Course Materials of the International Postgraduate Course on Flood Management

Edited by
Enikő Anna Tamás – László Mrekva – Jasna Plavšić

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Foreword

Floods are amongst the major issues in the Danube River Basin, if not the most important one. Several floods during the last decade were very huge, although there had been even larger ones over the long history of the region. In addition to huge floods in the Danube River in 2006, 2010, 2013 and 2014, even larger ones had happened along the major tributaries of the Danube in its middle course such as those in the Tamis River in 2005, the Tisza River in 2006, the Sava River in 2005, 2013 and 2014, as well as in the Sava River's principal tributaries: the Drina River in 2010, the Kolubara River in 2014. Damages topped billiards of Euros, and the casualties were even worse.

The management of floods is usually based on harmonised flood defence planning, forecast procedures and co-ordination on the national and international level, through the co-operation of different institutions. The EU Strategy for the Danube Region (EUSDR) recognises the importance of flood management. Thus, the Danube Strategic Project Fund (DSPF) supported the InterFloodCourse Project which aimed at the development of a curriculum and training material for the international, basin-wide course on flood management.

Basic documents related to flood issues developed so far in the EU are the EU Floods Directive and the Danube Basin Flood Risk Management plan. The curriculum within the InterFloodCourse Project is developed by distinguished experts from 7 countries basin-wide, having a long experience in water management education, research and engineering practice. They prepared a book which covers numerous phases within the flood management context, as well as a brief overview of impacts of climate change on floods and those of flood duration and magnitude on the environment, navigation, urban infrastructure systems and flood control structures. Various topics are complemented with practical experience on the Danube River and its tributaries in Hungary and Serbia.

The book supporting the course has been prepared, co-ordinated and edited by the two Project partners: The Faculty of Water Sciences of the University of Public Service and the Faculty of Civil Engineering of the Belgrade University.

The Course is offered at provision to interested parties and state agencies, who will benefit both from general and advanced knowledge in river hydrology and hydraulics, including statistics, sediment, soil and ground data, and different flood forecast aspects.

The Course will be divided into two parts in accordance with the curriculum. The first half of the course, which roughly covers 8 chapters of the book, will be held at the University of Belgrade – Faculty of Civil Engineering, and the second half, covered by additional 8 chapters, at the Faculty of Water Sciences in Baja. In addition to class trainings, one field trip is included in each host country.

It should be stressed once again that the course aims at the preparation of flood management professionals for the prevention and the decreasing of the damages and casualties in the entire Danube River Basin.

Baja–Belgrade, January 2019

Enikő Anna Tamás, Jovan Despotović

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Stevan Prohaska

Historic Floods on the Danube

The present paper stems from the draft of the project “Flood Regime of Rivers in the Danube River Basin”, which is being prepared by the Slovak National Committee for IHP UNESCO at the Institute of Hydrology, Slovak Academy of Sciences, where the author of the paper is involved as a member of the Steering Committee and is the nominated expert from Serbia, contributing two separate chapters.

General characteristics of the Danube River Basin

The Danube River is a unique international waterway flowing across Europe. It is 2,857 km long. The Danube runs from the heights of the Black Forest mountain range and ultimately empties into the Black Sea. Nineteen countries share the Danube River Basin (DRB): Germany, Austria, the Czech Republic, Slovakia, Hungary, Croatia, Serbia, Slovenia, Bosnia and Hercegovina, Romania, Bulgaria, Moldova, Ukraine, Switzerland, Italy, Poland, Montenegro, Albania and Macedonia.

The Danube, with a multiyear mean discharge of about 6,500 m^3/s, is the second largest river in Europe after the Volga (3,740 km long, multiyear mean discharge 8,500 m^3/s). It is the 21st longest river in the world and ranks 25th by drainage area (817,000 km^2).

The Danube flows from west to east and its basin extends from central and southern Europe to the Black Sea. The boundaries of the basin are determined by longitude 80° 09’ at the sources of the formative streams Breg and Brigach in the Black Forest and longitude 290° 45’ in the Danube Delta at the Black Sea. The southernmost point of the DRB is at latitude 420° 05’, at the source of the Iskar in the Rila Mountains. The northernmost point is the source of the Morava River at latitude 500° 15’.

The upper catchments of the Danube’s tributaries border on those of the Rhine to the west and southwest, of the Weser, Labe, Oder and Visla rivers to the north, and of the Dniester to the northeast, and the catchments of the rivers that empty into the Adriatic Sea and Aegean Sea to the south.

Based on the geologic framework and geographic configuration, the DRB can be divided into three regions, namely the upper, middle and lower Danube basins.

The Upper Danube Basin is the region from the source in the Black Forest to the Devin Gate east of Vienna.

The Middle Danube Basin is a magnificent and unique geographic unit. It extends from the Devin Gate to the mighty fault between the southern Carpathian Mountains and the Balkan Mountains near the Iron Gate Gorge. The Middle Danube Basin is the largest of the three regions.

The Lower Danube Basin comprises the Romanian and Bulgarian lowlands, the catchments of the Siret and Prut rivers, and the surrounding plateaus and mountains. It is bounded by the Carpathians to the north, the Bessarabian Plateau in the east, and the Dobrogea and Balkan mountains to the south.

Hydrometeorological information

The Danube River Basin is one of the most flood-prone regions in Europe. As such, there is a strong need for in-depth information on the flood regime so that generalisation is possible based on long-term observation across the basin.

From a floods perspective, the most significant hydrometeorological data on the DRB are mean daily discharges, daily precipitation totals and mean daily air temperatures, which are routinely collected and archived in all the Danube countries. Long time-series are of particular interest, to gain insight into historic floods. Such time-series of discharges are available from 20 gauging stations on the Danube and 77 on its main tributaries. There is a much larger number of weather (precipitation and temperature) stations; the study used those that provided the longest time-series.

Figure 1 shows a map of the DRB which illustrates the main stream and major tributaries of the Danube river basin and the gauging stations with the longest discharge time-series.

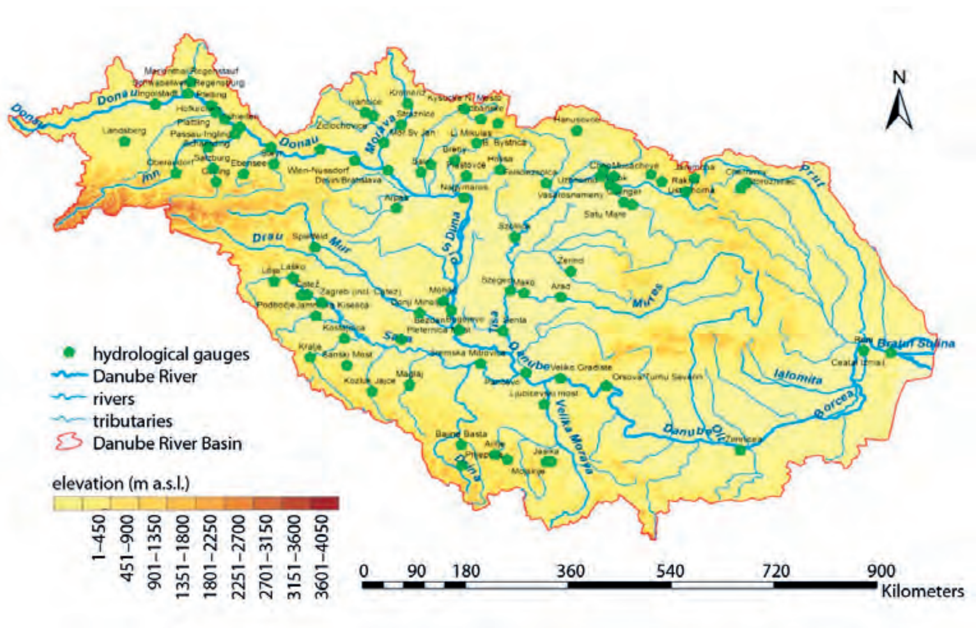


Figure 1. Gauging stations on the Danube and its tributaries [33]

Historic floods

Floods on the Danube have occurred throughout the river's history. Floods belong to the group of extreme natural phenomena. The first records of floods date back to the year 1012.

The Danube flood data can be classified as follows:

- archive data from 1012 to 1501
- registered historic traces (markers) from 1501 to 1820
- time-series recorded since 1821

Many authors have addressed DRB floods in written reports, including [14] [24] [10] [31] [9] [28] [29] [17] [21] [23] [27] [20] [26] [30] [16] [15] [18] [22] [25]. Most of them are related to the upper and middle DRB and archives were the main sources of information.

Historic floods from 1012 to 1501

Flood data are available in diverse historical documents, including original handwritten notes, newspaper articles, chronicles, formal letters, books, maps and photographs. The original handwritten notes usually include short descriptions of the floods, with an indication of peak water levels and cities located along the Upper Danube, where the floods were registered, such as: Passau, Linz, Mauthausen, Grain, Ybbs, Krems or Hainburg an der Donau.

According to [10] and [9], the oldest recorded flood on the Danube dates back to the year 1001. The other major floods in this group, according to historical data, occurred periodically, in several sequences – 1210, 1344, 1402, 1466, 1490, 1499 and 1501. [10] states that the 1501 flood peak was 11,000 m^3/s at Linz and 14,000 m^3/s at Stein Krems. [20] described in her doctoral thesis the floods during this period in the Austrian–Slovak–Hungarian sector. She highlighted the flood events in the summers of 1235, 1316, 1402, 1432 and 1490, shown in Figure 2.

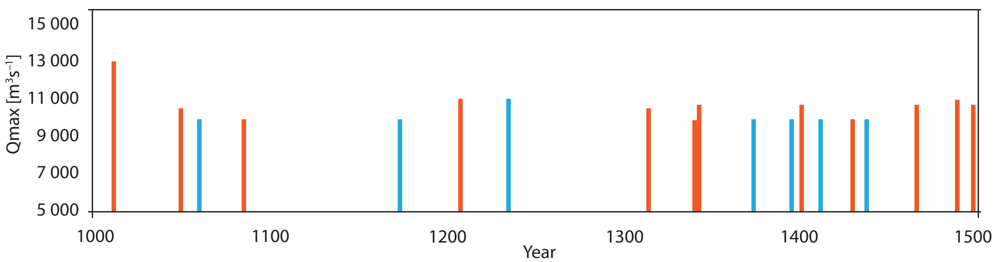


Figure 2. Historic floods on the Upper Danube up to Budapest from 1000 to 1500 AD according to [20]. Red bars – summer floods, blue bars – ice floods. [20]

The general conclusion was that most of the major floods occurred in the 15th century. [9] focused her research on Bratislava. Her published report describes flood occurrences based on a review of archive materials and concludes that many floods in Bratislava were caused by ice and ice jams under bridges. The city of Bratislava sustained considerable damage. In 1426, Sigismund of Luxembourg, king of Hungary, Croatia, Germany and Bohemia, ordered the damaged dikes along the Danube to be repaired. In 1439, he had a bridge built across the Danube in Bratislava, for which breakwaters and pontoons were used. Several floods damaged the bridge in the 15th century. For example, a flood on 20 March 1439 submerged a pontoon and another, on Easter Day of 1443, overtopped the entire bridge.

Matthias Corvinus, king of Hungary and Croatia, ordered a second bridge across the Danube in Bratislava to be built in 1472. The structure was similar to that of the first bridge. This bridge was damaged by a flood in early September 1478, and sustained further damage by an ice flood on New Year's Day in 1485. The bridge was also damaged in July 1485, and ultimately collapsed during the next flood event, on 1 September 1485. According to various chronicles, after the 1485 flood many people migrated from Bratislava to Bavaria. The bridge was damaged again in 1486 by an ice flood, at which time King Matthias Corvinus made considerable efforts to rehabilitate it. Major floods in Bratislava also occurred in 1490 and 1499.

Historic floods from 1501 to 1820

Water marks on old buildings remind us of the Danube's high stages in Germany and Austria. Some of the markers, recorded in cities along the Danube (Vilshofen, Passau, Linz, Mauthausen, Ybbs, Emmersdorf an der Donau, Durnstein, Spilz, Schönbühel, Stein-Krems, Hainburg and Budapest) are shown in [25]. The flood traces can only indicate the possible scale of the flood and serve as a basis for comparison over time. It should be noted, however, that many of the buildings were reconstructed at some point in time, so this information is not very reliable. Other sources of information also need to be used to assess the significance of the recorded floods.

The most significant flood event on the Danube, according to relatively authentic records, occurred between Passau and Bratislava in August 1501. This event has been studied by renowned hydrologists ([10] [28]). The flood wave was estimated to have peaked at 12,000 m^3/s in Linz and 14,000 m^3/s in Vienna. A discharge of 11,000 m^3/s at Ybbs exceeded the summer floods on 25 June 1682 and 31 October 1787, as well as the "rainy" flood on 3 February 1862. That flood caused enormous damage to bridges and swept away many fields and orchards, forcing farmers to migrate to safer areas. Figure 3 is a graphical representation of historic floods on the Danube between Kienstock and Bratislava from 1501 to 1876. Summer and winter floods are depicted separately. The figure also includes maximum annual discharges of the Danube recorded at Bratislava after 1876.

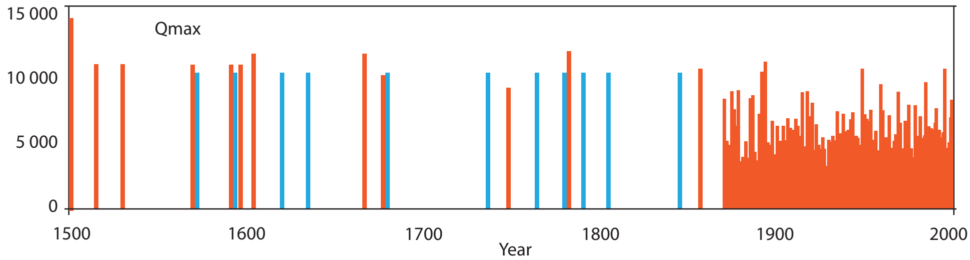


Figure 3. Historic floods on the Danube between Kienstock and Bratislava from 1500 to 1876. Red bars – summer floods, blue bars – winter floods; annual peaks, Qmax, recorded at Bratislava since 1876. [25]

Historic floods from 1821 to 2013

Several major flood events have occurred in the DRB in the past 15 years, particularly along the Upper Danube in August 2002 and June 2013, and the Middle Danube and Lower Danube in March–April of 2006 and 2014.

The highest discharge of the Upper Danube was recorded at Krems–Kienstock (11,900 m^3/s in 2013), followed by 11,306 m^3/s in 2002 and 11,200 m^3/s in 1899. At Bratislava, the highest peak was observed in 1899. Along the Lower Danube, in the Danube Delta, [7] estimated a discharge of 20,940 m^3/s during the July 1897 flood event.

The outcome of assessing a long-term trend in the time-series of maximum annual discharges (Qmax) is a highly questionable endeavour, given the quality of such an assessment of past events. Several disastrous floods occurred on the Upper Danube between 1840 and 1899, as shown in Figure 4.

The trends are indicative of increasing maximum annual discharges of the Upper Danube, upstream from Bratislava. Along the Middle Danube at Orsova – Turnu Severin, the multiyear trend of maximum annual discharges is constant. There are no sufficiently long time-series to assess the multiyear trend along the Lower Danube.

Contrary to the previous period, the years from 1901 to 1953 were relatively calm from a flood risk perspective. However, the period after 1953 was rather turbulent. Disastrous floods did not occur simultaneously on the Upper Danube (from the source to Bratislava), Middle Danube, and Lowr Danube (from the gauge at Orsova to the delta). The largest floods at Hofkirchen were recorded in 1845, 1862, 1882, 1854, 1999 and 2013; and between Passau and Bratislava in 1830, 1862, 1897, 1899, 1954, 2002 and 2013. There were similar occurrences along the Middle Danube in 1838, 1893, 1897, 1938, 1940, 1941, 1954, 1956 and 2006. According to [7], the largest floods on the Lower Danube occurred in 1845, 1853, 1888, 1895, 1897, 1907, 1914, 1919, 1924, 1932, 1940, 1941, 1944, 1947, 1954, 1955, 1956, 1958, 1962, 1965, 1970, 1975, 1980, 1981 and 1988. Some of these floods were caused by ice in winter and spring. The entire DRB experienced floods in 1897, 1965 and 2006. Figure 5 is a longitudinal representation of some of the extreme floods on the Danube.

