and maintenance costs tend to be very high.

Fig.4

The necessity to preserve precious water resources, protect against floods and against degrading valuable natural resources, the intense urbanization pressures, all in a period of clear climate changes already impacting many of the human activities, show that there is a clear need for a different approach. The answer lays in using an integrated, holistic strategy in urban water management, including specific techniques to address the quality of urban storm water runoff such as smart growth, low impact development, green infrastructure.

The other significant lesson learned is that the sooner in the planning process these new approaches are considered, the better it is. The design process should follow a treatment train methodology, focusing on retaining safely and infiltrating near the source as much runoff as possible, disconnecting the connected impervious areas. The planning process should start close to the source, at the individual lot level, progress to analyzing and implementing the various techniques at the project level, then sub-basin and watershed level with the aim to preserve the post development hydrology at the pre-development levels.

Traditional engineered solutions, which have proven their performance, are not precluded from use, under the new paradigm. While we move away from the traditional model of end-of-pipe, single site treatment, there are many instances where traditional engineered solutions have their role, and they augment the range of practical solutions available to practitioners in the field. Sedimentation basins, filters, detention and retention basins, and many other tools should be part of the menu of available practices that the specialist can use, depending on the particular situation.

Many countries and municipalities already recognized the need to change the paradigm in addressing the complex issues of diffuse pollution and are actively implementing the techniques that mimic natural processes, providing for the safeguard of limited precious natural and economic resources well into the future.

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Figure 2 Source: Schueler, TR. *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMP's* 1987; Metropolitan Washington Council of Governments, Washington D.C.

Role of polymer components of mixture as Non-Newtonian fluids

C. Moskalova, O. Popov, E. Rozhnyuk

***Abstract* – Analyzed are viscosity functions for model mixes – the emulsions of three polymers. Effective viscosity of the mixes is described with sufficient exactness by power rheological equation of Ostwald-de Waele. Parameters of the equations were determined by the data of rheological measurements for each mix according the design of experiment. To analyze the rheological behavior of the model mixes used the differences of logarithms of viscosity functions when mix proportions of emulsions are varied.**

***Keywords* - Concentration of polymer, difference of function logarithms, effective viscosity, emulsion, model mix, Ostwald-de Waele model.**

1. INTRODUCTION

Rheometry creates a base of information on the flow of Non-Newtonian fluids [1]. The results of the analysis and synthesis of information - part of rheology, which solves problems in the various fields of science, technology and medicine. As an analysis tool of structure of Non-Newtonian fluids the rheometry is used in science of building materials with the 50-ies (P.A. Rehbinder, N.B.Urev, N.N. Kruglitsky, Y.P. Ivanov [2-3]). For more than twenty years to assess the impact of compounding and technology (CT-factors) on the parameters of rheological models used experimental-statistically (ES) modelling and methods of computer materials science [4-6].

For compositions with different polymeric matrixes, depending on the factors for CT-factors by the model were analyzed effective viscosity η (Pa·s) at a constant shear strain rate γ' (s^{-1}) usually, $\gamma' = 1$, as η_1 = K – parameter of one of the most commonly used simple rheological models, the law of exponent . This is equation of Ostwald-de Waele (1), which, after taking the logarithm of (2) becomes linear in the parameters.

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$$\eta = K^{m\gamma'} \tag{1}$$

and also, $$\ln\eta = \ln K + m \cdot \ln\gamma' \tag{2}$$

In parallel, by the ES - model was described and analyzed parameter m <0 - the pace of destruction structured Non-Newtonian fluid under increase γ' .

Function (2) is make on the obtained on rotational viscometers values of effective viscosity by a change of shift speed and it well describes of building mixes with the maximum grain size of up to 2 – 2.5 mm in small and medium range $\gamma' \leq 20$ s^{-1} (it is typical, in particular for the mixing and pouring of solutions of dry building mixes (DBM)). The error of inadequacy in the area of γ' = 1 does not exceed 5%. This allows us to consider (1-2) as a model, admitting those or other engineering calculations and transformation.

The method of comparative analysis of variability under influence of CT - factors of viscosity using the differences of described models of logarithmic functions (2) is devised. One of the functions – for «E» of the mixture, regarded as a reference. For other mixtures «U» is defined by the represented straight functions (3) differences of the logarithms.

$$\exp([\ln K_U - m_U \ln\gamma'] - [\ln K_E - m_E \ln\gamma']) = \exp(\ln\eta_U - \ln\eta_E) = \eta_U / \eta_E \tag{3}$$

The methodology has applied in the multivariate comparative analysis of the influence components on the function of viscosity of model mixes for plaster from DBM in the early stages of hardening.

2. Experiment Description

The effective viscosity in the shear rate range of 0.045 to 8.406 s^{-1} was determined by for polymer emulsions. Polymer emulsions (index P) contained 3 polymer component in the saturated solution of $Ca(OH)_2$, which dosage (factors X, by weight parts per 100 w.p. solution) ranged at 3 levels (Table. 1) in accordance with a 15-point's plan of experiment B_3 in ranges wich produced in Ukraine DBM for plastering. The minimum concentration of polymer is 4.17% by weight solution $CaOH_2$; this mixture is considered as the reference «E». The maximum concentration of three polymers is 12.85%.

The viscosities of each model mixture were described by functions (2). The values of ln K and tempo | m | for each of the 15 types of the mixes allowed to build non-linear three-factor ES-model of dependency of parameter rheological model on the composition of the emulsion. Models describe field of rheological parameters in 3 coordinates of compositions.

The minimum values of effective viscosity for test solutions is correspond to the low level of polymer concentration (x_i = -1), while the maximum is correspond to the upper level ($x_1 = x_3 = 1$, $x_2 = 0.8 \approx 1$).

Table 1 Levels of varying the polymer components

Components		X_{imin} $x_i = -1$	X_{i0} $x_i = 0$	X_{imax} $x_i = +1$
X_1	Redispersible polymer powder Vinnapas 5034 (V) – ethylene-vinyl acetate copolymer (brand *Wacker Polymer Systems*)	3.5	8	12.5
X_2	Hostapur OSB (H) – multifunctional surfactants based on high molecular sulfonate olefin, sodium salt (*ShinEtsu*)	0.05	0.15	0.25
X_3	Tylose MH60010 (T) – methylhydroxyethylcellulose, water-soluble, nonionic cellulose ethers (*ShinEtsu*)	0.8	1.4	2

Within the field η_1 of emulsion increases 18 times. This is due, firstly, with the increasing concentration of polymer powders in 2.5-5 times by weight and with decrease of specific volume $Ca(OH)_2$; secondly - with increasing proportion of physically bound water (by dispersing, swelling and dissolution of polymer powders).

3. RESULTS AND SIGNIFICANCES

Effect of concentrations of the polymer components on lnK and |m| emulsions based on calcium hydrate. The total field of viscosity at a shear rate $\gamma' = 1$ in the coordinates of contents of the three polymer powders shows in Fig.1a. Along the direction of the average gradient of this field (dashed line on the diagonal of prescriptions cube), the viscosity η_1 increases by 18 times; the increasing almost proportionally.

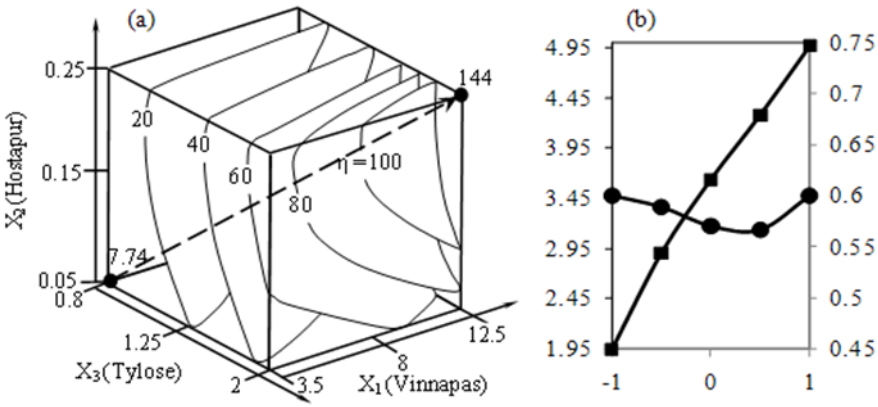

Fig.1. Field of emulsion viscosity by $\gamma' = 1\ s^{-1}$ in coordinates of polymer concentration (a) and their generalizing impact (b) on lnK (■) and |m| (●)

Figure 1b shows the integral influence of all three polymeric components on η_1 and the pace of destruction. Their individual influence in the area of maximum and minimum lnK is shown single-factor dependence to Fig.2. It is seen that the greatest influence on viscosity when $\gamma' = 1$ provides the introduction of water-holding Tylose; with an increase in a concentration of 0.8 to 2 w.p. the viscosity in the zone of minimum increases up to 6 times, and in the zone of maximum of 11 times, indicating a sufficiently "strong" frame which produced by cellulose ethers.

Sensitivity lnK to the amount of Vinnapas slightly lower; this parameter varies only in 2 - 3 times in maximum zone, and in minimum zone; viscosity increase is occur due to strong membrane formed by dispersing. Hostapur slightly increases the viscosity of the mixture; its plasticizing effect does not explicitly detected.

Figure 2 shows the extreme nature of the influence of content of Vinnapas on the $|m|$, most intensively destroyed under shear deformations a mixture with an average content V. Above this concentration Vinnapas begins to rapidly stabilize the mixture. Increased dosage Hostapur promotes destruction of the structure, and most intensively in mixes with a high viscosity.

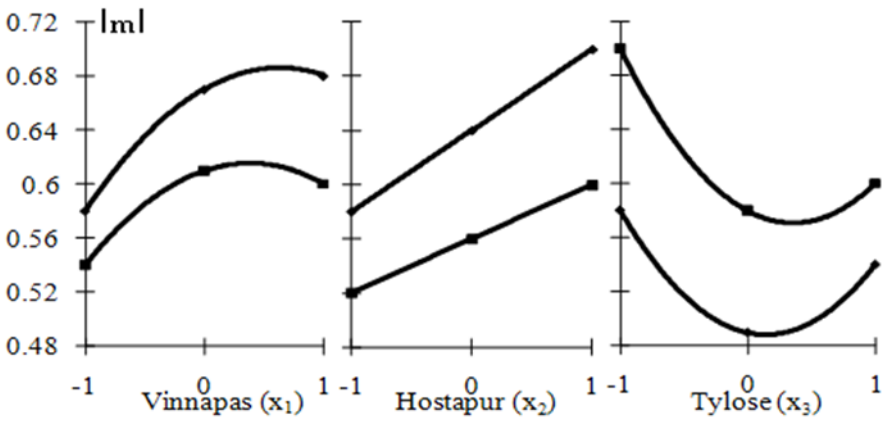

Fig.2. The single-factor dependency of concentrations of polymer in the emulsion from the rate of destruction $|m|$ and at zone of minimum and maximum

Fundamentally important results obtained for methyl cellulose - an increase in its concentration to X_3 = 1.25 w.p. plays a role of reinforcement and the pace of destruction of mixture structure is significantly reduced, and with increasing concentrations of | m | it remains constant or increases slightly.

In this way, it can be considered as a reliable conclusion that for polymer emulsions (to indicated in Table 1 dosage ranges.) the main factor determining the effective viscosity η_1 and the rate of destruction | m | is the content of Tylose.

The transformation of the emulsion viscosity functions during the transition of the concentrations polymer components from lower to the upper level. Extended information about the role of the components of the polymer

emulsion was obtained by using special methods of analysis of logarithmic viscosity function (2). In contrast to the previous analysis, it allows you to visualize the nature of viscosity as a processes associated with time through a gradient speed of deformation.

Logarithmic function which used in analysis are obtained from the shown in Table 2 ES - models by substituting x_i = const (i = 1, 2, 3). Discusses five boundary functions: emulsions with minimum (4.17%) and the maximum concentration (12.85% by weight) of the three components (reference P{E} and P{VHT} respectively) and with an extremely high content of a component (P{V}, P{H} and P{T}). The calculation of algorithm is reflects in the Table 3: it has five lines which correspond to the five functions of the boundary viscosity (2). Graphs of five logarithmic function P{U} (where U – E,V,H,T and VHT) has shown on Fig. 3 a, b.

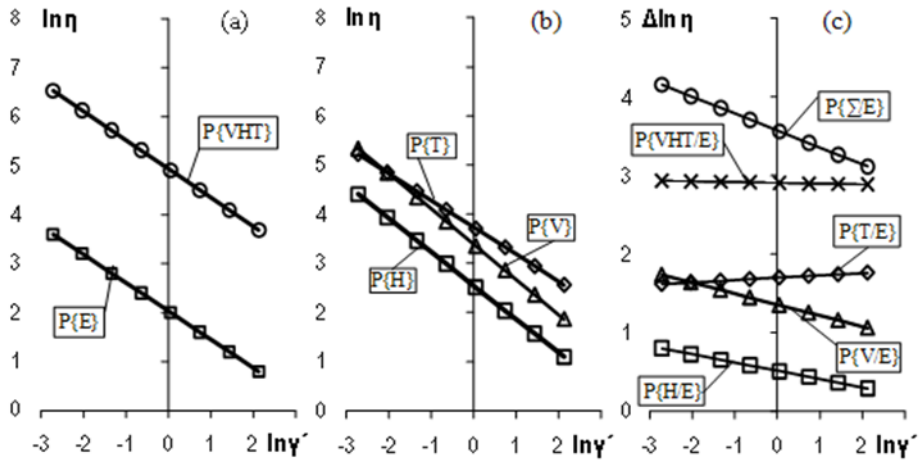

Fig.3. The logarithmic functions of emulsion viscosity (a, b) and the augmentation of logarithm of viscosity in relation to the reference emulsion P{E} (c)

The bottom line in Fig. 3a is a reference. Effective viscosity varies with shear rate increasing from 2.4 to 46, and falling almost 20 times. The top line corresponds to the maximum concentrations of the components. The viscosity decreased from 871 to 42.3 Pa·s - fall in 20.6 times.

Increasing η_1 the mixture P{VHT} compared to P{E} approximately 18 times this is presumably because the chemical reactions in the system P does not occur, it can be assumed that the reference emulsion has a mosaic structure with weak intermolecular bonds, which at higher concentrations of polymers are enhanced, but with growth γ' failure mechanism remains unchanged, due to the heterogeneity of the structure. Graphs of viscosity of emulsions with an increased concentration of one of the components (Fig. 3b) lie between the functions in Fig. 3a. Changes $\ln\eta(\gamma')$ in relation to the reference model mixture reflect graphs in Fig. 3c.

Table 2 Models of viscosity emulsion and augmentation of viscosity relatively lowconcentrated of the reference emulsion

Conditions	Model	Number of formula
The emulsion viscosity with different content of polymer		
The minimum concentration of polymers	$2.03-0.58\ln\gamma'$	(4)
The maximum concentration of polymers	$4.94-0.59\ln\gamma'$	(5)
The maximum content of Vinnapas (x_1 = 1)	$3.39-0.72\ln\gamma'$	(6)
The maximum content of Hostapur (x_2 = 1)	$2.55-0.68\ln\gamma'$	(7)
The maximum content of Tylose (x_3 = 1)	$3.73-0.55\ln\gamma'$	(8)
Relative augmentation of emulsion viscosity		
(5) – (4)	$2.91-0.01\ln\gamma'$	(9)
(6) – (4)	$1.36-0.14\ln\gamma'$	(10)
(7) – (4)	$0.52-0.11\ln\gamma'$	(11)
(8) – (4)	$1.70-0.03\ln\gamma'$	(12)
(10) + (11) + (12)	$3.58-0.21\ln\gamma'$	(13)

The biggest increase (3.6 times) was observed with increasing concentration of Tylose (while maintaining the two other components to the model level), this effect (12) does not depend on γ' (graphics is parallel graphics P {E}, Δ|m| for P {T} little different of zero). Increased viscosity connected to the increase in the proportion of physically bound water, while maintaining the mosaic structure and its destruction under shear deformations. Vinnapas, with small γ' is increases viscosity like Tylose, with increasing γ' structure collapses faster than the model blend. Hostapur gives a smaller increase η_1 (equal to exp (0.52) = 1.7 times) concerning to the model, which declining with increase γ'. Simultaneous administration of increased concentrations of components the increase (5) for P {VHT} and corresponding effective viscosity is lower than in the case of a hypothetical increase P {$\sum$ / E} from adding the individual effects (13); hypothesis of additivity is not confirmed.

4. CONCLUSIONS

In studies of multicomponent polimermineral mixtures, which when mixed with water, transporting, pouring and moulding begins intensive hardening mineral binders, useful information could be given by model liquid phase systems - emulsions. In their study, it is advisable to employ methods rheometry, the results of which are summarized and analyzed using methods of computer materials science. For systems, effective viscosity of which with sufficient engineering accuracy, describes the exponent functions of Ostwald-de Waele, useful analysis of the differences of logarithms functions at change of structure of emulsions.

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The Non-Linear Analysis of the Seismic Response for Dual Structures, Subjected to Equivalent Static Loads

Tepes Onea Florin, Caiteanu Mario-Claudiu

Abstract **–This work is about estimating the resistance of a building, which was first projected based on equivalent static loads, using P100/2006, after which a nonlinear determination, according to P100-2013, will be performed.**

The estimation is made by applying the nonlinear static analysis method, which provides what is commonly named a "resistance curve", expressing the Lateral load- Lateral Displacement Curve, while constanltly increasing the load, which reflects the overall behavior of the structure and relatively emphasizes the enrgy dissipation of the structure. This method allows a more précised accentuation of the αu/ α1 fraction which shows the redundancy of the structure, implying the values of the designed seismic forces.

Keywords **– seismic, nonlinear, lateral displacement.**

1. INTRODUCTION

The F –d Curve shows the evolution of the plastic mechanism of the structure and of course, the construction damages, while increasing the lateral loads which replicate the inertial forces, meanwhile the vertical load are being considered invariable.

The Bilinearization Method is used, by considering that the initial rigidity of the ideal system (SDOF) equals the flexural rigidity of the real structure (the sloap int the origin of the F –d Curve for MDOF) being the one suggested in P100-1 2013, Annex D and it is about to be presented next.

The F –d Curve for the real structure is acquired with the Nonlinear Static Analysis, using specialized calculation softwares which acknowledge the changes of the structural properties at every loading step. The gravitational loads, corresponding to the seismic calculation, are kept invariable.

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Lateral loads distribution is made by considering the two methods previously described and maintaining them alternately invariable, only increasing the value of the lateral loads, at every loading step.

This analysis allows the user to determinate the presumed order of the plastic hinges, therefore the determination of the breaking mechanism.

The breaking of the structure is correlated with the displacement of the structure, when it is no longer able to hold the vertical loads, therefore it is correlated with the breaking of an important element in the stability of the structure (a wall or a columns).

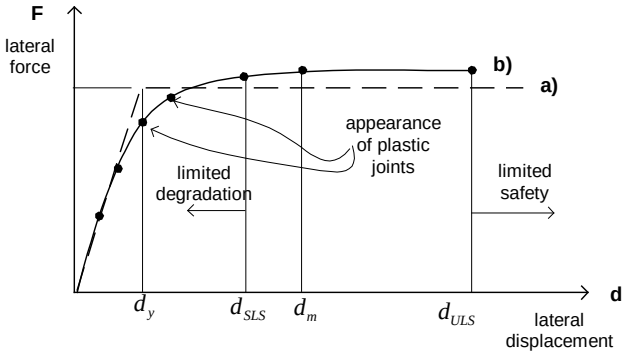

Fig. 1 The F –d Curve bilinearization, considering the initial rigidity of the ideal system(SDOF) as being equal with the flexural rigidity of the real MDOF structure (the tangent sloap in the origin) [2]

The nonlinear analysis is about knowing the physical properties of the reinforced concrete elements and also about selecting a fundamental effort-deformation theorem.

When designing new constructions, according to standards regarding reinforced concrete, the medium strength of the prestressed reinforced concrete is fcm = (fck + 8 N/mm2), in which fck is the natural strength of the concrete.

Even though the standard does not mention any similar connections with steel, in design practice is applied the following mathematical relation fym = 1,15 fyk between the medium and the natural strength, according to some statics studies regarding the current Romanian-made reinforcing steel [1].

2. PRESENTING THE STUDY REGARDING THE NONLINEAR STATIC ANALYSIS FOR DUAL STRUCTURES

The study is about verifying the nonlinear seismic response of a building with a hight regime, consisting in a basement + ground + 9 levels, which is situated in Bucharest City, it's destinated to be an office building and it's superstructure is dual (load-bearing walls and columns).

The structure of this building was designed during the year 2011, using the equivalent linear analysis method, according to standards that were applied then.

As a result of replacing the most important standards in building this type of structure (P100-1/2006 and CR 2-1-1.1/2005 had been replaced with newer versions in 2013),a brief parallel of the (seismic) hazard levels, associated to performance levels stipulated in both versions of the P100-1/2006 standard, the first one, applied when the structure was designed (P 100-1/2006) and the one applied today (P100-1/2013).

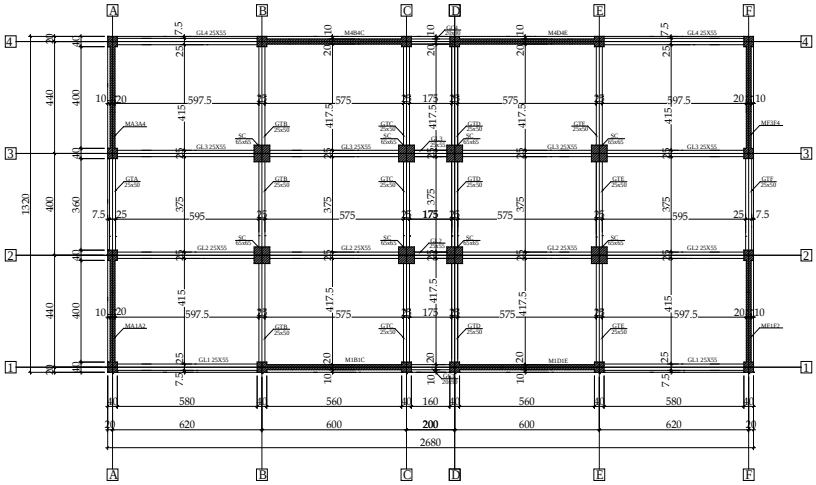

Fig. 2 Plan current level type structure of reinforced concrete walls analyzed

According to P 100-1/2013, the seismic performance level is the so called „designing earthquake" which, in Romania, for the seismic source from Vrance, it's the one with a 20% overcoming probability, within 50 years, respectively the earthquake with a 225 years recurrence interval. The top value of the designed ground acceleration suitable to this type of recurrence interval for the placement of this building, in Bucharest, is ag= 0,30g (g= gravitational acceleration) 1][1].

In the old standard (P100-1/2006) the „designing earthquake" was considered to be the one with a 40% overcoming probability, within 50 years, respectively the earthquake with a 100 years recurrence interval. The top value of the designed ground acceleration suitable to this type of recurrence interval for the placement of this building, in Bucharest, is ag= 0,24g (g= gravitational acceleration).

The main objective of this case study applied to the presented structure, is to punctuate the performances of a building, by analyzing its seismic response to the demands associated to two different seismic hazard levels: the present one and the one stipulated in the old standard that was used at the seismic designing of the building. These particularities will be highlighted by performing the following verifications:

-verifying the level of relative displacements;

-verifying the resistance of the brittle elements (if there are any);

-verifying the deformation capacity of ductile elements (if there are any);

-overall verifications of the structure (SLU) in displacement terms, comparing the required displacement with the capable displacement, which is defined as a lateral

displacement at the top of the structure, where will intervene the breaking of the first vertical element that is essential in the stability of the building;

-overall verification of the structure (SLU) in resistance terms, by comparing the maximum value of the registered shear, with the value of the designed seismic load, which is amplified using a quantification factor for the super-resistance of the structure.

The reinforced concrete load-bearing walls, provided with bulbs at every end and the connection beams had been measured in the ductility H-class.

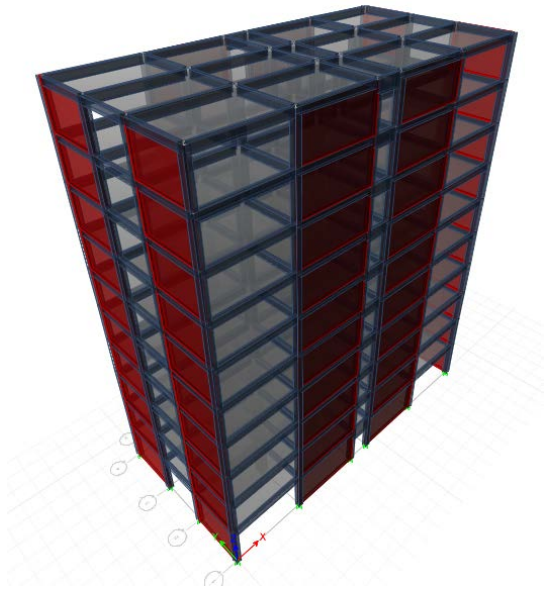

Fig.3 The structure modelled in ETABS

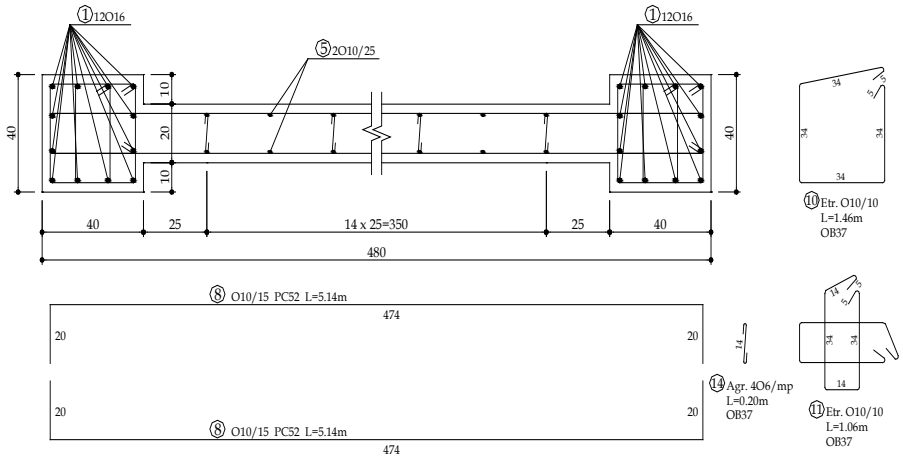

Fig.4 Section based on the ground floor concrete wall insulated X, Ax F

The interpretation of the base shear-top displacement curve is made in a nonlinear static calculation software (ETABS [6]), acknowledging the medium values of the resistance and deformation particularities of the used materials.

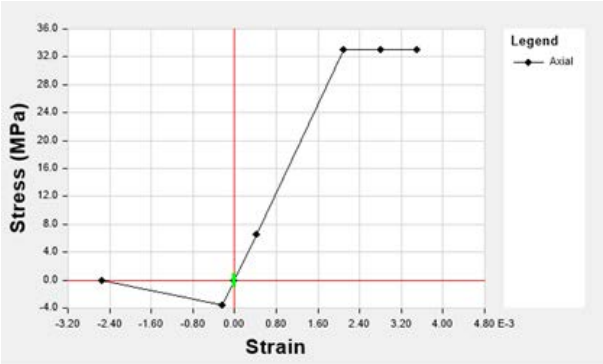

Fig. 5 Stress-strain curve Concrete C25 / 30

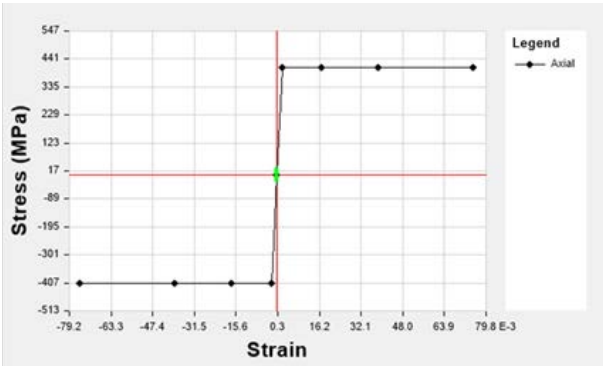

Fig.6 Stress-strain curve .Steel S355

3. RESULTS AND SIGNIFICANCES

For the joints of both, vertical (columns, bulbs) and horizontal (beams) elements, it's important to determine a plastic deformation (leakage) theorem, represented as a moment-rotation curve, that could assemble the behavior of the joints within the post-elastic domain.

When calculating the displacement demands, it is imperative to estimate the duration of the fundamental modulus of the SDOF system, on every direction. The displacement demand will be made according to the chapter containing the objectives of this study, for the two previously mentioned earthquakes, corresponding to the recurrence interval of 225 years, respectively 100 years. Meanwhile the displacement demands estimated for the SDOF transform into displacement demands which will correspond to the MDOF.

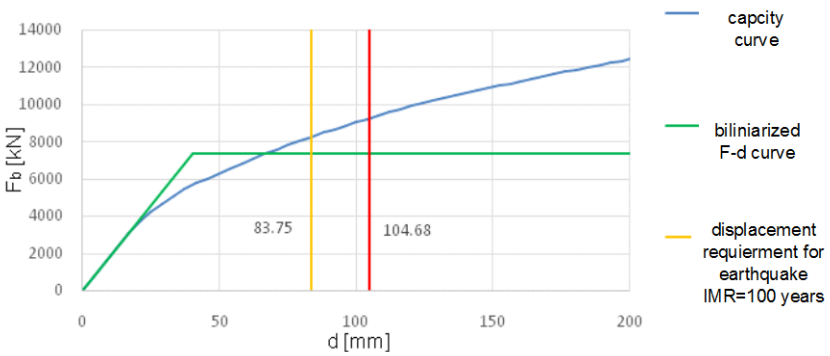

Fig. 7 F - d curve linearized and displacement requirements at peak direction OX

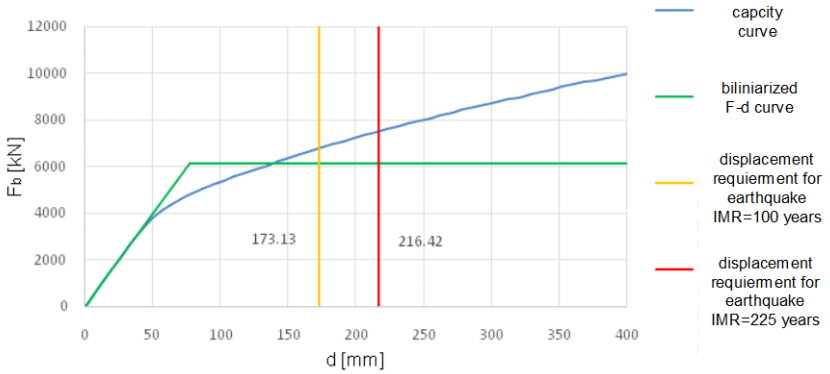

Fig. 8 F - d curve linearized and displacement requirements at peak direction OY

To evaluate the response of the analyzed structure to the seismic demands, indicated by the top displacement demands matching the two seismic hazard levels, the nonlinear static analysis was executed and the results were processed.

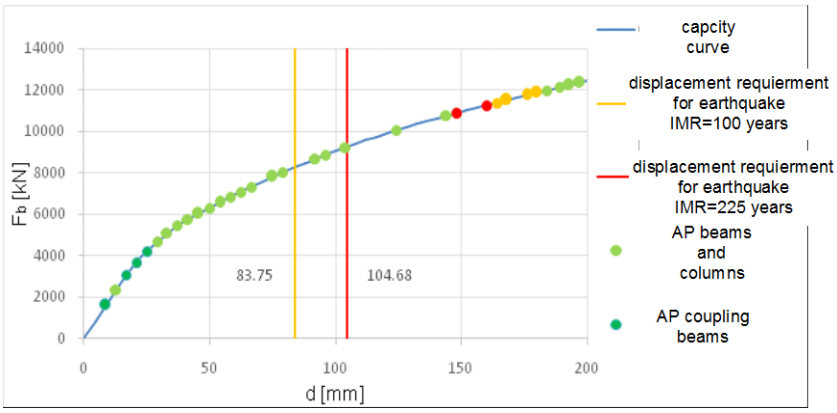

Fig. 9 Verification of the structure from the SLU overall in terms of displacement in the direction OX; Peak travel requirements; plastic hinges formation and movement capable

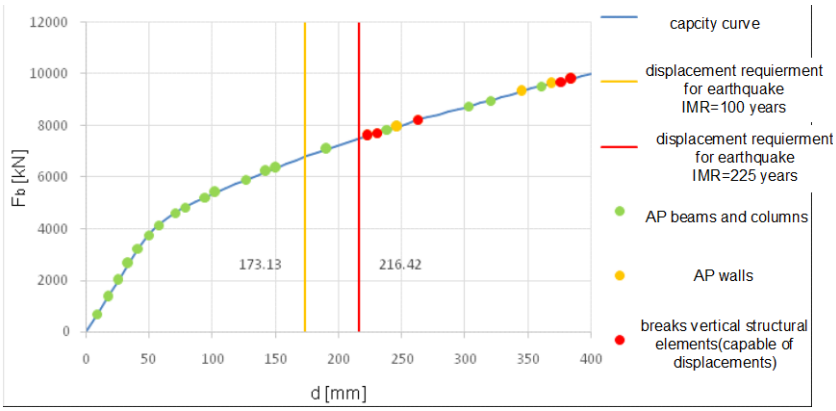

Fig. 10 Verification of the structure from the SLU overall in terms of displacement in the direction OY; Peak travel requirements; plastic hinges formation and movement capable

Tabelul 1 Checking the overall structure in terms of resistance to SLU on the OX and OY, for earthquakes with IMR=100 years and IMR = 225 years

				Earthquake design 225years				Earthquake design 100years			
Direction	m[t]	q	α_u / α_1	$F_{b,cap}$	$S_d(T_1)$	F_b	$F_{b,seism}$	$F_{b,cap}$	$S_d(T_1)$	$F_b[KN]$	$F_{b,seism}$
OX	3889,7	6,25	1,25	9300	1,177	3892,1	6081,3	8200	0,9418	3113,6	4865,1
OY	3889,7	5,0	1,25	7600	1,472	4865,1	7601	6800	1,177	3892	6081

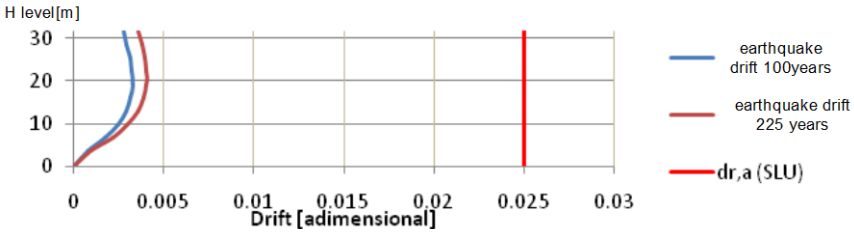

Fig. 11 Checking level relative displacements requirements on the SLU direction OX

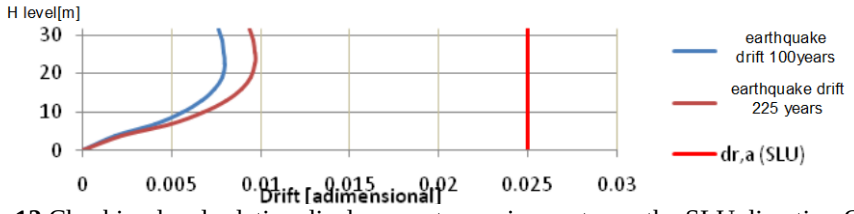

Fig.12 Checking level relative displacements requirements on the SLU direction OY

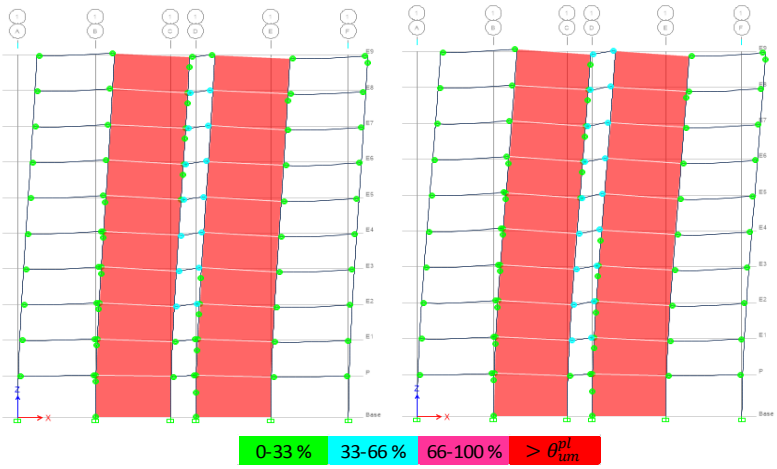

Fig. 13 The image of plastic joints for displacement requirements Ax 1,OX:
a) d = 83,7mm for earthquake 100 years; b) d = 104,7mm for earthquake 225 years

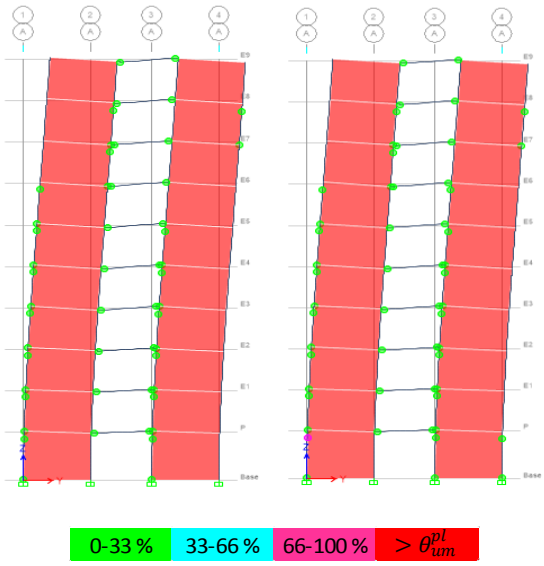

Fig. 14 The image of plastic joints for displacement requirements Ax A,OY:
a) d = 173,13mm for earthquake 100 years; b) d = 216,42mm for earthquake 225 years

4. CONCLUSIONS

After estimating the displacement results, associated to reference earthquakes, with a recurrence interval of 225 years (the present designing earthquake at SLU, according to P100-1) and an RI of 100 years (stipulated in the previous version of the

standard, which the initial project was based on) and verifying the deformation capacity of the structure, contrasting them with the specified demands, the following conclusions had been made.