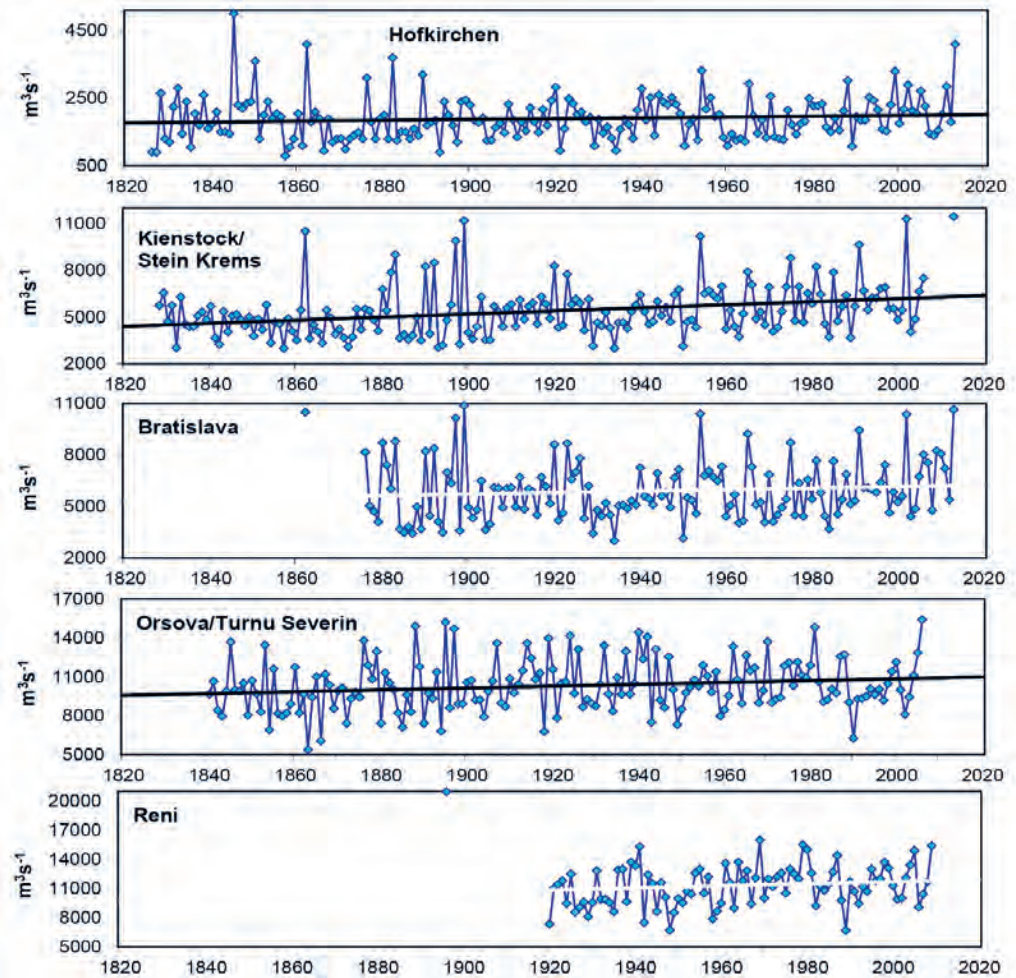


Figure 4. Maximum annual discharges at select stations along the Danube [1]

It is interesting to examine when floods on the Danube occur in a calendar year. Figure 6 shows annual hydrographs at output cross-sections of the gauging stations at Bratislava (Upper Danube), Orsova (Middle Danube) and Ceatal Izmail (Lower Danube). It is apparent that major flood events occur between April and July.

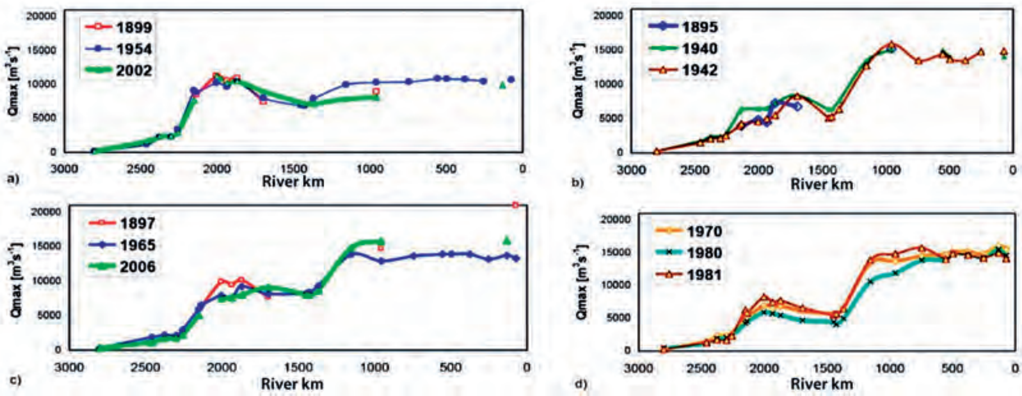
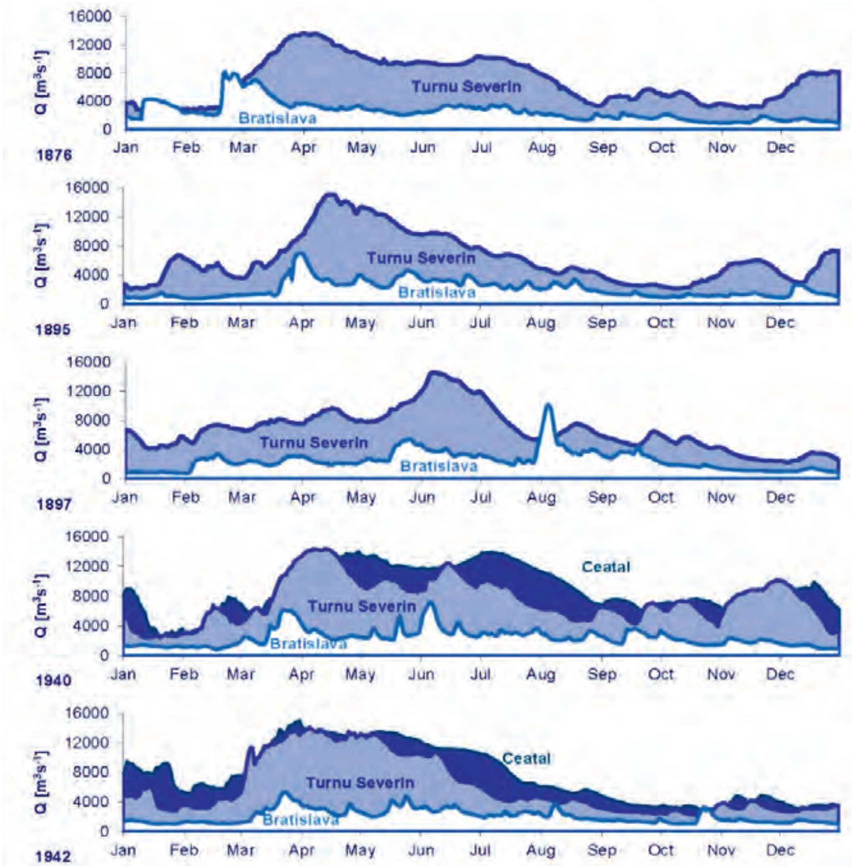


Figure 5. Extreme floods on the Danube [1]



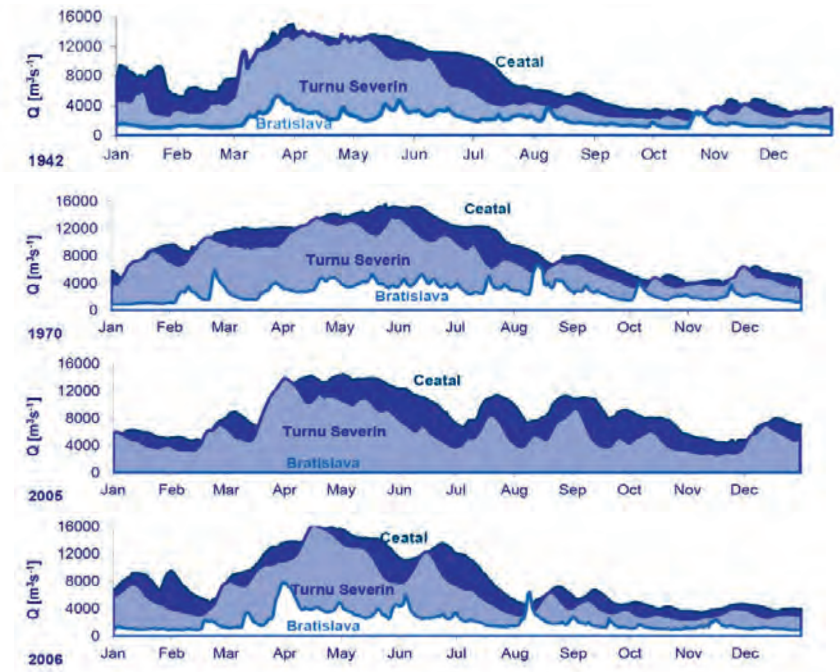


Figure 6. Daily discharges of the Danube at three gauges: Bratislava, Turnu Severin – Orsova and Ceatal Izmail (1940) [1]

Figure 7 is a longitudinal representation of maximum annual discharges along the Danube in the years 1964 and 2006.

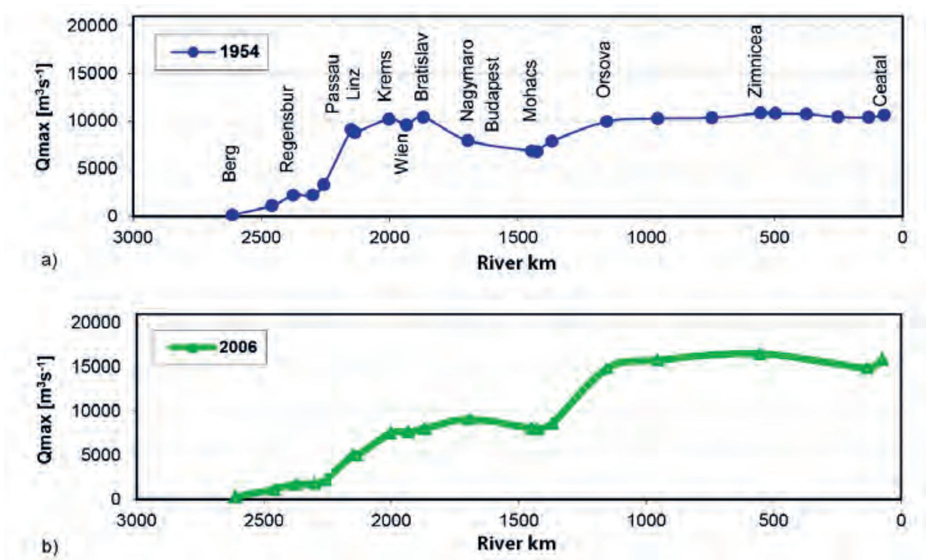


Figure 7. Flood peaks along the Danube in 1954 and 2006 [1]

Flood durations at Bratislava have been 5–10 days. High stages of the Lower Danube generally exceeded 40 days and at times lasted for as many as 200 days, as in the case of the 1965 flood.

The flood wave travel time from Hofkirchen (2,257 km) to Passau (2,226 km) is usually 25 hours, with a mean velocity of 30 km/hour. The travel time from Passau (2,226 km) to Bratislava (1,869 km) was 86 hours in 2002 (velocity 89 km/hour) and 130 hours in 1954 (velocity 66 km/hour). The travel time of the highest flood waves between Bratislava (1,869 km) and Orsova (955 km) has been about 16 days, at an average velocity of approximately 57 km/hour. The flood wave travel times from Passau to Nagymaros are shown in Figure 8.

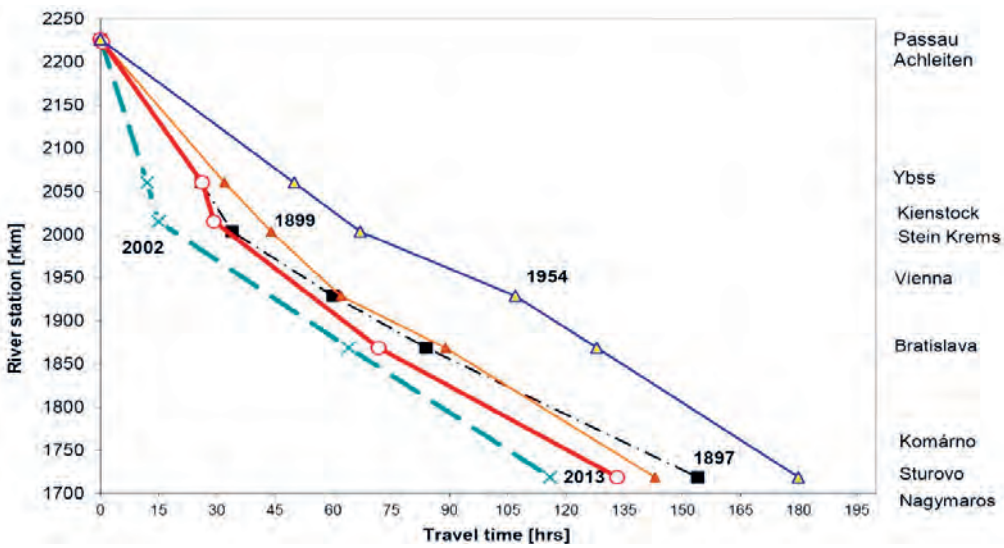


Figure 8. Travel times of the highest flood waves between Passau and Nagymaros [1]

According to [7], the travel times of the highest flood waves between Orsova and the Black Sea have been 15–20 days, at an average velocity of 53 km/hour.

The highest discharge of the Upper Danube (11,306 m^3/s) was registered at Krems–Kienstock in 2002, and the second largest (11,200 m^3/s) in 1899. On the Lower Danube, [7] estimated a discharge of 20,940 m^3/s in July 1897.

Figure 9 is a longitudinal representation of the most probable flood wave peak (50%) and the upper limit of the confidence interval – 99%. The graphic also includes estimated peak points of the highest flood wave on record (1501), at Kienstock, Bratislava and Ceatal Izmail.

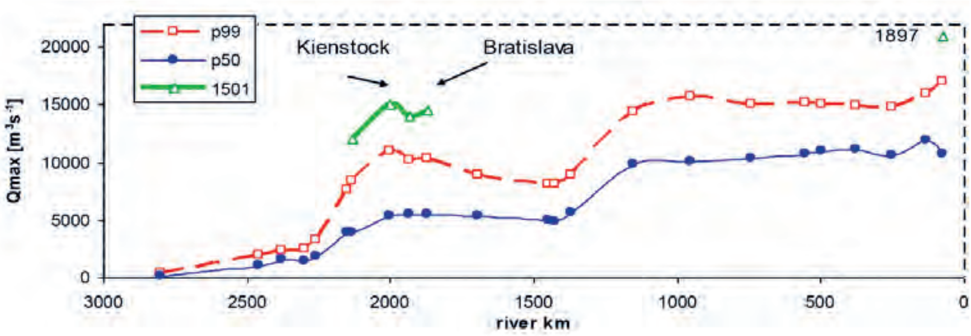


Figure 9. Percentiles of peak annual discharges of the Danube, 1876–2006, p99–99th percentile, p50–50th percentile, and historic flood in 1501 [1]

As an example, Figure 10 shows historic floods on the Vah, a tributary of the Danube, at Liptovsky Mikulas, where the highest peak was recorded in 1813 (1,100 m³/s), which is more than twice the maximum discharge registered from 1921 to 2010 (540 m³/s). The figure also includes the second largest historic peak, recorded in 1894.