As emphasized by the periods of the fundamental vibration mode on both main directions (OX: T1= 0,655s; OY: T1= 1,056s), the rigidity and the resistance of the structure are very different from the ones describing the two orthogonal directions. Thus the admitted displacement on the flexible direction (Fig. 8), OY is superior to he one describing the longitudinal direction (Fig. 7) OX, so the displacement demands are relatively doubled.

The conclusion, in this case, is that the different structural assemblyman and plan-development on both main directions very much affects the structural response in the behavioral linear domain and also the nonlinear one, the transversal direction OY being, in fact, the one which is vulnerable to lateral loads.

On the transversal direction OY, the structure does not reach its maximal displacement for none of the displacement demands, but it is submitted to advanced deterioration, according to the exigent demands (SLU) of the present P100-1 standard. Thus, when reaching the top displacement implied by the demands, the stretched bulbs of the walls present a high level of elasticity, in the section of the superior end of the element at the bottom of the structure, after which for a relatively slow increasing of the imposed displacement value, they reach the maximal rotation and so they would break (Fig. 10). The act is confirmed by the global verification of the shear on OY direction (Table 1), in which case it can be seen that the maximal shear, minimized for separating the effect of the material super-resistance, verifies the demand represented by the equivalent shear, calculated for the 225 years RI earthquake. (Fb, cap= 7600 kN ≈ Fb, earthquake= 7601,63 kN).

Although, for the displacement demand implied by the 225 years RI earthquake, the structure shows articulations to be aggregated in the unessential areas designed for dissipative elements, in this case are the beams and the relatively level displacement values are kept within the normal values (Fig.12).

The reinforced concrete frameworks, made of columns and beams, present the expected behavior, being made for bear the gravitational loads because of the lateral rigidity which is much more reduced than the lateral rigidity of the reinforced concrete poles. Therefore the joints are exclusively aggregated at the ends of the beams, neglecting the orientation of the lateral loadings Ox/Oy. (Fig.13, 14 for OX and OY).

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Obtaining of Structure Characteristics of the Building Composites by Computer Microscopy

Irina Dovgan, Andrey Kolesnikov, Svetlana Semenova

***Abstract* – The article deals with the algorithm of investigating the structure of building material on the basis of its photomicrographs by means of statistical analysis of point images. The obtained values allow defining the basic characteristics of the material structure.**

***Keywords* – building materials, image processing, Nih Image, point images, statistical geometry**

1. INTRODUCTION

One of the areas of science about materials received a significant development in the last decade, is a structure-based modeling and optimization of building composites [1, 2]. The success of this research area is largely due to the possibility of more effective control of formation of the desired performance based on structural information, evolving theory of self-organization of dispersed systems [3] and improves the overall technical level in the field of composite material research. The qualitative results about the self-organizing processes in the materials and their models have been obtained in a number of studies [4]. Transition to quantitative values and correlation is more productive in choosing the optimal control actions addressed at the composite using statistical procedures. In this connection it is necessary to analyze the possibility of obtaining such characteristics by simple means, considering their content component and the possibility of building statistical models "structure-properties". Such models as compared to traditional for material science models "composition-properties" have a number of advantages, i.e. clear quantitative and qualitative interpretation, the possibility of considering the received dependences of the broad classes of materials, the possibility of their use to express estimation of properties, easy transition to the physical models.

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In composite materials the interaction between their components – □ binder, filler and additives, is complex [5]. This, in particular, can manifest itself in the tendency to form particle distributions of the components, as well as pores and internal separation boundary in accordance with the laws that are different from purely random, given by Poisson law. At each fixed scale level such deviation can be of two types (Fig. 1).

If the attraction forces between the particles dominate, they form clusters. At the opposite tendency quasi-regular structures are formed based on the principle of maximum repulsion. This distribution is easy to get transmitting particles the same charges.

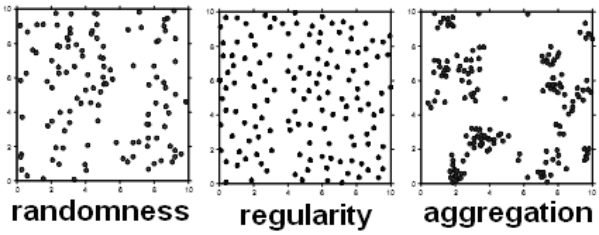

Fig. 1 Nature of the distribution of points on the plane

As a "reference point" for the degree of difficulty it is possible to accept the DLVO theory which takes into account the only Van der Waals and coulomb interactions. Particles obeying this theory are already characterized by structure organization and the complex interactions, therefore periodic colloidal structures [5], long and short coagulation contacts are formed. The transition to the particles of complex nature, the possibility of structural and mechanical barriers formation, and other causes significantly complicate the accurate examination of the interaction of particles in the composite binder test.

2. EXPERIMENT DESCRIPTION

One way to solve problems arising in the study of the interaction of composite particles is the transition to a simplified system - the original model. The distribution of macroscopic particles on the surface and in the bulk liquid phase [4] can in particular be used in such a model. In such systems, despite their relative simplicity and ease of the experimental methods the processes of structure formation occur. Due to capillary forces between particulates (2 to 5 mm), there is a preferential attraction, at close range should additionally take into account their electrostatic, van der Waals and other types of interactions. The considered method of physical modeling of processes in composites using particulate was used [4], introduction of a quantitative description increases its productivity.

One of the easiest ways to study the macrostructure of composites and their models based on optical methods of studying at a relatively low increase (10x-1000x), for which microscopic investigation in reflected light is used, for example, which has not lost its relevance in spite of the emergence and development of more advanced

techniques [6]. Enhanced with the corresponding means of fixing images (CCD-camera and associated equipment together with the software), the conventional microscopic study is transformed into the computer microscopy techniques.

For a variety of materials the structures observed using optical methods are characterized as small particles, pores, and other objects, their spatial distribution can be examined.

Consider possible options of point objects of this type:

1. The particle components of the composite. The research is facilitated if they are painted in different colors or different grayscale.
2. The pores. There is no need to stereological reconstruction of the porous structure of the observed object with the point approach.
3. Projections of cracks and internal borders phase boundary on chips or grinding. The point approach is possible to use in such objects only after the machine image segmentation of these lines and surfaces.
4. Mineral growths exhibited a special treatment (e.g., phenolphthalein for lime in the cement stone, tannin for cracks).

The process of investigation of these structures is as follows. With the help of an electronic microscope eyepiece, converted to macro mode webcams and similar equipment micrographs of thin section or a smooth flat cleavage of the material are obtained. Further, it will be assumed that a preparation produced by appropriate means if necessary (such as cement stone processed by phenolphthalein developer). As an example, consider the implementation of the algorithm processing micrographs of the plastering material on the basis of gypsum and perlite [7] (Fig. 2.).

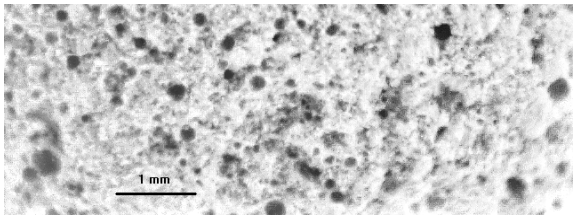

Fig. 2 Photomicrograph of material by subtracting the background light level (Scale mark - 1mm)

Further processing may be carried out by using free software such as Nih Image, ImageJ and its derivatives, Image Tools and many commercial ones (for example, Optimas). The Nih Image 1.62 Fat software [8] which has become a classic in the field of medicine and biology in emulator Executor 2.0 environment was used. The macro batch file for micrograph processing was written. After Background subtraction was carried threshold separation image (Threshold). Next, the filtering, which allows removing noise image produced by unrelated pixels. Next, after transfer to the binary image, the ultimate point of erosion (Ultimate eroded point) built. After the threshold separation and binarization small clusters of pixels appear on the screen corresponding to most of the central area of the original objects (Fig. 3.).

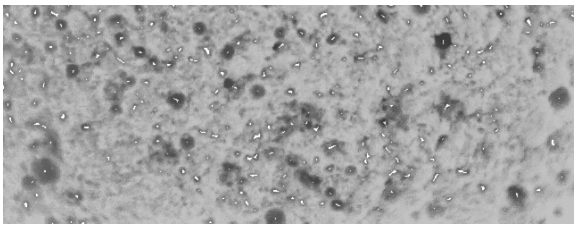

Fig. 3 Ultimate erosion points superimposed on the original image

The third phase of structure analysis is carried out using the points location on the plane and in space analysis software (PPA (DOS), Past – Paleontological software, Ppa (mac) and others. The latter is conveniently run on the same emulator. Consider the results of its work, displayed graphically.

3. RESULTS AND SIGNIFICANCES

Analysis of the distribution point can be made by square grid (squares analysis). The square grid with side **a** (1) is superimposed on the dot pattern.

$$a = \sqrt{\frac{\text{Preparation area}}{2 \cdot \text{Number of point}}} \tag{1}$$

Then the number of events (in this case - of the particles or pores) in each square is calculated and considered frequencies are statistically processed (Fig. 4).

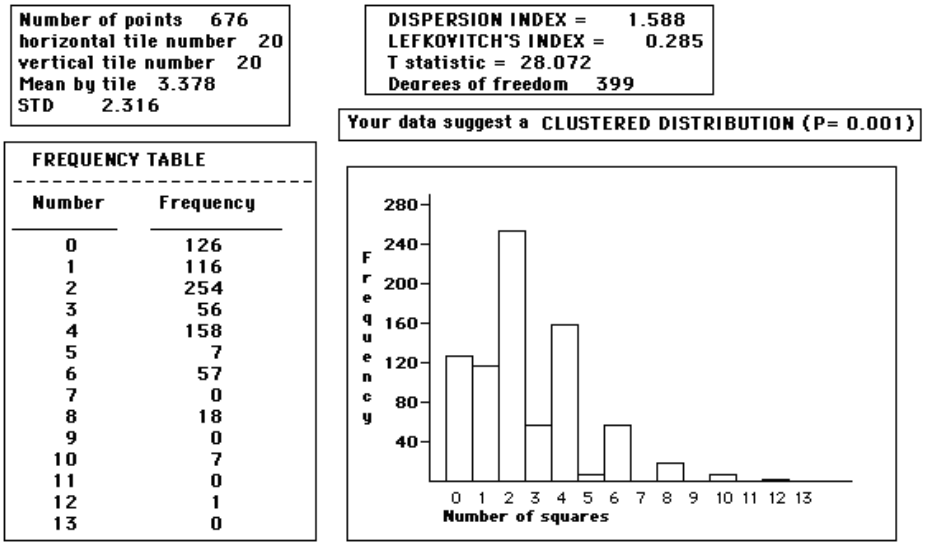

Fig. 4 Results of statistical analysis by squares method

Essential data to the decision making can be output as the analysis results (Fig.5)

1) General data

Number of points 676

Density 18 cells/mm2

2) Indices		Random	Meaning Regular	Clustered
Dispersion Index	2.389	=1	<1	>1
Lefkovith's Index	0.495	=0	-1	+1
Packing Factor	0.067	=0	+1	
Eberdhart Index	1.185	1.27	<1.27	>1.27
Means/STD Index	2.323	close to 0	>0	

Fig. 5 Results and the boundary values of statistical analysis of a point image

Dispersion index [9] or Fano factor defined as the ratio of the variance σ^2 to the mean frequency μ (2)

$$D = \frac{\sigma^2}{\mu} \tag{2}$$

If D = 1, the image is random. This means, the data set has no dominant trend for clustering or dispersion. If D <1, point image has a regular structure, i.e., the point spread in the observed region more or less regularly. Dispersion index depends on the values of the mean values, so use an additional criterion - Lefkovitch index [10] (3) (in radians):

$$\Delta = \frac{4}{\pi} arctg \frac{s^2}{\sigma^2} - 1 \tag{3}$$

Here s^2 - variation of the frequency in this sample, σ^2 - a variation of random distributions, the Poisson distribution is substituted for the value of the average, $\sigma^2 = m$. As the dispersion index, Δ = 0 corresponds to a random distribution, Δ = -1 - regular and Δ = 1- cluster.

Ebernharda Index [10] (4) also allows to the test assess the degree of grouping objects, S – the standard deviation and $\overline{x}$ the average value of the distance from random point to the test.

$$I_E = \left(\frac{S}{\overline{x}}\right)^2 + 1 \tag{4}$$

The corresponding limit values are shown in Fig. 5. All major statistical characteristics indicate a high probability of cluster formation in this sample.

The nature of the mutual arrangement of the particles can be detected from the diagram of distribution of distance between the particles (Fig. 6). Here on the values of the density maxima the approximate location of the coordination spheres (approximately 100 and 150 microns) and the mean value of the effective radius [11], characterizing the size of the "dead" space around the object under study, can be found.

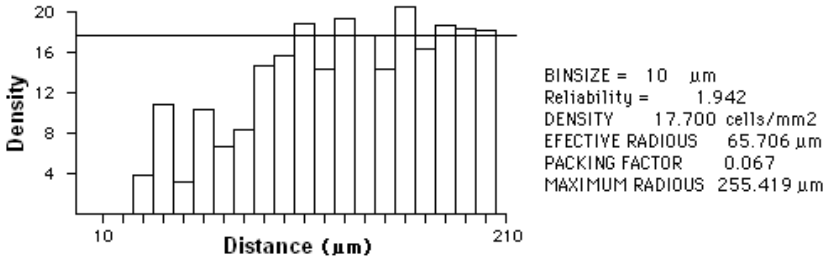

Fig. 6 Distribution of distance between particles and derivative values

4. CONCLUSIONS

Thus, a consistent application of computer image and statistical processing of the micrographs and similar images of material structure reveal the characteristic structure of the material and to specify the conditions of their non-random distribution. This, in turn, may be interpreted as the evidence of self-organization process of spatial structure of the material that occurs at different time stages.

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Asphaltic facings – Durability and long-term behaviour

M. Smesnik

***Abstract* – Asphaltic facings, which are frequently applied for reservoirs, channels and disposals have to be watertight and must be able to absorb strains without cracks. The good performance characteristics of asphalt deteriorate during its life by thermal and oxidative processes. The entire process is summarized by the term aging. To increase the durability of this type of facing it is important that the changing of the good performance characteristics of asphalt is minimized during the construction process. This paper provides a general overview about asphalt facings and points out the key aspects during the construction process in order to increase the durability and the long-term behaviour.**

***Keywords* – asphalt facing, asphalt, material aging, durability, hydraulic engineering, and dams**

1. INTRODUCTION

Asphaltic facings are mainly used for small and medium dams with heights up to 100 m. In case no appropriate dam material is available (material for embankment dams where the upstream dam body is under uplift), surface sealings are applied. A reason for choosing this type of facing is the short construction time due to the separation of different construction steps. The decrease of the upstream dam volume results in lower material costs due to the reduction of uplift forces. The waterside slope angle for dams over 20 m should consist of a maximum proportion of 1:1.5, otherwise the placing of the sealing is hampered. Applying a surface facing it is essential to consider exposure of external influences. One of the mayor advantages of surface sealings includes easy accessibility in case of maintenance work and inspections.

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2. DURABILITY AND LONG TERM BEHAVIOUR OF ASPHALT FOR HYDRAULIC ENGINEERING

The grain distribution, the air void and bitumen content of each asphalt for facings depends on several influences and varies between the projects.
Each project needs its own suitability test in order to optimize the material composition. This results in a positive long-term behavior of the facing. A typical asphaltic concrete consists of a bitumen content between 5 – 8(9) m. % and a grain distribution of 0 - 11(16) mm. Different bitumen types are used for material production. Fig. 1 displays the primary used standard bitumen types. Polymer modified bitumen can be used as well.

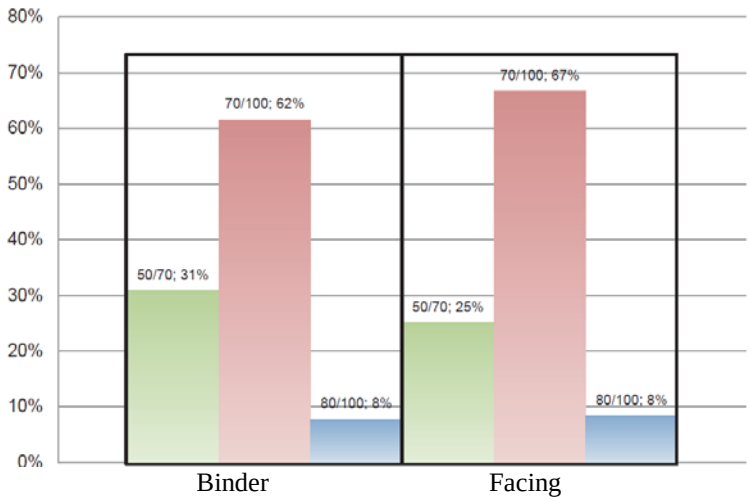

Fig. 6 Common standard bitumen types for asphaltic facings [1]

The good performance characteristics of asphalt deteriorate during its life by thermal an oxidative process. The entire process is summarized by the term aging. Asphalt is composed of a mixture of bitumen and aggregates, wherein the bitumen accredits for a major part of the impact on the deterioration of the performance characteristics during its life cycle. The aging process is divided into short-term (STA) and long-term (LTA) aging. The STA includes the manufacturing process of the mixed material and the placement. The LTA describes the aging process while in service. As an effect of the aging process, the material properties of the bitumen turn stiffer and more brittle. These procedures have a negative effect on the durability and the long-term behavior. The shift of the properties is progressive and irreversible. Equation (1) depicts a mathematical description of the material aging process.

$$A_{TOTAL} = A_{STA} + A_{LTA} \quad (1)$$

The changing of the material properties can be also described by the changing of the material viscosity. Fig. 2 shows the whole aging process (ATOTAL). It displays

that the STA includes the mixing, transportation and placing of the asphalt and LTA includes the aging process while in service.

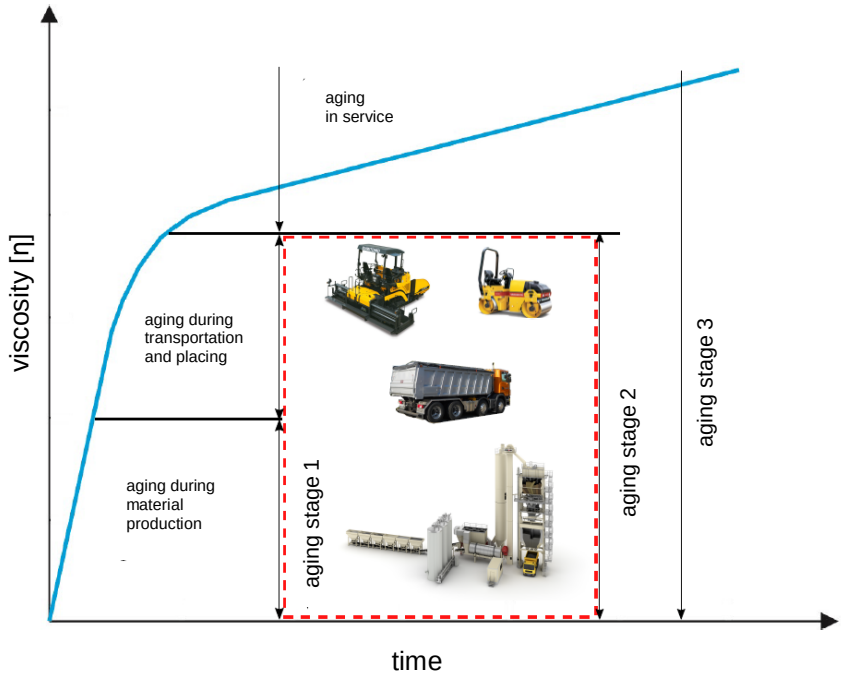

Fig. 7 Asphalt aging process – changing viscosity while in service [2]

The aging (A_{TOTAL}) can be divided in three different processes (see **Table 1**).

Table 1. Chemical and physical bitumen aging processes

Aging type	Occur (mainly)	Mainly influenced
Oxidative Aging	STA (production process)	Temperature + ROS*
Aging by distillation	STA (mixing, transport)	Temperature
Structural aging	STA + LTA	-

*…reactive oxidative species (oxygen, ozone etc.), see [2]

A combined comparison of **Fig.** ***1*** and **Table 1** displays the strongest material properties (viscosity) transformation during STA. This is predominantly caused by the elevated temperature during the manufacturing process, which is acting as a catalyst for the aging process. Asphalt is a thermo – viscoelastic material, which means the viscosity depends on the temperature of the material. Considering technical water tightness it is essential to keep the porosity after placing and compaction lower than 3 vol. %. Along with the decreasing material temperature the viscosity drops and the energy for a sufficient compaction increases. On the one hand a minimal material aging process requires an upper temperature limit during the production process however the temperature minimum is limited due to the compaction energy. These

circumstances indicate the importance of an exact planning of the manufacturing process and the corresponding temperatures. While the material is heated during the mixing process, the material temperature depends on external influences during the transportation and placing (climate - ΔT, machines on site etc.). Hence it is essential to identify the right temperature for the material production process considering the cooling during the time of transportation and placing. **Fig. 3** shows an exemplary temperature gradation during the production process for a standard bitumen 70/100. The red dashed line in **Fig. *3*** displays the minimal mixing temperature which is necessary in order to achieve the required temperatures of the guide lines [3] at the two marked points on 95 and 120 min.

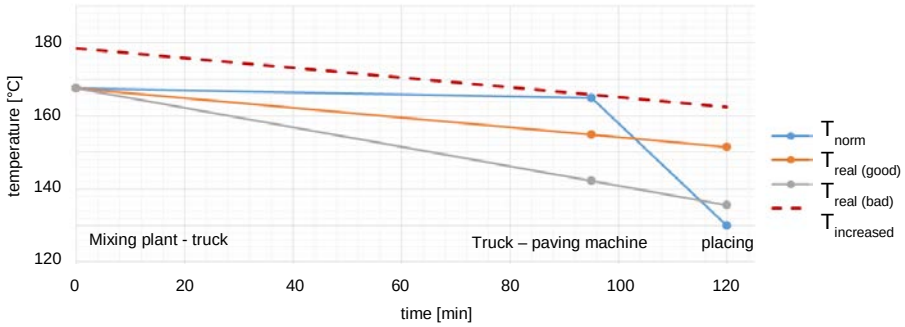

Fig. 8 Temperatures during the production process (70/100) [2]

In case the temperature is too high during the production, a strong material aging will occur (STA) consequently the material turns stiffer and more brittle. Equation (1) depicts the relation of the aging process. The equation can be switched to (2).

$$A_{LTA} = A_{TOTAL} - A_{STA} \quad (2)$$

(2) shows that by increasing of ASTA the remaining part of ALTA of ATOTAL decreases. Damages on the facing occur if a material dependent (bitumen type) boundary viscosity is reached.

2. CONCLUSIONS

If asphalt facings are constructed correctly they are a very secure and economic option for small and medium dams with a height up to 100 m. Literature reveals excellent experiences with facings being in service for more than 50 years. The displayed investigations in [2] prove a direct correlation of the production process and long-term behavior. To sum up, the durability and the long-term behavior of asphaltic facings is limited to the quality of the production process.

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Quantification of Volumetric Moisture in the Clay Loam Soil Profile by Measuring of Electrical Impedance

M. Gomboš, J. Pařilková

***Abstract* – The aim of this paper is to present the analysis of development of electrical impedance contour lines (R_x) in clay-loam soil. The development was tracked on the grounds of the results obtained from electrical impendance measurements performed by "Z-meter". Electrical impedance contour lines are presented in the form of log R_x. Contours show the development of electrical impedance during vegetation period in a soil profile up to 0.70 m. They are then compared with the contour lines of volumetric soil moisture (chronoizoplets) defined for the same period.**

Results of the analysis should show the apparatus potential for being used in soil water regime monitoring.

***Keywords* – soil profile, electrical impedance contour lines, Z-metr, volumetric soil moisture**

1. INTRODUCTION

"Z-meter" is based on electrical impedance spectroscopy and can be widely used both in practice and theoretical research [4], [5]. One of its possible uses is in the field of quantification of soil hydrological processes, namely quantification of soil water regime, soil water movement and identification of areas heterogeneous in terms of soil porosity, [2], [3], [6], [7]. In order to assess possible usage of the apparatus for the described purpose, "Z-meter" was used for electrical impedance measurements in clay-loam soil.

The aim of the paper is to present the results of electrical impedance development in time, measured along the vertical line of a soil profile during the

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vegetation period in question. The results are compared to volumetric moisture chronoisoplets defined for the same soil profile and time interval.

2. Experiment Description

The selected method is based on comparison between the developments of electrical impedance and volumetric soil moisture vertically along the soil profile during the same time intervals. The developments are shown as contour lines and contour intervals. Contour lines are lines along which volumetric soil moisture, or electrical impedance, have the same value. Contour intervals are intervals of values which are assigned different colour. Together they form a two-dimensional picture showing the development of analysed values in time.

Fig. 1 Localization of an observed area

Fig.2 Location of examined profile in the locality of Milhostov

The database used in the analysis is divided in two groups. The first group is used for defining development of real component of the electrical impedance contour lines in time "R_x". It contained data obtained from the measurements of R_x values by means of "Z-meter". The measurements were performed in Milhostov area on the East-Slovakian Lowland (ESL) during vegetation period (Fig. 1 and Fig. 2, Table 1).

The second group of data were obtained from the measurements of volumetric moisture "θ" by TDR probe.

Table 1 Characteristics of the location Milhostov lokality

Station	***Height (a.s.l.)***	***Geographical coordinates Latitude (to N)***	***Geographical coordinates Longitude (to E)***
Milhostov	102737 m	48° 40' 00,00"	21° 43' 47,45"

Fig.3 Field measurement of volumetric soil moisture by TDR and measurement of electrical impedance by "Z-metr"

Detailed characteristics of the area, type of measurements, way of excluding distant values from the measured data and processing and analysis of measured databases are described in [1]. Electrical impedance contour lines are modified by moving average of 3rd grade. For better illustration they are graphically represented in a logarithmic scale, i.e. log R_x. Spacially were selected directions which are most homogeneous in terms of soil porosity. Development of electrical impedance and volumetric moisture contour lines was processed in SURFER graphical environment.

3. RESULTS AND SIGNIFICANCES

Soil moisture regime of a soil profile measured by TDR is graphically shown in Fig. 4. It is represented by chronoisoplets. Each contour interval is different colour. Fig. 4 shows that intensive drying of soil started in late April. Its maximum was reached in late May and early June and the end of June meant the end of the intensive dry period. Another dry period started in the other half of August. Least volumetric moisture – 11.00% - was measured on august 25., 0.0 – 0.1m in depth. Highest volumetric moisture – 40.77% - was measured on august 2., 0.20 – 0.30m in depth.

Fig. 5 shows development of electrical impedance in different layers of a soil profile in time. Differences are obvious from the values indicated along the vertical line of a soil profile.

The highest values of electrial impedance are measured in the upper layers. It is probably related to their drying. The more soil dries, the lesser is the ability of soil to conduct electricity and the higher is electrical resistance. Volumetric soil moisture increases with depth (Fig. 4) and, on the contrary, its variability decreases. This is also reflected in the development of impedance which decreases with increasing depth. In around 0.6m under the surface the differences are minimal.

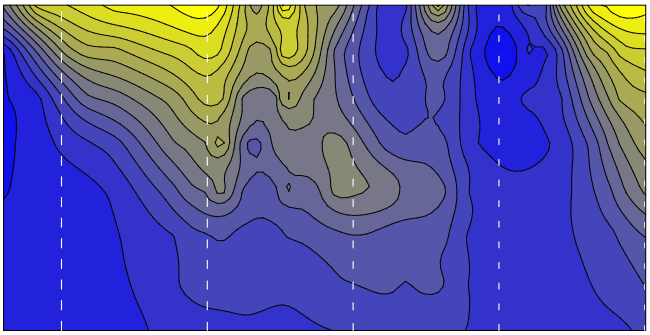

Fig.4. Volumetric moisture development in the soils of Milhostov area during 2011 vegetation period

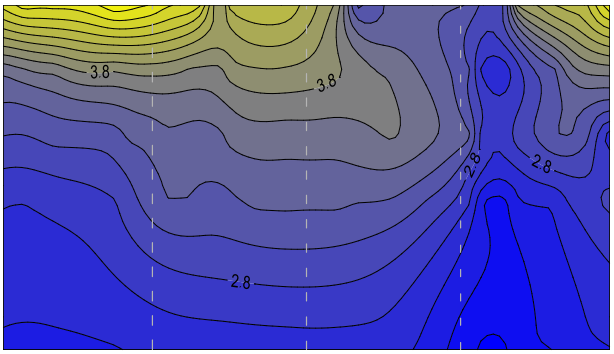

Fig.5 Eletrical impedance development in the soils of Milhostov area during 2011 vegetation period

Graphical representation of contour lines in Fig. 4 and Fig. 5 shows that electrical impedance captures soil water storage and its development in time. It is depicted in Fig. 6 which shows correlation between electrical impedance and volumetric soil moisture. The correlation is very tight. Correlation degree is R=0.81.

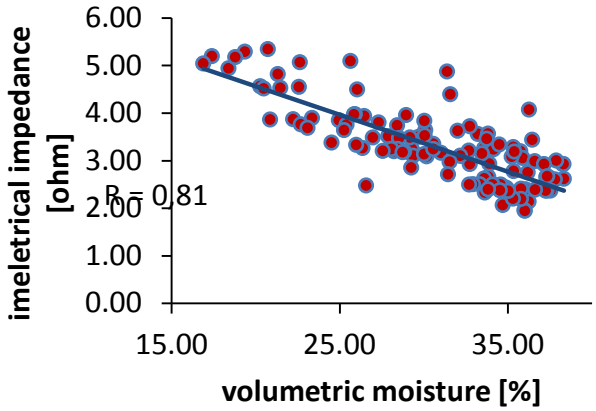

Fig.6 Relation between volumetric moisture and electrical impedance in clay-loam soil in Milhostov area

4. CONCLUSIONS

The aim of the paper is to evaluate the measurements of the real components electrical impedance development in soil. Its development is compared to the volumetric soil moisture development. Analyzed parameters were graphically represented as contour lines and contour intervals. Two-dimensional representation of electrical impedance and volumetric soil moisture development in time along the vertical line of a soil profile was gained. For assessing dependence of electrical impedance measurements on soil moisture, correlation analysis was used.

The results show that electrical impedance values measured during the analyzed vegetation period followed the development, volume and soil water storage in a soil profile. It has been proved that “Z-meter” has a great potential for field and research purposes for quantifying hydrological processes in soil. Further on, “Z-meter” should be used in different soil environments of various textures. On the basis of this, calibration equations will be developed for the individual soil types.

5. ACKNOWLEDGMENTS

The author would like to thank for the kind support of the project VEGA 2/0062/16

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Evaluation of Selected Locality of East-Slovakian Lowland According the Soil Water Capacity

A. Tall, D. Pavelková

***Abstract* – Most of present research in soil physics is oriented to numerical methods used for obtaining hydrophysical characteristics of the soil from data, which can be easily measured (in most cases from the particle size distribution of soil). Outputs are called "pedotransfer functions", which are useful for calculation hydrophysical characteristics of soils. This paper deals with application of pedotransfer function for regionalization of clay-loam soils in East Slovakian Lowland according to soil´s water capacity.**

***Keywords* – Clay soil, Pedotransfer function, Saturated water content, Soil´s texture.**

1. INTRODUCTION

East Slovakian Lowland (ESL) is characterized by high variability of soil types, when on relatively small area of land, there are soils with different textural composition. At ESL therefore occur soils from light soils (with high content of sand fraction), over loamy soils, to extremely heavy clayey soil with a dominant clay content. This heterogeneity is caused by difficult tectonic evolution of the area of ESL. The texture of soil is closely related to its water retention capacity. In general, with an increasing content of fine particles in the soil, also increases the ability to hold more water. Especially clay particles are characterized by their high ability to hold water in its structure and thereby highly increase retention capacity of heavy soils. Water capacity of the soil can be easily quantified using hydrolimit Θ_s (saturated water content). This hydrolimit determines the maximum amount of water that the soil can absorb. Θ_s is a maximum moisture from water retention curve (pF = 0). In general, saturated water content of soil, is almost equal to soil´s porosity. This assumption is not valid in heavy soils due to their volume changes and entrapped air

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in the pores. Saturated water content in heavy soils mainly depends on textural composition [3], [4].