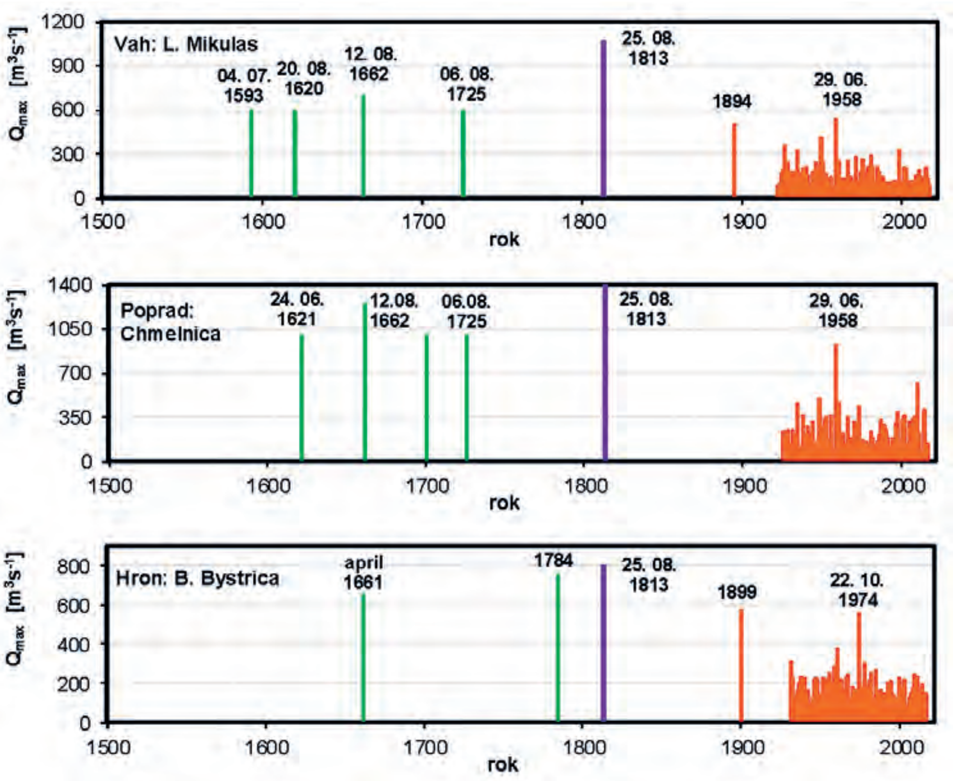


Figure 10. Maximum annual discharges of the Vah River at Liptovsky Mikulas during the observation period (1921–2016) and historic floods [1]

Long-term characteristics of hydrometeorological processes in the Danube River Basin

Long-term characteristics of hydrometeorological processes in the DRB can only be assessed on the basis of available data – precipitation and air temperatures. The longest time-series need to be examined. The objective is to determine the nature of variation in these parameters from year to year, in a multiyear time-series, and identify typical periods characterised by frequent floods or droughts.

With regard to precipitation, the most comprehensive analyses were undertaken for Slovakia, where 10-year averages from 1881–2016, based on 203 precipitation stations, and deviations from average annual precipitation totals were calculated for the entire territory of Slovakia. The results are shown in Figure 11, where a) identifies the wet and dry periods, and b) the 10-year average precipitation totals in Slovakia.

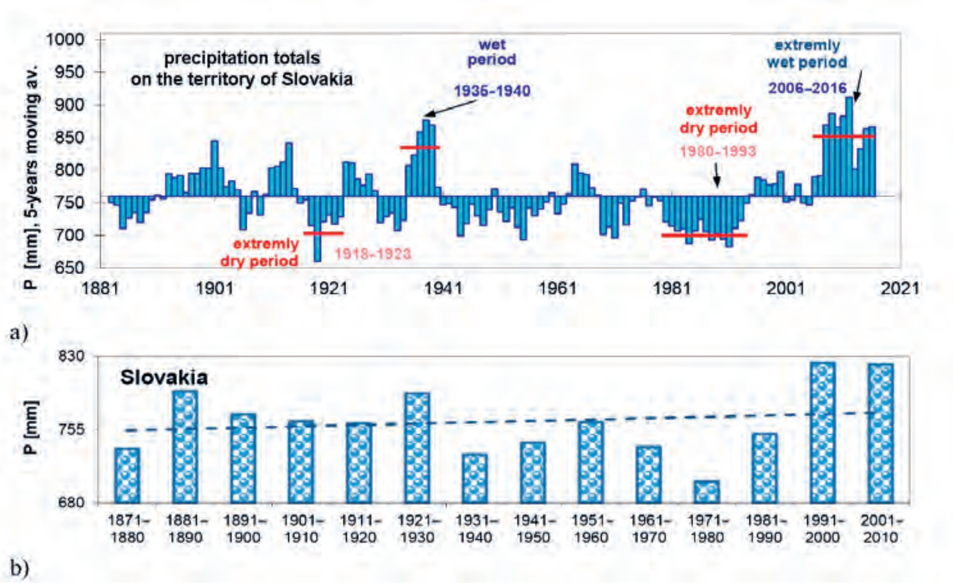


Figure 11. Moving averages of mean annual precipitation totals from 203 stations in Slovakia from 1881 to 2016, with 10-year averages [1]

It is apparent in the graphic that the periods from 1918 to 1923 and from 1980 to 1993 were extremely dry, and that the year 1938 and the period from 2006 to 2016 were wet. For example, in Slovakia, after 14 dry years (1980–1993) came a period of wet years, which began in 1996 and caused floods every year. These floods resulted in enormous damage to private and state property, and caused loss of life. There were a total of 47 fatalities during a disastrous flood in 1998 on the Mala Svinka River (eastern Slovakia), and another two during the summer floods in Slovakia.

The multiyear nature of the alternating dry and wet years was analysed at neighbouring weather stations – Hurbanovo (1871–2010), Mosonmagyaróvár (1861–2010), Vienna

(1841–2010) and Brno (1803–2010). The calculated 10-year average annual precipitation totals are shown in Figure 12.

The results lead to the conclusion that there have been two dry periods and one long rainy period, from 1871 to 1970 (nearly a hundred years) in this part of the DRB. The dry period from 1831 to 1870 was much drier than the other dry period, from 1971 to 2000. A multiyear trend in the precipitation time-series is the most obvious at Brno. The period from 1803 to 1830 agrees very well with the multiyear precipitation trend in the DRB lowlands. The long-term trend can be approximated by a 4th degree polynomial. Notable dry periods occur every 120–140 years.

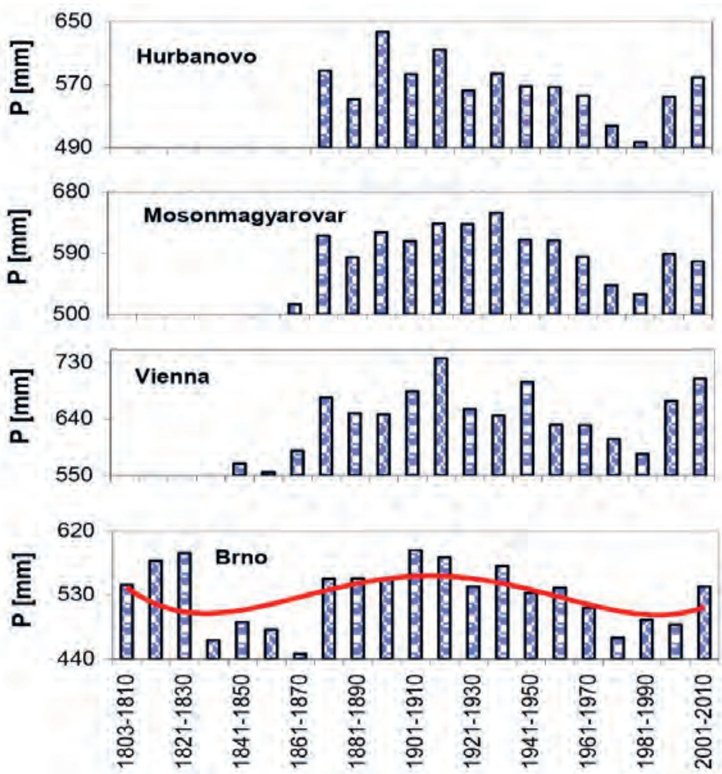


Figure 12. 10-year average precipitation at Hurbanovo (1871–2010), Mosonmagyaróvár (1861–2009), Vienna (1841–2009) and Brno (1803–2010) [1]

The multiyear nature of the air temperature regime in a large part of the DRB was also studied, based on mean annual air temperatures recorded by weather stations at Hohenpeissenberg, Vienna, Bratislava and Budapest. The DRB average annual air temperature ranges from –20 C° to +120 C°. The lowest value was observed at Sonnblick, whereas the highest mean annual temperature was recorded in a part of the Hungarian lowland (Figure 13) and on the Black Sea coast. DRB-wide, July is the warmest month and January the coldest [32].

For comparison, Figure 13 also shows mean annual discharges of the Danube at Orsova. The multiyear nature of air temperature variation is similar to all the considered weather stations. However, the nature of variation of mean annual discharges of the Danube differs, which means that there is no strong correlation between the Danube's discharges and air temperatures. The same applies to precipitation (Figure 11).

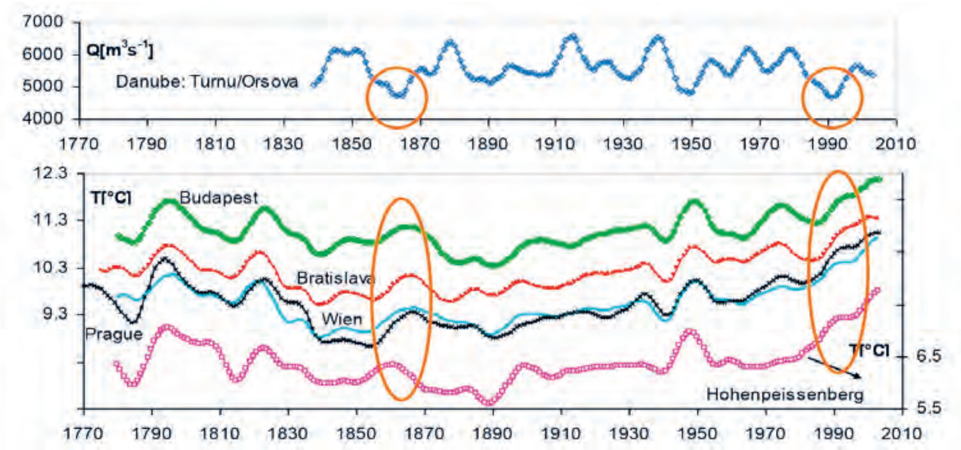


Figure 13. Filtered mean annual discharges of the Danube at Orsova and annual air temperatures, HP-filter lambda.50, Budapest, Bratislava, Prague, Klementinum, Vienna and Hohenpeissenberg stations, 1780–2004 [1]

The multiyear nature of discharge variation along the Danube is shown in Figure 14, via time-series of mean annual discharges and their 5-year moving averages. The 5-year moving averages were calculated for the selected gauging stations: Wasserburg on the Inn and Bratislava, Orsova and Reni on the Danube. The results are shown in Figure 13. They indicate that the nature of temperature variation is similar, from the mouth of the Inn through to the Danube Delta.

Data supplied by the gauging station at Bratislava from 1871 to 2006 was used for a detailed analysis of the nature of discharge variation of the Danube from year to year. Long-term 30-year discharges were examined in particular (Figure 15), as were the 7-year moving averages and long-term 10-year discharges (Figure 16).

The graphics lead to the conclusion that the wettest 30-year period was from 1916 to 1945, followed by 1886–1915 and 1976–2005. The driest 30-year period was 1946–1975. Analogous results for 10-year periods are: wettest 1911–1920, followed by 1961–1970 and 1891–1900.

All the statistical tests indicated that the time-series of the Danube's annual discharges were homogeneous. The trend analyses, whose results for select gauging stations (at Hofkirchen, Achleiten, Vienna, Bratislava, Orsova and Reni) are shown in Figure 17, indicated no statistical significance of the trends (equations also shown in the figure) along the entire course of the Danube.

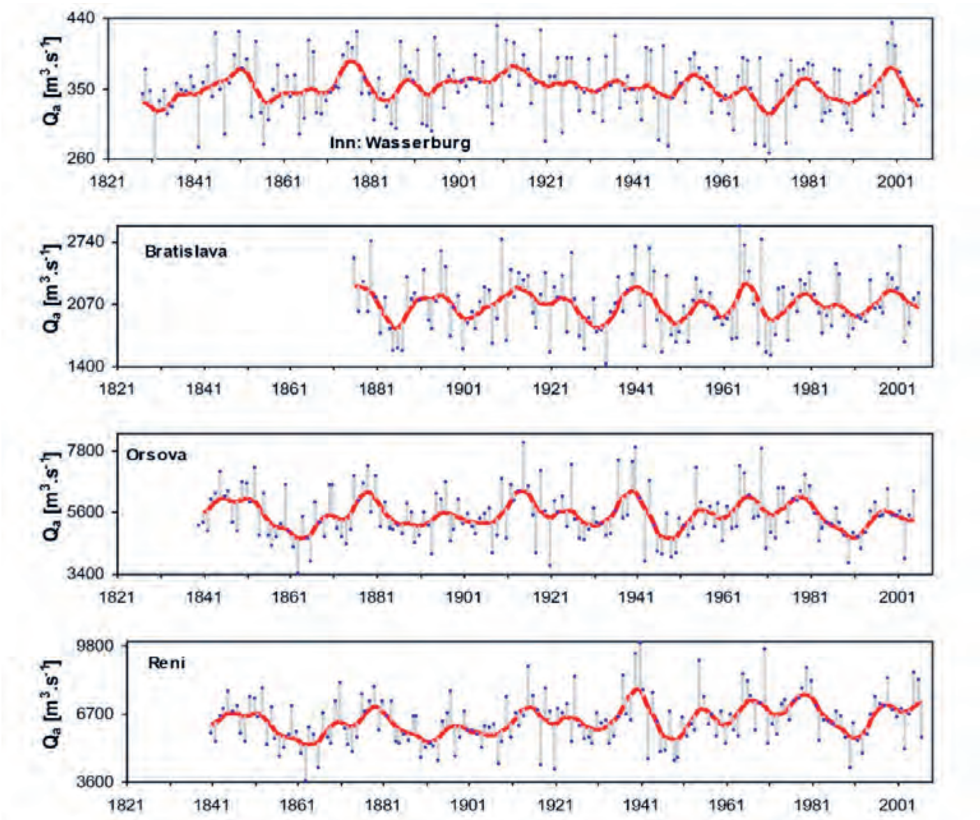


Figure 14. Average annual discharges of the Danube at selected points, deviation double 5-year moving average (hold time) [1]

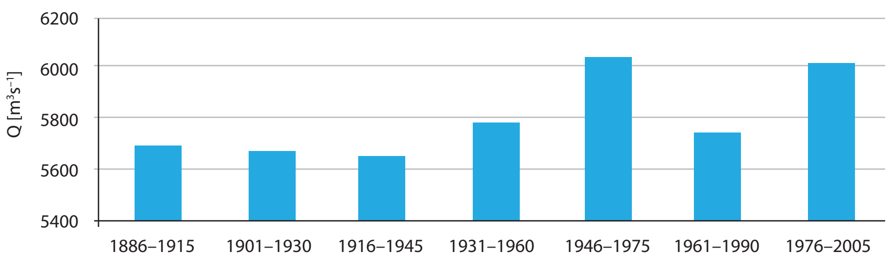


Figure 15. Long-term 30-year annual discharges of the Danube at Bratislava station [1]

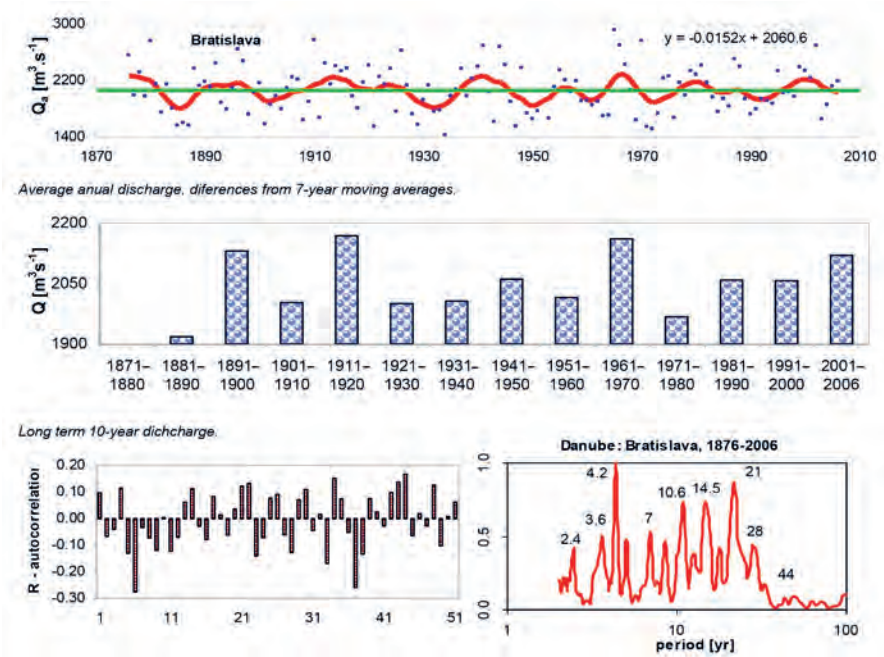


Figure 16. Annual discharges – differences from 7-year moving averages and long-term 10-year discharges of the Danube River at Bratislava station [1]

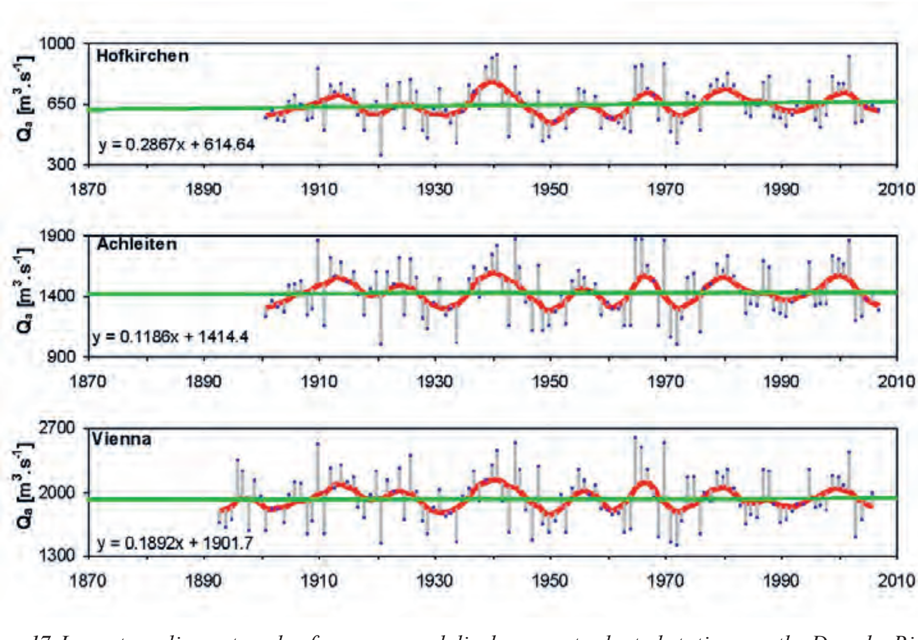


Figure 17. Long-term linear trends of mean annual discharges at selected stations on the Danube River [1]

A detailed analysis of the cyclical nature of the time-series of mean annual discharges of the Danube was conducted via calculations of the main stochastic characteristics: autocorrelation and spectral functions. The results for the gauging stations at Achleiten, Bratislava, Turnu Severin and Reni are shown in Figure 18. The computed autocorrelation functions suggest a multiyear cyclical and congruent nature of discharge formation on the Danube and its major tributaries.

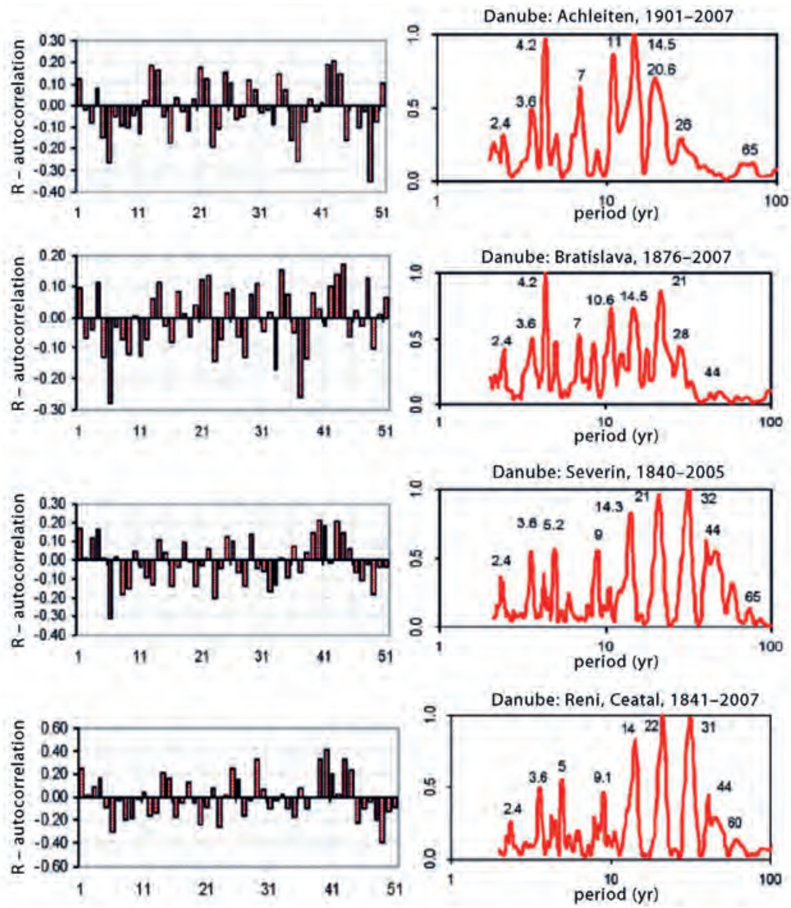


Figure 18. Autocorrelations (left column) and normalised periodograms (right column) of mean annual discharges of the Danube River, significant periods [1]

The spectral function provided a more detailed insight into the variation of cyclical periods along the Danube. The spectral function graphics identify the most prevalent periods in the time-series of mean annual discharges. It is apparent that the most common micro periods are 2.4, 3.6, 4.2 and 7 years and that the macro periods are 14, 22, 30 and 44.

According to [12], the 2.4-year period (cycle) is likely associated with the cyclic nature of the Quasi Biennial Oscillation (QBO) phenomenon. The cycle of about 3.6 years probably depends on the Southern Oscillation (SO), represented by the SO index. The 44, 22 and 11-year cycles are connected with solar activity. The cycle length of approximately 28–31 years is related to the Arctic Oscillation (AO), expressed by the AO index. Finally, the cycle of about 13 years is associated with the North Atlantic Oscillation (NAO), represented by the NAO index.

The analysis of the Danube's extreme annual discharge time-series indicates that the 1931–2005 time series is not representative, even though commonly used in the Danube countries. The last two decades of the 19th century abounded in disastrous floods across the DRB. Along the Upper Danube, there were only a few floods between 1900 and 1953. Since 1954, the flood variability has been higher and similar to the period 1876–1899. Consequently, the long-term trends tested for the period 1876–2005 at five gauging stations on the Danube, whose results are shown in Figure 19, indicate that there is no significant linear trend.

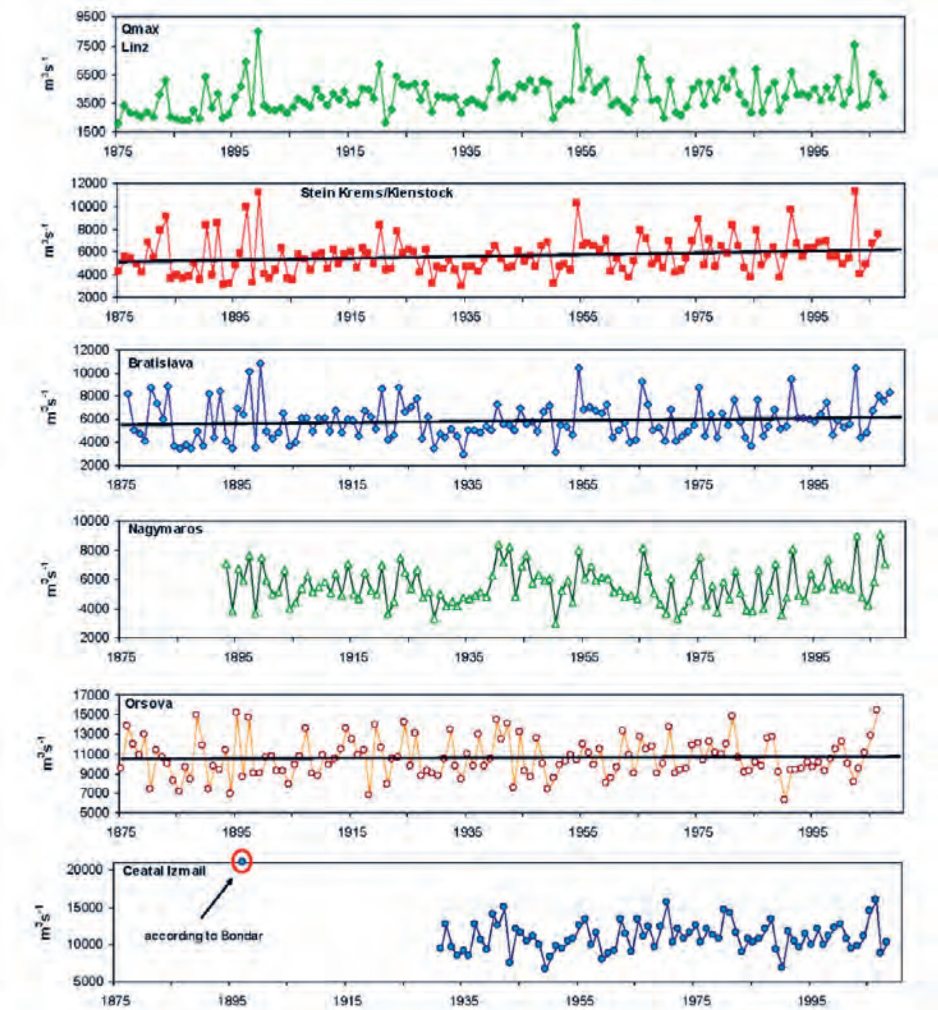


Figure 19. Long-term linear trends of maximum discharges at selected stations on the Danube River [1]

In general, the periods around the years 1915, 1940, 1965 and 1980 in the Danube River Basin were extremely rich in runoff. Contrarily, the period around 1947 was extremely dry and the period around 1863 even drier.

Modern approach to the assessment of statistical significance of historic floods

The statistical significance of historic floods is assessed in two ways, depending on the complexity of the river system:

- simple river systems with no significant impact of tributaries
- complex river systems, within significant impact of tributaries

In both cases the statistical significance of floods is assessed based on the theory of probability, assuming that floods on the main river (recipient) and a tributary are random events that adhere to the law of probability of one-dimensional and/or two-dimensional random variables.

Simple river systems

Simple river systems are those river sectors that are not affected by tributaries in the event of floods. The statistical significance is assessed using time-series of the basic flood hydrograph parameters, such as the peak hydrograph ordinate – Q_{max} and flood wave volume – W_{max}. These two parameters are assumed to be random quantities that adhere to a probability distribution theory. For example, the probability distribution function is calculated for Q_{max}, based on a multiyear time-series:

$$F(Q) = P(Q_{max} \geq q) = p$$

where p is the probability of occurrence of Q_{max}.

The probability of occurrence of a historic flood, Q_{hist}, is derived inversely – $p(Q_{hist})$ or its return period in years $T = \frac{1}{p(Q)}$, which will be described in more detail in Section *Flood Frequency Analysis.*

Complex river systems

Complex river systems are river reaches (sectors) where there is mutual influence of the recipient and a tributary in the event of a flood (flooding in the extended confluence area). There are several such sectors in the DRB, including the mouths of the Inn, Morava (the Czech Republic), Drava, Tisa, Sava, Velika Morava (Serbia) and Prut. Figure 20 is a schematic representation of such a confluence.

The symbols in Figure 20 are as follows:

QIN – discharge at the input cross-section in the zone of mutual influence of the Danube and a major tributary

QOUT – discharge at the output cross-section in the zone of mutual influence of the Danube and the tributary

qTR – discharge at the input cross-section of the tributary

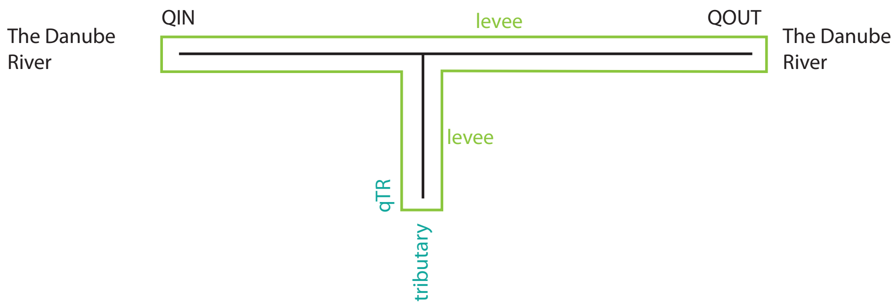


Figure 20. Schematic representation of the zone of mutual influence of the Danube and a major tributary (compiled by the author)

The extended area of the confluence of the Morava River and the Danube is used below as an example to illustrate the procedure for assessing the statistical significance of floods in complex river systems. In the specific case, the input cross-sections are gauging stations: QIN – Vienna on the Danube and qTR – Moravsky Jan on the Morava, and the output cross-section is QOUT – Bratislava on the Danube. The considered parameters are maximum annual discharges (Q_{max}) on these three locations.

The theoretical discharges of different probabilities of occurrence at the three stations were obtained by the conventional statistical-probabilistic approach:

$Q^{W}_{max,p}$ – theoretical maximum annual discharge of the Danube at Vienna, of probability of occurrence p

$Q^{B}_{max,p}$ – theoretical maximum annual discharge of the Danube at Bratislava, of probability of occurrence p

$Q^{MJ}_{max,p}$ – theoretical maximum annual discharge of the Morava at Moravsky Jan, of probability of occurrence p

The resulting probabilities (p) of maximum annual discharges at the three stations are shown in Table 1.

Table 1. Theoretical maximum annual discharges of the Danube and the Morava at different probabilities of occurrence – $Q_{max,p}$ *(m^3/s) (compiled by the author)*

p (%)		Danube	Morava
	$Q^{W}_{max,p}$	$Q^{W}_{max,p}$	$Q^{MJ}_{max,p}$
0.1	12,922	13,760	2,170
1.0	10,309	10,906	1,541
2.0	9,519	10,042	1,362
5.0	8,463	8,890	1,131

In case of a flood in a complex river system, such as the confluence of the Morava and the Danube, the coincidence (simultaneous occurrence) of flood waves on both the recipient and the tributary is very important. A bivariate distribution of concurrent flood waves on the recipient and the tributary needs to be defined (i.e. the coincidence calculated).