This paper presents the results of regionalization of depressed area of ESL according to saturated water content of soil.

2. EXPERIMENT DESCRIPTION

A typical area of ESL with various soil types was chosen for the regionalization according to the soil´s water capacity (**Fig 1**). The area is bounded on its west side by the river Laborec, south by the river Uh and east by water channel Revištia - Bežovce. Northern border is formed by south coast of Zemplínská širava dam together with road no. E50. It is a lowland area with central depression. In the midst of this depression are located Senianske ponds, which are subsidized by the water from the Zemplínska šírava dam through the channel Čierna voda. Circuit of examined area is 72.3 km and its area is 278 km^2. The average altitude of the area is 103 m above sea level (ASL). Minimal altitudes (98 m ASL) are in the central part of the depression around Senianske ponds and maximal altitude (133 m ASL) is in the northern edge of the area (see **Fig. 2**).

Fig. 1 Situation of experimental site

In the studied area was performed a field survey and picked up were 106 disturbed soil samples from a depth of 0.5 m. Sampling density in the area was 1 sample to 2.6 km^2. Coordinates of each sampling site were recorded using GPS. The situation of sampling sites is shown in **Fig. 3**.

From the samples was in laboratory performed particle size analysis by the method of Cassagrande and were classified soil types. Results from particle size analysis are shown in **Fig. 4** and **Fig. 5**. According to these figures we see that the textural composition of investigated soils ranging from loam to clay with a dominant position of silty - clay loams.

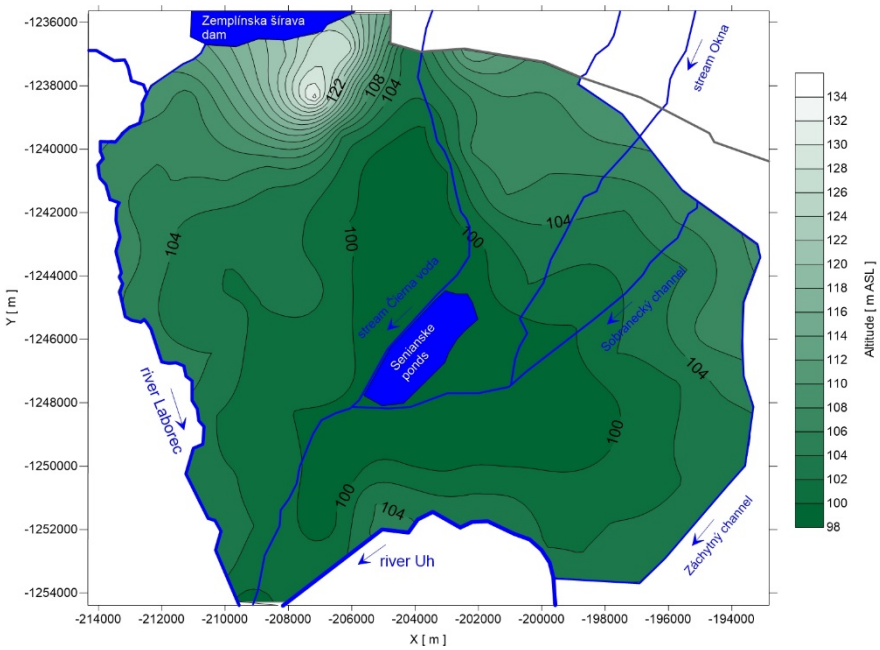

Fig. 2 Terrain map of observed area

Fig. 3 Map of sampling sites

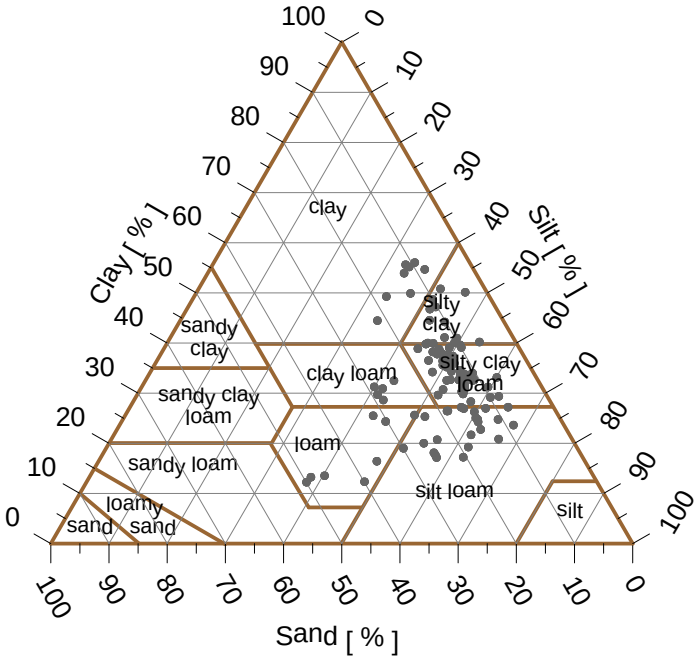

Fig. 4 Soil texture triangle according the USDA

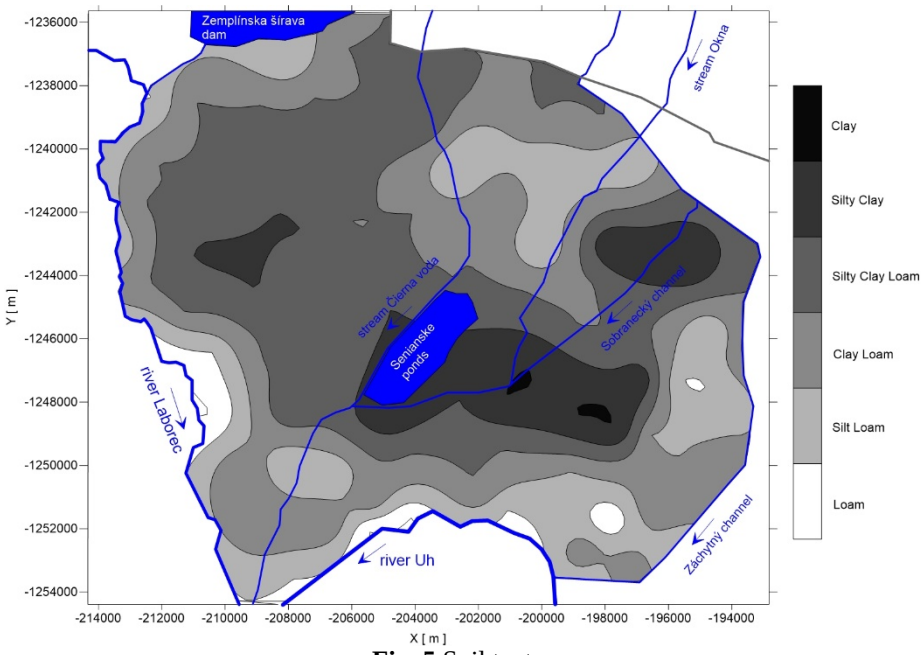

Fig. 5 Soil texture map

The next work was focused on evaluating of retention capacity of soils. Used was hydrolimit Θ_s – saturated water content. Direct determination of Θ_s in the laboratory is time-consumpting process. Necessary is pick up undisturbed soil samples in the field, then saturate and dry of samples, and measure of weight and volume. In the laboratory of Department of Lowland in Michalovce were over the past years determined many values of Θ_s from ESL. Based on these determinations was created pedotransfer function, which makes it possible to indirectly determine the value of Θ_s using textural analysis of soil. In this work was for the calculation of Θ_s used next pedotransfer function [1], [2]:

$$\Theta_s \text{ (vol \%)} = -242{,}025 + 3{,}09953*(\%\ \text{I.fr.}) + 2{,}91079*(\%\ \text{II.fr.}) + 2{,}62629*(\%\ \text{III.fr.}) + 2{,}85083*(\%\ \text{IV.fr.}) \quad (1)$$

To investigate the influence of textural composition on the value of Θ_s were used statistical methods. This indirect method of determining the Θ_s of the soil is preferred because it is quick and easy. Reliability of this method is proved by very high degree of correlation (r = 0.966) between the measurements and the equation (1).

3. RESULTS AND SIGNIFICANCES

Calculated results of regionalization of soils according the Θ_s are graphically shown in **Fig. 6**.

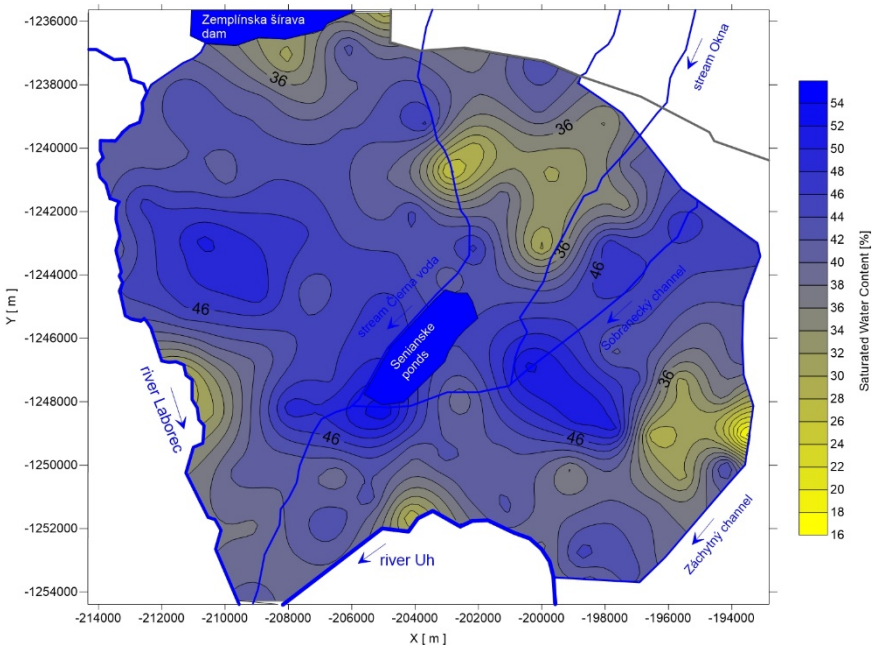

Fig. 6 Regionalization of depressed area of ESL according to soil´s water capacity

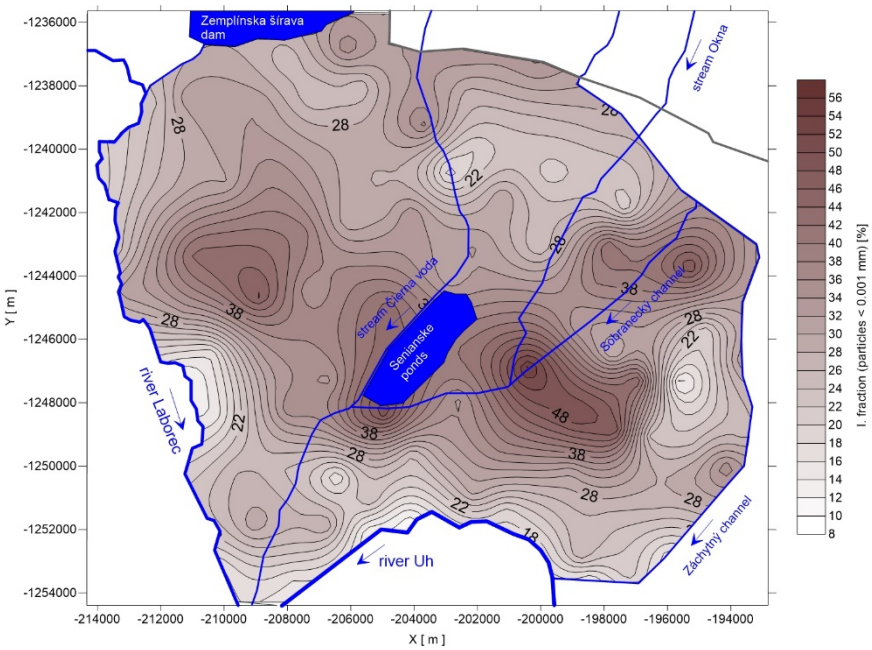

Fig. 7 Map of colloidal clay distribution (particles < 0,001 mm)

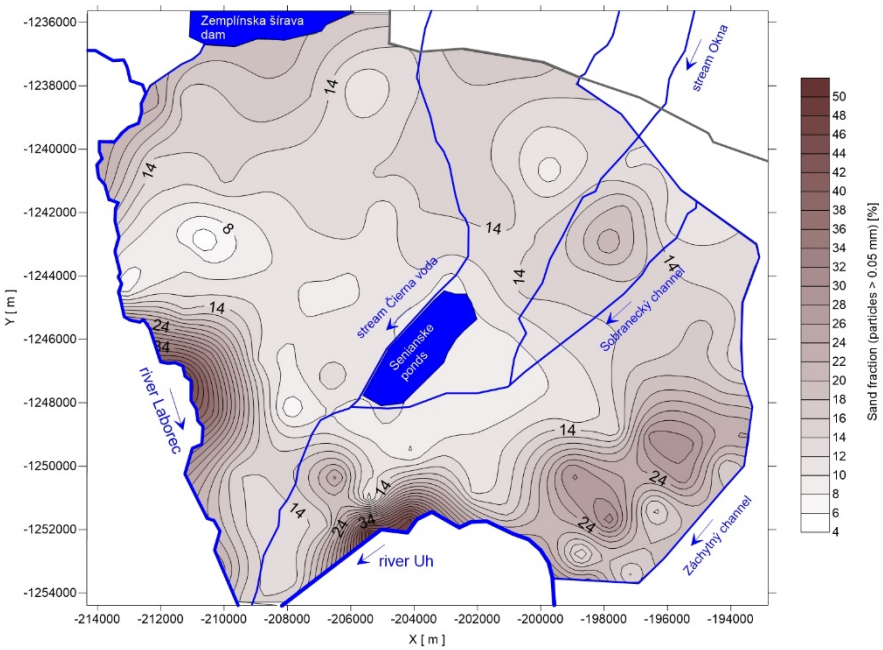

Fig. 8 Map of sand distribution (particles > 0,05 mm)

Θ_s values are expressed in percentages and their values vary from 16.39% to 53.57%. The average value of Θ_s is 40.72%. For the illustration, there is in **Fig. 7** shown spatial distribution of colloidal clay particles (particles < 0.001 mm – (I. fraction)). Particles < 0.001 mm (I. fraction) have the highest influence to the ability of soil to hold water. From the **Fig. 7** it is evident the high areal variability of the clay particles and thus hydrophysical properties of soils.

Areas with the highest values of Θ_s and hence the places with the highest occurrence of clays are found in the central part of the investigated area. This part is the depression of the investigation area with the lowest altitudes (see **Fig. 1**). This is given by the genesis of clays, which have their origin in the water meadows in depressed parts of the area. On the other hand, the places with the lowest values of Θ_s are located near rivers Laborec and Uh, and correlate to the content of sand fraction, which has its origin in the transport activities of these rivers. Map containing sand fraction is shown in **Fig. 8**.

4. CONCLUSIONS

In recent years, a lot of research work is focused on the development of computational methods and procedures for effectively obtaining the necessary hydrophysical soil characteristics from the relatively easy obtained data, such as the data of soil texture. This paper gives an example of using pedotransfer function for regionalization of depressed area according to the value of saturated water content. For this research were picked up 106 disturbed soil samples from the predefined area of ESL. Based on the particle size analysis was used pedotransfer function for quantification of Θ_s. According the values of Θ_s was performed regionalization of investigated area of ESL. Areas with the highest values of Θ_s strongly correspond to a depression in the central part of the investigated area. The lowest values of Θ_s were identified in soils close to rivers, where dominates the sand fraction.

5. ACKNOWLEDGMENTS

The authors would like also thank for the kind support of the project VEGA 2/0062/16.

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Soil Water Storage Modelling with Aspect to Volume Changes

B. Kandra, D. Pavelková

***Abstract* – In the present contribution was through the soil water storages quantified the impact of soil volume changes on the results of numerical simulations of its water regime. Two approaches were compared. The previous approach disregarded the changes in the volume of soil samples in the process of drying and a subsequent determination of volumetric moisture. This resulted in inaccurate determination of the course of soil retention curves which was also reflected in the numerical simulations. The second approach took into consideration the change in volume and allowed more accurate calculation of the volumetric moisture of samples. Following, after obtaining more accurate courses of retention curves were by means of numerical simulations calculated water storages which correspond better with real conditions.**

***Keywords* – heavy soils, soil volume changes, water retention curve, water regime modelling.**

1. INTRODUCTION

Knowledge about the water regime of soils is acquired by fieldwork monitoring or by calculations. The calculation methods are based on the numerical simulation by means of mathematical models [1]–[3]. At present, numerical simulations and numerical experiments are applied in dealing with hydrological processes. Experimental methods allow actively participate in the analysed hydrological processes. In numerical experiments, the inputs are systematically altered and outputs are subsequently analysed within hydrological system. It is also possible to meet the requirement of selected inputs repeatability.

Obtaining the input data for mathematical models is often difficult. One of the crucial input data is a soil water retention curve (WRC). WRC significantly affects the results of calculations and analysis of water regime of heavy soils. Measurement,

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evaluation and application of WRC in heavy soils have its specific problems. The source of these problems is a high content of clay minerals. These particles cause the volume changes in soils during the moisture changes [4], [5]. In the conditions of Eastern Slovakian Lowland (ESL), it was demonstrated that the total volume of soil shrinkage is up to 40% shrinkage in comparison to saturated state. The volume changes are three-dimensional processes. Under natural conditions, this process is manifested by vertical movement of the soil surface (in ESL up to 0.13 m) and by formation of cracks [6]. The result is a formation of two-domain soil structure which consists of soil matrix and cracks. The formation of two-domain soil structure significantly alters the dynamism of hydrological processes in unsaturated zone. Moreover, the formation of this structure may be also manifested with transport processes and during the sudden rainfall occurring in drought seasons. In the ESL conditions, the cracks may capture up to 50 mm of rainfall due to their retention capacity. In the laboratory conditions, the volume changes are manifested by the change of geometric dimensions of examined soil samples. During the WRC measurement of heavy soils, it is necessary to measure respective volume changes of soil samples. When volume changes are not taken into account, it causes a distorted determination of WRC course. The application of such distorted inputs leads to errors in numerical simulations. Consequently, such errors affect the results of analysis.

The aim of the present paper is to quantify the effect of soil volume changes on the results of numerical simulation of its water regime. A degree of impact of volume changes was studied through the model outputs. Two courses of WRC were compared. In the first case were the volume changes reflected while in the second case they were neglected.

2. EXPERIMENT DESCRIPTION

To investigate the effects of volume changes on the results of numerical simulation were selected the Milhostov locality situated in the ESL (Fig. 1). The selection was based on the texture of analysed soil profile, with two material layers (within 0.7 m and over 0.7 m). Material layers have a slightly different texture. In both, there was an assumption of the large volume changes for a greater percentage of clay fractions.

Fig. 1 Localization of the observed area

The volume changes were measured in the process of drying of intact soil samples using digital vernier calliper (Fig. 2).

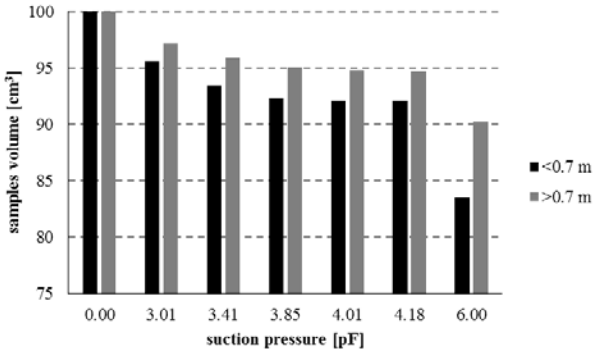

Fig. 2 Average samples volume according to pressure

The undisturbed soil samples were collected from the soil surface divided into depth intervals of 0.1 m to 1 m. The process of drying the soil samples was conducted in the equipment designed for measuring the WRC by hyperbaric method. Methodology for measuring was based on the ISO standard [7]. Selection of pressures for drainage of samples followed the rules of international ring test laboratories [8]. Samples were continuously weighted due to the measurement of moisture loss. At the end of the measurement, volumetric moisture Θ_{100} and Θ were calculated. Θ_{100} presents a previous method of measurement when the volumetric moisture content is expressed per 100 cm^3 (Kopecky ring). Θ is moisture related to the actual volume of the soil sample after shrinkage. These measurements were used in the analytical expression of WRC using the RETC program. RETC program is based on the calculations of pedotransfer functions. As an input the program calculation uses the data about the grain size composition of soil and WRC points measurement. Based on the measured points, two courses of WRC were calculated. An unreal course Θ_{100} which did not take into account the volume changes and a real one Θ which counted with the volume changes (Fig. 3).

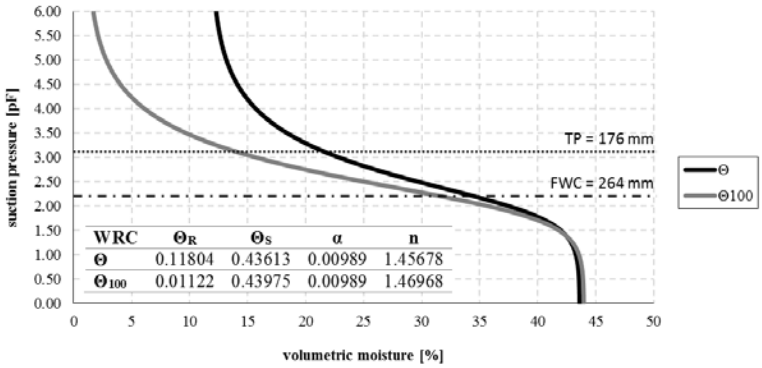

Fig. 3 Real and unreal courses of WRC

Thus determined retention curves were applied in numerical simulations on a mathematical model GLOBAL. For the purpose of simulations 6 growing seasons

(GS) were selected (2001, 2003, 2004, 2006, 2007 and 2010). From the selected seasons were available a field measurements of soil volumetric moisture to a depth of 0.8 m. On these periods was based the earlier verification of used model GLOBAL. Data from field measurements of soil moisture allow better interpretation of model output differences. Meteorological and crop characteristics in the simulations were based on real Milhostov conditions [9].

The impact of volume changes on the results of numerical simulations was evaluated through the soil water storage (WS) to a depth of 0.8 m. In comparison figuring both extremely dry and extremely wet growing seasons.

3. RESULTS AND SIGNIFICANCES

Soil profile of Milhostov is not homogeneous, but formed from two material layers. WRC in Figure 3 characterize first of them. The first layer is more important for the evaluation and is defined to a depth of 0.7 m. The other one is defined by a depth more than 0.7 m. Material layers have a slightly different texture. The figures shows two courses of WRC. The real WRC Θ, based on laboratory measurements which counted with the volume changes and unreal Θ_{100}, which did not take into account the volume changes. Thus constitutes an error in WRC estimation. In unreal WRC, the volumetric moistures are related to the volume of Kopecky ring (100 cm^3).

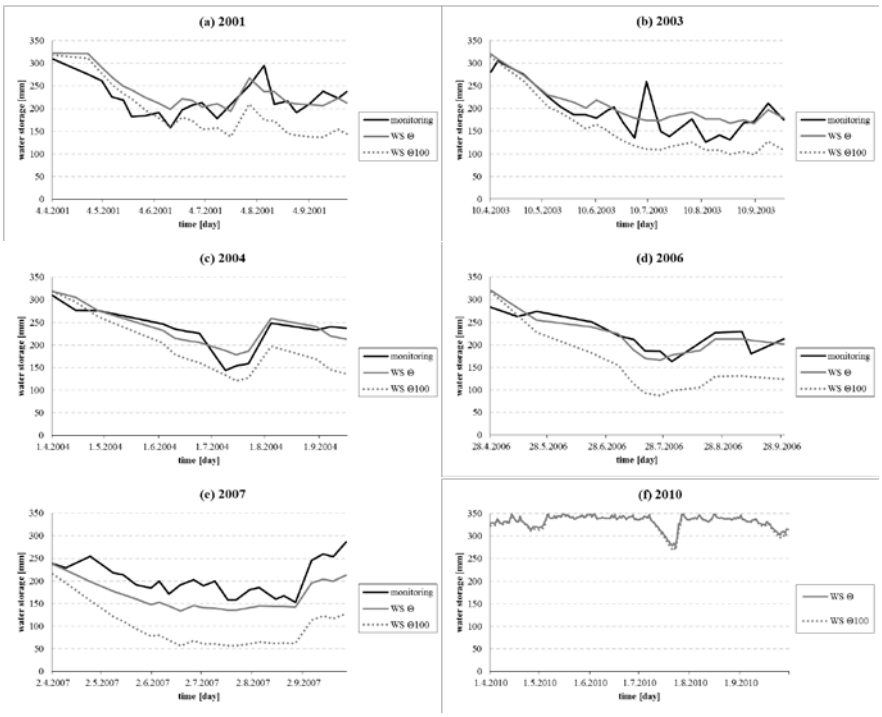

Fig. 4 Soil water storage in 0.8 m depth

The Figure shows the parameters values (Θ_R – residual moisture content, Θ_S – saturated moisture content, α and n parameters) of WRC analytical expression by the RETC program. The differences in WRC courses are quantified through the WS to a depth of 0.8 m (Fig. 4).

The average differences in water storage between WS Θ and WS Θ_{100} in evaluated GS are as follows: 2001 (44,6 mm), 2003 (52,2 mm), 2004 (45,2 mm), 2006 (68,33 mm), 2007 (70,73 mm) a 2010 (1,23 mm). The highest differences were during the driest years 2007 (e), 2006 (d) and 2003 (b). Within the GS were smallest differences in the WS in the spring months (April, May) and highest differences in the summer and autumn months. The particular case was the year 2010 (f). This year was significantly wet and differences in WS were minimal. The lowest value of WS in this GS was 264 mm, which corresponds to 33% of volumetric moisture. On WRC this moisture corresponds to the field water capacity (FWC), with suction pressure (pF) of 2.1 - 2.5. Significant differences in water storages between WS Θ and WS Θ_{100} start at a value of around 200 mm and grow with the decrease in WS below this level. On the WRC this value corresponding to 25 % soil volumetric moisture, near threshold point (TP) hydrolimit (pF 3.1 to 3.3) with 22 % of soil moisture and 176 mm WS (Fig. 3). At this point begins the influence of soil samples shrinkage on the WRC estimation accuracy and hence the accuracy of the model output. The Table 1 shows the values of the linear regression coefficients (R^2) between monitored and calculated soil water storages Θ and Θ_{100}. Effect of volume changes was reflected in the regression coefficients too.

Table. 1. Linear correlation between monitored WS and calculated WS Θ, Θ_{100}

Linear regression coefficient R^2										
GS	**2001**		**2003**		**2004**		**2006**		**2007**	
Model	**Θ**	**Θ_{100}**	**Θ**	**Θ_{100}**	**Θ**	**Θ_{100}**	**Θ**	**Θ_{100}**	**Θ**	**Θ_{100}**
Monitoring	0.51	0.31	0.68	0.68	0.77	0.69	0.80	0.79	0.73	0.54

4. CONCLUSIONS

The paper has analysed the problem of measuring WRC in heavy soil. The results present the differences of WRC measured in soil samples taken from Milhostov locality situated in ESL. Soil profile in this locality is characteristic of high content of clay particles, i.e. in average 30.94 %. During the drainage process, the results have proven the raising rate of shrinkage in soil columns which was dependent on the content of clay and moisture in samples. The rate of shrinkage has affected the results of the WRC measurement and consequently its analytical determination using the RETC program.

The differences in courses of WRC are visible in Figure 3. Shrinkage of the samples had an impact on the calculation of their volumetric moisture. Shrinkage of samples volume caused the increase in volumetric moistures Θ when comparing these results with the volumetric moisture calculated for initial volume of samples 100 cm^3 (Θ_{100}).

The importance of measuring the volume changes in heavy soils in determining the WRC was quantified through the WS to a depth of 0.8 m. The most significant differences between Θ and Θ_{100} were shown on the WS in dry and extremely dry GS

and months. On these moisture values it was demonstrated significant volume changes of the soil. For the evaluated soil profile was estimated a limit of soil moisture and pF pressure at which the differences between WS at a level of moisture Θ and Θ_{100} are significant. This value corresponding to the value near TP hydrolimit. The resulting differences were reflected in the coefficient of linear regression.

This approach brought more accurate methodology for measuring soil retention curves of heavy soils. Accurate determination of soil hydrophysical characteristics is essential for proper function of mathematical models which is confirmed by the obtained results.

5. ACKNOWLEDGMENTS

The authors would like to thank for the kind support of the project VEGA 2/0062/16.

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Landslide Hazard Assessment in Regional Scale: Implementation and Reliability

N. S. Klimis, K. A. Papatheodorou and B. N. Margaris

***Abstract* – Albeit Landslide Hazard Assessment (LHA) at a regional scale is a powerful tool for rational decisions for strategic planning, it has been used very scarcely by local, regional and national authorities. Main reasons for that, are: lack of data and metadata, lack of accessible landslide inventories, restricted budgets and the multitude of methods and results for LHA. In the present work, three different methods for LHA, scientifically sound and internationally well recognized, have been tested in Greece and other Black Sea countries. We refer to the pilot implementation areas in Greece, where those methods have been extensively tested between them and with field evidence, in terms of feasibility, reliability, accuracy and usefulness to the end-user.**

***Keywords* – Landslide Hazard Assessment, Regional Scale, Physically based Models, Factor of Safety, FEMA (HazUS), Mora and Vahrson.**

1. INTRODUCTION

Landslide Hazard Assessment (LHA) on a regional scale can provide useful information which when combined with a preliminary risk assessment can support decisions regarding strategic planning for disaster prevention. Landslide Hazard maps can be used to assess the potential risks, prioritize areas in terms of the necessity to apply preventive measures and plan site-specific investigations (slope stability analyses), which require a more detailed planning for funding and implementation. Such a strategic planning can provide the State, Regional and Local Administration with a useful and efficient tool to effectively plan landslide disaster mitigation measures in both their financial and technical aspects.

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Numerous methods exist for assessing Landslide Hazard on regional scales each with its own advantages and pitfalls. The multitude of methods used, results into non comparable outputs, a fact which, especially in cross-border areas, forms a block for cross-border cooperation. Among the major problems that a large number of countries and their authorities face when dealing with this issue is: lack of accessible landslide inventories; lack of data and meta-data and restricted budgets available for research and investigation. Within this context, feasible LHA methods are suggested, in order to develop a harmonized basis of communication for this specific issue across the Black Sea area countries or other bordering countries within the EU.

Three different methods for LHA have been tested and evaluated; all three of them are scientifically sound and are used internationally. The outputs of the selected methods were compared to field observations, in order to assess their adaptability to local specific conditions and to evaluate them by comparing their outputs to actual landslides recorded in the field, as landslide inventories are not accessible for most of the EU and the Black Sea area countries.

An additional target of this procedure was the development of landslide hazard maps which can support informed decisions made by the State, Regional and Local Administration regarding strategic planning for landslide disaster prevention.

2. DESCRIPTION OF METHODS SELECTED FOR REGIONAL LHA

A large number LHA methods used worldwide, was extensively reviewed and evaluated according to their data requirements, their complexity in implementation, their flexibility on adaptation to local conditions, their cost of implementation and finally, the completeness of provided results as well as the reliability and accuracy of their outputs. The methods finally selected to be further tested and evaluated include:

A. The Mora & Vahrson method (1994) [1]: a macrozonation method for landslide hazard determination.

B. The method proposed and used by the Federal Emergency Management Agency of the USA, widely known as HazUS (1999) [2] and

C. The calculation of Factor of Safety (FoS) based on the Infinite Slope Model (ISM) for planar and the Deterministic Model for circular landslides.

The above methods have been applied in two Pilot Implementation areas (PIA) in Greece: the broader area of Serres (Fig.1a) and the broader area of Nymfaia (Fig.1b), focusing on the vertical road axis Komotini - Nymfaia - Hellenic/Bulgarian border. The scale of implementation is 1:50,000, as geologic and topographic maps available of the Hellenic PIAs exist in the above scale.

The data required for applying the above selected methods, include: a) Digitized Topographic Maps at a scale 1:50,000 with a contour interval of 20m, b) Digitized Geologic Maps (faults and dip and dip direction of geologic planes were also digitized), c) Road network, urban areas, general information, d) Geologic Maps at a scale 1:50,000, e) Engineering Geologic reconnaissance results f) Rainfall Data (30 years time series, where available) including Mean Monthly rainfall (mm) and maximum daily precipitations from meteorological stations within and around the examined areas, g) Ground Motion: Peak Ground Acceleration (PGA) for different

mean return periods (100, 200, 475 and 1000 years), h) the Geological Strength Index, GSI, (Marinos et al., 2005) [3], when geologic formations are rocks.

All raster files developed had a spatial resolution (pixel size) of 15x15m. Data were harmonized, georeferenced and used as input into an OpenSource GIS developed for the LHA.

Fig. 1 Pilot Implementation Areas in Greece: Serres and Komotini-Nymfaia areas

A) The method proposed by Mora and Vahrson (1994) [1] for the prediction of susceptible zones was based on case studies of slope failures triggered both by earthquakes and by heavy rainfall. According to this method, three factors are considered as the factors influencing the susceptibility to landslides: relative relief, lithological conditions and soil moisture. In addition, two factors: seismicity and rainfall intensity, are incorporated as the triggering factors.

By combining those factors, a degree of slope failure hazard (H_ℓ) is defined as:

$$H_\ell = Susceptibility * Trigger \quad \text{or} \quad H_\ell = (S_r * S_\ell * S_h) * (T_s * T_p) \qquad (1)$$

where,

H_ℓ : landslide hazard index

S_r : value of relative relief index, $R_r = (h_{max} - h_{min})/km^2$

S_ℓ : value of lithological susceptibility

S_h : value of index of influence of natural humidity of the soil

T_s : value of influence of seismic intensity

T_p : value of influence of rainfall precipitation intensity

B) The FEMA method (HazUS, 1999) [2]

The procedure proposed and used by the Federal Emergency Management Agency –FEMA (USA) to assess Landslide Hazard on regional scales is a three step procedure and it applies only when the triggering factor is a seismic event:

1. Assess Landslide Susceptibility under static conditions

2. Assess the Critical Acceleration (A_c), where "critical" is the peak seismic horizontal acceleration applied on a slope which produces a pseudostatic Factor of Safety equal to one (F_S=1.0).