To assess the statistical significance of a historic flood in the specific case, first the coincidences of all combinations of maximum annual discharge Qmax,p and corresponding (simultaneous) discharges QCOR,P of the recipient and tributary need to be defined for various probabilities of occurrence. In other words, a set of six coincidences [1] is identified.

1. $P\,[(OUT_{max} > qOUT_{max}) \cap (QTR_{cor1} > qTR_{cor1})] = p$ and $f\,(QOUT_{max}, QTR_{cor1}) = p$
2. $P\,[(QTR_{max} > qTR_{max}) \cap (QOUT_{cor2} > qOUT_{cor2})] = p$ and $f\,(QTR_{max}, QOUT_{cor2}) = p$
3. $P\,[(OIN_{max} > qIN_{max}) \cap (QTR_{cor1} > qTR_{cor1})] = p$ and $f\,(QIN_{max}, QTR_{cor1}) = p$
4. $P\,[(QTR_{max} > qTR_{max}) \cap (QIN_{cor2} > qIN_{cor2})] = p$ and $f\,(QTR_{max}, QIN_{cor2}) = p$
5. $P\,[(OUT_{max} > qOUT_{max}) \cap (QIN_{cor1} > qIN_{cor1})] = p$ and $f\,(QOUT_{max}, QIN_{cor1}) = p$
6. $P\,[(QIN_{max} > qIN_{max}) \cap (QOUT_{cor2} > qOUT_{cor2})] = p$ and $f\,(QIN_{max}, QOUT_{cor2}) = p$

where:

p – probability of occurrence.

Table 2 shows the coincidence calculation results for the extended zone of the confluence of the Morava and the Danube.

Table 2. Design discharges of different flood coincidence probabilities of the Danube and the Morava (compiled by the author)

p (%)	GS at Vienna			GS at Bratislava			GS at Moravsky Jan		
	$Q^{W}_{max,p}$	$Q^{B}_{cor1,p}$	$Q^{MJ}_{cor1,p}$	$Q^{B}_{max,p}$	$Q^{W}_{cor1,p}$	$Q^{MJ}_{cor2,p}$	$Q^{MJ}_{max,p}$	$Q^{W}_{cor2,p}$	$Q^{B}_{cor2,p}$
0.1	12,922	6,000	31	13,760	6,500	22	2,170	2,100	2,500
1.0	10,309	5,800	30	10,906	6,100	19	1,541	1,700	1,800
2.0	9,519	5,500	29	10,042	5,700	17.5	1,362	1,550	1,600
5.0	8,463	5,300	28	8,890	5,100	16	1,131	1,250	1,500

In the present case it is of interest to analyse the return periods of exceedance coincidences of the floods in Bratislava in July 1954 and June 2013, or, in other words, to define their statistical significance. Only the significance of recorded simultaneous combinations of discharges on the Danube at Bratislava and the Morava at Moravsky Jan is addressed here, as follows.

Table 3. Recorded simultaneous combinations of discharges on the Danube at Bratislava and the Morava at Moravsky Jan (compiled by the author)

Year	Q Bratislava (m^3/s)	Q Moravsky Jan (m^3/s)
2013	10,640	52.34
1954	10,400	130

The probability of exceedance of the constellation of maximum annual discharges of the Danube at Bratislava and the corresponding discharge of the Morava at Moravsky Jan in 2013 is (Figure 5):

$$P\{(Q^{B}_{max} \geq 10640) \cap (Q^{MJ}_{cor2} \geq 52.34\} = 0.009,$$

or the return period is:

$$T = \frac{1}{P} = \frac{1}{0.009} = 111 \text{ years}$$

The probability of exceedance of the constellation of maximum annual discharges of the Danube at Bratislava and the corresponding discharge of the Morava at Moravsky Jan in 1954 is (Figure 21):

$$P\{(Q^{B}_{max} \geq 10400) \cap (Q^{MJ}_{cor2} \geq 130)\} = 0.005,$$

or the return period is:

$$T = \frac{1}{P} = \frac{1}{0.005} = 200 \text{ years}$$

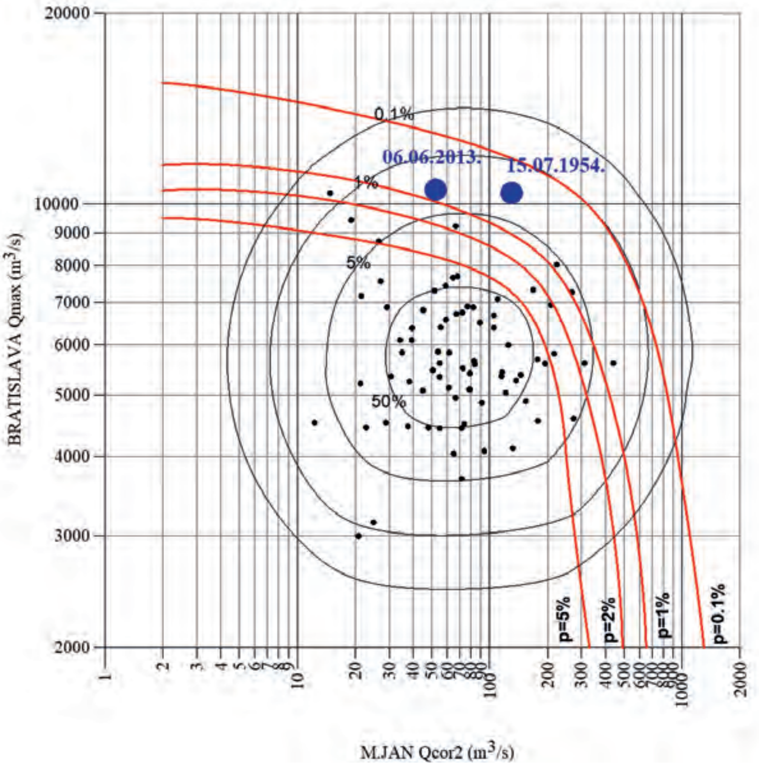


Figure 21. Coincidence of maximum annual discharges of the Danube at Bratislava and corresponding discharges of the Morava at Moravsky Jan, indicating floods on the Danube at Bratislava [1]

Consequently, from the standpoint of statistical significance of simultaneous occurrences of maximum annual discharges of the Danube at Bratislava and the corresponding discharges of the Morava at Moravsky Jan, which are highly relevant to flood protection, the most significant flood waves were registered in July 1954 (200-year event) and June 2013 (100-year event), even though when viewed separately both maximum discharges of the Danube at Bratislava were below the 100-year return period.

Flood marks of historical floods along the Danube river

The analysis of the historical floods occurrence on the upper part of the Danube is in the first part of this paper. It is based on the historical flood marks in Passau, Linz, Mauthausen, Ybbs, Melk, Spitz, Krems, Hainburg, Bratislava, Šturovo and Budapest. The oldest evidence of floods on the Danube goes back to 1012 (see Figure 2). Other floods with severe consequences, as documented in historical annals, occurred in 1051, 1060, 1086, 1173 and 1210.

The occurrence of the Danube medieval floods on its Austrian–Slovak–Hungarian stretch has been described in detail by the dissertation of [20]. As very high floods were denoted those in years 1235, 1316, 1402, 1414, 1432 and 1490. In general, the 15th century is known by a high flood occurrence. From the 15th century, mainly the references about the Bratislava ice floods (ice jams, ice barriers) damaging the bridge are preserved. These floods damaged seriously also the city buildings by ice floes.

Figure 22. Building with the Danube flood marks in Schönbühel (Photo taken by Pavla Pekárová, 2010.)

The water level marks of the highest Danube floods (after 1500) remained on historical buildings in Germany, Austria, Slovakia and Hungary. Such examples are shown on the following photos, taken from [2], for cities located close to the river (Passau, Melk, Emmersdorf an der Donau, Spitz, Schönbühel and Bratislava). These marks make it possible to imagine a real Danube water level elevation, and to compare them each with the others. It should be taken into account that the Danube River channel morphology changed several times in the course of the centuries.

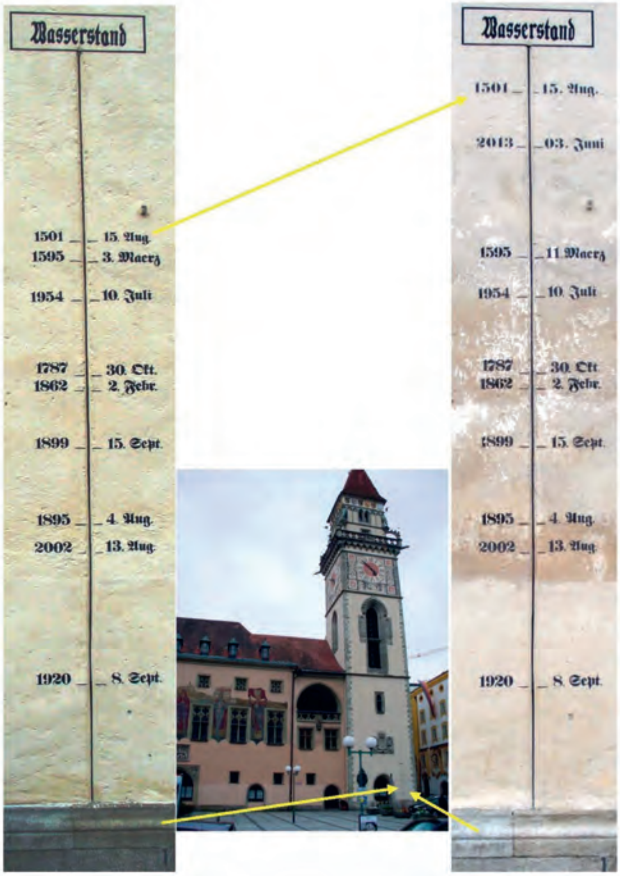


Figure 23. The Danube flood marks, Passau (Photo taken by Pavol Miklánek, left [2010], right [2014]). After the June 2013 flood the mark of 1501 was increased

Figure 24. The Danube flood marks, Melk, detail (Photo taken by Pavol Miklánek and Pavla Pekárová, 2014)

Figure 25. The Danube flood marks, Emmersdorf an der Donau (Photo taken by Alexander Szép, 2014)

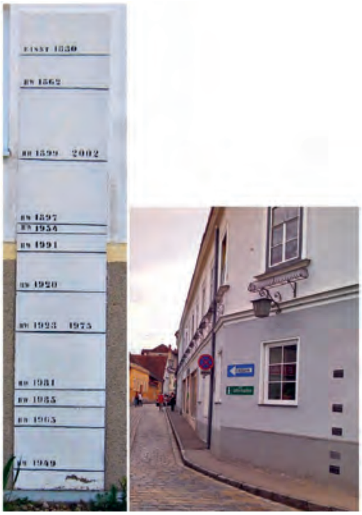


Figure 26. The Danube flood marks, Spitz (Left photo taken from the internet, right photo taken by Pavol Miklánek, 2014)

Figure 27. The frozen Danube at Bratislava in winter 1928–1929 (Archives of the City of Bratislava, Photo Hofer)

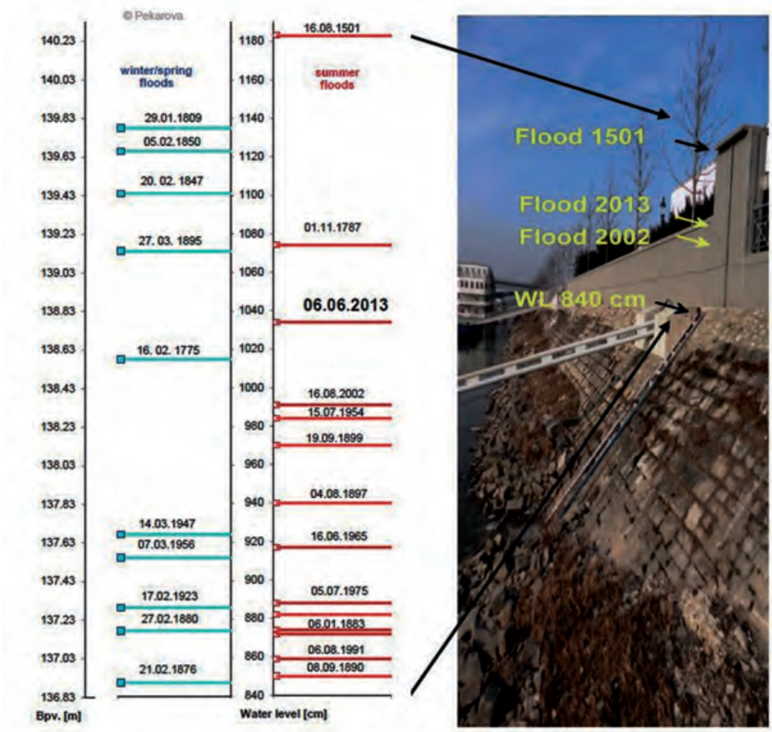


Figure 28. Measured and estimated water levels of significant floods at Bratislava gauge. Left column (blue points) – ice floods, right column (red points) – summer floods (Photo taken by Pavla Pekárová, 2012)

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Lidija Tadić and Tamara Brleković

Flood Frequency Analysis

Hydrological processes are present in time and space in a manner that is partly predictable and partly random. We call them stochastic processes. In that case, the value of certain observations is not correlated with adjacent observations. This type of approach is appropriate for observations/measurements of extreme hydrological events such as floods, droughts or hydrological data averaged over longer time period such as annual precipitation. Statistical methods based on mathematical principles are only tools, which can be used to describe their random variations. These methods are focused on data, not on physical processes [7].

Flood is a natural event which cannot be prevented. It is usually defined as a temporary covering by water of land not normally covered by water. This shall include floods from rivers, mountain torrents, Mediterranean ephemeral water courses, and floods from the sea in coastal areas, and may exclude floods from sewerage systems [5]. It is a stochastic event the probability of which may be derived from a number of different sources. It may be derived directly from historic data on water levels or it may be derived indirectly from modelling. In both cases some form of historic data is needed. If modelling is used, then the historic data can be rainfall or river flow. The length of the available record is important in assessing the magnitude of events with small probabilities. Thus it is important to collect data routinely on both rainfall and river flow.

If a sufficiently long length of record is available, then it is possible to estimate the magnitude of floods with different probabilities directly from a historic record. Such historic data cannot be used to assess the impact of proposed works, so if this is required, then some form of modelling would have to be undertaken.

Flood frequency analysis is a hydrological procedure used to determine high flow values of certain probabilities in successive river cross-sections or hydrological profiles (stations).

Flood frequency estimates of recurrence of floods which is used in designing hydraulic structures such as dams, bridges, culverts, dykes, highways, sewage systems, waterworks, etc. In order to achieve the optimum and safe design of hydraulic structures, and to avoid over designing or under designing, it is necessary to apply statistical methods to determine flood frequency. It is also helpful in flood insurance, physical planning of a certain area or maintenance of the hydraulic structures.

If we have sufficiently long data series of flood flows, therefore, the calculation of empirical frequency distribution could be relatively precise under the assumption that natural and anthropogenic processes did not change relationships relevant for flood occurrence. In that case frequency is equal to determination of maximum measured annual discharge over a longer time period which can be relevant for design of flood protection structures. In most of the stations (rivers), there are not many measured data series which could be reliable for optimum and safe designing. Besides, there is always a chance of occurrence of flood greater than the maximum historical flood [9]. Problems arise when the substantial hydraulic structure in the watercourse has been constructed or any other change in the basin has been introduced which significantly changes the hydrological regime and discharges [8].

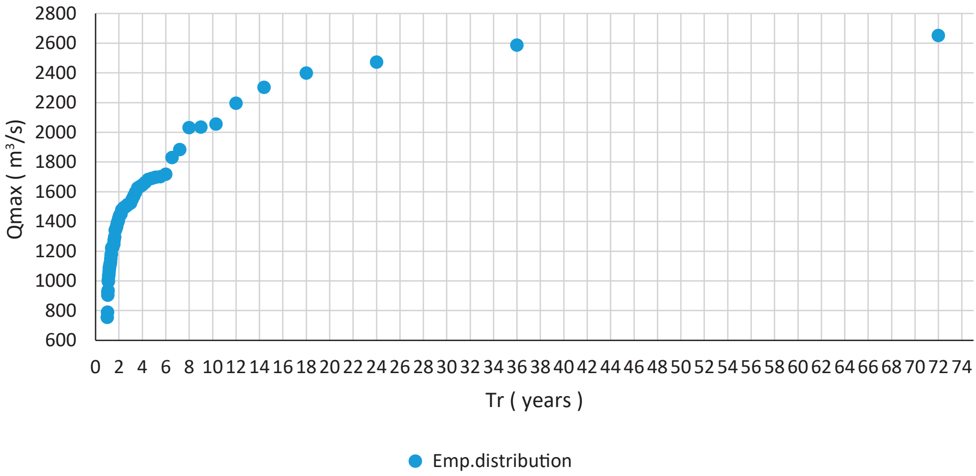

Figure 1. An example of empirical frequency analysis (compiled by the authors)

An example of empirical frequency distribution is presented in Figure 1 calculated on the basis of 71 years long data series of maximum annual discharges recorded on Botovo station (the Drava River in Croatia). In the observed period (1926–1998), the highest discharge occurred in July 1972 (2,652 m^3/s); this is a flood of about 72 years return period.

In order to make flood protection systems safe as much as possible, hydrologists have to use different statistical methods and apply statistical procedures on available data records. Usually only one parameter has been involved in the analysis (water level or discharge) with the following characteristics:

The magnitude of an extreme event is inversely related to the frequency of occurrence. In other words, the most severe floods occur less frequently.

Hydrological data are assumed to be independent and identically distributed [7].

The hydrological regime that produces floods is considered to be stochastic, time and space independent.

The flood frequency curve is used to relate flood discharge values to return periods to provide an estimate of the intensity of a flood event. The discharges are plotted against return periods using either a linear or a logarithmic scale. Generally, the frequency of maximum discharges is more reliable than water levels, because they are less dependent on riverbed deepening or other changes of the watercourse [8].

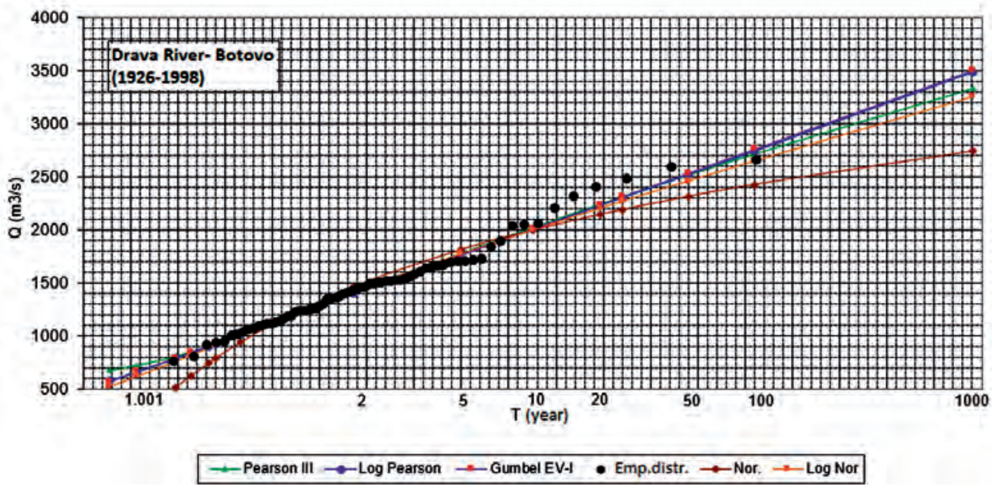

Figure 2. An example of theoretical frequency analysis (compiled by the authors)

An example of application of theoretical frequency distributions is presented in Figure 2. Five of the most common theoretical distributions were tested (Pearson III, Log-Pearson, Gumbel EV-I, Normal and Log-normal) and Log-normal distribution was selected as the most precise. Compared to results presented in Figure 1, the maximum recorded discharge is flood of about 100 years return period. According to presented Log Normal theoretical distribution, flood of 1,000 years return period would have a discharge of 3,245 m^3/s.

As it was illustrated, the result of statistical calculations are floods of different return period. Return period, also referred to as 'recurrence interval' is a term adopted by scientists and policy makers to estimate the likelihood and severity of extreme events (such as cyclones/hurricanes, flooding and earthquakes). It is based on the statistical analysis of data (such as historical climatic records, flood measurements), to provide a probability that an event of any given magnitude will occur in any given year. This probability is often used to assess the risk of these events for human populations. The concept is based on the magnitude-frequency principle, where large magnitude events (such as major cyclones) are comparatively less frequent than smaller magnitude incidents (such as rain showers).

In this approach, which is common in modern flood frequency analysis, it is essential to understand the concept of return period. The theoretical definition of return period is the inverse of the probability (generally expressed in %) that an event will be exceeded in a certain year. For example, the return period of a flood might be 1,000 years, expressed as its probability of occurring it would be 1/1,000, or 0.1% in any year. It means that, in any given year, there is a 0.1% chance that it will happen, regardless of when the last similar event was. Or, it is 10 times less likely to occur than a flood with a return period of 100 years (or a probability of 10%).

The most common misunderstanding about return periods, for example, the 100-year return period is that the flood of this magnitude will only occur once in 100 years. It is essential to understand that if a flood with a 100-year return period occurs now, it does not mean that another flood of this magnitude will not occur in the next 100 years.

EU Flood Directive (Directive/2007/60/EC)

Different countries used to have different approaches to flood frequency analysis as a basis of flood protection measures. The most common return periods used in flood protection are 2, 5, 10, 25, 50, 100, 1,000 and 10,000 years.

Since 2007, members of the EU accepted the common document, Flood Directive [5]. The main reason is the fact that flood risk is best managed on a basin level, not at individual member state level. Without going deeply into the articles of the Flood Directive, its major tasks are determining flood hazard maps of the geographical areas which could be flooded according to the following scenarios:

- floods with a low probability, or extreme event scenarios
- floods with a medium probability (likely return period ≥ 100 years)
- floods with a high probability, where appropriate

For each scenario referred to in the previous paragraph the following elements are important:

- the flood extent
- water depths or water level
- where appropriate, the flow velocity or the relevant water flow

These scenarios are important regarding the problem of return period. According to the Flood Directive, floods of high probability are all flood events with a return period of <100 years. For example, the Danube Flood Risk Atlas (2012) recognised areas along the Danube River affected by floods of a 30 years return period (HQ30). These areas along the river are frequently flooded. Generally flood plains, wetlands, forests and agricultural areas are affected. Usually the inundation areas of a 30-year-flood should serve for retention purposes in order to reduce the overall flood risk and be kept free of settlements buildings. These retention areas are often valuable biotopes, such as in Hungary and the Danube Delta.

The flood event with ≥100 years return period (HQ100) is widely accepted as the design level for flood protection measures along the Danube River. Normally, flood hazard in the areas between the limits HQ30 and HQ100 is known mainly to the residents having lived there for a long time. Agricultural land use is predominant; permission for settlement use should only be given exceptionally and with the provision of preventive construction measures.

During very rare events (HQ1000), flood extents and depths are distinctly larger, respectively higher than what has been observed so far. Existing flood protection works might be overtopped or might fail to perform, thus describing a residual risk scenario. For the areas between a HQ100 and HQ1000 flood, no direct restrictions of land use arise; however, preventive flood strategies and emergency planning should be accounted for, especially regarding vulnerable objects. As potential preventive measures (such as evacuation plans) are highly dependent on flood depth, not only the limits of flooded areas, but also flood depth classes are illustrated.

According to the Flood Directive, the first scenario is defined as: determining floods with a low probability, or extreme event scenarios are given as a framework, which means that each country can make its own choice within it. The choice usually depends on previously established reference values.

Illustrations of hazard maps of the geographical areas which could be flooded according to chosen return periods are given in Figure 3.

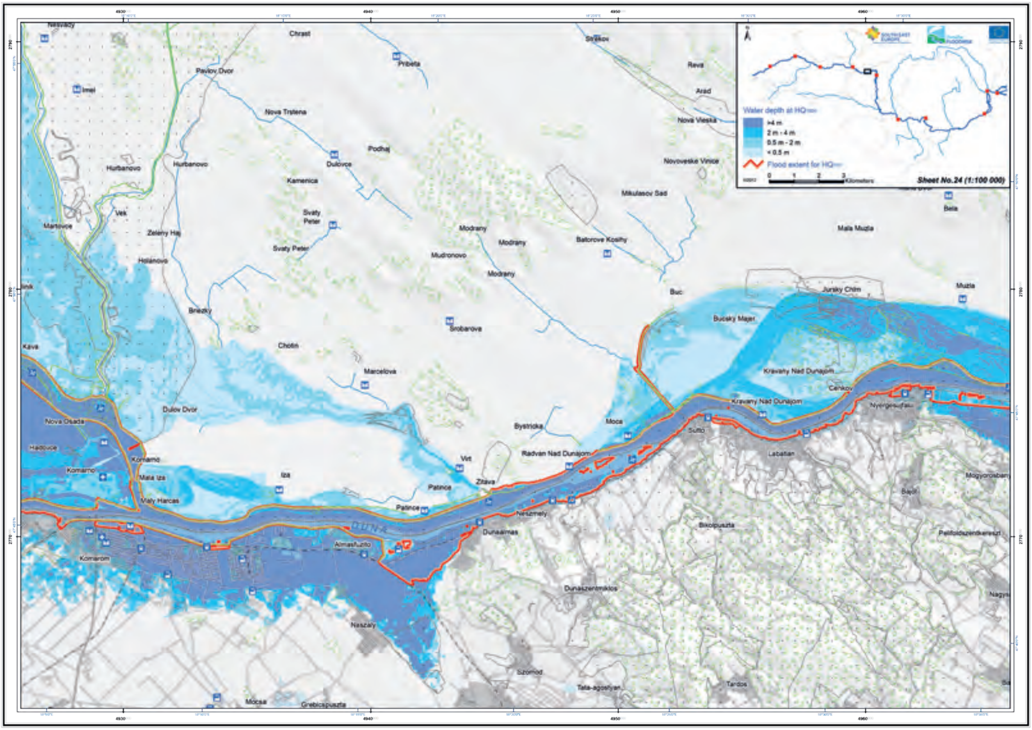

Figure 3. An example of flood hazard map at HQ100 and HQ1000 for one section of the Danube River [4]

Figure 3 presents one section of the Danube River and possible water depth related to discharges of 100 and 1,000 years return period (HQ100, HQ1000). There are dykes along both riversides and their height keeps water in the river flood plain until discharge exceeds HQ100 (red line), or flood with a medium probability (likely return period ≥100 years). Discharges of low probability (HQ1000) will cause floods of an adjacent area with water depth 0.5–4 m, depending on topographical conditions.

Figure 4 presents the relationship between the return period and related flooded areas in the Danube countries. It is clear that floods of low probability (extreme flood scenarios) have the greatest affected area according to the Danube Flood Risk Atlas.

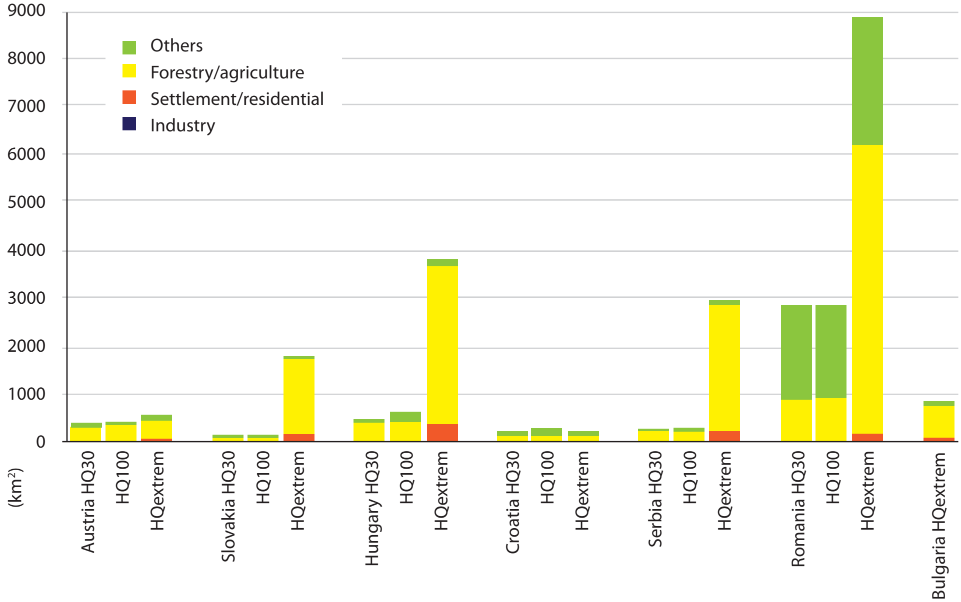

Figure 4. An example of the relationship between the flooded area and return period [4]

Empirical flood frequency

Flood frequency analyses are used to predict design floods for sites along a river. As it was illustrated in Figure 2, in order to estimate the return period of a given discharge or vice versa, the observed data is fitted with a theoretical distribution using a cumulative density function (CDF). This helps the users in analysing the flood frequency curve.

Using the annual peak flow data available for a number of years, flood frequency analysis is used to calculate statistical information such as mean, standard deviation and skewness which is further used to create frequency distribution graphs.

The mostly used frequency distributions in hydrology of extreme events are: Gumbel distribution (in the United Kingdom), Normal distribution, Log-normal distribution and Log-Pearson III distribution (in the USA). After choosing the probability distribution that best fits the annual maxima data, flood frequency curves are plotted. These graphs are then used to estimate the corresponding design flow values.

Procedure:

– using the observed annual maximum discharges of a period as long as it is possible to calculate basic statistical information such as mean values, standard deviations, skewness, etc.

- calculation of recurrence intervals (Figure 1) by using empirical equations such as Weibull equation which is one of mostly used empirical distribution and according to some authors it is the most accurate [6]

It is not the only one, there are a number of empirical distributions as it is presented in Table 1.

Table 1. Several methods of empirical distribution [6]

Method of "RI"	Proponent
m/N + 1	Weibull (1939)
(m-0.31)/(N + 0.38)	Beard (1943)
(m-0.44)/(N + 0.12)	Gringorten (1963)
(m-0.5)/N	Hazen (1914)
(m-0.3)/N + 0.4	Čegodajev (?)

Data records of maximum annual discharges are sorted in descending order and each annual peak have a certain rank, called the magnitude number, "m" (the highest value is ranked as m = 1, and the smallest value is valued as N). The number of items (data points) in the record is "N". The recurrence interval (RI) for a particular river profile (station) gives us information, how often we expect the river to exceed a certain discharge.

After the calculation of basic statistical parameters it is necessary to calculate the maximum annual discharge for different return periods by applying distributions.

Each distribution has its own characteristics and mathematical basis.

Normal (Gauss) distribution

In spite of the fact that Normal (Gauss) distribution is a symmetric two-parameters distribution and flood waves ordinarily have non-symmetric distribution, this distribution is very often used in flood frequency analysis.