3. Compare the A_c to the expected PGA by calculating the ratio A_c/PGA

All the above parameters are calculated for two different moisture/groundwater conditions: "*dry*" meaning that the groundwater level is below the level of sliding surface and "*wet*" meaning that the groundwater level is at ground surface fully saturated.

Landslide susceptibility under *static conditions* is evaluated taking into consideration the engineering geologic conditions and the slope angle for the two predefined moisture conditions (wet and dry).

Landslide susceptibility under *seismic conditions* is based on the limit equilibrium principle where an earthquake is considered as a horizontal force (seismic coefficient * weight of the potentially sliding mass of a slope). The crucial parameter is A_c which is calculated as a complex function of slope, geologic group, steepness, water table, type of land sliding and history of previous slope performance (Wilson & Keefer, 1985) [4]. There are certain bounds that limit the slope values for which a critical acceleration can be defined as shown in Fig.2.

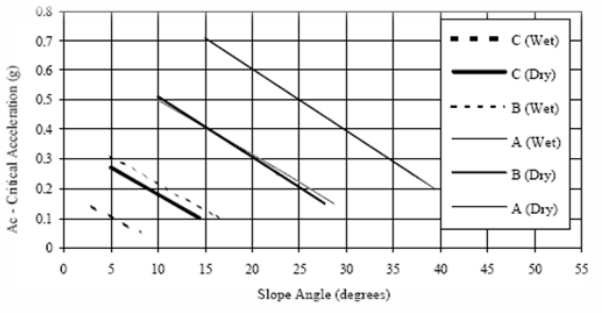

Fig. 2 A_c as a function of slope and geologic group [4]

As per the third step of the FEMA method, a probabilistic seismic hazard analysis is deemed necessary. In this way values of PGA are attributed in the examined area on a grid, based on local Ground Motion Prediction Equations (GMPEs) which take into account the site conditions. In Greece the most recent local GMPEs are those proposed by Skarlatoudis et al., 2003 [5] in "2" (when focal depth is unknown).

$$logPGA = 1.07 + 0.45M - 1.35xlog(R + 6) + 0.09F + 0.06S \pm 0.286\varepsilon \qquad (2)$$

where,

M : magnitude of the earthquake

R : epicentral distance in Km

F : parameter dependent on the type of seismic fault (F=0 for a normal fault; F=1 for a strike-slip fault; F=2 for a reverse fault)

S : parameter dependent on the soil class (class B, S=0; class C, S=0.058; class D, S=0.125)

ε : parameter denoting the number of times that the standard deviation will be considered in the equation (the last term of the equation is the standard deviation)

LHA under seismic conditions according to FEMA method is based on the calculation of Permanent Ground Displacements – PGD (Goodman and Seed, 1966) [6]. The method is applicable to the LHA for "shallow' landslides (depth of slip surface less than10m max). The idea behind this method is the fact that each accelerogram may cause a permanent displacement of a sliding mass on a slope, in case PGA exceeds A_c. For each cycle, there is an expected permanent displacement ($E[d/A_{is}]$, so for a number (n) of cycles the total expected permanent displacement is:

$$E(PGD) = E\left[{}^{d}/_{A_{is}}\right] * A_{is} * n \tag{3}$$

where,

A_{is} : is the induced acceleration (in decimal fraction of g's); it equals PGA for "shallow" landslides, whilst A_{is}=(2/3)PGA for deep and large landslides

$E[d/A_{is}]$: is the expected displacement factor per cycle

n : is the number of cycles which is calculated as a function of the Earthquake Moment Magnitude (M_w) (Seed & Idriss, 1982) [7]:

$$n = 0.3419M_w^3 - 5.5214M_w^2 + 33.6154M_w - 70.7692 \tag{4}$$

The expected displacement factor per cycle is given as a function of the ratio A_c/PGA [6]. Once this number is calculated, as well as the M_w, then the expected PGD is estimated according to "(3)".

C) The method of FoS

Calculation of FoS of natural and cut slopes falls into the physically based LHA methods which are based on modeling of slope failure processes. This method is applicable over large areas provided that geological and geomorphological conditions are fairly homogeneous and landslide types are relatively simple. It can be implemented in areas with incomplete or even non-existing landslide inventories. Physically based LHA methods can be applied using the infinite slope concept to model shallow landslides or the deterministic model for circular failures. Those methods take into account as triggering factors, rainfall and transient groundwater response or the ground motion induced by earthquakes.

LHA method is based on *the infinite slope model* in order to describe the failure mechanism of shallow landslides, triggered by precipitation under static conditions:

$$F_s = \frac{c' + (\gamma_{app} - m * \gamma_w) * z * (cos\beta)^2 * tan\varphi'}{\gamma_{app} * z * sin\beta * cos\beta} \tag{5}$$

where,

φ' : effective angle of friction of geomaterial (^{0})

c' : effective cohesion of geomaterial (kPa),

γ : specific weight (kN/m^3),
β : slope angle (Deg),
γ_w : specific weight of the water (kN/m^3),
z : normal thickness of the failure slab (m),
m : percentage of the water saturated failure slab (%)
$\gamma_{app} = \gamma*(1-m) + \gamma_{sat}*m$, if slope is dry then $\gamma_{app} = \gamma$ (m=0%), if completely saturated $\gamma_{app} = \gamma_{sat}$

The same physically based model (infinite slope) is used when the triggering factor is the earthquake. In this case the driving equation is modified as follows:

$$F_S = \frac{c' + (z*\gamma*(cos\beta)^2 - z*\rho*a*cos\beta*sin\beta - \gamma_w*z_w*(cos\beta)^2)*tan\varphi'}{z*\gamma*sin\beta*cos\beta + z*\rho*a*(cos\beta)^2} \quad (6)$$

where,
φ' : effective angle of friction of geomaterial (^{0})
c' : effective cohesion of geomaterial (kPa),
γ : specific weight of geomaterial (kN/m^3),
ρ : bulk density (Kg/m^3)
β : slope angle (Deg),
γ_w : specific weight of the water (kN/m^3),
z : normal thickness of the failure slab (m)
m : percentage of the water saturated failure slab (%),
a : earthquake acceleration (m/sec^2)

The geotechnical parameters are crucial parameters as they affect largely the value of FoS. The shear strength parameters of rock formations outcropping in the examined area were calculated using the Hoek & Brown failure criterion combined with the M-C failure criterion. For each geologic formation, two pairs of effective cohesion (c') and effective angle of friction (φ') for low and high normal stresses were calculated in order to adopt the minimum values from each approach and to come up with a "conservative" pair of shear strength parameters (φ', c').

The most difficult parameters to assess in order to implement the infinite slope model are: i) the normal thickness (z) of the sliding slab and ii) the percentage of saturation (m%) of the sliding slab. The normal thickness (z) was defined as a parameter with a single value of either 1, 5 or 10m, whilst alternatively, the "z" parameter was calculated using a physically based model that links it to soil and regolith development on natural slopes as suggested by previous researchers (Saulnier et al, 1997) [8].

3. Results and Evaluation Of Methods

The evaluation of the methods selected above, to assess Landslide Hazard in two Pilot Implementation Areas in Greece, was based on the comparison of field

work data with model predictions thus estimating the reliability, accuracy and quality of results by comparing them to actual facts recorded in the field (Figs. 3 & 4).

Quality of results is always related to intrinsic weaknesses of the methods and the level of assumptions and generalizations during each of the processes followed.

Additional parameters were also considered included the models ability to provide detailed information, spatial resolution of their outputs, compliance of their outputs with standing procedures and regulations and their requirements in terms of data and processing.

Finally, to assess their potential for dissemination and broader use by a network of scientists including personnel of state authorities, their “complexity” was also considered (Klimis et al., 2015) [9].

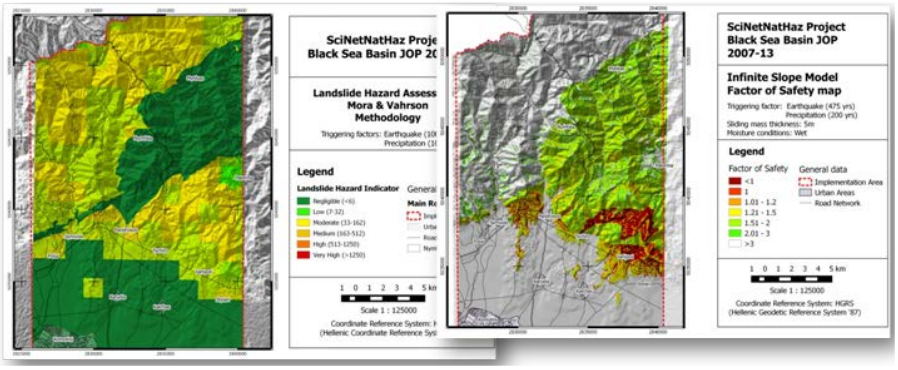

Fig. 3 Comparison of outputs from Mora and Vahrson method (left) to FoS method (right) based on the Infinite Slope Model in PIA of Serres

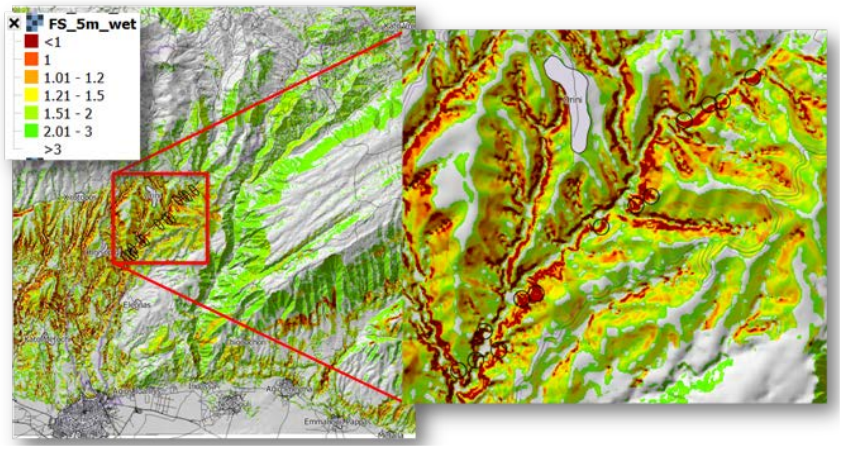

Fig. 4 Comparison of outputs from FoS method for "*wet*" conditions, with a thickness of a sliding slab, z=5m, on natural slopes in PIA of Serres. The black cycles (right part of the figure) are locations of landslides on natural slopes.

Evaluation of the selected methods has also been extended in the PIA of Nymfaia, where regional LHA assessments were significantly improved by using information produced using remote sensing techniques. In fact, those techniques offered the possibility to detect fractured / weathered zones, as lineaments represented with a buffer zone of 30m, where mechanical characteristics of soil and rock masses have been appropriately modified. Consequently, rock masses initially treated as homogeneous and isotropic as given in the geological maps of scale 1:50.000, have been differentiated by modifying their mechanical, physical and hydraulic characteristics and resulted in different FoS (Fig. 5), which proved to be close enough to reality.

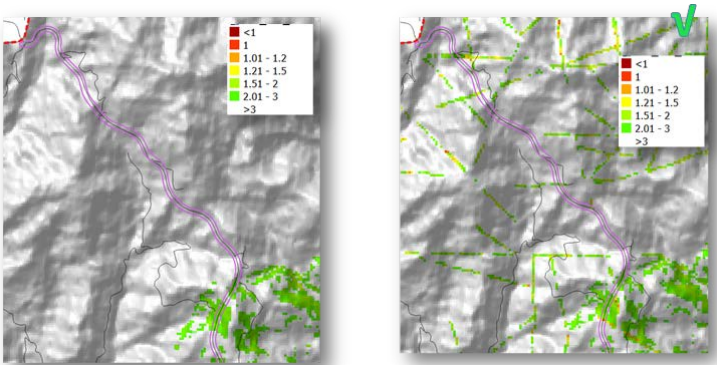

Fig. 5 Prediction of FoS method for a rainfall of 50 years, with a sliding slab thickness, z=5m on *natural slopes* in Nymfaia PIA *without* (left) and *with* (right) fractured / weathered zones, located by remote sensing techniques used. Gray color corresponds to F_S>3.

The very close relationship between the predicted values of F_S to actual slope failures was evident in many high cut slope along the Nymfaia-BG border road axis (Fig.6). Almost all of the predictions of examined cut slopes along this road axis, were both qualitatively (rock mass conditions) and spatially (locations within ±15m) very accurate, a fact indicative of the high reliability and accuracy of the landslide hazard assessment. This is an impressive performance especially when considering that this level of special accuracy was achieved using 1:50.000 scale maps (Fig. 6) and readily available Landsat TM and ETM+ data available through the Earth Science Data Interface (ESDI) at the Global Land Cover Facility (http://glcfapp.glcf.umd.edu:8080/esdi/).

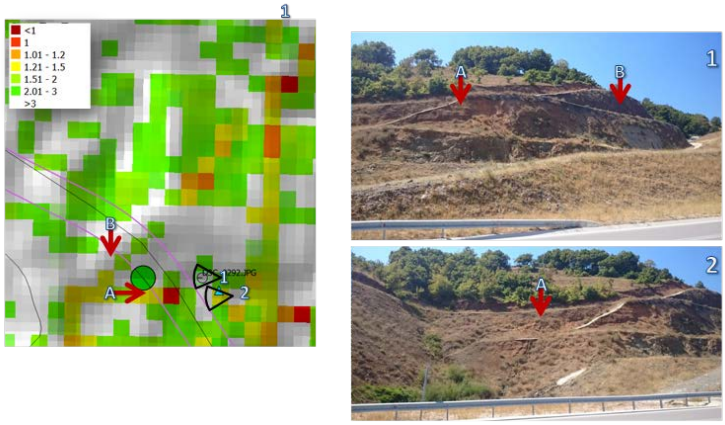

Fig. 6 Location along the vertical road axis from Komotini - Nymfaia to Hellenic-Bulgarian borders, where predicted F_S values are evaluated by in-situ observations. Point B: High Fs value denotes a safe cut slope; Point A: very low Fs values indicating potential failure, seen in the respective photo.

4. CONCLUSIONS

Different worldwide used, scientifically sound methods to assess LH at regional scales were evaluated, based on their complexity of use, completeness of their outputs and adaptability to local conditions, feasibility of implementation in terms of data and hardware requirements. Their final evaluation was based on comparison of their predictions to field observations. As resulted:

1. the method of *Mohra and Vahrson* could be considered as a crude and approximate method to assess regional LHA in a rather qualitative way for both triggering factors (water and earthquake), since the calculated hazard indicator is an arbitrary index denoting rather susceptibility than hazard to slide.

2. The method proposed and used by *FEMA (HazUS)* is restricted to LHA only if the triggering factor is an earthquake; it is a rather demanding method in terms of data needed for its application and an important number of intermediate "products" (maps) has to be calculated in order to assess Permanent Ground Displacemens (PGD), which is the end-product of this method. Despite difficulties in application, complexity and understanding, this method can provide results in terms of permanent seismically-induced displacements, which is actually a realistic way to perceive the phenomenon of land sliding.

3. The method of *FoS* is the most comprehensive among the three methods, as a physically based method based on a simple slope failure process. This method applies to both static and seismic conditions, where water or earthquake are respectively the triggering factors. The results, i.e. maps with F_S values, are well perceived by end users (usually engineers and geologists). The FoS method outputs were also, as compare to outputs of the other methodologies, more spatially analytical and by far the closest to field observations. Therefore, this method is considered to be the most

efficient as compared to the other ones since it appears to be the most feasible (easy to implement, with average data requirements) and the most accurate and reliable as has been seen by field observations in Serres and Nymfaia areas.

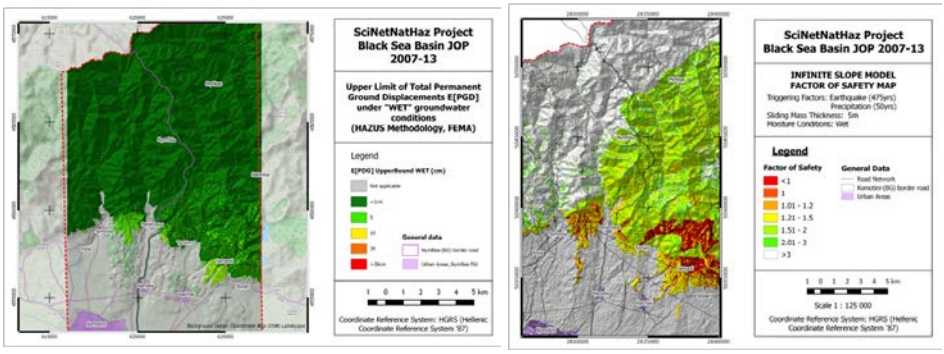

Fig. 7 The FEMA method (left part) and the FoS method (right part) applied at the PIA of Nymfaia. The seismic event with a mean return period of 475 years is the triggering factor. The Hazard maps are given in PGDs (left) and in values of F_S (right)

Nevertheless, the combined knowledge of maps resulting from both FEMA method and FoS method (Fig. 7.), based either on the infinite slope model for shallow landslides or the deterministic model for circular landslides, is a promising tool that deserves further exploitation and research.

5. ACKNOWLEDGMENTS

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New areas of activity on the market in Romania: Facility Management

Gabriela Mehedintu, Cristian Vasiliu

***Abstract* – The graduates' chances of insertion on the facility management market is high due to the fact that this field is at the beginning, with great opportunities and safe future trends because of the demand for integrated services such as those in this field. This paper presents the high potential of the facility management sector in Romania and analyses the students' degree of insertion on this market.**

***Keywords* – facility management, services, market insertion.**

1. INTRODUCTION

The market dynamics of the facility management field at national and international levels, the market demands, the service providers' and the customers' expectations of integrated support services determine the graduates' degree of insertion on the labour market of facility management. The field of facility management is new, booming and with clear continuing growth trends. This entire dynamic requires providing specialized services and staff to match. Facility management is a new area of activity. The Romanian educational system is not yet adapted to the evolution of this market, but practitioners can easily develop requirements for the future employees. Based on the definitions of facility management, the paper will present a study showing the young graduates' insertion opportunities on this market.

2. DEFINITIONS AND APPROACHES OF FACILITY MANAGEMENT

The concept of facility management (FM for short) was introduced in the specialized language of economics around the 1970s.

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In 1982, the first definition of facility management was proposed, a definition which integrated the human factor into the processes, technologies and spaces from the built environment. According to the European standard in the field (EN 15221) facility management is defined as the „integration of processes within an organization to maintain and develop the agreed services which support and improve the effectiveness of its primary activities" [3].

This definition emphasizes all the services of an organization, lives alone the built environment and focuses on satisfying customers whose requirements are grouped into two main categories:

- Space & Infrastructure: refers to the requirement for space services (e.g. space planning, workplace, design, construction, maintenance, technical infrastructure, cleaning);
- People & Organization: refers to the demand for health and organizational services (e.g. health services, catering, event management, hospitality, security, safety, logistics, document management).

Among these fields, facility management contributes to the sustainable development of a company through the three areas: economic, social, and environmental.

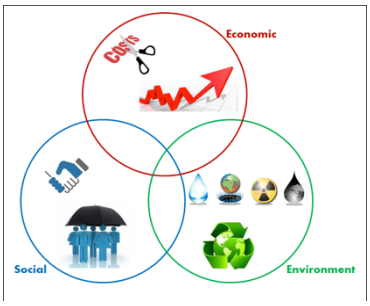


Fig. 1 Sustainability through FM on three directions

In other words, the companies' practices of sustainability, to achieve long-term growth, are determined by environment friendly behaviour with a positive impact on the whole society.

In facility management cost-effectiveness, productivity improvement, efficiency, employee quality of life, and social impact are key concepts.

3. RESULTS AND SIGNIFICANCES

Not only have the definitions given by the different institutions of facility management over time shown an evolution in this area, but also the structure of the covered services. From the 1980 one may notice a tendency to depart from the domestic services strictly linked to the space of a building and to head to outsourcing, first as specialized, individual (1990) services, then as grouped services and then integrated ones, with a strong character oriented towards partnerships and the creation of added value.

The figure below presents in detail each stage of evolution [4]:

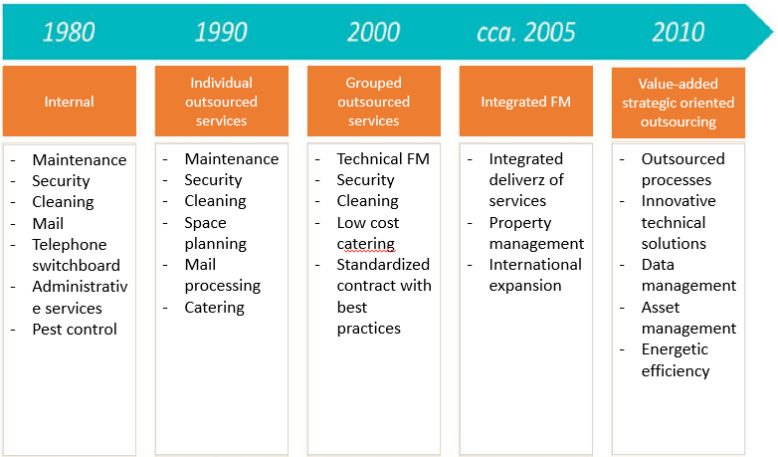


Fig. 2 Stages of evolution of facility management

The development of the facility management field is correlated with the exponential technological developments, with the dynamics of the global economy and the businesses environment which are oriented towards strategies of economic efficiency by reducing the overall costs (through outsourcing and the acquisition of specialized and integrated services) and by an efficient management of the assets (especially of the buildings that have a construction cost of approximately 20% of the total lifecycle costs, while the costs of administration, operation and maintenance account for the remaining 80% of the costs).

The specialists in the field believe that there are opportunities to expand services to the support areas of an organization which, in the near future, will determine the expanding of facility management towards the quasi-totality of the support services of an organization.

4. THE FACILITY MANAGEMENT SECTOR IN ROMANIA. FACTS AND DEVELOPMENT TRENDS

The facility management activity ignited during the early 2000s real estate and construction frenzy. The new buildings required managing, maintaining and operating at high standards. Also, the avalanche of foreign companies on the Romanian market, which were familiar with the facility management field, led to a demand for professionalism in this field both on the service side and the qualified personnel side. The answer to these demands was the creation of ROFMA (the Romanian Facility Management Association) in 2009.

The typology of the companies on the Romanian market, depending on the number and types of services provided, include:

- companies providing only one type of service (Single service) – for example, cleaning services, technical maintenance services; catering services, security services, etc.;
- companies providing a small number of interconnected services (Bundled services) – cleaning / pest control / waste management;
- companies providing several types of interconnected or not services (multi services);
- companies that have and offer a portfolio of integrated services, variable and flexible.

The number of companies from Romania, active in the facility management field at the end of 2014, was of 6,299, representing an increase of 70% compared to 2010. The total turnover of these companies represents about 0.9% of Romania's GDP, and the number of employees approximately 2.6% of the total employees of the national economy.[5]

In 2014 a study was addressed to the specialists in the field of Facility Management (FM), both to consumers and service providers, the aim of the study being to assist in clarifying and understanding the specifics of the Romanian market, the phenomenon of outsourcing, the importance of the added value and innovation in facility management.[6] A number of over 200 specialists working in three main categories of players in the sector of facility management were interviewed: building owners, providers of service, and end users (tenants).

The most important elements of facility management identified by them are:

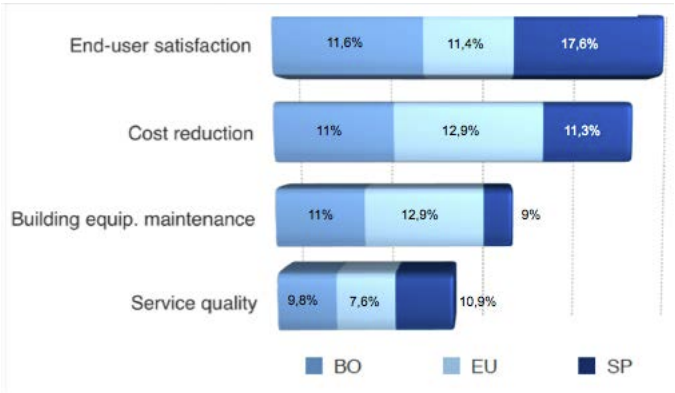


Fig. 3 Important elements of facility management

The main trends that will characterize the facility management market in Romania at the horizon of the following years are considered to be:

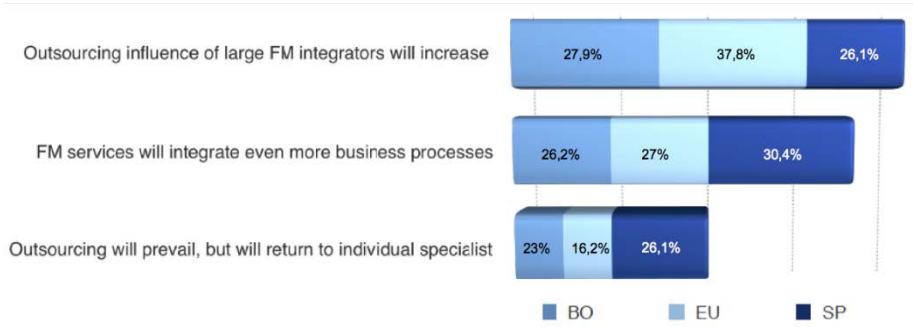


Fig. 4 Trend in the facility management market in Romania

5. FACILITY MANAGEMENT - OPPORTUNITIES FOR HIGHER EDUCATION IN ROMANIA

In Romania, because facility management is a relatively young field, currently there are no academic programs in facility management, but there are initiatives for launching such programs in the near future. At the moment, the technical universities, especially the civil engineering and technological equipment faculties offer a growing number of graduates who are recruited on the labour market by firms specialized in facility management.

In 2015, through the EU funded POSDRU/161/2.1/G/132723 project, ROFMA started the 'Study on the socio-professional insertion of the university graduates on the labour market' to know the reality of the labour market, the opportunities that it offers and the requirements of the employers for the graduates candidates in order to successfully insert them socio-professionally as a prerequisite to increase the competitiveness and attractiveness of higher education is its adaptation to the Romanian realities and labour developments [7].

The main objective of the study is the analysis to facilitate the socio-professional insertion in the area, based on a survey among companies that are active in facility management, or which have specialized departments for facility management.

The lot was selected as a pilot sample, non-random, quasi-representative, being composed of the following types of respondents (participating companies).

- Companies whose exclusive activity object, primary and / or secondary, is the following facility management services;

- Companies that have their own departments of facility management, or their equivalents.

The questionnaire was sent to a number of 173 companies that met the selection criteria listed above. A number of 53 companies agreed to answer the questionnaire totally or partially. Based on the facility manager's specific skills existing in the occupational standard, and on the respondents' options a rank of the importance of these specific skills was carried out, the result being presented in the table below:

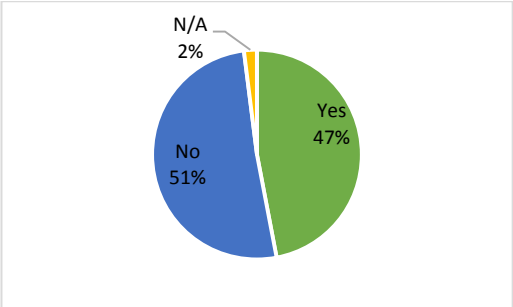
Table 1. FM competencies

Top	FM competencies
1	Develops facility management policies
2	Manages budgets in facility management
3	Develops strategies or efficient use of energy in facility management
4	Manages facility management projects
5	Understands facility management and its place in the organization
6	Provides support for the organization's business continuity in emergency situations
7	Develops working relationships with stakeholders/interested partners
8	Contributes to the organization's sustainability
9	Develops the facility management team
10	Manages facility management performance
11	Optimizes the utilization of the space
12	Participate in the acquisition of products and services for facility management
13	Manages the exploitation of the organization's facilities throughout their life cycle
14	Promotes facility management services
15	Manages facility management services

Regarding the specific competencies of facility management, the most important are considered to be: the ability to develop facilities management policies; the ability to manage budgets in facility management; the ability to develop strategies for the efficient use of energy in facility management. This ranking highlights the need to place the facility manager among the specialists within an organization, who can and should have something to say about the designing and founding of the general development strategy of the organization and about its implementation through the facility management policies.

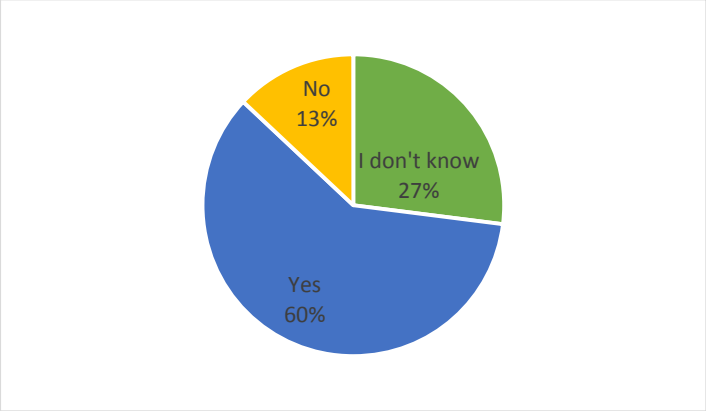
A number of questions of the survey highlighted the interest of the facility management field for the insertion of young graduates from higher education and the employers' needs for the next 2-3 years.

1. Have you recently hired young graduates from higher education for FM activities?



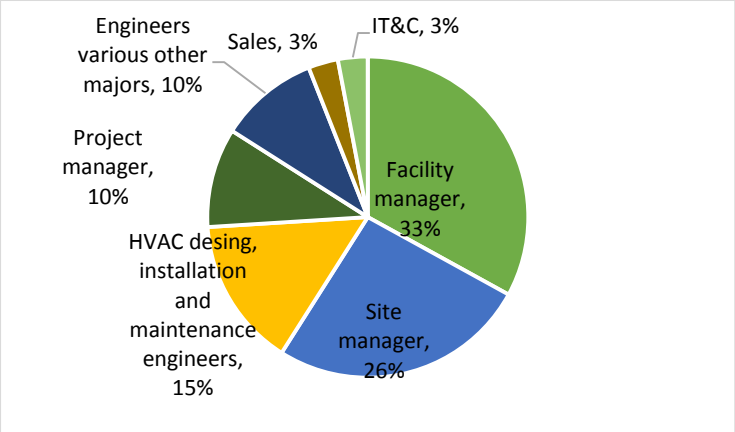
The dynamics of the graduates' employment in activities belonging to the FM field indicate that about half of the companies from the sample have recently (2013-2014) hired such personnel, The high percentage of 47% proves that companies are interested in this area and that they understood the need for young staff on a new market.

2. Do you think that in the near future (2015-2016) you will hire young graduates for FM activities?



A positive trend can be seen regarding the intent of employing young graduates, 60% of the respondents planning to hire people in the near future, which again proves the development potential of this area.

3. For what jobs (higher education) can you hardly find candidates in your recruitment process?



In the context of the insertion degree on the labour market, the jobs in the field of facility management with the most difficult recruitment are: facility managers and site managers. A possible explanation for this situation is that the two jobs are recent entrants into the active professional field and therefore the number of those prepared for these professions is still low.

From the results of this study, one may find the ideal profile (at this time) of the young university graduate, who applies for a job in the field of facility management:

- Education: Graduate of a higher education institution with an engineering profile with a major in: construction installations engineer, civil engineer, thermotechnics engineer;
- Behaviour: positive attitude, good communication, honesty, desire for improvement;
- General skills: technical and organizational skills as well as expertise in the context of an experience (relevant) in the areas related to facility management.

7. CONCLUSIONS

Facility management is a relatively new field at an international level and a very new one at the national level. It emerged in Romania in the early 2000s and since then it has been in a continuous development, due to the increased demand. The potential is high, and the companies providing facility management services are open to collaborate with the graduates from the engineering field who have technical skills and a positive attitude. The desire for improvement, training and communication are essential in this field, being the basis of the services offered by facility management and the main elements that help the sustainable development.

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Life-cycle cost of a building in the new Romanian legislation

G. Mehedinţu, N. Postăvaru

***Abstract* – At European and Romanian level, the public procurement law is changing, in order to stimulate quality of international markets. The regulations have introduced a new bid evaluation system: the life-cycle cost, a complex method, currently used only partially in-betweens but very useful in the investment decision-making.**

***Keywords* – public procurement law, life-cycle, life-cycle cost, net present value.**

1. INTRODUCTION

In recent decades we can notice more and more, both on a national and international level, the service consumers' high attention given to values as competitiveness, transparency and quality of the received services. These values are promoted by harmonizing the legislation to the European one.

In the construction field, in order to take correct investment decisions, it is useful and necessary to have a global picture of current and future costs. Current practices require as adaption element, the introduction of the term “life-cycle cost”.

The new European regulations on public procurement have introduced a new bid evaluation system: the life-cycle cost of the construction.

2. UPDATING THE EUROPEAN AND NATIONAL LEGAL FRAMEWORK

The European Commission has requested changes in the public procurement law, to harmonize the legislation at European level, in order to develop the international public procurement markets and to stimulate the trade. In this way, the non-discriminatory practices are encouraged and the solid and qualitative public

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investments are promoted. Romania has implemented the European Commission's recommendations on strengthening the public procurement system, through the Directive 2014/24/EU, whose implementation falls under Article 346 of the TFEU (Treaty on European Union) [1].

The draft law on public procurement was approved by the Government on 27th October 2015 and adopted by senators on 9nd February 2016. On 15th January 2016, the Government published draft rules for the application of the law on public procurement. On 10th February 2016, the Government adopted the Decision which amends GD. 925/2006.

The law's purpose is to increase competitiveness, equal treatment, non-discrimination, transparency and obtaining favourable results for both of the parties, based on achieving the best price-quality ratio. The new law is improved by enhancing the quality of the procurement projects through the changes in the procurement criteria for awarding contracts. It aims to increase quality, adopting as award principle, "most advantageous tender" based on 3 criteria:

- The lowest price (criterion used in most cases under the old legislation);
- The best price-quality ratio (criterion determined on the basis of some evaluation factors, including qualitative, environmental and/or social aspects regarding the subject of the public procurement contract/framework-agreement;
- The lowest cost (criteria which are determined by considering profitability, using as calculation factors such as the life-cycle cost).

The third criterion is a complex one and, as it is very little used in the Romanian practice, the paper presents further the life-cycle calculation.

3. THE CONCEPT OF LIFE-CYCLE COST

The implementation of the new public procurement law is applicable mainly in the construction sector to boost competitiveness. For this reason, it is required the study and knowledge of the life-cycle costs, i.e. the knowledge of all the costs involved in a construction, even from the investment decision until its decommissioning. The use of this criterion helps the investment decision making, as it provides the ability to compare two or more alternatives and thus the ability to choose which one reveals the minimum life-cycle cost.

In general, for the proper consideration and understanding of any construction project, the whole history of the life-cycle should be taken into account, from initiation, until its demolition or change of its construction use.