The analytical expression is given in the form of frequency density function:

$$p_{(Q_M)} = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(Q_M - \overline{Q_M})^2}{2\sigma^2}} \qquad (1)$$

p(x) = normal distribution density function

s2 = standard deviation (variance) of the distribution

$\overline{Q_M}$ = mean value of the distribution

Introducing of transformation

$$z = \frac{Q_M - \overline{Q_M}}{\sigma} \qquad (2)$$

gives an equation of standard normal distribution:

$$p_{(x)} = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{z^2}{2}} \tag{3}$$

If the general variable x designates discharge Q, flood discharge QMp of different return periods (p) can be calculated as:

$$Q_{Mp} = \overline{Q_M} + z\sigma \tag{4}$$

Figure 5 illustrates a standardised normal distribution and its properties. It is clear that the density function is symmetric about the mean value $(\bar{x})$ and the function mode coincides with the mean value. The variance of standardised normal distribution s2 = 1 and the mean value $\bar{x} = 0$. The maximum value of density function is:

$$p(y) = \frac{1}{\sigma\sqrt{2\pi}} = 0.399 \tag{5}$$

$$\sigma = \sqrt{1}$$

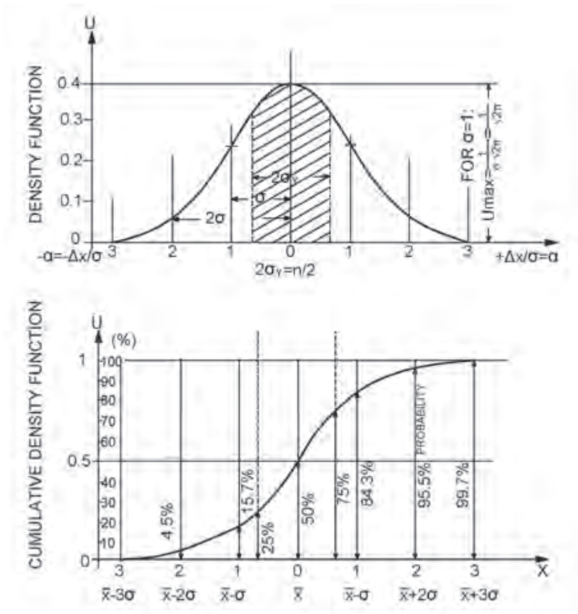

Figure 5. Cumulative density function [8]

The area below the curve presented in Figure 5 equals 1 and represents the number of values N. The total area divided in 2 parts defines 50% of the sample group in the interval:

$$[\bar{x} - 0.6745s, \bar{x} + 0.6745s] \tag{6}$$

The maximum deviation in this case is $s_{max} = 3s$.

Log-normal (Galton) distribution

If some data from the given data series are expressed as log values, in that case Normal distribution changes into Log-normal or Galton frequency distribution.

$$p_{(x)} = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}\frac{(\bar{q}-\bar{q}_o)^2}{\sigma^2}} \tag{7}$$

$$\bar{q} = \log Q_M \tag{8}$$

$$q_o = \frac{\sum \bar{q}}{n} = \frac{1}{2}\log\left(\frac{\overline{Q_M}^4}{Q_M^{\,2} + \sigma^2}\right) \tag{9}$$

$\bar{q}$ = log value of maximum discharge

qo = mean value of logarithms of log QM series

In Galton distribution, the reduced deviation is:

$$z = \frac{\bar{q} - \bar{q}_o}{\bar{\sigma}} \tag{10}$$

And the logarithm value of the maximum discharge (of different return periods) will be:

$$\bar{q}_p = \bar{q}_o + z\bar{\sigma} \tag{11}$$

The anti-logarithm of qp will give us the maximum discharge of the given return period (Annex: Table I).

Gumbel distribution

The Gumbel distribution is non-symmetric and two-parametric. According to the Gumbel probability of maximum (flood) discharge occurrence is defined by exponential function:

$$p(QM) = ae^{-a(Q_M - Q^*)} e^{-a(Q_M - Q^*)} \tag{12}$$

where Q* and a presents parameters of the Gumbel distribution. Q* is a mode of Gumbel's curve:

$$Q* = \overline{Q_M} - \frac{0.577}{a} \tag{13}$$

Number 0.577 is Euler's constant and a is the parameter defined as:

$$\frac{1}{a} = 0.780\sigma \tag{14}$$

As in the previous distribution, the introduction of

$$z = a(Q_M - Q^*) \tag{15}$$

into the equation (10) will give

$$p(QM) = ae^{-z}e^{-e^{-z}} = p(QM) = e^{-e^{-z}} \tag{16}$$

For different return periods, the Gumbel distribution has defined the relationship between p(QM) and z (listed in the Annex: Table II) which can easily lead us to maximum discharges of any return period with solving an equation:

$$Q_{Mp} = Q^* + \frac{1}{a} z \tag{17}$$

Pearson III distribution

The Pearson III distribution is a non-symmetric three-parametric distribution. The original form of this contribution is very complex and using it in practice is time-consuming, so in hydrological problems its modification is more often used proposed by Foster-Ribkin (Annex: Table III) that [8] defined as:

$$Q_{Mp} = (c_v\varphi + 1)\overline{Q_M} \tag{18}$$

$\overline{Q_M}$ = mean value of the distribution

cv = variation coefficient

ϕ = function defined as ϕ = f(cs,p) where cs presents the skew coefficient. Values of ϕ function are listed in Annex: Table IV for different return periods p and skew coefficients cs assuming that the variation coefficient cv = 1.

Testing

The presented frequency distributions will give different maximum discharges for the same return period. An example presented in Figure 2 shows a wide range of Q50 (maximum discharge of 50 years return period), between 2,300 and 2,550 m^3/s.

The decision of the most appropriate, or the most accurate method depends on the result of statistic tests which has to be applied on calculated theoretical distributions to determine if a calculated theoretical distribution matches measured values. There are many statistic tests but, frequently used tests are the Kolmogorov-Smirnov test and the χ^2 test.

Kolmogorov-Smirnov test

The Kolmogorov-Smirnov Goodness-of-Fit Test (K-S test) compares data with a known distribution and lets you know if they have the same distribution. Although the test is nonparametric – it does not assume any particular underlying distribution – it is commonly used as a test for normality to see if your data is normally distributed. More specifically, the test compares a known hypothetical probability distribution (e.g. the normal distribution) to the distribution generated by your data – the empirical distribution function [22].

Measure of tolerance DN is given by equation:

$$D_N = \max\left|\Phi_N(x) - F(x)\right| \quad -\infty < x < +\infty \tag{19}$$

where φ N(x) presents empirical distribution and F(x) is the theoretical distribution.

Predefined confidence level is usually α = 0.05 (5%). Table 2 presents critical values of Do related to the number of data in series (n).

Table 2. Critical values (Do) of K-S test [8]

n	5	10	15	20	25	30	35	40	45	50	>50
Do	0.56	0.41	0.34	0.29	0.27	0.24	0.23	0.21	0.20	0.19	$1.36/n^{1/2}$

If the calculated value of DN<Do theoretical distribution is acceptable. If not, the tested theoretical distribution should be rejected. In a case of testing several distributions, the best one is the one with the smallest DN value.

χ^2 test (Chi-squared test)

The Chi-squared test can also be used to determine how well theoretical distributions, such as normal, binomial, etc. fit empirical distributions, obtained from measured data [10].

In the Chi-squared test, it is necessary to define the nul-hypothesis (Ho), which confirms and the alternative hypothesis (Ha), which rejects the statement. If under hypothesis Ho the computed value of the χ^2 test is greater than some critical value alpha (α), we would reject Ho. Otherwise we would accept it. The critical value is chosen by the researcher. The usual alpha value is 0.05 (5%), but it could also have other levels like 0.01 or 0.10.

In equation:

$$\chi^2 = \sum_j \frac{(o_j - e_j)^2}{e_j} \tag{20}$$

symbols oj and ej are representing respectively observed (measured) data and expected frequency in th j-th cell [10].

Calculation models

In order to perform the flood frequency analysis, the first step is to get the time series of discharges and/or water levels for the hydrological station, which is of interest from water authorities. This type of analysis is usually performed on time series of maximum annual discharges. Then, the record of maximum discharges, which was previously sorted in descending order is fitted with a theoretical distribution using a cumulative density function. This can be done in Excel, but some knowledge of statistic and probability functions is necessary. There are many theoretical distributions integrated within Excel functions. For example, input window for Gamma distribution from Excel is shown in Figure 6.

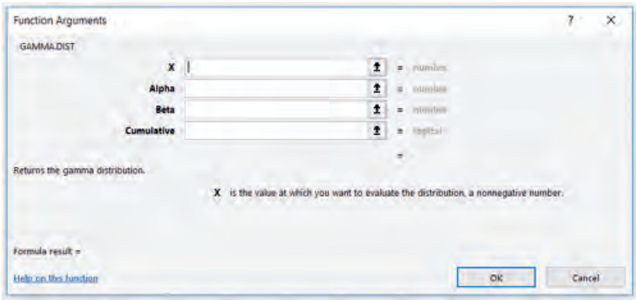

Figure 6. Input parameters for Gamma distribution in Excel (compiled by the authors)

Besides basic functions in Excel, there are many programs or stand-alone applications or add-inns for Excel. One of them is EasyFit. This applications includes goodness-of-fit tests and more than 50 distributions. For each distribution, EasyFit provides several functions to be used in Excel sheets. After the distributions are fitted, EasyFit will display the Fitting Results window (Figure 7) for the distributions comparison and selection of the best model. EasyFit supports all the most popular goodness-of-fit tests, including the Kolmogorov-Smirnov, Anderson-Darling and Chi-squared tests. Once the distributions are fitted, EasyFit displays the goodness-of-fit reports which include the test statistics and critical values calculated for various significance levels.

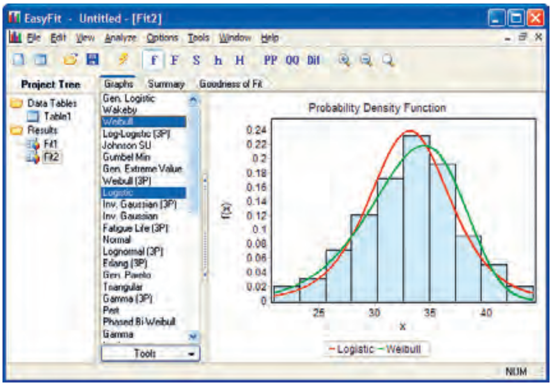

Figure 7. Fitting results window provided by EasyFit (compiled by the authors)

Another software which can be used for distribution fitting is ModelRisk. ModelRisk is a Monte Carlo simulation Excel add-in that allows to include uncertainty in their spreadsheet models. A ModelRisk user replaces uncertain values within their Excel model with special ModelRisk quantitative probability distribution functions that describe the uncertainty about those values. ModelRisk then uses Monte Carlo simulation to automatically generate thousands of possible scenarios. It contains more than 130 probability distributions. The distribution's parameters are estimated using maximum likelihood estimates (MLE) [23].

The fitted distributions are ranked according to the SIC, AIC (Akaike) and HQIC information criteria. For these holds: the lower an information criterion, the better the fit. The advantage of AIC and the other Information Criteria is the fact that they take into account the number of parameters estimated, and penalise for overfitting: a model that has a good fit using fewer parameters is preferred over the one that needs more parameters.[1] The AIC is the least strict of the three in penalising for more parameters, while SIC is the strictest.

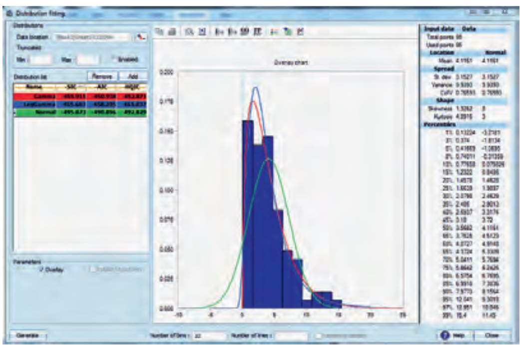

Figure 8. Distribution fit window from ModelRisk (compiled by the authors)

[1] For more information see www.vosesoftware.com/riskwiki/ComparingfittedmodelsusingtheSICHQICorAICinformationcritereon.php

Statgraphics is another software which contains several procedures for manipulating statistical probability distributions. 45 distributions may be plotted, fit to data, and used to calculate critical values or tail areas (Figure 9). Random samples may also be generated from each of the distributions with this stat software. Goodness-of-fit tests used in Statgraphics are ShapiroWilks, the Kolmogorov-Smirnov and Chi-squared tests.

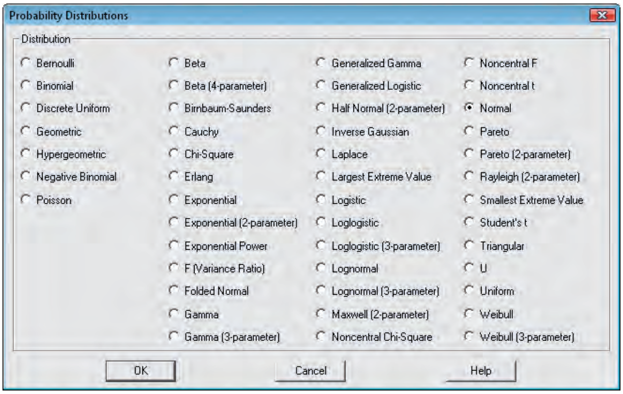

Figure 9. Probability distributions available in Statgraphics (compiled by the author)

These are just some of the software used for distribution fitting. There are also R software, CumFreq, Minitab and others.

When the appropriate function for analysed dataset of maximum annual discharges is obtained, it is necessary to calculate the maximum annual discharge for different return periods. Those values, together with the riverbed and floodplain geometry can be input parameters for hydraulic analysis which can be done in software like HEC-RAS or MIKE. Results of hydraulic analysis can be plotted on maps using GIS software in order to obtain flooded areas according to different return periods, or flood risk maps just like the one shown in Figure 3.

This was done, for example, in the hydrological research of the Kopački rit Nature Park with analysis of its flooding frequency depending on the Danube River water levels and discharges for different return periods [20]. Time series of maximum annual discharges of the Danube River were analysed from the Bezdan station (Serbia) in the period from 1951 to 2008. These data were analysed by several distributions in order to achieve the Danube River discharges and water levels of 5, 10, 25, 50 and 100 return period (Figure 10). As the Log-Pearson III distribution is recommended for flood frequency analysis, it was chosen to be the most appropriate.

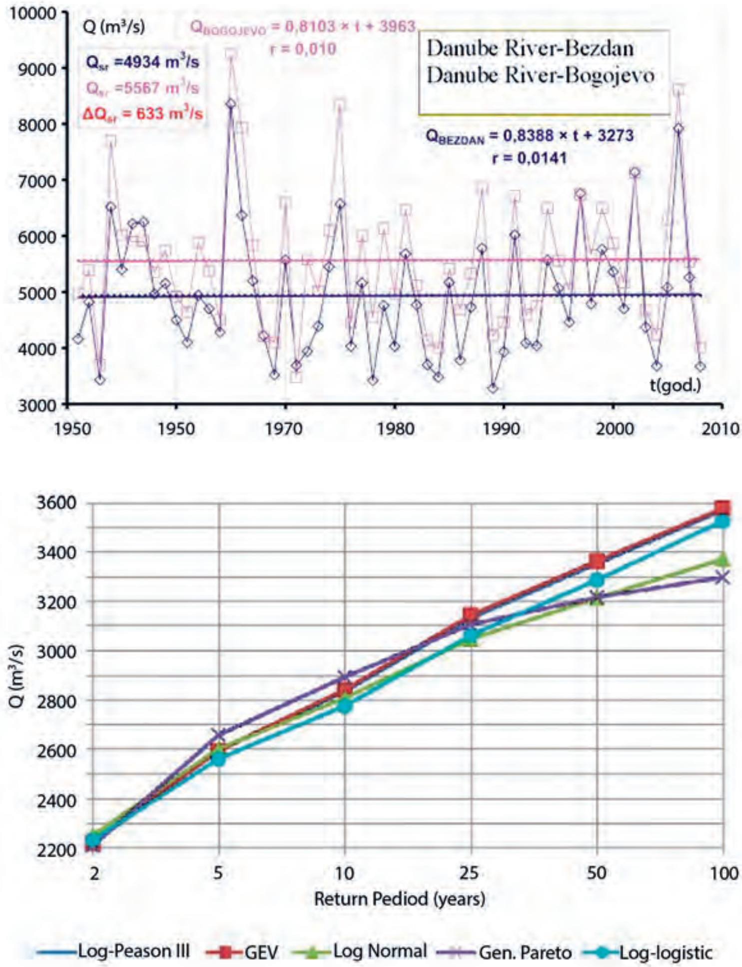

Figure 10. Discharges according to different distributions for different return periods [20]

Hydraulic analysis performed with software HEC-RAS included, besides discharges, water levels at Aljmaš station, discharges of the Drava River, which is the tributary of the Danube River, riverbeds and floodplain geometries and land cover. Obtained results were analysed with GIS in order to see flooded areas according to different return periods. In Figure 11 are shown maps of the flooded area for 5 and 100 years return periods.