According to GEFMA (German Facility Management Association) Directive 100-1, the life-cycle phases are divided into the following steps [2]:

1. Initiation,
2. Design,
3. Construction,
4. Commissioning,
5. Purchase,
6. Operation and use,

7. Refurbishment/reuse - renovation/modernization,
8. Vacancy,
9. Recovery.

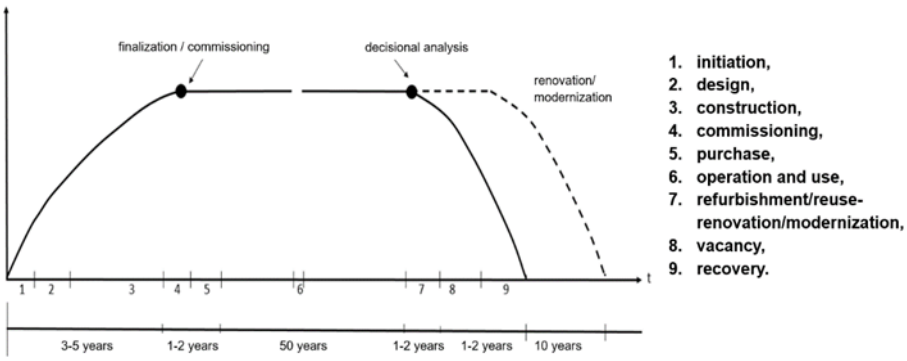

Fig. 1 The life-cycle phases

The concept of life-cycle was introduced in the '60s by Logistics Management Institute and it comes from the military field, being introduced in the construction field during the ’70s.

ISO 15686-5: 2008 defines life cycle costing as „a valuable technique which is used for predicting and assessing the cost performance of constructed assets. Life cycle costing is one form of analysis for determining whether a project meets the clients’ performance requirements.” [3]

This approach was implemented to facilitate the work of decision managers, providing them the assessments of present and future costs related to equipment and facilities, with the aim of developing long-term perspectives.

The definition, as emerges from the Standard ISO 14040 aims to reach the following targets [4]:

- Setting up a records of relevant inputs and outputs of a system;
- Evaluating the potential environmental impacts associated with these inputs and outputs;
- The interpretation of evidence and impact phases in relation to the study objectives.

There are three major components of the life cycle analysis. They are: evidence analyzing, impact assessment and possible improvements evaluation.

4. LIFE-CYCLE COST ANALYSIS

According to ISO/DIS 15686:2008-5 entitled „Buildings and constructed assets -- Service-life planning -- Part 5: Life-cycle costing”, the structure of life-cycle costs includes the following categories:

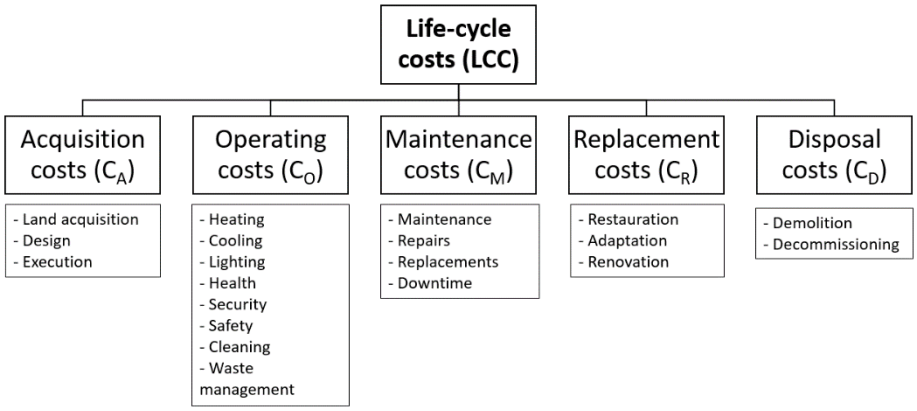

Fig. 2 The structure of life-cycle costs

Life-cycle costs of the building (LCCB) are calculated using the following formula:

$$LCCB = C_A + C_O + C_M + C_R + C_D \quad (1)$$

A) Acquisition costs refer to the land acquisition, design and execution, all according to the offers. To determine the execution cost, in the Romanian practice it is used the General Estimate, according to GD 28/2008., in the design phase. This is done on the feasibility study which is assumed already as the design theme, the urbanism certificate, the purchase design services, i.e. time and money, two elements that the investors do not generally have.

In many developed countries such as USA, UK, Germany, Italy, these acquisition costs are estimated in the initiation phase by the investor's project manager.

In Germany, as a database for forecasting investment costs of a building already in the phase of project management, they use cost parameters of BKI - Baukosteninformationszentrum deutscher Architektenkammern (Buildings Cost Information Centre of the German Chamber of Architects). BKI is a center that provides services to more than 100,000 architects throughout Germany. The center includes several thousand projects for new buildings, old buildings and outdoor facilities. They are the basis for specialized information program in the cost calculation. The database provides real information about all new buildings, old and outdoor installations and is updated annually.

The BKI documentation is classified into three groups corresponding to the following levels of detail according to DIN 276 [5]:

- Part 1: Buildings - 1. Level of group costs (e.g., Construction group costs 300 - Gross structure)
- Part 2: Coarse elements: 2. Level of group costs (ex. Groups of costs 300 - exterior walls)

- Part 3: 3. Levels of elements (ex. Groups of costs 331 – supporting elements) with information on the individual cost service areas (e.g., Excavation, drainage etc.)

DIN (Deutsches Institut für Normung - German Institute for Standardization) is designed to encourage the use of standards, which are not legal norms, they are only private, technical regulations of an advisable character.

DIN 276 classifies large construction activities as follows [6]:

Table 1. Din 276 cost groups

Name as per DIN 276	
300 Building – Construction shell	400 Building – Technical equipment
310 Foundation pit	410 Residual water, water, gas
320 Foundation	420 Heating systems
350 Ceiling	430 Air treatment systems
	440 Power installations
330 Outside walls	450 Telecommunications equipment
340 Inside walls	460 Conveying system
360 Roofs	470 Use specific systems
390 Other constructions	480 Building automations
	490 Other equipment
370 Internal installations	

The steps in forecasting the investment costs of buildings are [7]:

- Identification of the cost groups in Table 3;
- Classification of the building into three quality categories: "simple”, "medium”, "high”;
- Inclusion into the quality standard: each type of activity is given a score based on the three levels of quality;
- Summing up the score for each item into a result that also represents a standard assignment at a quality level;
- Framing the whole project in one of the three quality categories;
- Assignment of a cost value for each level of quality and detail, according to the information provided by the BKI table;
- Identification of the price ranges, according to the tables provided by BKI.

The calculated amount represents the building’s cost, without the project management phase. Further, there can be determined the percentages costs, except for the costs group 100 – Land, because in this case, the decision is based on the land price, so there are no costs to be calculated.

This method is efficient due to the fact that cost estimations can be done quickly and easily in the project management phase, based on already existent data and based on the desired quality requirements.

B) Operating costs: heating, cooling, power, lighting, health, security, safety, cleaning, waste management;

C) Maintenance costs: annual planned services and maintenance, repairs, routine replacement of components, costs of downtime and activity interruption;

D) Replacement/reuse costs: restoration, unexpected costs arising from legislation, adaptation, renovation;

E) Disposal costs: demolition and decommissioning costs.

Operating, maintenance and replacement costs are known as facility management costs. Facility management is a new economic aria belonging to services, i.e. all services which are necessary for a building. The building is an investment that brings revenues like the investment made on the stock market, and which requires a change of thinking of the construction school in our country. The building has become a bank that is profitable and must be treated as such.

Currently, there are no international structures of life-cycle costs. Each country uses its own cost estimating, forecasting and planning systems that are grouped and classified differently.

It is impossible to know precisely the exact cost of the whole life-cycle of a building, because future cost can be only estimated by different degrees of security. Future costs are usually linked to a level of uncertainty arising from a variety of factors such as:

The forecast of the use of the asset in time;

- The nature, extent and trend of operating costs;
- The impact of inflation;
- The opportunity cost of an alternative investment (the best alternative foregone when the decision to use limited resources to produce or procure a certain economic good was taken);
- Forecasting the life of the building.

The main purpose in life-cycle cost assessment of a building is to generate a reasonable approximation of costs (derived from all possible alternatives), not to try to obtain the perfect answer.

In Romania, there isn't a recognized method of classifying the building costs depending on its destination, height, structure etc. In 2010 it was introduced a cost standard for construction costs carried out from public money, costs set by the central public authorities. Thus, three laws regulating these standards were adopted:

- GD. 363/2010 regarding investments objectives financed of public funds;
- Order no. 2748/2010 regarding construction types subject to funding under GD. 1680/2008 for the establishment of a state aid scheme for ensuring sustainable economic development;
- GD. 1394/2010 regarding the investments objectives financed from public funds from transport infrastructure field.

The initial purpose of these standards was to reduce the State expenditure for available funds and to reduce the disparities between different prices the authorities used to sign for in the construction contracts. These standards are vulnerable because they are not generally mandatory and do not contain differentiated, but total costs. In the meantime, there is a limitation of cost standards to projects similar to the standard ones, because they are described in detail and develop a very small field of similar projects.

At national level there has not been achieved so far a unified approach of the life-cycle cost calculation issue, there are only certain individual practices, experienced only partially in intermediate phases of the construction life-cycle.

The use of life-cycle cost analysis of constructions on a national level will be mandatory when the new legislation on public procurement is implemented, thus enhancing competitiveness, the services quality and development of the national economy. It is therefore very important that the initial structure of the life-cycle cost includes all cost categories with major impact on the whole life-cycle cost.

The analysis of the life-cycle cost of a building provides a basis of a better investment decision and purchase, as it gives an insight into the entire life of the building by summing up the initial costs (acquisition costs) and the future costs (facility management costs and disposal costs).

In current practices, the investment decision is based only on the comparison of two or more alternatives evaluated upon the current cost which is actually the initial acquisition cost.

Taking for example the current year 2016, the following comparison of the two alternatives of decisions is considered: investment I vs. investment II:

$$LCCB_{I\,(2016)} = C_{A\,(I)}\ vs. LCCB_{II\,(2016)} = C_{A\,(II)} \tag{2}$$

A better analysis is the calculation of the life-cycle cost of the building. Assuming that the life of the building is 50 years, it is necessary to know the total cost for the year 2066.

The literature shows a wide variety of economic methods for analyzing the life-cycle cost and shows that the most adequate approach is the net present value (NPV) method. Because the time value of money changes (basically depending on the inflation rate), in the life-cycle cost analysis, all relevant current and future costs are summarized in present values. [8]

$$NPV = \sum_{t=1}^{T} \frac{C_t}{(1+r)^t} \tag{3}$$

NPV = net present value
C_t = total cost
r = discount rate
T = number of analyzed periods

It is obvious that by a comparison of the costs at different times of the project, the variable time value of money must be considered. They must be discounted back to their present value. Costs must first be converted into their time-equivalent value at the base date before the life-cycle cost is calculated.

Based on the example in which the life-cycle of a building is 50 years, the value of the life-cycle cost of the two alternatives I and II will include all costs:

$$LCCB_{I\,(2066)} = C_{A\,(I)} + C_{O\,(I)} + C_{M\,(I)} + C_{R\,(I)} + C_{D\,(I)} \tag{4}$$

$$LCCB_{II\,(2066)} = C_{A\,(II)} + C_{O\,(II)} + C_{M\,(II)} + C_{R\,(II)} + C_{D\,(II)} \tag{5}$$

It is important to know these variables from the early stage, in order to determine the optimal items for the investment, such as materials and facilities, items which determine the performance and therefore the life-cycle of the building.

5. CONCLUSIONS

To improve the quality of the services on the market, to develop competitiveness, to support the economic development, the public procurement legislation and the regulations in the construction industry experience changes. They promote business decisions based on as correct parameters as possible, thus promoting the method of calculating the life-cycle cost.

The life-cycle cost analysis is advantageous because it takes into account both initial and later costs, determined ultimately by the design method. Hence, the advantage of knowing them even from the early stages of a construction project. A system like the German one should be implemented in our country.

The results of the life-cycle cost analysis however require elaborate explanations including information about the variation in costs according to surface, functional units, time, cost categories etc. It must be taken into consideration the fact that the life-cycle cost calculation methods are not accurate because there are based on assessments and assumptions of time and costs. Estimations related to inflation, operating costs etc. can't be approximated correctly, which gives the life-cycle costing a degree of subjectivity that should be taken into account when analyzing and making strategic decisions.

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Pricing Policy of the Construction Firm

Aneta Marichova

***Abstract* – The pricing policy is a major activity in each company. The price is this element that only brings revenue, unlike other corporate activities – product, advertising, innovations that are related to costs. The classical economic theory proposes analytical tool for determining the price as a function of demand, supply and the competitive structure of the market. Nowadays the market dynamics and general economic conditions, making it difficult to determine the demand for the product in a market, the marginal revenue and marginal cost and ultimately on that basis determining the final price. For these objective reasons, it is known that construction firms applied in their pricing policy principle of "pricing on costs." At the same time, experience shows that they are not mechanically apply this principle because they have a "feeling" about the price at which you can sell and make additional assessments of their services. This is defined as the leading concept of "pricing on the analysis of the product." The aim of the study is to prove that in determining the market price of construction market apply intermodal pricing strategies, taking into account the specificity of construction products and the various market segments.**

***Keywords* – Construction market, Construction firm, Pricing policy, Cost-based, Competition-based and Demand-based price strategy**

1. INTRODUCTION

The pricing policy is a major activity in each firm. The price is this element that only brings revenue, unlike other corporate activities - product advertising innovations that are related to costs. The classical economic theory proposes analytical tool for determining the price as a function of demand, supply and the competitive structure of the market. Nowadays, however, market dynamics, changes in demand and general economic conditions, make construction firms to create different products for different markets. They transfer resources from the market of building construction to the market of civil construction, from one region to another country or outside its territory. Every company strives to maximize its profit from the

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overall activity which realized. In practice this means difficulties in determining the demand for the product in a market, the marginal revenue and marginal cost and ultimately on that basis determining the final price. For these objective reasons, it is known that construction firms applied in their pricing policy principle of "pricing on costs." The price attributed on the cost the equivalent of full costs and most often in their recovery and receipt of a satisfactory profit. On the other hand it is clear that only a large company monopoly in a given market can impose its price determined on the costs included profit. The analysis of the market structure and the level of competition in the different market segments show that the construction market employs a large number of relatively small companies, suggesting strong competition. This means that such a market pricing on the costs is irrational approach. In such dynamic, competitive conditions in order to survive the construction firm should act flexibly by not adhere to the principle of cost pricing, but on the principle of "pricing on the analysis of the product." In this case the price of the final product is determined on certain product characteristics that are important for the buyer, taking into account the quality of performance, technology, system engineering [1]. The pricing policy of construction firms is therefore approaching closer to pricing in services where the price is determined on the costs and market-oriented. In other words, the actual pricing practices in the construction occupy an intermediate position and differ from the principle of cost pricing and the principle of market prices. At first glance, the two theories (price formation on the market and on the costs) are mutually exclusive. From a practical viewpoint however in both approaches there is no substantial difference [2].

The implementation of the cost-based method of price formation only at first glance exclude ignore the demand and market. The practice of construction firm shows that their prices are strongly influenced and account changes in demand, therefore both theories not exclusive, but complementary. It is particularly important to clarify how this combination and complementarity of the two approaches and how to form the final price. The aim of the study is to prove that in determining the market price of the construction product is applied combined pricing strategies, which include the following three components, taking into account the specifics of the construction output and market (Fig. 1):

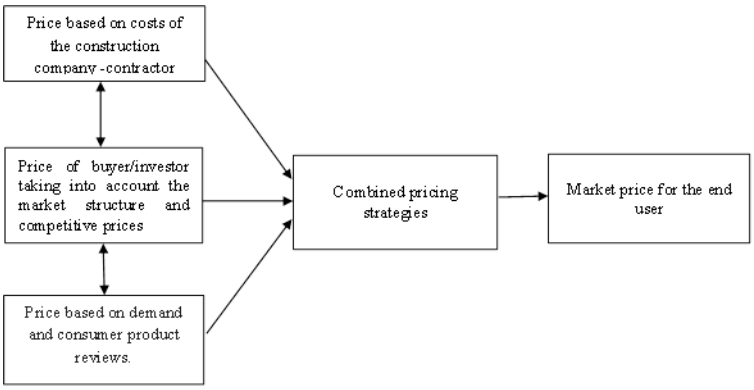

Fig.1 Combined pricing strategies for determining the price in the construction market

1) Formation of the price of construction product based costs ("cost plus" or "price mark-up") of the construction company contractor and / or subcontractor of the construction object.

2) Formation of the final price of construction product based on competitive prices on the market of civil engineering.

3) Formation of the final price of construction product based on demand and consumer evaluations of product to market of building construction.

2. THE THEORETICAL BASIS OF STUDY

2.1. Specifics of the pricing policy of the construction market

The specifics of the pricing policy of the construction market is a result primarily of the characteristics of the construction product and secondly the characteristics of the construction process and relationship of the various actors involved in it.

• Specificity of the product that is created in construction. The majority of today created products are available between clean products and clean services. This is particularly valid for construction. The creation of the final building product begins with placing requirements, preferences of the investor (client) with respect to the construction object under certain frames - developed area, quality and price. All these claims are first place in the creation of the project (service) by architects and designers. The final decision of the investor to start or refuse this project is the result of its assessment of expected return, anticipated changes in economic conditions, expected changes in the market, etc. If he decided to proceed with its implementation, it now turns established product at the service level in the final product with specific characteristics.

• The specifics of the construction activity is expressed in individual character of each project (service) / object (good), its unique location, close relationship with the land (most scarce resource), high costs of material, human and technical resources, that also part of the final price.

• Participation of the various subjects in the whole process (investor, company-contractor, company / companies-subcontractors, final buyer/user) and a complicated management structure of construction firm [3]. Executive company carries out its activities with subcontractors such as seek, collect, analyze various offers from them and choose one with which to work. This is related to high transaction costs, it can reduce if it decides to implement vertical integration with subcontractors and include them in its structure. The company subcontractor (independent or in structure of the performer) implemented the project against payment managed by one company, the main contractor. The goal of both companies is profit and long-term interests of survival and growth. The winning of all objects from the subcontractor is not based on the constant appearance of the tender and on the basis of long-term relationships that create security and stability for both countries, which limits the role and influence of the market, and excludes price competition.

2.2. Impact of market demand and supply and market structure on the pricing policy of construction firm

Key determinants of demand for each product are: own price of the product, prices of other products, level of income, elasticity demand of price, cross elasticity of demand, tastes and preferences of the consumer, population/consumers, government policy, the opportunities for obtaining credit, the level of demand in the previous period, the level of income in the previous period, expectations of changes in income and prices. Supply of each product is a function of its price, production costs of the firm, which depend on the price at which to buy the necessary resources, production capacity, level of technology used in the company, prices of other products must that the company can create the same resources, indirect taxes and subsidies, number of manufacturers of the product and the level of competition, expectations for change in the market and general economic conditions in the country. The demand determines revenues (total revenue-TR, marginal revenue-MR), which every firm can get the realization of its products. The supply of every firm depends first of its costs (total cost -TC, marginal cost -MC), which include costs for producing, distribution, product realization and others. Therefore, demand determines the "ceiling" of the price that the company can determine for their product and cost of the firm set "floor", i.e. the floor price.

According to the classical theory the companies apply marginal analysis - comparing marginal costs - MC and marginal revenue - MR, to determine the price level and the optimal production volume, which should offer the market to maximize its profit. The optimum amount,which company will provide the market is determined by the equality of marginal revenue and marginal cost. The price at which the firm will sell depends on the market structure of a given market that works. If it is a perfect competitor price at which to sell is equal the designated equality of marginal revenue and marginal cost. If it is imperfect competitor, the price is higher than so determined equality of marginal revenue and marginal cost.

Construction is a sector known for its "high birth rate and high mortality", with low barriers to entry and relatively high exit. In this market has different market segments with different market conditions as a function of differences of buyers, sellers, regions, type of construction activity, type of project sites, complexity, location, additional services, etc. In an open market, the intensity of competition is determined by the total number of permits for construction / projects and tenders for the implementation of each project. The higher general economic activity and growing demand stimulate the development and expansion of the construction companies increase projects but reduce competing offers for their performance and vice versa. In reducing economic activity and shrinking demand for construction production decreased projects that customers want to be realized, but steadily increasing number of submitted bids for executed them. Therefore, there is an inverse relationship between the intensity of competition and market demand expressed through projects and objects for realization. At low construction activity, job search, competition and the intensity of the rivalry between the companies is particularly high and in those conditions the companies become more and more new markets. The

change in demand and respectively the price of a market (increase in demand and price increase and vice versa) forces companies to shift resources from one to another market segment (product, territorial, etc.). These actions in turn lead to increase the supply of a given market and to reduce the price. The result of all this can be described as "constant ebb and flow of the market" where customers seek to maximize the high competitive intensity and construction firms contractors to reduce and minimize. Overall, however, the individual fluctuations in demand are higher than the general fluctuations in demand in the construction market, and a major factor in increasing the supply and influence prices is the availability of necessary resources (financial resources, mainly skilled workers). Due to the specifics of construction activity in the construction market has less flexibility in supply and therefore the firms have more limited opportunities to respond to dynamic changes in demand.

3. FORMING THE MARKET PRICE OF THE CONSTRUCTION PRODUCT BY APPLYING THE COMBINED PRICING STRATEGIES, TAKING INTO ACCOUNT THE SPECIFICS OF THE CONSTRUCTION MARKET

The study was developed based on data collected from meetings with managers and employees in several randomly selected construction firms, as well as publications and analyzes in specialized journals.

3.1. Formation of the price of construction product based costs ("cost plus" or "price mark-up") of the construction company contractor and / or subcontractor on a construction object

Of the construction market each participant in the chain of activities has a different rolecreates a different additional valueand there are different costs of their activities. Therefore, the formation of the cost of construction product passes through different stages and includes specific costs of each participant.

Firstly determine the costs of construction firm-contractor who form factor price. The factor price includes the cost of purchase of inputs (materials, labor, equipment, transportation, electricity, machinery and equipment). Every expense is related to the specific technology and every client and contractor should in each case to choose and define the technology and rate of related costs. Influence on the determination of the price factor have an organization - the contractor, technical and staffing, relationships with its suppliers and others.

Thus determined factor price is a price that the client (the investor) pays a construction company. It is the basis for determining the producer price, which includes additional requirements of the investor for level of performance used technology and profit of the contractor. Therefore, factor price includes costs, at constant technology and a fixed amount of construction work specified in a contract in accordance with existing standards and norms, and producer price measured movements and changes in prices as a major account changes in productivity and technologies used. The difference between the factor price and producer price is higher when price is based on the expected tender / competitive prices that can vary

from time to time and place and in the condition of competition and market conditions. Moreover, construction companies into account the impact of these additional factors when negotiating with investors and determining the producer price [4]:

1) The need for a job.
2) Expectations about the behavior of competitors.
3) Previous experience in a project.
4) Type and size of project and site.
5) Characteristics of the client.
6) Conditions of Contract.
7) Past realized profit on similar projects.
8) Expected risk in the activity.
9) The general economic conditions.
10) Qualification of staff

The third stage is the formation of the end /selling price is equal to the cost, which pays the end user. It includes producer price to which is added the cost of land, architectural services, administrative costs, infrastructure, indirect taxes and profit for the distributor.

The stages in the formation the price on the construction market (factor price, producer price and selling price) indicate that they are the result of market relationships between investors and performers, contractors and subcontractors and suppliers and so on. The established practice is the price of construction product are made based on specific analyzes for each building service, which is negotiated between the parties involved of the investing process. The contractor must comply with very circumstances that have an impact on the price, such as the object location provided communications, fixed term, fixed technologies or technologies and materials preferred by the investor and so on. Another part of the factors that affect price are external and beyond the contractual relationship the investor and contractor. These are for example: tax contributions, duties, rights to use communications, general economic influences such as inflation, strikes and more. The complex system of gradual formation of the price of construction product enable any company-contractor to seek ways to decrease or cost optimization of each element of the final price or moderate increase, but as a result of differentiation of products, higher quality, performance, use of new raw materials, material technology, effective vertical linkages, specific market conditions (relations buyer and seller) and competitive structure.

3.2.Formation of the final price of construction productbased on competitive prices on the market of civil engineering

In the market of civil engineering has a clear monopsony buyer of a specialized production and this is usually the state and seller are several large companies (oligopoly) specializing in the construction of large infrastructure projects. Besides state, monopsony buyer of particular specific production may be large companies with specialized production. Sales in this market is a great company with

differentiated assets and specializes in the construction of just such large objects (monopoly). The practice proves extremely close and lasting relationships that are built between the buyer and the seller of such a specialized market that guarantees security, stability in the activity of both parties. The enduring links between the two market players can reasonably grow into even closer relations along the lines of vertical links or vertical integration by combining, merging the companies working on the chain, which reduces the overall monopoly power of the participants and increases economic efficiency market. However, the fact that the buyer and seller are imperfect competitors could affect the final price upwards if the seller is stronger than the purchaser or downwards if the buyer is a strong from the seller, but not excluded the possibility of collusion between firms and sellers to maintain high prices.

Companies that operate in the market of civil engineering produce such similar products close substitutes and have similar average costs. In this market there are usually one or two dominant firms, and other firms silently follow the leader in its pricing, product and promotion policy. The dominant company is the company with the lowest cost, which means relatively lower prices and realized gains. This limits the access of new firms and for smaller companies operating maintenance of price leadership is often a matter of survival. These are the conditions that allow companies to conduct conscious policy of agreements and arrangements between the companies on the basis of secret collusion or policy "follow the leader" or establishing joint ventures or strategic alliances to win an auction object. Participation of companies in these agreements provides higher bid price, fuller capacity utilization and higher profits. So construction firms can compensate their costs of participation of the tender and benefits of future contracts. Because of the low barriers to entry in this market and its dynamics, these agreements are usually short.

Basic strategy in pricing is the principle of tender pricing. Transactions between buyer and seller to conclude after the announcement and winning the auction, usually based on the lowest price or the most economically advantageous tender in which the emphasis is on quality, timeliness and guarantees. The auction normally won by the company offered the lowest bid. The price is based on an expected price that will involve other competing companies, and its levels generally depend on the expected intensity of competition in the auction. The number of participants in each auction is a function of so-called "hunger for work." This means: 1) hunger for work as a result of the change in demand through good and bad years for construction, and 2) hunger for the job because of unused resources, production capacity of construction companies, even in good years for the industry. The number of participants in an auction is only an indirect indicator of the intensity of competition because high intensity may even only two strong players competitors. The large number of bidders leads to a reduction of the price at which some firms may with draw or as a last alternative investor offers a lower price acceptable to him.

At the tender pricing participating firm has the typical strategic behavior - actions and reactions to expectations of actions and reactions of other competitors. The decisions of each company are the result of the conscious recognition and interdependence of intellect, knowledge of the competitive firm. Each company chooses to play with the price based on its assumptions about the price, which will play the other participants and evaluationthe probability of winning of the tender at

different volumes of profit contained in the various offers. The company plays with the offer and hence price and ensures the highest percentage chance of winning the tender and that her choice is a reaction to the expected actions by other firms. Eventually the company will win the tender, if all its expectations for the behavior of others bidders be confirmed. Practice shows that public contracting (municipal/state) works with one or maximum two firms that annually earn tender or renew their contract. Several leading companies win orders with the lowest price that can offer thanks to optimize our costs resulting from diversification and inclusion of all activities along the chain of value creation in its own structure, efficient organization, management, spatial location near constituent objects owned raw materials and realized economies of scale and scope. Despite the high specialization of these firms performed and stable relations between buyer and seller cannot exclude doubts opacity at auctions in place and preliminary agreements between the companies. In the public procurement system are not rare cases where the company won the tender with the lowest price, use their monopoly position to contracting authority and the begin updating the contract in order to increase the price as it highlighted a number of unexpected subsequently occurred problems requiring additional activities and expensive object or extend the period of performance.

3.3. Formation of the final price of construction productbased on demand and consumer evaluations of product to market of building construction.

On the market for building construction demand is formed by two or three large investor (oligopsony) or one large investor (monopsony), or multiple buyers (households). The offering is formed by a large number of companies-contractors / subcontractors of a standard object or objects with specific characteristics - residential buildings, business buildings, commercial, administrative and industrial buildings. Therefore, in this market the price is formed under conditions of monopolistic competition with differentiated product and by definition it is significantly higher than the equality of marginal cost to the buyer and marginal revenue to the seller. The main focus is on pricing on demand and satisfaction to consumers and a major cost factor is the effectiveness of the product. Prices are differentiated by product characteristics and its modifications, characteristics of the purchaser, quantity, time, place of purchase, etc., without much relation to costs. This suggests that firms are relatively free in choosing the price in a short period. They seek to gain monopolistic advantages over other competitors with its differentiated product with higher quality or additional services. Based on non-price competition the construction companies can maintain a higher price to other companies offering a standard product and realize supernormal profits in a short period. In the a long period possibility of entry of new companies means and opportunity to reduce individual demand, and hence price reduction or maintain demand and prices in constantly demonstrating the effect of the product and its improvement by offering customers and additional benefits and rewards. The company's success requires constant process of differentiation of the product and adapt to fluctuations in demand.

Basic condition for differentiation on the market of building construction are customer preferences for the location of the construction object and infrastructure,

which determines the uniqueness of the created final product. This is a key feature for product differentiation, even when identical or similar qualitative characteristics with other comparable product. In big cities, the development of transport infrastructure and the security of individual neighborhoods is crucial for the assessment of consumers and choice of residence. These two factors are in fact indirect evidence for differentiation of the product offered.

Differentiation of the product except for its characteristics can be implemented and placement firms and create a final product, i.e. a horizontal differentiation. Construction companies carried out their main activity in the region in which they are localized. Their decision to locate a function of demand for their end products, while the search itself is a function of economic activity in the region. To differentiate maximum product, companies are located in a given market closest to consumers and as far away from competing companies, which guarantees them an opportunity to offer and sell a product unique in its location at a high price. The large number of companies that offer in bigger cities, increasing competition between them and reduces the firm differentiation of product placement. Any company can build anywhere succeed in obtaining land and building permit where consumer choice is largely randomly, guided only by the preferences available to the home and the price you can pay.

Horizontal product differentiation, except placement can be done placement and price is a function of product characteristics - size, interior, exterior, additional services offered by the company. These essential characteristics of the product may lead the buyer to overlook the good location on account of their better realization. This is a factor that allows construction firmsto offer the market the entire range of different characteristics sought by consumers. This is a sure guarantee for product differentiation in the eyes of consumers and combined with the location affects the level of the final price. If all companies offer products with comparable same characteristics in terms of consumers and are located close to each other, the differentiation of the product is minimal competition between them is fierce, which means a lower final price. If you offer a specific product and demand is high, the location of the company is unimportant because its product is differentiated in the eyes of consumers, which guarantees sales.

Guideline differentiation the company's product is the vertical differentiation associated with quality used raw materials, technologies, know-how [5]. The high quality of a product, which is a guarantee for the high price that is willing to pay user connects with exclusive functionality, durability, reliability product with exclusive offer to sell of the firm. Construction companies can differentiate your product in a basic, crucial function and most often it is functionality, convenience of the proposed housing. They can differentiate their product in several different functions, each of which has a large range of characteristics or all features but offer a more limited range of characteristics.

Some of the major companies working in the market of building construction can divide it into separate smaller segments according to the characteristics of their buyers that they know well, according to the time of purchase (a deal) or volume of purchase and also according to the location of the object, i.e. implement price discrimination in all its forms. Firms applying this price discrimination generally do

not penalize their customers, some of them earn, but the seller was able to realize all their production and to maximize its profit.

Specific activities and specific costs allow construction firms to raise the final price for end customers by raising the cost of research on their part. The cost of construction output includes expenses (purchase of land, design work, architectural services), which can hardly be familiar to the end user. The specifics of these costs means that consumers can hardly make exploration and evaluation because it would cost them too expensive. Therefore, in this market is a typical form of price dispersion - the cost of the study (search cost), are higher than the payable costs. Under these conditionsIt can not be expected,that consumers will react immediately to changes in the price - in increasing its purchases to shrink and vice versa. On the other hand, the high cost of construction output, long life time and specifics related to the fact that this is a product whose characteristics are visible after its consumption, forced users to carefully seek information to compare different alternatives, proposals and analyze posts others cautiously to make their final choice. Therefore, any buyer in this market there are high costs for making a final decision, which determines the high cost of changing the company offering the corresponding desired output and hence lower propensity to shift the selected supplier. This means that for consumers price is a key indicator symbol for the quality offered, a fact that can be used by companies. They may increase the price without any objective reasons for this,just to convince the user that his offer exclusive deal. Buyers of the desired product, will accept this higher price as a symbol of quality, extra benefits, especially if price elasticity, given their sensitivity to price is lower. Conversely, the price reduction would cause confusion, uncertainty and the user is willing to postpone or even abandon this purchase. Therefore, product differentiation, the result of objective factors such as the specifics of construction, immobility and connectivity with the land as a specific, scarce resources and the ability to build high consumer loyalty are key factors influencing the formation of the final price. Product differentiation becomes users of the firm in its customers with high loyalty, which is a guarantee to increase sales revenues into the future and market share. This is a factor that blocked the entrance and shrinking demand for other companies. In practice this means increasing market power of existing firm and the ability to impose higher prices in this market, considered a sufficiently long period of time.

This conclusion is made for the segment of housing construction where demand is formed by all households as potential users with numerous subjective tastes and desires can with even greater certainty to make the segment of non-residential construction. Large investors in this market (especially the construction of the prestigious business and commercial building) have distinct preferences and requirements for the desired object and usually work with several companies, based on repetitive contracts and specificity of transactions. The experience gained in these long-term relationships are a guarantee for the realization of yet another object of the required quality and within the prescribed period and it became one of construction firms preferred partner with assured business for years to come. The effect of this specialization of companies is differentiation of the firm and the proposed product strong positions in each subsequent deal, maintaining high price and implementing strategic corporate goals.

4. CONCLUSION

In this paper the author explores the pricing policy of the construction firm. Economic theory offers analytical tool for determining the price as a function of demand, supply and competitive structure of the market. Nowadays, however, the dynamics of the market creates significant difficulties in determining the demand for the product in a market, the marginal revenue and marginal cost and ultimately determining the final price. For these objective reasons, actual pricing practices in the construction occupy an intermediate position and differ from the principle of cost-based pricing and the principle of market prices. The aim of the study is to demonstrate how to carry out complementary and combined the two theories (price formation on the market and on costs). According to the author, in its pricing policy construction firms apply combined pricing strategies that take into account the specifics of the construction product (both goods and services), the construction process (participation of different actors, investor, contractor, subcontractor, final customer) and the construction market (different market segments with different levels of competition, various buyer-seller relationship). Firstly, the cost of the construction product is formed based on the cost of the investor, contractor and end user, then read the level of supply and demand, but also the impact of specific external factors that are strictly for each individual construction project / object. Customers (investors) and end users of construction products have different specific preferences, requirements. Therefore, companies offer standard projects, but with a distinct personality and specific characteristics. In practice, this means significant variation in the cost of implementation of individual projects resulting from the use of various technologies and therefore different prices of labor, equipment, different time, place of realization. The price agreed between the contractor and the client determines the revenues of the construction company. Therefore, each project costs and the final price (revenue) are different.

The formation of the final price is affected by competitive conditions. In the market of civil engineering with oligopolistic structure, the principle of tender pricing is a fundamental principle of price formation. Pricing decisions of any company depends on their expectations and the actions and decisions of other firms. In the market of building construction, the main focus is on pricing on demand and satisfaction of consumers. A major pricing factor are marketing tools -searches beneficial effect of the prod uct, which is a function of perceptions, evaluations, desires, attitudes, product characteristics that are important for the buyer.

The final price reflects and shows deviations as a result of changes in market conditions locally and nationally. The difference between the market price and the cost is particularly pronounced in situations where markets are unstable equilibrium. At equilibrium the difference between them is inessential. The problem is that the demand and supply on a constant influence and various other non-price factors that violate the reached equilibrium on different markets (factor, commodity, etc.). Changes in the price of the final product responds to changes in market conditions: uncertainty in the industry and the economy as a whole, product heterogeneity (regardless of typification in several projects, which is a factor for increasing price), uncertainty about future contracts, increasing demands of investors, behavior of

competitors and fluctuations in the activity of construction companies. Specifics in the process of pricing in the construction market suggest that any company could realize an effective pricing policy by adapting flexibly to changes in demand, building long-term and stable relations with investors, architects, suppliers, customers and their active involvement and participation in the process design and realization of the desired object, which is a factor of differentiation of the company's product.

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Effective Organization of Construction Firm

Aneta Marichova

***Abstract* – Today construction firms work in conditions of increasing globalization, increasing scarcity and cost of resources, geographical isolation, the result of localization of the firm and insufficient capacity to adapt to smaller, fragmented markets. This practically means that the traditional sources of competitive advantage by differentiation based on costs resulting from realized economies of scale or scope of activities have increasingly limited influence. New economic realities, however, create new opportunities and new sources of competitive advantage. They are related to the growing importance of intangible assets and their effective management and use. These new "natural" resources develop as a function of management, organizational activities built internal relations, relationships with suppliers, customers, institutions, etc., and are the result of history, culture, experience, routine and specificity of corporate development. In the study author aims to explore the connection "Resources - Firm organization - Competitive advantages" and on this basis: 1) To define company resources that provide opportunities to build competitive advantages and performance, 2) To evaluate the company resources and opportunities for developmentin construction firm and to develop a model of effective organization in which skillful combining of resources ensures implementation of competitive advantages.**

***Keywords* – Construction market, Construction firm, Evaluation of internal and external firm resources, Effective organization**

1. INTRODUCTION

Today construction firms work in conditions of increasing globalization, increasing scarcity and cost of resources, geographical isolation, the result of localization of the company and insufficient capacity to adapt to ever-smaller, fragmented markets. This practically means that the traditional sources of competitive advantage by differentiation based on costs resulting from realized economies of scale or scope of activities have increasingly limited influence. New economic realities,

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however, create new opportunities and new sources of competitive advantage. They are related to the growing importance of intangible assets, ownership, effective management and use.

In recent years, research on the construction market and the behavior of the company is directed to the analysis capabilities of the firm to develop intangible assets (knowledge, innovation), its ability to adapt to changes in the competitive and business environment and on this basis to achieve competitive advantage. Analyzing the internal resources of the firm, most authors pay particular place of a reputation resulting from history, experience. The specifics of construction determines the first place competitive advantages as a function of the established specific organizational structure as a system of strategic actions, decisions in which an important place occupied vertical links, development of human resources, specialization in specific projects and managerial skills for managing these projects. The success of any construction firm is related to reporting and adaptation to specific national, institutional factors in the country and the specific corporate resources that together define their value and uniqueness. The biggest problem, according to most analysts in construction is related to education, quality of the workforce and the lack of effective corporate organization. Growing requirements of investors, consumers, technology development and the creation of new materials, have a very strong impact on the company and impose the need for accelerated development and incorporation of intangible resources in the company's organization.

Intangible resources (knowledge, technological know-how, innovation) should be combined effectively with other intangible and tangible resources / assets that can be defined as complementary in the company and this process creates new opportunities for company growth and build competitive advantage. Therefore, the effectiveness of each intangible asset of any company is determined by its specific management capabilities, combining efficient allocation and use of this resource and not from ownership. To respond to constant changes in the external environment, the In the study author aims to explore the connection: "Resources -Firm organization - Competitive advantages " and on this basis: 1) To define company resources that provide opportunities to build competitive advantages and performance, 2) To evaluate the company resources and opportunities for developmentin construction firm and to develop a model of effective organization in which skillful combining of resources ensures implementation of competitive advantages.

2. THEORETICAL FRAMEWORK OF THE STUDY

The firm resource is defined as "all assets, capabilities, organizational processes, firm attributes, information, knowledge, which are controlled by it and allowing it to implement strategies that are not implemented by current and potential competitors," [1]. The capabilities of firm to realize the competitive advantages associated with the possibility of firm to create a resource that corresponds to the four characteristics - value,rarity, difficulty imitating and no substitutes, briefly described as VRIN-frame [2]. The uniqueness and value of the resource is a function of the uniqueness of physical inputs or from a unique combination in between. Therefore, the unique

resource that has given firm and that it provides a competitive advantage is not a separate resource, but a result of the capabilities of the firm to develop their skills, knowledge, technology and they in their unity and interdependence make unique resource. The difficulty of imitating established resource is the result of imperfection of factor markets, which are characterized by information asymmetry, high cost of establishing a specific combination of them or both. The absence of substitutes is the result again from the imperfection of factor markets, the cost of the combination and use of resources and the cost of creating a new combination between them. Imperfect mobility of the resource is strong specialization of a company and inability other companies to apply the same strategy.

Reason lies in the fact that each company has resources and opportunities as a combination of real assets that can be bought from the market and are directly related to its activities, and another part of intangible assets that are inherent only to her and can not be bought/sold. Such resources are reputation, image of the company, history, culture, values built, specific knowledge and experience. Each company has unique knowledge, training system developed on its experience and history, which allows the creation of a strategic resource [3]. The increase in value is the result of established specific combinationthe ability of the firm to recombination under the influence of changes in the external environment, as well as the specific links that have any business with companies and resources that are beyond the line of creating complementary products. Therefore, intangible resources/assets are the basis for sustainable competitive advantage for the firm because these are resources which are difficult to trade, it is difficult to transfer and change ownership and ultimately difficult to imitate [4].

The main reason for this specific characteristicand feature of intangible resources is the fact that their functioning in the company is always associated with other complementary its resources/assets, as other companies do not have. With these resources and combination of them has one single firm, and in such aspect they are always unique. On the other hand, implicit characteristic of these resources is the difficulty of their assessment, valuation and therefore they rarely (strictly speaking never) appear in the balance sheet and finances of the company. It is important to note that, even assuming that these conceptual resources can be estimated to be subject to sale and who have to pay high price specifics and their uniqueness for each company can not guarantee the desired success in another company-buyer. Therefore, the effectiveness of each intangible asset of any company is determined by its specific opportunities to build effective corporate organization and management rather than ownership.

To analyze and evaluate in practice, the resources of the company and its ability to build a competitive advantage using the theoretical framework known as VRIO [5], which is an acronym of the four questions that company managers must respond (assessment of value, uniqueness - rarity, the possibility of imitation - imitability and willingness of the firm to use this resource - organization). In this VRIO-frame, the resources of the firm are divided into groups that include tangible and intangible assets (financial, physical, technical, human, reputation and organization) that provide specialization, human capital development and their effective combination through appropriate marketing and management actions. Organizational resources / assets of

Table 1. Firm resources (internal and external) and evaluation indicators

Firm resource / asset	Evaluation indicators
I. Internal resources / assets	
1. Material resources / assets	
Product	Specialized, differentiated or standardized, homogeneous Quality of product
Production	Diversification / Specialized Scope (incl. Three streams: supply of raw materials, design, construction)
Technological resources	Effective location Evaluation of used machinery, equipment Development and adoption of new technologies, new materials, of corporate knowledge, experience and combining with other resources
Complementary resources	Innovations in product / process Create strategic alliances among all stakeholders in the vertical value chain
Financial resources	Cost,Price, Revenue Stability of financial flows Value of the firm Financial opportunities for development and expansion
2. Intangible resources/ assets	
History, mission	
Human resources	Experience, knowledge.Innovative features Specificity, uniqueness of human capital Relationships with universities, research units
Motivation and stimulation of human resources	Using different systems to encourage company employees and linking the final result with the payment company
Reputation of the firm	Trademark / name, strength and value Evaluation and opinion of consumers about the company Processing and analysis of data on consumer valuation for the firm Evaluation and opinion of the suppliers about the company (accuracy, reliability, indebtedness) A steady stream of information from the market, consumers, competitors, suppliers, changes in demand Feedback with the client
Organizational resources / assets	Effective system of corporate organization to use and transform inputs into desired final product Firm competencies and capacity for combining tangible and intangible assets Management skills Effective strategic management and planning, evaluation and control
II. External resources /assets	
1. Market Resources	Monitoring, analysis and assessment of demand and forecasting its changes Monitoring, analysis and evaluation of the competitive environment Analysis and assessment of profit potential in the market Analysis of the processes of mergers, acquisitions, diversification
2. Institutional Resources	General macroeconomic conditions Changes in technological, demographic, social, political and legal environment State requirements for the activity Barriers to entry into the relevant market Market regulation

the company are related to its ability to develop and expand existing resources through a more effective combination, improve organization and management which increases the final result. Using famous VRIO-frame in their study author divides the company's resources into two groups - external and internal resources, each of which includes several components and related indicators. The inclusion of external factors puts emphasis on the dynamics of the external environment and the need for each company to monitor, assess changes in it and adapt by assimilating and integrating external and internal knowledge, continuous learning and reconfiguration of internal resources (Table 1. Firm resources (internal and external) and evaluation indicators).

In this theoretical framework main place occupies the connection of external and internal resources with the organization and management of the firm, which in its unity can provide a sustainable competitive advantage. Basis for the realization of corporate goals is the creation of effective organizational and managerial process in which managers have the following functions:

1) Reducing the uncertainty of the environment in which the company operates through constant monitoring, collection, analysis of information and on that basis develop informed decisions about strategic changes to the internal organization and its behavior.

2) Objective assessment of opportunities and threats and the ability of the firm to cope with the challenges of the external environment.

3) Evaluation of business opportunities and the search for new knowledge, training in line with changes in technology, demand, competition.

4) Integration of new knowledge in the company.

5) Building an effective corporate organization through necessary coordination inside, reducing the contradiction between the various entities and reconcile the different objectives of the various groups and the company as a whole.

6) Creation of new opportunities through reconfiguration of resources, which is in line with its other capabilities and evaluation of results.

3. APPLICATION THE THEORETICAL MODEL FOR THE EVOLUATION OF RESOURCES IN THE CONSTRUCTION FIRM AND DEVELOPING A MODEL OF EFEECTIVE ORGANIZATION THAT ALLOWS THE REALIZATION OF COMPETITITVE ADVANTAGE

The empirical analysis is made on management estimates of company resources (internal and external) of the surveyed construction companies. All firms operating in the market of building construction and are selected at random. They have different market positions expressed by financial results, market share, customer evaluation, etc. Collected and processed data allow the author to make an overall assessment of company resources (internal and external) of "successful" construction firm (as a focal feature) and "typical construction firm" (also as a focal feature) which has serious problems in its activities. On the basis of this analysis we will draw conclusions on the need for strategic change and establishing an effective organization of corporate activities that improve the situation of "typical" construction firm.

3.*1. Evaluation of internal (tangible and intangible) and external resources of the "successful" construction firm*

"Successful" construction firm has a long history and its mission is quality, honesty, efficiency, full satisfaction of customers at ensuring safe, healthy working conditions and protection of the environment. The firm performs a wide range of activities in the construction market, has significant personnel with subsidiaries and sites in different parts of the country. It creates highly specialized product with high quality, which is differentiated and focused on the client's wishes. The main objective of the activity is sustainable development in all its aspects. Customers of the successful company are big investors and customers, households. The firm focuses on the wishes of its customers and builds with them partnership. Strives to find the optimum solution for clients and works so that they feel employee engagement to each their individual projects.

In its production, firm ensures a reliable and efficient integrated solutions in construction, offering flexible solutions to each individual problem. It is focused on: Quality control and customer feedback. In its production, firm ensures a reliable and efficient integrated solutions in construction, offering flexible solutions to each individual problem. It is focused on: Quality control and customer feedback. The scope of its production includes all activities: supply of raw materials, design and construction of objects in the market of building construction. In practice, implementing new efficient technologies, materials and innovations allowing transformation of knowledge, intangible assets in the final result - a complex and specific construction sites performs firm. The firm achieved rapidly and quality performance of the objects in harmony with the environment. Adherence to normative regulations is the foundation of all business strategies and plans.

"Successful" construction firm has its own production of raw materials and/or long-term stable contracts with suppliers, in practice build an effective integrated vertical supply chain. Effective vertical connections provide the desired quality rhythm, timeliness and stability in cost and final price. The firm impose its requirements for quality products and services delivered by suppliers and subcontractors. "Successful" firm has relatively high costs in its operations as a result of differentiation of product offerings, development of individual projects according to specific customer requirements and the requirements of sustainable construction. These costs are within the production capacity and the ability to optimize them. In the dynamic, competitive conditions, successful firm operates flexible by not adhering to the principle of cost pricing. The price of the final product is determined to offer the company additional product characteristics which are important for the buyer - quality of performance, technology, system engineering. The firm can continue and deepen the differentiation of their product on important major for the buyer characteristics and their relationships with him, and as a result increase the price. This stabilizes its market position and increases its profits because in this market of durable goods high price is a symbol of quality, which is especially important for customers. The growing reputation is the result of growing image in society, increasingly recognizable brand, and activities based on the principles - quality, reliability and efficiency, stable relationships with customers, suppliers, investors.

"Successful" construction firm operates in an efficient, monopolistic competition with increasing market share as a result of its strong specialization in the market of building construction. The company has built lean organizational structure that includes project managers, structures associated with the development of new products, technical managers and other specialized units. All employees, officers, managers have personal involvement in achieving strategic corporate goals. In successful firm introduced integrated management system that includes quality management, environmental management and ensure healthy and safe working conditions. Qualifications and professionalism of each employee is a guarantee for the quality of the finished sites. The company uses appropriate methods and monetary and non-monetary measures and incentives to increase staff motivation for participation in all processes related to achieving the goals of the organization. The main task is investing in staff by providing and ensuring appropriate opportunities for training and development, and objectively evaluate the work of staff.

3.2. Evaluation of internal (tangible and intangible) and external resources of the "typical" construction firm

"Typical" construction firm has a relatively short history. In high demand in the market performs small and medium sized orders, objects, while reduced demand - main finishing and /or repairs activities. Its mission is: "To have a job and survive." The offered product is standardized, with limited features based on the type of construction projects, with an average quality performance. The firm has no clear profile of specialization and far from the requirements of modern, sustainable construction. Customers usually are households with fewer opportunities, with limited information and requirements for housing. Relatively rare clients of such firms are larger investors. The firm does not build long-term relationships with customers and after transmission of object these connections and relationships are terminated. None of the surveyed companies do not maintain a system for customer feedback. In terms of the decline in construction, however, consumers are more clearly express their preferences, impose their own requirements and increase its market power against the construction company.

"Typical" construction firm offers individual actions related to either the design or the construction or both, at a standard level. In realization apply traditional technologies with low productivity and quality. Using traditional materials, with high energy consumption and polluting the environment. Expenditures are reduced, usually as the result of "savings"made in breach of regulatory requirements, technological discipline, expenditure norms or reduction of wages. The firm has no own production bases for raw materials, no long-term stable contracts with suppliers. It works on the principle of short-term contracts that provide needed supplies for each object implemented, which creates problems with quality, rhythm and timeliness.

"Typical" construction firm usually has an inefficient organizational structure with a large enough staff, of the scale of production. Qualitative characteristics of personnel usually do not meet the requirements necessary knowledge, experience and professionalism. Often there is also an organizational structure in which included people with very narrow specialization, knowledge and limited job opportunities.

Extremely complex problem for any "typical" firm is the provision of human resources and their training. In the first place it is a result of the peculiarities of the construction market: the company created a separate organization, employs temporary workers, equipment for the implementation of each object and then released them. Therefore, such a firm harderwould find motivation and resources to invest in development, qualification, professionalism of employed workforce, which in turn determines the lower quality end product creation. The company has a low reputation because of lack of stable relationships with clientsof stable relationships with customers, suppliers and hence loyalty on their part. It is far from the requirements and the ability to obtain a certificate for management quality system to ensure safe and healthy working conditions, and environmental protection is not a priority.

The problems in the activity of "typical" construction firm prove the need for deep structural changes in the organization. They are a function of the management team, which must be performed three permanent actions: monitoring of market developments and product offering competitorsassessment of development opportunities (strengths and weaknesses of the company). The collected information and its analysis is the basis for strategic decisions and actionsa reconfiguration of resources to adapt to the dynamic external environment in the following ways:

- Identification of potential for future development based on internal resources and their ability to neutralize the influence of the external environment.
- Creation of an integrated vertical supply system with all entities involved in the construction process, which is the condition for cost optimization.
- Increasing specialization by type of product, type of customer or product segment type or scope orders -services / commodity and creating a differentiated product that customers want.
- Differentiation of the product and the company's activities towards sustainable construction - compliance with the requirements for protection of the environment, reducing operating costs, providing aesthetic appearance, comfort, health and tone.
- Increase the supply additional value for customers compared to competitors by offering additional features, additional services, facilities for the user, or increase the added value created in the integrated supply chain.
- Establishment of corporate reputation as a result of efficient operations and customer relations. The reputation of the firm is supported by a constant flow of information from the market, consumers, competitors, suppliers,changes in demand and on this basis quick feedback with clients. The aim is to develop and maintain a competitive advantage on price, timeliness, quality and satisfaction to all customers and partners.
- Development of intangible assets - knowledge, training and motivation of human resources, continuous training and qualification of personnel, "investment in the future", i.e. training of necessary personnel, before they have entered into practice.
- Enhanced quality control project implementation and compliance with "green" standards and norms.
- Creation of an effective system for planning, selection, integration, development and personnel management, through which achieves integration of personal goals

of each employee with the company's strategy and which is the main factor for stable and sustainable company growth.

• Creating an effective organizational structure which skillfully combines and coordinates decentralization, control and management and allows timely adaptation to technological and market change by developing long-term strategies and flexible solutions in the short term.

• Optimization of company activities on the principles of adequacy, appropriateness and compliance with regulatory requirements (norms, standards in construction) better coordination, integration of internal and external resources, which increases their value, uniqueness and creates the potential for realization of competitive advantages.

• Effective management and organizational structure of construction firm must be built on the following principles: Specialization in specific project and create a differentiated product, building vertical integrated system supply, customer proximity and constant feedback, compliance with "green" standards and norms and control the performance, continuous development of the training system, education and qualifications (dynamic link between the external environment and the development of corporate resources).

The establishment of organizational structure on those principles will enable of the firm to achieve sustainable competitive advantage by offering greater value to customers on the principles of sustainable construction which is in the interest of consumers and society as a whole and also to increase their efficiency (realized profit) and market performance (Fig. 1).

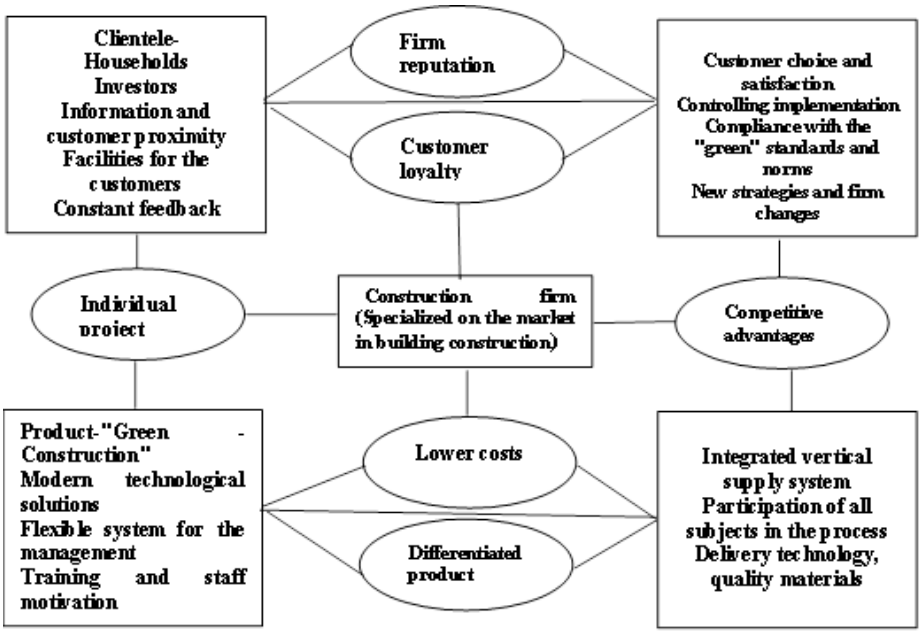

Fig.1. Efficient organization in the activities of a construction firm specializing in the market of building construction

4. CONCLUSION

Dynamic changes in the external (often and internal) environment put companies with major new and unknown challenges that make the increasingly difficult the process of anticipating and adapting to them. Under these new conditions known traditional models of corporate behavior have a more limited role and chance of success. Development prospects are associated primarily with the development of intangible assets - knowledge, technological know-how, innovations that allow dynamic firm to differentiate its activities and realize competitive advantages. Intangible assets alone may not create the desired additional value and provide competitive advantages. The unique combination of intangible assets other internal company resources as a result of managerial and organizational decisions can turn this resource with high value that provides a competitive advantage. According to the author, each company must seek opportunities for successful development, both in their internal resources (technology, training of the workforce, raw materials used, innovation, financial resources) and their combination as a value, uniqueness, nomobility an efficient corporate organization as well as external resources (market structure, vertical connections dynamics of demand, technological changes, etc.). From this point of view in the study used the famous VRIO- framework, but the author divides the company's resources into two groups - external and internal resources, each of which includes several components and related indicators. The inclusion of external resources focuses on the dynamics of the external environment and the need for each firm to monitor, assess changes in it and adapt by assimilating and integrating external and internal knowledge, continuous learning and reconfiguration of internal resources.

This VRIO- framework was applied for evaluation of internal and external enterprise resources of the surveyed construction companies. All firms operating in the market of building construction have different market positions and are selected randomly. Collected and processed data allows the author gave the following summary evaluation of company resources (internal and external) of "successful" construction firm (as the collection feature) and "typical construction firm" (also as the collection characteristic):

- "Successful" construction firm creates highly specialized product which is differentiated and focused

on the client's wishes. Strives to find optimal and flexible solution for each individual project. The main objective of the activity is sustainable construction in all its aspects. Effective vertical connections are a factor for the implementation of new efficient technologies, materials and innovations that provide the desired quality rhythm, timeliness and stability in cost and final price. Adherence to normative regulations is the foundation of all business strategies and plans. The firm has built a lean organizational structure and introduced an integrated management system that includes quality management, environmental management and ensure healthy and safe working conditions. The main task is to increase the skills and professionalism of every employee, investing in staff by providing and ensuring appropriate opportunities for training and development, as well as an objective assessment of final outcome of each.

- "Typical" construction firm offers a standardized product with limited characteristics based on type

construction projects, with an average quality performance. It works on the principle of short-term contracts that provide the necessary raw materials for the given implemented object, which creates problems with quality, rhythm and timeliness. "Typical" construction firm usually has an inefficient organizational structure with a large enough staff, of the scale of production. The firm has no clear profile of specialization and far from the requirements of modern, sustainable construction. The firm has a low reputation due to lack of stable relationships with customers, suppliers and hence loyalty on their part. Apply traditional technologies with low productivity and quality. Extremely complex problem is the provision of human resources and their training because such a firm difficult to find motivation and resources to invest in development, qualification, professionalism employed workforce, which in turn determines the lower quality of creating a finished product.

The existing problems in the operation of a typical construction firm show the need for structural changes in its activities that should result from responses to the following questions: 1) Who are its customers?, 2) What product they want ?, 3) How can build effective vertical relationships? 4) What is the objective evaluation of its internal and external resources?, 5) What additional value can offer to its clients based on their specialization and created unique resource? These corporate decisions must be supported by the efforts of the management team to create efficient management and organizational structure on the following principles: Specialization in a particular project and create a differentiated product, building integrated vertical system of the deliveries, customer proximity and constant feedback, compliance with "green" standards and norms and control performance, continuous development of the training system, education and training, relationship between the dynamic external environment and the development of corporate resources.

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Comparison of ISO 21748 and ISO 11352 Standards for Measurement Uncertainty Estimation in Water Analysis

Tony I. Venelinov

***Abstract* – A comparison between ISO standards – ISO 21748:2010 and ISO 11352:2012 for the measurement uncertainty estimation in the field of water analysis is presented. Several Certified reference materials (CRMs) with certified values for several water quality parameters were measured and measurement uncertainties were evaluated for each of them according to the above mentioned ISO standards. Results show excellent agreement for chemical oxygen demand (COD), total organic carbon (TOC), ammonium nitrogen (NH_4^+-N), nitrate nitrogen (NO_3^--N), total phosphorus (PO_4^{3-}-P) and chloride (Cl^-). Based on the experiment design and the comparison results, ISO 11352:2012 is recommended for measurement uncertainty estimation in the field of water analysis during method validation.**

***Keywords* – ISO 21748, ISO 11352, measurement uncertainty, uncertainty estimation, method validation**

1. INTRODUCTION

Chemical analysis of a set of water quality indicators is the most powerful tool for the determination of water suitability for human consumption or to be released back into nature after treatment. Numerous analytical methods exist for measuring COD, total organic carbon (TOC), ammonium nitrogen (NH_4^+-N), nitrate nitrogen (NO_3^--N), total phosphorus (PO_4^{3-}-P) and chloride (Cl^-) at laboratory scale. They are usually carried out according to the national and international standard methods. These methods are well studied and documented, but they are often too sophisticated and time consuming for operational control and require considerable practical experience and skill to get reproducible results. Furthermore, most of them require the use of toxic substances, which can be harmful to the analyst and their subsequent utilization can be dangerous to environment. Spectrophotometric methods for water

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analysis, based on ready-to-use cuvette tests, can be an alternative to the time-consuming reference methods. Practically all compounds and indicators of water environment could be measured directly or after suitable preliminary treatment using contemporary spectrophotometers and portable photometers. The cuvette tests quality is demonstrated by the fact that for the first time a COD cuvette test has been accepted as a reference method [1].

The aim of this study was to compare the measurement uncertainty estimation according to two different ISO standards – ISO 21748 : "Guidance for the use of repeatability, reproducibility and trueness estimates in measurement uncertainty estimation" [2], and ISO 11352: "Water quality - Estimation of measurement uncertainty based on validation and quality control data" [3] for the determination of parameters in water samples.

2. EXPERIMENT DESCRIPTION

Apparatus

DR 3900 - portable spectrophotometer (Hach Lange GmbH) with 320 to 1100 nm wave length range (tungsten halogen lamp) and referent ray to compensate lamp wear and power fluctuation was used. The devise has integrated system for barcodes reading of the prepared tests, with ten measurements for rotation and elimination of wrong reading caused by prepared cuvettes wasting.

LT 200 - thermo-reactor (Hach Lange GmbH) with thermo block was used for heating up the cuvettes up to 1500C and TenSette plus - Electronic pipette 0.2 to 5 mL (Hach-Lange GmbH) was used for dilusions.

Materials

CRM (RTC COD 500-500) with certified value for COD of 500.00 ± 7.65 mgO2/L -(LOT No. 016203), CRM (CertiPrep) with certified value for NO3--N of 1005 ± 3 mg/L (LOT No. 2-78NO3N-2), CRM (RTC TPO 1000 to 500 ML) with certified value for PO43--P of 1000.0 ± 15.5 mg/L (LOT No. 017605), CRM (SPEX Ammonium Standard) with certified value for NH4+-N of 1002 ± 3 mg/L (LOT No. 2-95NH4N-2), CRM (RTC QC3130-500ML) with certified value for TOC of 38,0 ± 0,9 mg/L and TOC1K-XXX with certified value for TOC of 1000 ± 5 mg/L, CRM (Fluka BCBC2167 with certified value for Cl- of 1000 ± 4 mg/L. (LOT No. 39883), were used for the validation studies.

Cuvette tests (Hach-Lange GmbH) LCK 114 and LCK 314 were used for the determination of COD in range 150-1000 mgO2/L and 15-150 mgO2/L, respectively. LCK 339 was used for the determination of NO3--N in the range of 0.123-13.5 mg/L.. LCK 303 and LCK 305 were used for the determination of NH4+-N in the range of 2.0-47 mg/L and 1.0-12 mg/L, respectively. LCK 350 was used for the determination of PO43--P in the range of 2.0-20 mg/L. LCK 380 and LCK 381 were used for the determination of TOC in the range of 2-65 mg/L and 65-730 mg/L. LCK 311 was used for the determination of Cl- in the range of 1-1000 mg/L.

Sample preparations for all the parameters are described elsewhere [4].

Experimental design

Uncertainty is best estimated during method validation. The experimental design according to ISO/IEC 21748:2010 was set up in a way so that the repeatability, reproducibility and trueness estimates are used for measurement uncertainty estimation [2].

Trueness was proven by measurement of three independent samples of a certified reference material on two different days. From these data the uncertainty of trueness and method bias were calculated:

$$u_t = \frac{s_t}{\sqrt{n_t}} = \sqrt{\frac{s_t^2}{n_t} + \frac{\sum u_{mat}^2}{n_{mat}^2}} \tag{1}$$

where st is the standard deviation, nt is the number of replicates, umat is the uncertainty of the certified value of the CRM used and nmat is the number of the CRMs used.

Repeatability and intermediate precision were determined by replicate analysis and assessment of between-day effects. This was achieved by preparation of three independent samples of a certified reference material on three extra days. Combination from these data and the data obtained for trueness were used for calculation of the uncertainties of repeatability and due to intermediate precision.

Measurement uncertainty components of repeatability and due to intermediate precision can easily be calculated using the ANOVA function in the Microsoft Excel:

$$u_r = \frac{s_r}{\sqrt{n}} = \frac{\sqrt{MS_{within}}}{\bar{y}} \tag{2}$$

where ur is the uncertainty of repeatability, sr is the standard deviation of all the repeatability measurements, and n is the number of replicates and is the mean of all measurements performed.

$$u_{ip} = \frac{s_d}{\sqrt{d}} = \frac{\sqrt{\frac{MS_{betweengroup} - MS_{withingroup}}{n_{pergroup}}}}{\bar{y}} \tag{3}$$

where uip is the uncertainty due to intermediate precision, sd is the day-to-day variation, d is the number of measurement days, and n is the number of replicates and is the mean of all measurements performed.

Equation 4 is used for calculation of the combined uncertainty (uc). Expanded uncertainty, U, or the relative expanded uncertainty, Urel, is calculated using a coverage factor of $k = 2$. This approximately corresponds to a symmetrical confidence interval of 95 %.

$$u_{c,rel} = \sqrt{u_{r,rel}^2 + u_{ip,rel}^2 + u_{t,rel}^2} \tag{4}$$

ISO11352:2012 focuses in the field of water analysis and specifies methods for the measurement uncertainty estimation of chemical and physicochemical methods in single laboratories based on validation data and analytical quality control results. Experimental design according to [3] was set up in such a way that the uncertainty components for the within-laboratory reproducibility, uRw, and the uncertainty component from method and laboratory bias, ub, form the basis for the estimation of the measurement uncertainty:

$$u_{Rw} = s_{Rw} \tag{5}$$

where s_{Rw} is the standard deviation of the quality control results.

$$u_b = \sqrt{b^2 + \left(\frac{s_b}{\sqrt{n_M}}\right)^2 + u_{Cref}^2} \tag{6}$$

where b is the difference between mean measured value and an accepted reference value; sb is the standard deviation of the measured values of the reference material; nM is the number of bias measurements on the reference material; uCref is the uncertainty of the reference value.

Equation 7 is used for calculation of the combined uncertainty (uc). Expanded uncertainty, U, or the relative expanded uncertainty, Urel, is calculated using a coverage factor of k = 2, which corresponds approximately to a confidence interval of 95 %.

$$u_{c,rel} = \sqrt{u_{Rw}^2 + u_b^2} \tag{7}$$

3. RESULTS AND SIGNIFICANCES

A measurement result of a laboratory is an estimate of the value of the measurand. The quality of this estimate depends on the inevitable uncertainty that is inherent to the measurement result. In principle, the measurement uncertainty is a property of individual measurement results. During method validation there are different quality parameters, which characterize the method and must be established: accuracy, precision, quantification and detection limits, linearity range, sensitivity, robustness. Laboratories have to design strategies for assuring internal quality control – to prove that data produced is fit for their intended purpose, especially relevant for laboratories envisaging accreditation processes according to ISO/IEC guide 17025 [5] for their competence.

ISO/IEC 21748:2010

Data for the measurement uncertainty estimation according to ISO/IEC 21748:2010 is presented in Table 1. The mean of the 15 measurements over a five-days spread of a respective CRM for all the parameters was used to calculate method bias and repeatability. The uncertainties of trueness, repeatability and due to intermediate precision are based on ANOVA-single factor calculation. They are expressed as relative uncertainties and summarized below.

Table 1 Measurement uncertainty components and estimation according to ISO/IEC 21748:2010 for several water parameters

Parameter	u_r [%]	u_{ip} [%]	u_t [%]	U [%] (к = 2)
LCK 114	0,9	0,8	0,6	**3**
LCK 314	5,0	4,2	1,9	**12**
LCK 339	2,2	2,0	0,5	**6**
LCK 303	3,0	0,8	1,4	**7**
LCK 305	2,1	1,8	5,5	**12**
LCK 350	7,0	1,8	1,9	**15**
LCK 380	3,0	1,1	2,3	**8**
LCK 381	3,2	2,4	6,9	**16**
LCK 311	0,6	1,0	0,7	**3**

Table 2 Measurement uncertainty components and estimation according to ISO/IEC ISO11352:2012 for several water parameters.

Parameter	u_{RW} [%]	u_b [%]	U [%] (к = 2) ISO 11352:2012	U [%](к = 2) ISO 21748:2010
LCK 114	0,9	1,6	**4**	**3**
LCK 314	4,5	1,9	**10**	**12**
LCK 339	2,3	0,5	**5**	**6**
LCK 303	3,1	2,1	**8**	**7**
LCK 305	2,1	9,5	**12**	**12**
LCK 350	5,0	3,0	**12**	**15**
LCK 380	3,1	4,1	**10**	**8**
LCK 381	6,4	10,6	**17**	**16**
LCK 311	1,5	1,2	**4**	**3**

ISO/IEC 11352:2012

If stable quality control (QC) samples that cover the whole analytical process, including all sample preparation steps, are analysed regularly and if these QC samples are similar in matrix and analyte concentration levels to test samples, then the uncertainty component for the within-laboratory reproducibility, uRw , at this concentration and for this matrix, can be estimated from the standard deviation of these QC results (5). A minimum number of eight measurements is required for the estimation of each uncertainty component.

To evaluate the uncertainty associated with method and laboratory bias, the difference from the certified value and the uncertainty of the certified value are estimated.

If only one reference material is available, the results of analyses of this reference material are treated as the best available estimate for the measurement uncertainty component associated with method and laboratory bias. A minimum number of six measurements is required for the estimation of each uncertainty component.

The estimation of measurement uncertainty is based on analytical quality control results and validation data which represent the within-laboratory reproducibility, and the method and laboratory bias. Measurements were combined, so only eight measurements in a one-day run were used for the calculations. The measurement uncertainty components, expressed as relative uncertainties are summarized in Table 2.

4. CONCLUSIONS

Two different approaches for the estimation of measurement uncertainty in the field of water analysis were compared. The results for the measurement uncertainty estimations according to ISO/IEC 21748:2010 and ISO11352:2012 are in a close agreement and statistically indifferent (Table 2). Based on the results obtained and the experimental design, the use of ISO11352:2012 for the measurement uncertainty estimation, is recommended. This International standard employs fewer determinations (minimum number of eight measurements is required, compared to fifteen), fewer working days (one compared to five), easier measurement uncertainty components' calculations and even gives opportunity for routine laboratories to use standard solutions over the more expensive CRMs for the measurement uncertainty estimation during the method validation.

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Data Driven Estimation of Monthly Streamflow Based on Observation Ranges

F. Dikbas

***Abstract* – This paper presents the application of Frequency Based Imputation (FBI) method for the estimation of missing monthly streamflow data. Value estimations are made based on the determined ranges with the highest frequencies. The FBI method is applied on the monthly discharge data of Aswan station on Nile River and tested by annually removing and estimating the existing 1382 monthly observations made between 1869 and 1984. Ten missing observations are also estimated in the process. The obtained results and the sensitivity analysis show that the developed method performs very well in estimating all portions of the dataset even though the construction of Aswan High Dam between 1960 and 1970 significantly influenced the flow regime of Nile.**

***Keywords* – Frequency based imputation, monthly streamflow, Nile River, sensitivity analysis.**

1. INTRODUCTION

Data driven modeling (DDM) has become a rapidly developing area with the increase of available observed data and the developments in computational power. DDM is based on methods of computational intelligence and machine-learning and uses the information contained in the observed data mostly without considering the physical processes. Artificial neural networks [1]-[3], fuzzy rule-based systems [4]-[5], l-moments [6], ARIMA models [7]-[8], k-nearest neighbor method [9], genetic algorithms and support vector machines [10] are among the many examples of data driven methods used for the estimation of streamflow.

This paper presents the estimation of streamflow values of Nile river at Aswan station (Fig. 1) by using a novel two-dimensional frequency based data driven modeling approach named Frequency Based Imputation (FBI) [11] which provides probable value ranges for missing data by considering the frequencies of the value pairs close to the missing data in the two-dimensional data matrix. A sensitivity analysis was also performed and the results of the method were compared with Expectation Maximization (EM) and regression methods.

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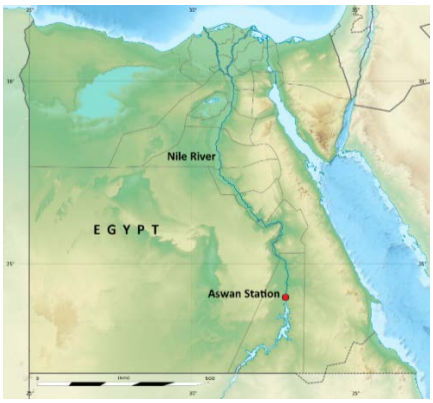


Fig. 1 The location of Aswan Station [12]

2. MATERIAL AND METHODS

The FBI method is a data-driven model based on extracting information from available data. The obtained information is used for estimating missing values by determining the value range frequencies of neighboring data pairs in a matrix. The logic behind the method is that if a data set is not completely random, there are sectors resembling and representing other sections of the data set. The details of the implementation of the FBI method will be presented below in the Results and Discussion section with a step-by-step description of estimation of a missing value.

2.1 Data Preparation

The data used in this study are the monthly discharge observations of Aswan Station on Nile River. The whole data series consists of 1382 monthly discharges through the 116 years between 1869 and 1984. The minimum (34.0 m^3/s) and the maximum (1066.6 m^3/s) monthly mean discharges were observed in April 1900 and September 1878 respectively (highest flows in Nile are observed in August-October range). The mean of the whole series is 239.0 m^3/s. The FBI method is tested by deliberately removing and estimating flows for each year. The flows of the Nile at Aswan shows very high degrees of fluctuations. The lowest recorded annual discharge was 42 billion m^3 in 1913 and the highest was 150 billion m^3 in 1878 [13]. A significant regime change is observed in the data series after the construction of the Aswan Dam between 1960 and 1970. The heatmap of the monthly discharges clearly shows this situation in Fig.2. More information on the long term hydrologic trends in the Nile basin might be found in the studies made by Tesemma [14] and Awadallah [15].

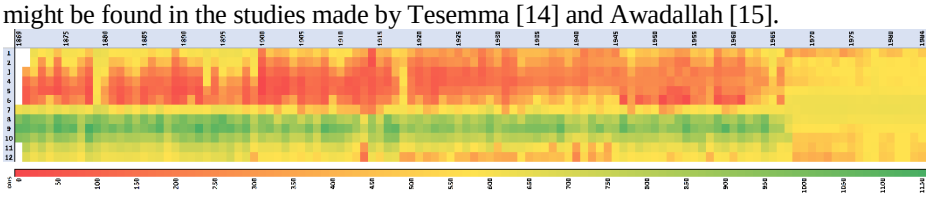


Fig. 2 Heatmap of the monthly discharges of Aswan Station on Nile River

3. RESULTS AND DISCUSSION

A software was developed by the author for the implementation of the FBI method and will be freely provided upon request. The steps of the method will be explained for a randomly selected month (October 1933). First, the observed value of October 1933 (487.1 m^3/s) is removed from the input data matrix. The pairwise numerical relationships within the 7x7 neighborhood of the missing node are searched throughout the whole map and the range frequencies are determined as explained below.

After removing the value of October 1933, the remaining 1381 observations are sorted ascending and divided into clusters. The number of clusters to be used starts from 2. The highest preferred number of clusters depends on the range, the total number and the distribution of observed values and should be selected so that the required level of significance in the estimations is obtained. For this example, the highest number of clusters is chosen to be 20.

When the data is divided into two clusters, the first one contains the lower values and the other one contains the higher values. Then each data is assigned a cluster number; 1 for the data in the first cluster (the lower values) and 2 for the data in the second cluster (the higher values). Then all the cluster pairs in the neighborhood of the missing data are searched through the whole data matrix and the frequencies of the two clusters are determined by counting the cluster numbers at the relative location of the missing data. The two cluster frequencies constitute the first column in the cluster frequency table in Fig. 3. After completing the process for two clusters, the same procedure is repeated for 3 to n clusters and the whole frequency table shown in Fig. 3 is obtained.

For example, the first column on the left in Fig. 3 shows the frequency values obtained for the 1st (the lower values) and the 2nd (the higher values) clusters when the data series is divided into two clusters. The frequency value of the 2nd cluster (62165) is much higher than the frequency value of the 1st cluster (21209). This shows that it is much more probable that the data of October 1933 is in the 2nd cluster indicating that its value is most probably higher than the median.

The 20th column shows the range cluster frequencies obtained by dividing the data into 20 clusters. The highest frequency among the 20 clusters was 486 which was obtained for cluster 17. The value range of cluster 17 is between 406 m^3/s and 529 m^3/s when the data series is divided into 20 clusters. The actual observed value in October 1933 is 487 m^3/s. This value falls within the boundaries of cluster 17. The second highest frequency value is 275 and it was obtained for cluster 16. The total frequency of clusters 16 and 17 is 761. This value is higher than the total frequencies of the remaining 18 clusters (605). This finding suggests that the value in October 1933 most probably falls within the boundaries of clusters 16 and 17 (272 m^3/s to 529 m^3/s). It must be noted that the determined frequency values decrease with the increase in cluster numbers and the cluster frequency table in Fig. 3 is generated only for the missing value in October 1933.

The developed software applies conditional formatting separately to each column of the frequency table so that the lower values have tones of red and the higher values have tones of green. The direction of the dark green region intersects the data range columns on the right around the value observed in October 1933 which was deliberately removed for estimation.

Cluster numbers	The number of clusters to which the data series is divided																				
	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	Min	Max
1	21209	2874	486	266	115	59	38	14	4	6	6	2	2	1	0	3	0	1	0	0	50.98
2	62165	9507	1733	811	337	144	117	42	28	19	14	7	10	3	1	6	4	0	2	51.24	60.91
3		30705	6014	1113	602	292	146	67	51	24	15	14	15	9	7	5	3	7	1	61	69.64
4			19513	5065	1131	324	181	245	81	65	36	28	20	20	4	3	7	9	5	69.98	76.72
5				10968	5862	906	360	160	106	118	75	17	21	6	10	6	9	4	0	76.72	84.15
6					5219	5458	849	316	136	71	56	70	83	58	9	9	1	7	8	84.59	96.77
7						2388	4783	944	191	122	82	25	24	52	37	39	13	27	1	96.77	108.3
8							1491	3892	1148	181	136	108	35	13	8	27	51	38	32	108.6	118
9								1065	2572	1069	226	109	74	59	31	11	4	11	26	118	128.3
10									828	1923	1353	310	82	46	69	35	15	10	4	128.3	139.3
11										620	1201	1431	375	83	35	40	34	14	7	139.3	150
12											322	932	1386	478	92	24	28	35	28	150	165.6
13												186	724	1129	495	133	33	43	23	166.1	189.6
14													164	484	857	494	157	69	19	190	218.3
15														163	395	691	566	211	63	218.3	269.7
16															131	307	433	423	275	271.6	403.1
17																92	208	380	486	406.4	529
18																	63	174	222	532.2	626.7
19																		55	111	629	761.3
20																			53	764.5	1067

Fig. 3 The cluster frequencies obtained by dividing the data series into 2 to 20 clusters

3.1 Estimation of the Missing Data

The developed approach for the estimation of the missing data is based on the density of the cluster frequencies. Three consecutive clusters with the highest total frequency are determined for each clustering step. The estimations for each clustering step are calculated by averaging the observations generating the frequency of the clusters. For example, when the data range is divided into 20 clusters, the average value of the 983 (the total frequency for the clusters 16, 17 and 18) observations determined in the cluster pair search constitutes the estimation (467.3 m^3/s) for the missing value. The estimations obtained with this approach are stored in the "Correlation Tables" and the "Cumulative Statistics" output files.

Obtaining a successful estimation for only a single value does not show that the method will make successful estimations for the remaining data. For this reason, the explained steps used for estimating the missing value in October 1933 are applied for making estimations for all the remaining observations in the data set. Instead of removing the observations from the data one by one, annual groups of 12 observations were removed and estimated.

3.2 Statistical evaluation of the model performance

Fig.4 shows the time series graphs comparing the observations and the best estimations obtained from the output files of the developed FBI software. The estimations for the missing values in 1869 and 1870 are also presented.

To statistically evaluate the model performance, some widely used parameters in hydrologic data driven modeling studies are calculated. The estimations obtained for each year are compared with the observed values and the statistical parameters are calculated for each year separately (Fig. 5). Then all estimations are compared with all observations to obtain the parameters in the "Whole" row of Fig. 5 which summarizes the results of the statistical evaluation of the estimation performance of the proposed model. Coefficient of correlation (r), Nash-Sutcliffe efficiency

coefficient (E), normalized root mean square error (NRMSE) and mean absolute percentage error (MAPE) values are calculated. The cells with tones of green have the best performances and the cells with tones of red have lower performances.

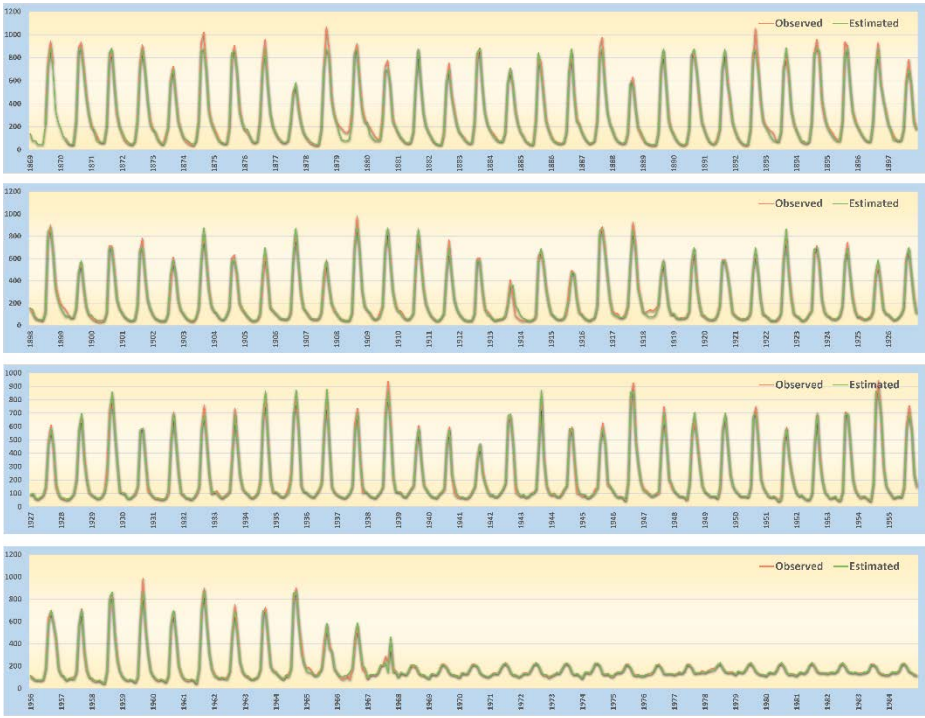


Fig. 4 Time series graphs of the observed and estimated discharge series at Aswan station

	1869	1870	1871	1872	1873	1874	1875	1876	1877	1878	1879	1880	1881	1882	1883	1884	1885	1886	1887	1888	1889	1890	1891	1892	1893	1894	1895	1896	1897
r	0.999	0.999	0.997	0.997	0.998	0.997	0.996	0.998	0.993	0.995	0.995	0.990	1.000	0.997	0.999	0.998	0.994	0.997	0.999	0.996	0.999	0.998	0.995	0.985	0.992	0.994	0.996	0.998	0.991
E	0.961	0.983	0.989	0.992	0.991	0.979	0.991	0.987	0.985	0.971	0.938	0.964	1.000	0.989	0.956	0.994	0.986	0.992	0.987	0.990	0.997	0.994	0.990	0.965	0.971	0.987	0.983	0.993	0.976
NRMSE	0.074	0.031	0.035	0.031	0.029	0.051	0.033	0.035	0.047	0.058	0.063	0.063	0.004	0.034	0.069	0.025	0.040	0.030	0.039	0.033	0.019	0.029	0.033	0.061	0.057	0.040	0.044	0.027	0.048
MASE	0.201	0.117	0.127	0.129	0.123	0.185	0.141	0.121	0.147	0.181	0.428	0.294	0.018	0.122	0.264	0.113	0.149	0.126	0.115	0.109	0.073	0.103	0.114	0.206	0.240	0.150	0.176	0.118	0.182

	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	1911	1912	1913	1914	1915	1916	1917	1918	1919	1920	1921	1922	1923	1924	1925	1926
r	0.995	0.989	0.999	0.993	0.987	0.998	0.993	0.997	0.999	0.989	0.997	0.997	0.995	0.998	0.997	0.892	0.994	0.978	0.997	0.997	0.989	0.994	0.999	0.997	0.994	0.998	0.998	0.993	0.999
E	0.989	0.970	0.995	0.985	0.968	0.995	0.985	0.972	0.995	0.969	0.991	0.993	0.987	0.992	0.993	0.749	0.987	0.948	0.995	0.986	0.949	0.988	0.997	0.994	0.986	0.993	0.994	0.982	0.997
NRMSE	0.036	0.054	0.027	0.041	0.056	0.024	0.042	0.051	0.024	0.054	0.031	0.027	0.037	0.028	0.029	0.146	0.042	0.081	0.026	0.038	0.068	0.035	0.020	0.027	0.039	0.030	0.025	0.044	0.019
MASE	0.162	0.194	0.131	0.116	0.150	0.086	0.142	0.179	0.087	0.144	0.093	0.106	0.152	0.070	0.112	0.498	0.199	0.251	0.107	0.167	0.276	0.114	0.096	0.089	0.138	0.105	0.087	0.157	0.066

	1927	1928	1929	1930	1931	1932	1933	1934	1935	1936	1937	1938	1939	1940	1941	1942	1943	1944	1945	1946	1947	1948	1949	1950	1951	1952	1953	1954	1955
r	0.995	0.996	0.991	0.996	0.999	0.999	0.999	0.997	0.997	0.996	0.996	0.999	0.995	0.994	0.996	0.998	0.997	0.991	0.997	0.996	0.998	0.994	0.999	0.998	0.998	0.998	0.999	0.993	0.999
E	0.990	0.993	0.980	0.992	0.996	0.990	0.992	0.988	0.987	0.987	0.983	0.990	0.989	0.985	0.988	0.992	0.990	0.975	0.992	0.991	0.991	0.983	0.997	0.994	0.992	0.994	0.998	0.987	0.992
NRMSE	0.034	0.029	0.050	0.031	0.021	0.033	0.027	0.036	0.039	0.036	0.043	0.033	0.032	0.039	0.034	0.033	0.030	0.053	0.030	0.031	0.030	0.044	0.017	0.027	0.029	0.024	0.017	0.040	0.032
MASE	0.133	0.100	0.152	0.099	0.081	0.117	0.103	0.125	0.134	0.102	0.151	0.106	0.109	0.121	0.130	0.120	0.101	0.187	0.114	0.117	0.173	0.156	0.066	0.077	0.117	0.066	0.063	0.132	0.103

	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984
r	0.995	0.997	0.997	0.995	0.998	1.000	0.998	0.996	0.993	0.980	0.987	0.921	0.993	0.997	0.998	0.998	0.997	0.996	0.998	0.992	0.995	0.987	0.965	0.992	0.995	0.997	0.998	0.996	0.997
E	0.990	0.994	0.994	0.984	0.996	0.997	0.985	0.991	0.986	0.937	0.951	0.801	0.977	0.990	0.995	0.994	0.993	0.991	0.994	0.981	0.986	0.966	0.892	0.983	0.988	0.991	0.982	0.990	0.992
NRMSE	0.037	0.024	0.029	0.038	0.021	0.018	0.040	0.032	0.041	0.077	0.068	0.120	0.050	0.031	0.025	0.025	0.027	0.033	0.022	0.044	0.039	0.063	0.107	0.043	0.034	0.028	0.047	0.034	0.029
MASE	0.131	0.099	0.109	0.100	0.078	0.074	0.153	0.122	0.158	0.280	0.254	0.398	0.144	0.136	0.110	0.112	0.133	0.145	0.093	0.194	0.199	0.226	0.419	0.183	0.161	0.131	0.193	0.186	0.135

Whole: r = 0.994; E = 0.987; NRMSE = 0.025; MASE = 0.144

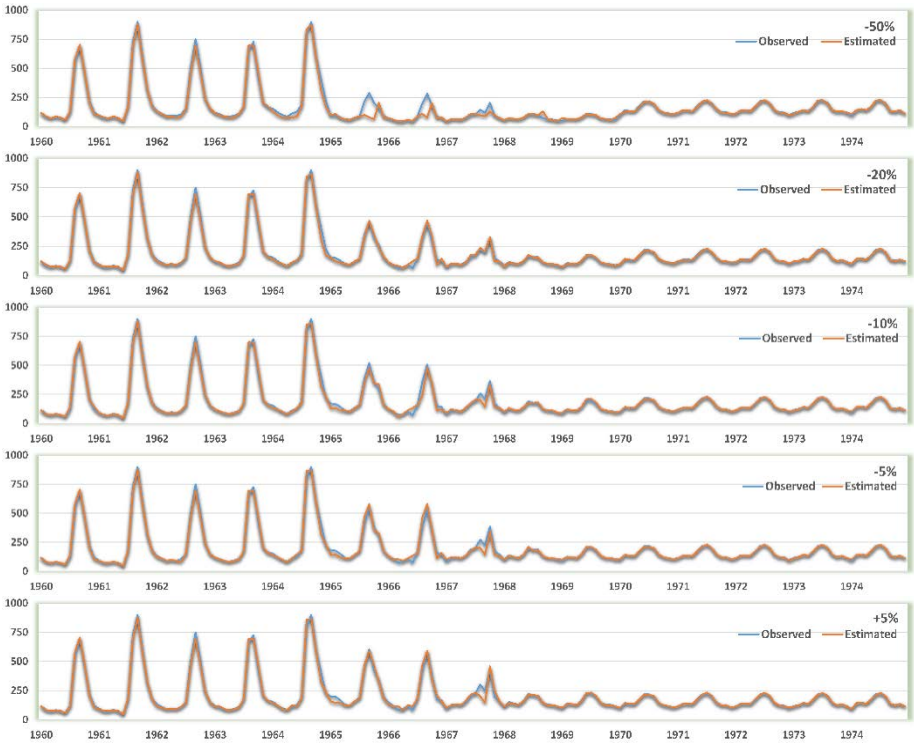
Fig. 5 Statistical evaluation of the estimation performance

The obtained results and the statistical evaluations show that the FBI method is very successful in the estimation of all monthly flow values of the Nile River. The yearly evaluations revealed that the method shows the best performance in the

estimations of 1881 series and the lowest estimation performance for the values in 1913. It is obvious that the decreased performance in 1913 is related with the fact that most of the historically lowest unexpected flow values in the data set were observed in 1913.

3.3 Sensitivity Analysis

The sensitivity analysis of the method is made by changing the values of the observations from 1965 to 1969 which seem to be more challenging to estimate as the flows significantly decrease in that period because of the construction of the Aswan Dam. The data between 1965 and 1969 are deliberately changed with the ratios -50%, -20%, -10%, -5%, +5%, +10%, +20% and +50% and estimations are repeated for the range 1960-1974 for testing the sensitivity of the proposed method against the changes in the dataset. Fig. 6 shows the time series graphs of observations and estimations after the values in 1965-1969 range are changed. Fig. 7 shows the statistical evaluation of the performance of the model when the changes are made. It was found that the model is highly sensitive to the changes in the dataset and the new estimations are generally close to the altered values; but, as anticipated, a low fraction of the estimations are not very close to the changed values when the new values are extremely low and are very rare in the dataset.



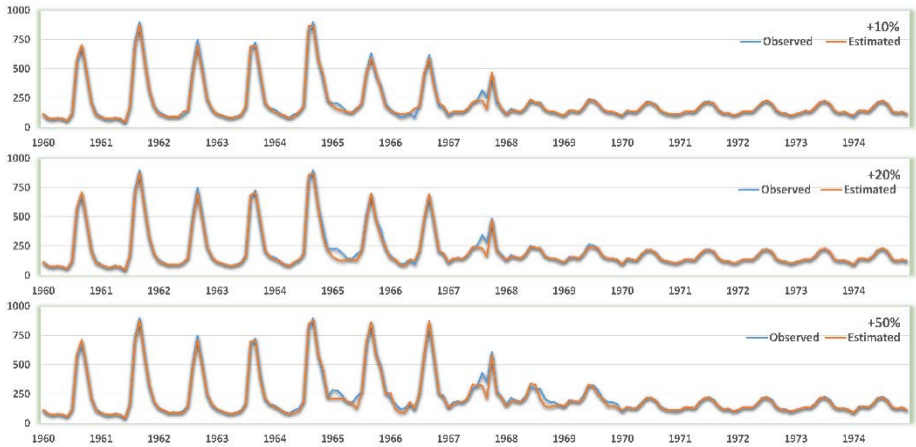


Fig. 6 The comparisons of observations and estimations when the discharges from 1965 to 1969 are changed by -50%, -20%, -10%, -5%, +5%, +10%, +20% and +50%

	Correlation								
	-50%	-20%	-10%	-5%	Obs	+5%	+10%	+20%	+50%
1960	0.999	0.999	0.998	0.999	0.998	0.999	0.999	0.999	0.999
1961	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.999	0.999
1962	0.999	0.999	0.999	0.999	0.998	0.999	0.998	0.999	0.999
1963	0.996	0.996	0.996	0.996	0.996	0.996	0.996	0.997	0.996
1964	0.995	0.993	0.994	0.992	0.993	0.993	0.996	0.995	0.997
1965	0.199	0.992	0.983	0.988	0.980	0.989	0.990	0.983	0.989
1966	0.578	0.981	0.951	0.982	0.987	0.989	0.984	0.997	0.995
1967	0.868	0.989	0.945	0.937	0.921	0.881	0.904	0.907	0.925
1968	0.734	0.985	0.990	0.991	0.993	0.991	0.987	0.990	0.859
1969	0.883	0.996	0.994	0.993	0.997	0.996	0.998	0.986	0.963
1970	0.993	0.998	0.996	0.997	0.998	0.997	0.997	0.993	0.986
1971	0.996	0.995	0.995	0.997	0.998	0.997	0.994	0.995	0.994
1972	0.998	0.997	0.997	0.996	0.997	0.996	0.996	0.994	0.995
1973	0.994	0.995	0.995	0.996	0.996	0.996	0.996	0.995	0.995
1974	0.991	0.993	0.993	0.994	0.998	0.997	0.994	0.992	0.998

	Nash-Sutcliffe Coefficient								
	-50%	-20%	-10%	-5%	Obs	+5%	+10%	+20%	+50%
1960	0.997	0.997	0.996	0.996	0.996	0.997	0.997	0.997	0.996
1961	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.998
1962	0.984	0.987	0.987	0.987	0.985	0.987	0.986	0.989	0.992
1963	0.991	0.992	0.991	0.991	0.991	0.991	0.992	0.992	0.992
1964	0.979	0.982	0.986	0.984	0.986	0.986	0.991	0.988	0.992
1965	-0.330	0.983	0.953	0.957	0.937	0.968	0.971	0.923	0.953
1966	0.242	0.948	0.894	0.931	0.951	0.975	0.962	0.989	0.985
1967	0.596	0.977	0.850	0.799	0.801	0.750	0.801	0.772	0.823
1968	-0.072	0.969	0.976	0.982	0.977	0.980	0.974	0.978	0.482
1969	0.760	0.991	0.951	0.983	0.990	0.992	0.994	0.955	0.871
1970	0.982	0.996	0.989	0.992	0.995	0.990	0.983	0.985	0.970
1971	0.992	0.990	0.989	0.994	0.994	0.992	0.988	0.990	0.988
1972	0.994	0.994	0.992	0.992	0.993	0.991	0.989	0.986	0.985
1973	0.988	0.990	0.990	0.991	0.991	0.993	0.992	0.989	0.982
1974	0.956	0.983	0.986	0.987	0.994	0.995	0.986	0.982	0.989

	NRMSE								
	-50%	-20%	-10%	-5%	Obs	+5%	+10%	+20%	+50%
1960	0.018	0.017	0.022	0.020	0.021	0.018	0.018	0.019	0.021
1961	0.018	0.017	0.017	0.017	0.018	0.018	0.017	0.017	0.016
1962	0.040	0.037	0.036	0.037	0.040	0.036	0.038	0.033	0.028
1963	0.032	0.031	0.032	0.032	0.032	0.032	0.031	0.031	0.031
1964	0.049	0.045	0.040	0.043	0.041	0.040	0.033	0.037	0.030
1965	0.353	0.040	0.066	0.063	0.077	0.055	0.052	0.085	0.066
1966	0.265	0.069	0.099	0.080	0.068	0.048	0.060	0.031	0.038
1967	0.170	0.041	0.104	0.120	0.120	0.134	0.119	0.128	0.113
1968	0.339	0.058	0.050	0.044	0.050	0.046	0.053	0.048	0.236
1969	0.158	0.030	0.070	0.041	0.031	0.029	0.024	0.067	0.113
1970	0.045	0.022	0.035	0.030	0.025	0.033	0.044	0.041	0.059
1971	0.029	0.034	0.034	0.027	0.025	0.030	0.037	0.033	0.036
1972	0.024	0.026	0.028	0.028	0.027	0.031	0.034	0.037	0.039
1973	0.039	0.035	0.035	0.033	0.033	0.030	0.032	0.037	0.047
1974	0.422	0.039	0.036	0.034	0.022	0.022	0.036	0.041	0.032

	MASE								
	-50%	-20%	-10%	-5%	Obs	+5%	+10%	+20%	+50%
1960	0.062	0.067	0.084	0.079	0.078	0.068	0.073	0.068	0.075
1961	0.066	0.070	0.071	0.070	0.074	0.071	0.070	0.072	0.071
1962	0.147	0.127	0.130	0.140	0.153	0.133	0.150	0.126	0.112
1963	0.121	0.118	0.115	0.117	0.122	0.124	0.114	0.111	0.113
1964	0.228	0.171	0.150	0.153	0.158	0.153	0.143	0.139	0.128
1965	1.105	0.165	0.288	0.257	0.280	0.223	0.236	0.361	0.279
1966	0.609	0.251	0.327	0.297	0.254	0.186	0.241	0.125	0.168
1967	0.495	0.143	0.357	0.411	0.398	0.411	0.354	0.370	0.334
1968	0.700	0.207	0.169	0.149	0.144	0.175	0.163	0.168	0.818
1969	0.512	0.143	0.287	0.165	0.136	0.134	0.098	0.274	0.451
1970	0.180	0.093	0.148	0.129	0.110	0.151	0.194	0.186	0.250
1971	0.142	0.156	0.153	0.115	0.112	0.136	0.150	0.141	0.155
1972	0.105	0.131	0.142	0.144	0.133	0.158	0.171	0.183	0.188
1973	0.149	0.149	0.153	0.144	0.145	0.127	0.130	0.159	0.187
1974	1.458	0.164	0.134	0.131	0.093	0.093	0.141	0.167	0.130

Fig. 7 Statistical measures for the estimations between 1960-1974 when the discharges from 1965 to 1969 are changed by -50%, -20%, -10%, -5%, +5%, +10%, +20% and +50%

3.4 Comparisons with other methods

The performance of the proposed method and the developed software is evaluated by comparing the best and worst correlated five years (a total of 10 years) with the estimations of EM and Regression methods. Fig. 8 shows the time series graphs of the observations and the compared methods. It was found for the compared years that the FBI method outperformed the EM and Regression methods for all compared years. The statistical measures are presented in Fig. 9 in which the greens

indicate the best and the reds indicate the worst performances. The results show that the FBI method has the best performance for all years for all statistical measures except for the somewhat pathologic and spurious correlations of 1913. It might be noticed from the time series graphs that the estimations of the FBI method in 1913 are much closer to the observations than the estimations of the other methods. Nash-Sutcliffe, NRMSE and MASE values of FBI in 1913 also validate the time series graph that FBI performs better than EM and Regression also in 1913 even though the correlation is lower. This situation shows why correlation alone is not always sufficient for making statistical comparisons and generally other measures are also required. It was shown in literature that correlation is misleading [16], good correlation does not automatically imply good agreement [17] and the risk of producing spurious correlations when analyzing non-independent variables is very large [18].

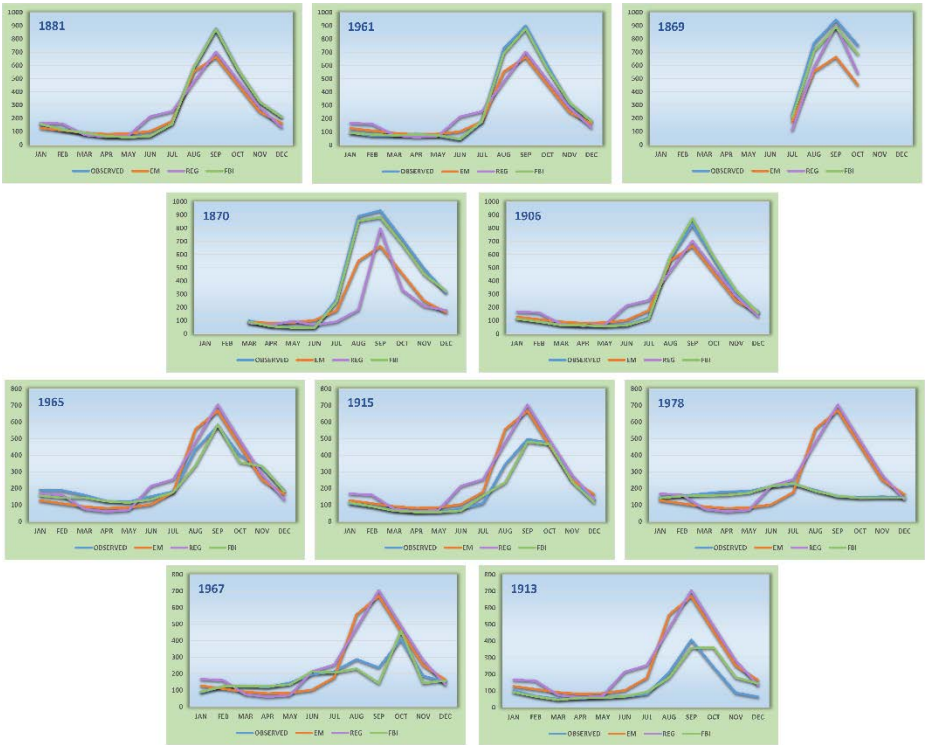


Fig. 8 Time series graphs of the observations and the estimations of FBI, EM and Regression methods

	Correlation (r)			Nash-Sutcliffe (E)			NRMSE			MASE		
	FBI	EM	Reg.	FBI	EM	Reg.	FBI	EM	Reg.	FBI	EM	Reg.
1881	1.000	0.988	0.965	1.000	0.910	0.886	0.004	0.093	0.105	0.018	0.351	0.464
1961	1.000	0.997	0.954	0.997	0.878	0.828	0.018	0.116	0.138	0.074	0.450	0.600
1869	0.999	0.980	0.973	0.961	0.255	0.685	0.074	0.323	0.210	0.201	0.847	0.548
1870	0.999	0.979	0.760	0.993	0.686	0.284	0.031	0.211	0.319	0.117	0.863	1.119
1906	0.999	0.992	0.966	0.995	0.938	0.896	0.024	0.081	0.104	0.087	0.320	0.463
1965	0.980	0.979	0.966	0.937	0.773	0.768	0.077	0.146	0.147	0.280	0.707	0.736
1915	0.978	0.952	0.947	0.948	0.716	0.628	0.081	0.190	0.217	0.251	0.692	0.869
1978	0.965	-0.221	-0.166	0.892	-67.695	-68.262	0.107	2.698	2.709	0.419	9.907	9.367
1967	0.921	0.733	0.749	0.801	-2.273	-2.342	0.120	0.485	0.490	0.398	1.521	1.611
1913	0.892	0.945	0.952	0.749	-1.224	-1.531	0.146	0.435	0.464	0.498	1.685	1.943

Fig. 9 Statistical measures for the performances of the FBI, EM and Regression methods

4. CONCLUSIONS

The application of the FBI method, a novel data driven method and software for forecasting and estimating missing data, has been presented. The method makes estimations based on the frequencies of data pairs throughout the available data. A range based clustering approach is implemented to determine the cluster frequencies. It has been demonstrated that the developed method is very successful in estimating all monthly flow observations of Nile Aswan station even though there is a significant regime change after the construction of the Aswan Dam. The approaches of the method are not limited with hydrology and the method might also be applied quite reliably in other fields dealing with nonrandom data. Future work will mainly cover the development of additional features for the method in order to improve estimation of extreme values. Future studies will also involve the application of the proposed algorithm to data from fields like finance, social sciences and biostatistics.

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Trends in Yearly Precipitation and Temperature on the Aegean Region, Turkey

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***Abstract* – The values of a precipitation and air temperature parameters are very important issues in climatological, meteorological and hydrological events. In this study we examined precipitation and air temperature trends in Aegean Region in Turkey for minimum 50 yearly time scale in 95% of confidence intervals. This study aims to determine trends in the long-term annual mean precipitation and air temperature series using the Linear regression analysis, Mann–Kendall and Sen's tests. Results of the temperature data trend show a general increasing trend. But result of the precipitation data trend show a general decreasing trend.**

***Keywords* – Linear regression, Mann-Kendall, Precipitation, Sen's, Temperature, Trend analysis, Turkey**

1. INTRODUCTION

Climate of the Earth varies across temporal and spatial scales throughout the planet [1], [2]. Climatic variability can be described as the annual difference in values of specific climatic variables within averaging periods such as a 30-year period [3].

Turkey possess risk of climate change is one of the countries [4]. The extreme and unpredictable climate variability is enormous pressure on water resources, drought and land degration. In climatic elements in terms of time and space parameter is the maximum amount of precipitation and temperature variability and climate change increases and decreases observed in this direction has the characteristics of the most important evidence [5]. Air temperature and precipitation are principle element of weather systems. Therefore. recently. the focus on climate variability bases mostly on the detection of trends in records of precipitation and temperature [4], [5], [6]. The effects of climatic change and variability have been analyzed by many researchers in a variety of geophysical fields.

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There are many studies on year by changes in annual and monthly and seasonal rainfall, temperature, streamflow, water quality and lake levels in Turkey. Reviews of relevant recent researches include. For some example: Kadıoğlu et al. [7], Partal and Kahya [8], İstanbulluoğlu et al. [9], Turkes [5] and Zhang et al. [10] for precipitation; Kadıoğlu [11] temperature; Tağıl and Danacıoğlu [12], Kızılelma et al.[13] precipitation and temperature; Içağa and Harmancıoğlu [14], Kalayci and Kahya [15] for water quality variables; Önöz and Beyazıt [16], Kahya and Kalayci [17] and Cıgızoglu et al. [18] for streamflow; and finally Cengiz et al. [19] for lake levels. Partal and Kahya (2006); January, Ferbruary, September decreasing trend is observed on annual precipitation data in Turkey between 1929-1993 [8]. Karabulut (2012) was found which maximum and minimum temperature there are obviously increasing trend in Mediterranean Region [20]. Tağıl and Danacıoğlu (2012) found which as a result trend of temperature data is determined to increasing way in all stations in Aegean Region [12]. Kızılelma, Çelik and Karabulut (2015), found which maximum and minimum temperature trend is increased in Central Anatolia Region [21]. There are many studies in different countries applied the trend anlysis tests to temperature and precipitation data in India by Arora. et al. [22]; in Lisbon. Portuguese by Santos and Leite [23]; in Canadian Prairies by Lawson [24]; in South Brazil by Sansigolo and Kayano [25]; in India by Pramod Kumar Meena. et al. [26]; in Pakistan by Ijaz Ahmad. et al. [27].

In this study we examined precipitation and air temperature trends in Aegean Region in Turkey for minimum 50 yearly time scale. The purpose of this study was to determine changes in the annual variations of precipitation and air temperature. The results of this study are important for Aegean Region with its many agricultural areas.

2. STUDY AREA

Turkey's diverse regions have different climates because of irregular topography. Aegean region with its 79.000 square kilometers of land. The Aegean region occupies 11% of the total area of Turkey (Figure 1). The Aegean coastal plain has an exceptionally mild climate. The Aegean region has perpendicular mountains to its shores and many valleys between them. Although some of the provinces inland show also characteristics of continental climate [28], [29]. Annual rainfall amounts in the Aegean regions are with the means ranging between 600 and 800 mm in the Aegean Region; west to east elongated deep valleys. Observed annual precipitation and temperature data records from meteorological stations located in Aegean Anatolia in Turkey have been selected for this study. The length of available records at these stations is minimum 50 year. The evaluated monthly rainfall data were measured by the Turkish State Meteorological Services [30].

Fig. 1 Aegean Region

Table 1 Raingauge Stations in the Aegean Regions [30]

Station Name	Latitude	Longitude	Elevation (m)	Station Name	Latitude	Longitude	Elevation (m)
Denizli	37.7620N	29.0921E	425.29	Bodrum	37.0328N	27.4398E	26.47
Aydın	37.8402N	27.8379E	56.3	Datça	36.7093N	27.6919E	28
Kuşadası	37.8597N	27.2652E	25	Fethiye	36.6266N	29.1238E	3
Nazilli	37.9135N	28.3437E	84	Köyceğiz	36.9700N	28.6869E	24
Bergama	39.1098N	27.1710E	53	Milas	37.3027N	27.7804E	52
Çesme	38.3036N	26.3724E	5	Muğla	37.2095N	28.3668E	646.07
Dikili	39.0737N	26.8880E	3.4	Kütahya	39.4171N	29.9891E	960
İzmir	38.3949N	27.0819E	28.55	Simav	39.0925N	28.9786E	809
Ödemis	38.2157N	27.9642E	111	Acıpayam	17.4337N	29.3498E	941
Akhisar	38.9118N	27.8233E	92.034	Güney	38.1515N	29.0587E	825
Manisa	38.6153N	27.4047E	71	Selçuk	37.9445N	27.3673E	17
Afyon	38.7380N	30.5640E	1001.49	Salihli	38.4831N	28.1234E	111
Dinar	38.0597N	30.1531E	864	Emirdağ	39.0098N	31.1463E	983
Uşak	38.6712N	29.4040E	919.22	Tavşanlı	39.5439N	29.4917E	833

3. METHODS

3.1. **Mann Kendall Method:** The non-parametic Mann-Kendall test (MK) [31], [32] has generally been used to determine the significance of a trend at a site. This test is based on the statistic S. S is calculated by using

$$S = \sum_{k=0}^{n-1} \sum_{j=k+1}^{n} \operatorname{sgn}(x_j - x_k) \quad \operatorname{sgn}(x) = \begin{cases} +1, & x > 0 \\ 0, & x = 0 \\ -1, & x < 0 \end{cases} \tag{1}$$

In Eq.(8). where n is the length of the data; xj and xk are the data values in time series k and j (j>k). respectively. In cases where the sample size n≥10. the mean and variance are given by

$$E(s) = 0 \tag{2}$$

$$Var(s) = \sigma_s \sqrt{\left(n(n-1)(2n+5) - \sum_{1}^{r} t_i (t_i - 1)(2t_i + 5) \right) / 18} \tag{3}$$

In Eq.(10). where. n is the number of tied groups and ti denotes the number of ties of extent i. A tied group is a set of sample data having the same value. The standard normal test statistic Z is computed as:

$$Z = \begin{cases} \dfrac{s-1}{\sigma_s}, & s > 0 \\ 0, & s = 0 \\ \dfrac{s+1}{\sigma_s}, & s < 0 \end{cases} \tag{4}$$

According to Eq. (11). a positive values of Z inform increasing trends while the negative Z inform decreasing trends. Testing of trends is made at a specific α

significance level. The significance level of α = 0.05 was used in this study. At the 5% significance level. the null hypothesis of no trend is rejected if IZI>1.96 [31], [32].

3.2. **Sen Method:** The non-parametric procedure developed by Sen [33]. The slope estimates of N pairs of data are predicted by Sen's estimator.

$$Qi = \frac{(x_j - x_k)}{j - k} \quad (i = 1, \dots, N) \tag{5}$$

In Eq.(12). where xj and xk are the data values in time j and k (j>k). respectively.

The N values of Qi re ranked from smallest to largest and the median of slope or Sen's slope estimator is computed as

$$Q_{med} = \begin{cases} \left[Q_{\frac{(N+1)}{2}}\right] N\ is\ odd \\ \frac{1}{2}\left[Q_{\frac{N}{2}} + Q_{\frac{(N+2)}{2}}\right] N\ is\ even \end{cases} \tag{6}$$

Qmed is computed by two sided test and then a true slope can be obtained by the non-parametric test.

3.3. **Linear Regression Method:** A linear regression method is to check whether there is a significant relation between the variables under consideration. The regression lines are used to estimate a slope. The slope indicates the mean temporal change of the studied variable. Positive slope values of the slope show increasing trends. while negative slope values of the slope indicate decreasing trends. A linear regression line has an equation of the from

$$y=a+bx \tag{7}$$

where x= the explanatory variable. y= the dependent variable. b= the slope of the line and a=the intercept [36].

4. RESULTS AND SIGNIFICANCES

Article I, Article II, Article III, Article IV

Annual precipitation and temperature were made trend analyzed for Aegean Region in Turkey. An increasing trend and change of annual precipitation and temperature of the İzmir station is shown Figure 2 and 3.

Linear Regression analysis was made of the value of annual precipitation and temperature and presented in Table 3 and Table 4. According to Linear Regression statistically significant (%5 risk) negative trend are found for annual precipitation in Ödemiş ve Muğla. Statistically significant positive trend are found for annual temperature in all stations.

As shown in the Aegean Region stations of the significant trends at the 90% and 95% significance levels in the precipitation data were not significant for Mann-Kendall (Table 4). In the temperature data the significant increasing trend was found at the all stations for Mann-Kendall and Sen's Methods (Table 5). In the precipitation data the significant increasing trend was found at the Denizli, Kuşadası, Nazilli, Çeşme, İzmir, Afyon, Dinar and Uşak stations and the significant idecreasing trend was found at the another all stations for Sen's Methods (Table 5).

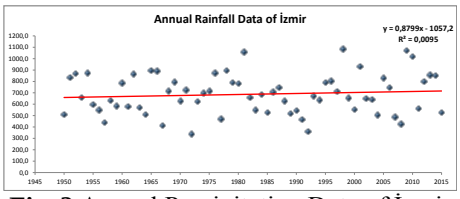

Fig. 2 Annual Precipitation Data of İzmir in Aegean Region, Turkey

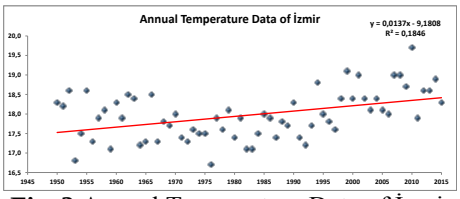

Fig. 3 Annual Temperature Data of İzmir in Aegean Region, Turkey

Table 3 Annual Trend analysis results of precipitation for the study area.

Precipitation		Mann-Kendall		Sen's Slope		Lineer Regression	
Station	Data	z	Trend	Qmedian	Trend	t	b
Denizli	1957-2015	0.89	—	0.84	▲	0.74	0.66
Aydın	1950-2015	-0.50	—	-0.59	▼	0.80	-0.77
Kuşadası	1950-2015	0.84	—	1.14	▲	0.89	1.24
Nazilli	1950-2012	0.39	—	0.54	▲	0.17	0.20
Bergama	1963-2012	-2.52	▼	-4.46	▼	2.12	-3.72
Cesme	1963-2015	0.76	—	0.87	▲	0.93	1.25
Dikili	1950-2015	-1.56	—	-1.82	▼	1.61	-1.64
İzmir	1950-2015	0.46	—	0.56	▲	0.78	0.88
Ödemis	1950-2012	-1.60	—	-1.81	▼	2.21	-2.46
Akhisar	1950-2015	-1.47	—	-1.56	▼	1.50	-1.28
Manisa	1950-2015	-1.20	—	-1.45	▼	1.01	-1.14
Afyon	1950-2016	0.79	—	0.37	▲	0.69	0.38
Dinar	1963-2012	0.33	—	0.29	▲	0.22	0.25
Uşak	1950-2015	0.25	—	0.19	▲	0.21	-0.17
Bodrum	1950-2015	-1.17	—	-1.51	▼	0.91	-1.06
Datça	1965-2015	-0.19	—	-0.47	▼	0.19	-0.32
Fethiye	1950-2015	-1.02	—	-1.64	▼	0.21	-0.32
Köyceğiz	1963-2012	-0.79	—	-2.26	▼	0.61	-1.04
Milas	1960-2012	-1.69	—	-3.10	▼	0.86	-2.33
Muğla	1950-2015	-0.13	—	-0.13	▼	2.19	-3.12
Kütahya	1950-2015	-0.91	—	-0.56	▼	1.11	-0.71
Simav	1959-2012	-2.33	▼	-4.95	▼	2.37	-4.44

Table 4 Annual Trend analysis analysis results of temperature for the study area.

Temperature		Mann-Kendall		Sen's Slope		Lineer Regression	
Station	Data	z	Trend	Qmedian	Trend	t	b
Acıpayam	1966-2015	4.53	▲	0.04	▲	2.73	0.04
Denizli	1956-2015	5.20	▲	0.03	▲	4.25	0.05
Güney	1963-2015	3.99	▲	0.02	▲	4.18	0.03
Aydın	1950-2015	2.09	▲	0.01	▲	2.29	0.01
Kuşadası	1950-2015	5.90	▲	0.05	▲	5.70	0.07
Nazilli	1950-2015	1.28	—	0.01	▲	1.44	0.01
Bergama	1961-2015	4.92	▲	0.03	▲	3.38	0.09
Çeşme	1963-2015	3.41	▲	0.02	▲	2.70	0.02
Dikili	1950-2015	2.11	▲	0.01	▲	2.35	0.01
İzmir	1950-2015	3.30	▲	0.01	▲	3.81	0.01
Ödemiş	1950-2015	1.32	—	0.01	▲	1.37	0.01
Selçuk	1963-2015	5.20	▲	0.03	▲	6.21	0.04
Akhisar	1950-2015	3.34	▲	0.02	▲	3.58	0.01
Manisa	1950-2015	1.92	—	0.01	▲	2.08	0.01
Salihli	1959-2015	4.27	▲	0.03	▲	3.40	0.05
Afyon	1950-2015	3.70	▲	0.02	▲	3.93	0.02
Dinar	1959-2015	4.62	▲	0.03	▲	4.70	0.06
Emirdağ	1963-2015	1.66	—	0.01	▲	2.16	0.03
Uşak	1950-2015	3.25	▲	0.01	▲	3.69	0.01
Bodrum	1950-2015	3.14	▲	0.01	▲	3.73	0.02
Datça	1965-2015	5.18	▲	0.02	▲	6.09	0.03
Fethiye	1950-2015	2.19	▲	0.01	▲	2.08	0.01
Köyceğiz	1959-2015	3.24	▲	0.01	▲	3.59	0.04
Milas	1960-2015	5.91	▲	0.04	▲	3.70	0.07
Muğla	1950-2015	2.42	▲	0.01	▲	2.73	0.01
Kütahya	1950-2015	2.86	▲	0.02	▲	3.16	0.02
Simav	1959-2015	2.42	▲	0.02	▲	2.32	0.01
Tavşanlı	1965-2015	4.78	▲	0.03	▲	5.45	0.03

5. CONCLUSIONS

In this study was to analyze annual precipitation and temperature trends in selected stations in the Aegean region of Turkey for the minimum 50 yearly period. For this analyses were used Linear Regression, Mann-Kendall and Sen's methods.

Statistically increasing trend are found for annual precipitation in all stations except Ödemiş ve Muğla stations for linear regression. But in the precipitation data were not significant trend expect Bergama and Simav for Mann-Kendall. In the precipitation data the significant decreasing trend was found in all stations except

Denizli, Kuşadası, Nazilli, Çeşme, İzmir, Afyon, Dinar and Uşak stations. In the temperature data the significant increasing trend was found at the all stations for Mann-Kendall except Nazilli, Ödemiş, Manisa, Emirdağ and increasing trend was found at the all stations for Sen's Methods. Results of the temperature data trend show a general increasing trend, but result of the precipitation data trend show a general decreasing trend. This results were indicator of effects of climate change and drought in Aegean Region. After the operating, managing and planning of water resources in the region should be taken into consideration of this effects of climate change and drought.

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Modeling Tools for Flood Mitigation in Urban Areas

Mihai STANCU, Maria CHEVERESAN, Tudor POIENARIU, Valentin ZAHARIA

***Abstract* – Floods are the most destructive natural accounting for more than 45% of the total material damages and loss of lives caused by natural disasters [1]. There is a great focus on flood mitigation in urban areas taking into consideration the amount of vulnerable assets as well as the impact upon people's lives. This paper presents a case study which applies modern software technology to model urban flood events in order to identify the flooding hazard areas and to test the proper constructive mitigation solutions for preventing these phenomena to take place as well as to protect the inhabitants living in the urban areas.**

***Keywords* –Coupled 1D and 2D modelling, urban areas, flood mitigation**

1. INTRODUCTION

Urban flooding is a very important aspect which has to be taken into consideration in urban planning because of the high potential of producing high damages and loss of lives. Flooding in urban areas most often occur due to high intensity or long period rainfall events which exceeds the capacity of the drainage system. In the last decade the urban areas are going under a steep development process, especially due to the increase of the number of inhabitants, leading to an expansion of the impervious areas, followed by a reduction of the infiltration capacity and increase of the pluvial discharge response into the collection system.

Bucharest, with a drained surface of approximately 228 km^2 has one of the biggest and most complex drainage system in the country. The capital's sewer is a combined system which collects the wastewater and the runoff volume through more

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than 2250 km of sewer pipes, out of which 250 km are main collectors. The age of the drainage network (approximately 22% of the drainage system is more than 60 years old [3]) leads to the increase of water transported by the sewer due to infiltration from the groundwater and implicitly the increase of the water volumes that need treatment in the wastewater treatment plant. Certain parts of the system are subject to high sedimentation processes obstructing the flow ultimately leading to floods in the respective area.

The Bucharest collection system has been designed to completely drain the rainfall water for events with a return period of 1:3 for the main collectors and 1:2 for the secondary pipes [2]. The extreme rainfall events in the last years have been highlighted some particular areas to be more susceptible to flooding.

This paper describes the flood assessment and the application of constructive mitigation measures against such events through mathematical modeling in a central area of Bucharest, called *Tineretului*. In order to fully describe the flooding phenomena and to precisely represent the field conditions into the models it has been concluded that two models are needed: 1D wastewater collection system model for the accurate representation of the drainage system in the area of interest and 2D model for propagation of the spilled discharges over the terrain surface and assessment of the flood extension. The two models are coupled in order to continuously simulate the phenomena, without the need to manually transfer the boundary conditions in between them.

2. EXPERIMENT DESCRIPTION

Tineretului area has been flooded repeatedly over the years because of the high intensity rainfalls and the incapacity of the collection system to drain the runoff volume and transport it downstream. More than 5 flooding events occurred in this area in the past 5 years, which led to the necessity to identify a series of measures in order to mitigate the flooding.

Tineretului area is located in the south-eastern part of Bucharest, near the Vacaresti natural park and in the vicinity of the Dambovita River which crosses the city from north-west to south-east. Underneath the Dambovita river is located the main sewer collector of the capital (further referred to as *CASETA*) with varying dimensions between 3.3 meters wide by 2.45 meters in height and 4.5 meters wide by 3.6 meters in height [4]. This main collector drains the wastewater from the collection network and routes it to the Glina wastewater treatment plant, as it can be observed from the picture below.

In order to properly assess the impact of flooding over Tineretului area it was necessary to create both a two-dimensional and a one-dimensional flow model (see fig. 2). The input data (discharges) into the 2D model is provided by a 1D model which schematizes the wastewater collection system. One particularity of this approach is that the 2 aforementioned models are coupled so the exchange of information is bi-directional, therefore if the collection system is put under pressure the discharges are routed to the surface through the manholes and as soon as the pressure in the pipes drops the water from the surface re-enters in the sewer.

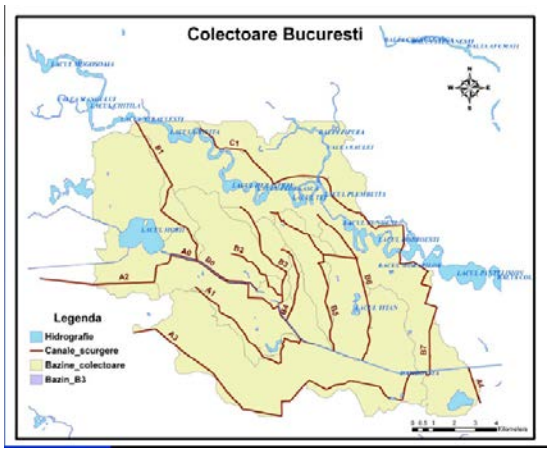

Fig. 1 Layout of the main collectors in Bucharest [2]

This allows to properly model the processes and to obtain results with a higher degree of confidence.

Fig. 2 Map of the 1D model drainage area and the area of interest for the 2D model

Mike by DHI solutions have been used in the modelling activity. Mike Urban was used for the 1D collection system model, whereas Mike 21 Single Grid was used for the 2D flow model. The 2 model were then coupled using DHI's solution Mike Flood which enables a bi-directional communication and data transfer between the two models.

The one-dimensional flow model covers an area of 689 ha and drains the rainfall water and the wastewater from the hydrographic basin of the ‘A1’ collector. The total length of the sewer pipes included in the model is approximately 102 km distributed along a number of 2847 pipes. Over the 689 ha area of the drainage basin there are

distributed over 2760 nodes which leads to a drainage area of 2491 m^2 per node (calculated based on the data provided by Apa Nova Bucuresti).

Regarding the physical characteristics of the pipes, over 93% are made of concrete, while the rest of 7% are made of PVC and PAFSIN and over 80% have a circular cross-section. Approximately 78% of the sewer pipes have a diameter under 500mm [5].

The A1 collector (red color) discharges into the A0 collector (green color) (see fig. 3). Because of this connection and because of the terrain characteristics, a steep transition from high to low grounds equivalent to a 17% slope, during high intensity or long period rainfall events, the A1 collector starts operating under pressure which leads to flooding the area of interest (see fig. 4) [5].

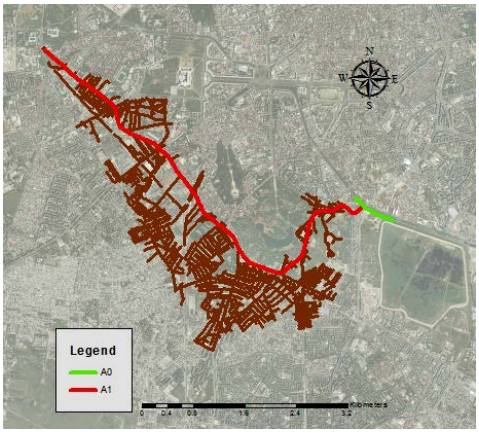

Fig. 3 Connection point between the A0 and A1 collectors [5]

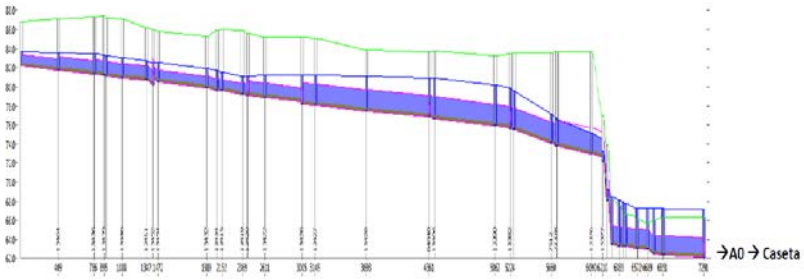

Fig. 4 Longitudinal profile along the A1 collector [5]

An analysis over the percentage of impervious areas has shown that from the total of 689ha more than 30% are completely impervious while less than 20% are green areas (see fig. 5).

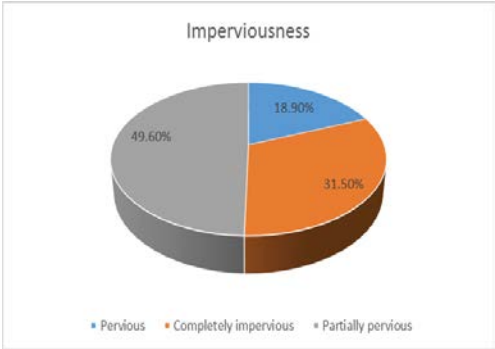

Fig.5 Chart of imperviousness percentages [5]

In order to ensure that the results are as close as possible to the measurements a calibration and validation process has been conducted. In order to calibrate the model a series of simulations were carried out using two rainfall events from September 2013, respectively April 2014. The differences between the results of the model and the measured data were between 7% and 17% for maximum discharge and between 4 and 14% for runoff volume.

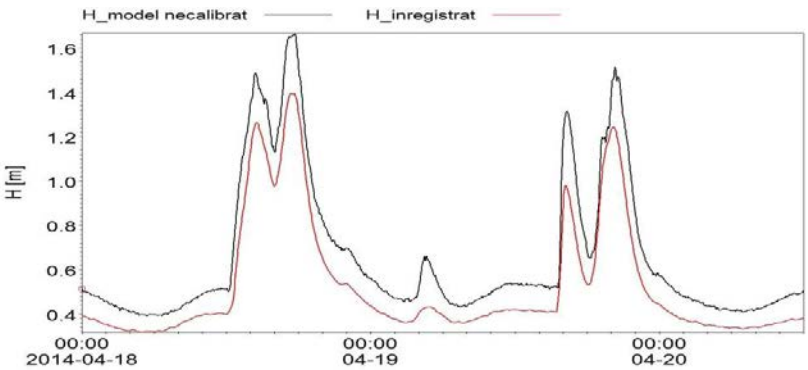

Fig 6 Uncalibrated model. Water level (Red – measured, Black – simulated)

Although the shapes of the simulated and measured water level hydrographs are similar there is a difference of approximately 15% in between the two types of values which needed to be reduced through a calibration process. In order to calibrate the model the impervious coefficients have been adjusted and the rainfall reduction factor (coefficient that takes into account the infiltrations, the small imperfections of the roads that cause water to be stored there, etc.) was decreased. These modifications led to smaller differences between the simulated and measured values.

DHI's solution Mike 21 was used to create the 2D flow model using a highly detailed resolution computational single grid comprised of rectangular cells with a size of 1m by 1m per cell resulting a discretization of 1775000 computational cells for the whole area of approximately 500 ha.

In comparison to the area taken into account for the modelling of the collection system behavior, the 2D model extent is smaller covering the downstream part of the hydrographic basin of the A1 collector because during extreme rainfall events only the downstream part of the area was flooded.

For an accurate representation of the Tineretului area two alterations of the Digital Terrain Model were made: DTM was lowered with 20cm based on the streets layout and raised with about 5m based on the footprints of buildings (see fig. 8).

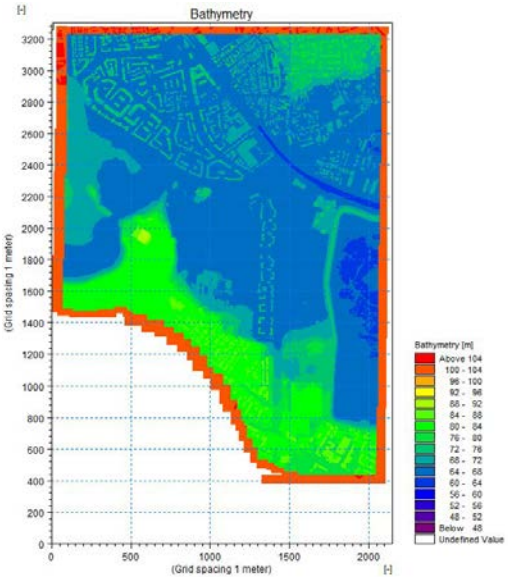

Fig. 8 Mike 21 Computational Grid

The coupling of the 1D and 2D hydraulic models was done using Mike Flood. The link between the 2 models is able to transmit data in both directions, therefore in the eventuality of an extreme rainfall event which causes the collection system to operate under pressure, water is discharged from the sewer to the surface through the manholes causing flooding. As soon as the collection system stops operating under pressure, the water a part of the water returns into the pipes while the rest may be stored in various locations due to the configuration of the terrain.

Two scenarios have been proposed for reducing the amount of water discharged at the surface. The first solution consists in a separate collector connected to the 'A1' collector and equipped with a weir that starts operating when the water level equals and overtops the weir crest. The second scenario consists of one retention basin that should retain water for a sufficient period of time in order to have the collection system operating normally.

Both the coupled model and the 1D scenario were simulated using a third rainfall event (higher intensity than the previous two rainfall events used for the calibration and validation processes) that took place between 30.06.2014 and 01.07.2014 with a total rainfall amount of about 120mm.

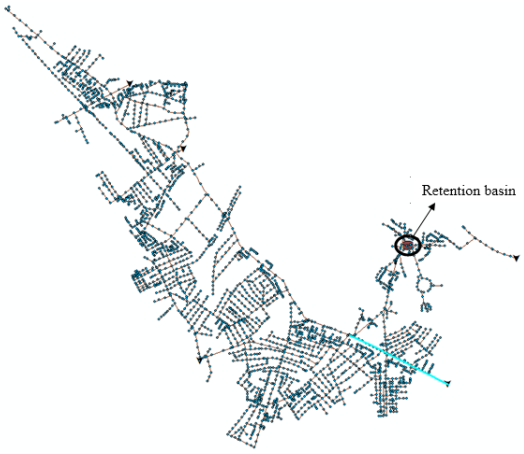

Fig. 9 Mike Urban – Proposed solutions

Using a third event for this simulation provided valuable information regarding the calibration and validation processes, as the model results were very close to the measured data.

3. RESULTS

As it can be seen from the figure below the results from the 1D simulation can provide valuable information regarding the approximate area where flooding can occur for a specific rainfall event. It can be observed that the downstream area of the A1 collector's hydrographic basin, at the connection with the A0 collector is flooded, the blue nodes representing manholes where the simulated water level is higher than the ground level.

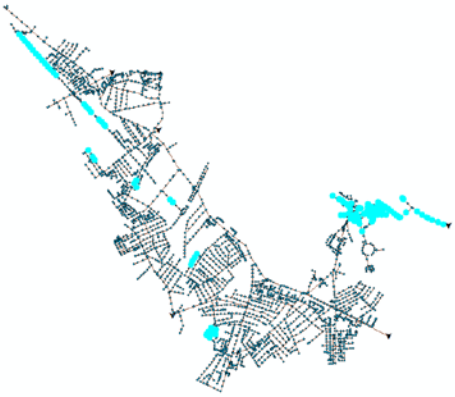

Fig. 10 Mike Urban - Nodes where the water level is higher than the ground level of the manhole

Although it might be sufficient just to analyze the 1D results, the 2D modelling results give a much more better understanding of the phenomena, as the results can be viewed either as static or dynamic layers. As it can be seen in the picture below the water spreads primarily on the streets where the terrain level has been lowered, in order to represent the road layout. Also, a small part of the green areas is flooded. The water depths in the area are between 2-3mm (light blue color) and reach a maximum of 85 cm (red color), depending on the terrain, with an average value of 23 cm. The surface of the flooded area is around 93000m^2.

Fig. 11 Flood extent. Water depths

Also, one noticeable difference is the simulated water level along a longitudinal profile in both the 1D and 1D-2D coupled modelling. It can be seen in the image below that the water level obtained in the coupled model (red line) is lower than the one resulted from the 1D simulation (blue) because of the dynamic interaction regarding the exchange of data during the simulation between the two models.

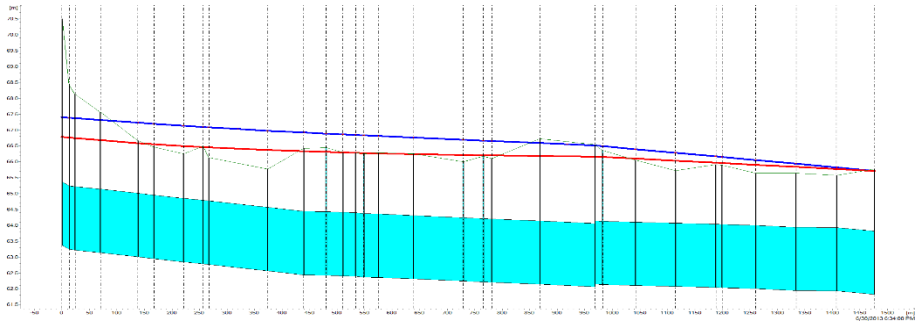

Fig. 12 Water level along the A1 collector (Red – 1D-2D coupled model, Blue – 1D model)

A couple of simulations were made in order to quantify the results of the scenarios containing both structural measures. It resulted that only 23 nodes in the system had the simulated water level higher than the ground level (compared to the initial number of 93 nodes before implementing the constructive measure).

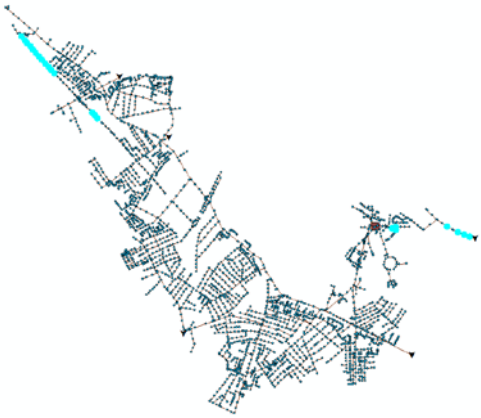

Fig. 13 Manholes with the simulated water level higher than the ground level – after implementation of measures

4. CONCLUSIONS

Due to several reasons that include the increase of the occurrence of high intensity rainfall events, the terrain characteristics and the layout of the collection system, Tineretului area is flooded at every significant rainfall event more than 5 times in the past 5 years).

The 1D model created to simulate the collection system is able to accurately represent the flow conditions in the system and to provide high precision results regarding the water levels, discharges and velocities. Also the model can provide an idea of the approximate area that is being flooded by analyzing the manholes where the simulated water level is higher than the ground level.

Due to the fact that the 1D model doesn't contain information regarding the terrain configuration, the water is discharged, from the moment when the system starts operating under pressure, in artificial basins corresponding to each individual manhole (process which doesn’t occur in reality). The water level in these artificial basins can continuously increase until the collection system stops operating under pressure. The water volumes stored in the artificial basins flow back into the sewage system.

The latter aspect is corrected using the 2D model which takes into account the discharges from the sewer system and determines the real flow paths, flooding limits and water levels and the surface of the ground based on the Digital Terrain Model.

One disadvantage of the 2D model would be the lack of data needed in order to calibrate the 2D flow model (water levels, extent of flooded area measured for a particular flood event which occurred in the area of interest).

A separate collector linked to the A1 collector upstream of the analyzed 2D area would significantly improve the behavior of the collection system and reduce the extent of the flooded area.

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