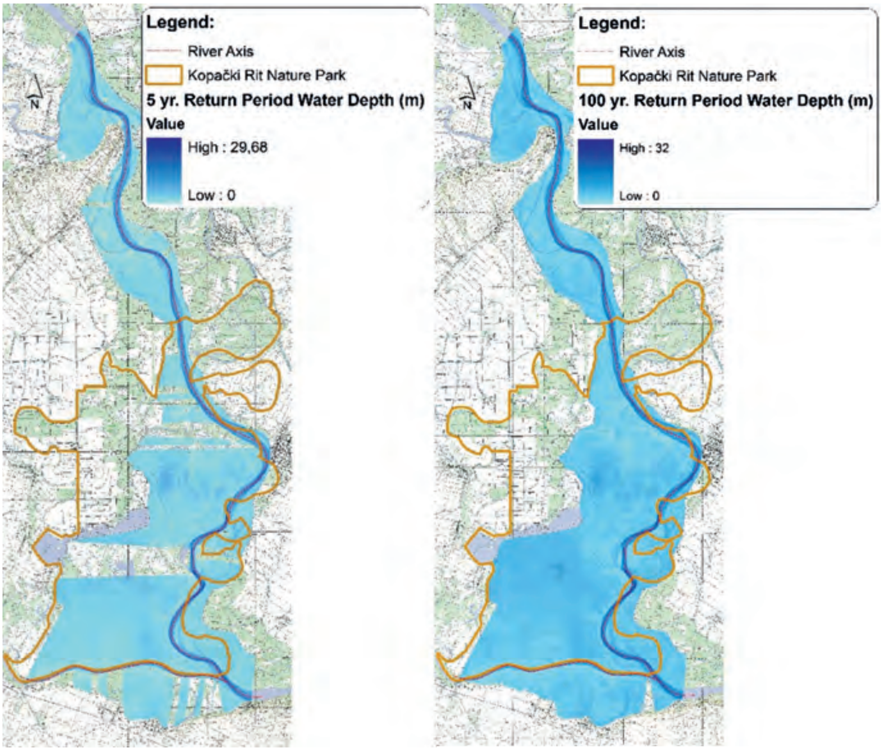

Figure 11. Maps of the flooded area for 5 and 100 years return periods [20]

Flood coincidence

In the previous sections, only the problem of one parameter flood frequency has been analysed, usually the maximum annual water level or discharge assumed to be stochastic. Unfortunately, there are many locations in the world where the flood of one river coincides with the flood of its tributary which enormously increases damages in the given area. In this case, the term "coincidence" presents the occurrence probability of two stochastic events X and Y at the same time (simultaneously), where X presents the event in the main watercourse and Y is the event in its tributary [2].

Only in the Danube River basin there are several potentially critical profiles. In the zone of significant interaction between the mainstream and its tributary, it is recommended to apply flood coincidence methodology which gives a statistically sound analysis concerning an important feature of flood genesis. To judge this, the evaluation of historical data is of great importance. In the case of a complex river system, limited by two inlet profiles (on the mainstream and its tributary) and one outlet profile (on the mainstream), without a significant influence of inflow from the inter-catchment, the relevant combinations of maximum annual discharges and their corresponding (synchronous) discharge values has to be calculated [1].

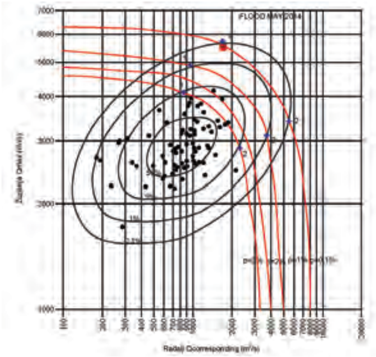

Figure 12. An example of maximum discharges coincidence at Sava and Drina rivers [2]

Figure 12 presents the probabilities of coincidence of two flood waves – on the Sava River (main watercourse) and the Drina River, its tributary. Analysis was done on the basis of historical data. An extreme flood event, occurred in May 2014, was a flood of 1,000 years return period (0.1%).

Statistical calculation is rather complex and it will not be explained in details (see references).

Copulas

The coincidence of two flood waves, of a main river and its tributary, represents a multivariate hydrological event. Another similar problem in flood frequency analysis is to determine the relationship hydrograph – return period. Besides information on flood peak (maximum dischargers), great influence on magnitude of floods have also volume and duration of flood wave. In such cases, the consideration of more than one variable in analyses is reasonable. In order to comprehend and connect these variables, joint cumulative distribution function (cdf) and probability density function (pdf) of involved variables are needed. Because of this, multivariate statistical analyses have to be applied. Some multivariate approaches were introduced in flood frequency analysis during last years, but they all had three limitations [12]:

- all univariate marginal distributions have to belong to the same family, but analysed variables could show different margins
- mathematical formulations become complicated when increasing the number of variables
- it is not possible to distinguish marginal and joint behaviour of studied variables

Copula functions overcome these limitations and present a useful tool in the field of multivariate analyses. The copula actually 'couples' the marginal distributions together to form a joint distribution. In analysis of coincidence two flood waves, distribution of maximum discharges of main river represents one marginal distribution and distribution of maximum discharges of tributary represents another one. The copula connects marginal distributions to one joint distribution and gives the probability of their coincidence.

The advantages in using copulas to model joint distributions are [13]:

- flexibility in choosing marginal distributions
- analysis of more than two variables;
- separate analysis of marginal distribution

When analysing two variables, which is the simplest analysis because of the small number of variables, bivariate copula is used. A bivariate copula C is the joint distribution function of two uniform random variables and can be written as [14] [15] [18]:

$$C: [0.1]2 \rightarrow [0.1] \tag{21}$$

Two following conditions must be fulfilled: C(1,u) = C(u,1) = u and C(u,0) = C(0,u) = 0 and the second one C(u1,u2) + C(v1,v2) – C(u1,u2) – C(v1,v2) ≥ 0 if u1 ≥ v1, u2 ≥ v2 and u1, u2, v1, v2 ϵ [0.1]. The link between copula and the joint distributions is based on the theorem of Sklar:

$$FX,Y(x,y) = C[FX(x), FY(y)] \tag{22}$$

where

FX,Y(x,y) are the joint cumulative distribution function of the random variables, and FX, FY are marginal distribution functions.

There are two groups of copulas: elliptical and the Archimedean family. Elliptical copulas are the copulas with elliptical distributions, which have an elliptical form and therefore symmetry in the tails. Important copulas in this family are the Gaussian and the student's copula. The Gaussian copula is often used because of his simple form. Archimedean copulas are widely applied, because they are not difficult to construct. Archimedean copulas have only one dependency parameter, instead of a dependency matrix. The most important Archimedean copulas are Gumbel, Clayton and Frank [16]. Different types of copulas are shown in Figure 13.

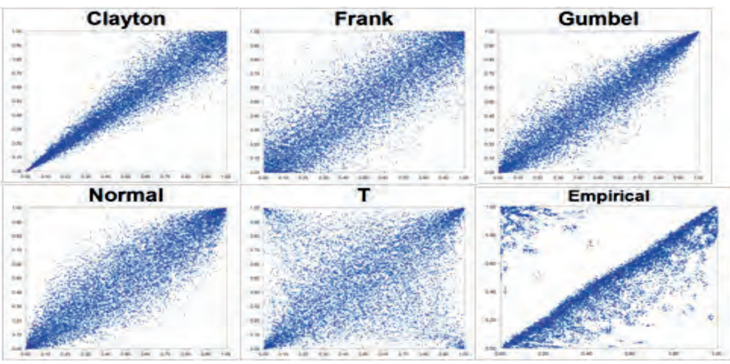

Figure 13. Some types of copula functions [17]

It is evident from Figure 13 that different copulas have different appearances and characteristics. The main characteristic is tail dependence. It is a very important feature of copulas, which has a great effect on how well the joint distribution captures the behaviour of extreme events. The tail dependence is a measure of extreme correlation, which marks the probability of an extreme occurrence for a variable when the extreme value of another variable occurs. When the variables of the marginal distribution in the upper tail (top right corner) of a copula are dependent on each other, we can say that the upper tail of the copula is dependent. From Figure 13 it can be seen that the Clayton copula can capture only lower tail dependence, the Frank copula family cannot exhibit any tail dependence and the Gumbel copula can only capture upper tail dependence [19].

There are different criteria in order to determine which copula is better suited for the analysed problem. The Kolmogorov-Smirnov test and the $\chi 2$ test, which are previously mentioned and explained, can be used. The Anderson-Darling goodness-of-fit and the Bayesian copula selection method can also be mentioned. Statistical measures of fit called information criteria such as the Schwarz Information criterion (SIC), known as the Bayesian information criterion or BIC, Akaike information criterion (AIC) and Hannan-Quinn information criterion (HQIC) can also be used [17].

As mentioned earlier, flood frequency analysis based on copula is mostly used in areas where confluences of tributaries can be found. For example, in [11] bivariate frequency analysis using a copula function is used to calculate the probability of coincidence of maximum water levels in the Drava and the Danube rivers. Results showed a 0.7% probability that highest water levels occur simultaneously in both rivers, which can be seen in the upper right corner of Figure 14. This is important information for future flood risk management because their coincidence would be disastrous for citizens in surrounding areas.

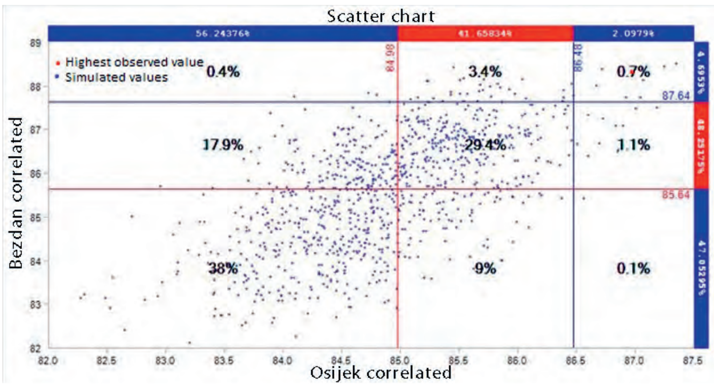

Figure 14. Probability of coincidence of maximum water levels of the Drava and Danube rivers [11]

Another example is from the Sava River basin [21]. The Sava River is the right-hand tributary of the Danube. Two tributaries of the Sava River were selected for analyses and it was shown that the proposed copula approach estimates recent flood events more accurately than the univariate flood frequency analysis based on the observation data.

Case study

In order to show how to fit distribution to observed discharges and how to calculate discharges for different return periods, time series of the Danube River discharges measured at Bogojevo station in Serbia will be analysed.

Maximum annual discharges of the Danube River in the period from 1950 to 2017 are analysed (Figure 15). Basic statistical parameters of this time series are shown in Table 3. Measured values for the years 1996, 1997 and 2010 are missing, so frequency analysis is done without them.

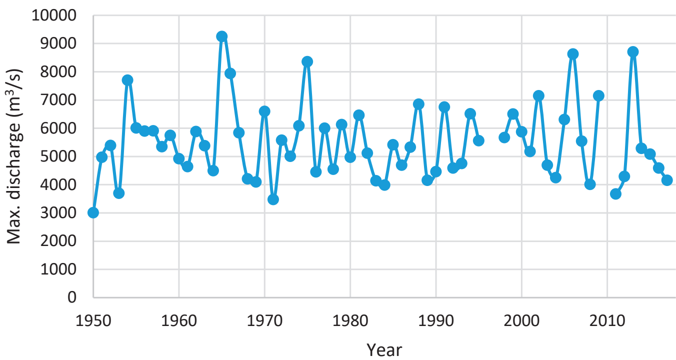

Figure 15. Maximum annual discharge of the Danube River at Bogojevo station (Serbia) (compiled by the authors)

Table 3. Basic statistical parameters of maximum annual discharges (compiled by the authors)

Number of observations	65
Mean value	5,493.862
Minimum value	3,010
Maximum value	9,250
Sum	357,101
Standard deviation	1,325.226
Variance	1,756,224
Skew	0.82733

The next step is to put discharge values in descending order and assign each value a rank. The highest value (in this case 9,250) has rank 1 and the smallest one, 3,010, has rank 65 which is the total number of values. After this, it is possible to fit distributions to observed time series.

Two empirical distributions, Weibull and Čegodajev, and three theoretical are calculated. Normal and Gamma distributions are calculated using Excel functions only and the Log-Pearson III distribution is determined also in Excel but with formulas and coefficients for this distribution.

Empirical distribution

Probabilities of occurrence for Weibull and Čegodajev distribution are calculated according to formulas shown in Table 1.

Results are shown in Figure 16. Probabilities of occurrence according to these two empirical distributions are the same.

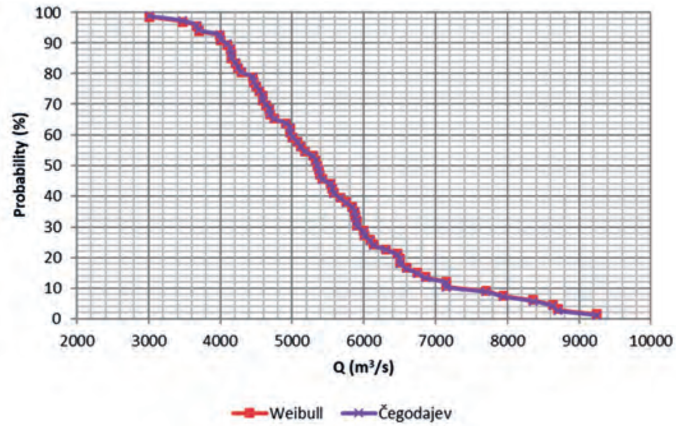

Figure 16. Weibull and Čegodajev distributions

Theoretical distribution

Log-Pearson III distribution is highly recommended for flood frequency analysis. The main advantage of this method is successful application on relatively short time series in order to obtain floods for much longer return periods. The equation and parameters of Log-Pearson III distribution are:

$$log\ x_{PR} = \overline{log\ x} + K \cdot \sigma\, log\ x \tag{23}$$

where

x_{PR} – is the variable value relevant for different return periods,

x – is the random variable (discharge, water level),

$\overline{\log x}$ – is the mean value of logarithms of random variables,

K – is the frequency coefficient; it is in a function of the skewness coefficient Cs and return period (Annex: Table V),

σ – is the standard deviation.

This distribution is based on logarithmic values of discharges, and not discharges themselves, so the first step is to calculate logarithmic values of discharges and then the skewness coefficient Cs and standard deviation of this logarithmic time series. After this, it is easy to calculate discharges for different return periods. Results are shown in Table 4 and in Figures 17 and 18.

Table 4. Results of Log-Pearson III distribution (compiled by the authors)

Probability	Return period	K1 (C = 0.1)	K2 (C = 0.2)	K	logQ	Q(m^3/s)
100	1	–2.252	–2.178	–2,1826	3.5065670	3210.45848
50	2	–0.017	–0.033	–0.0319	3.7247979	5306.37492
20	5	0.836	0.83	0.83037	3.8123042	6490.89067
10	10	1.292	1.301	1.30043	3.8600018	7244.39027
4	25	1.785	1.818	1.81592	3.9123096	8171.64876
2	50	2.107	2.159	2.15573	3.9467904	8846.88533
1	100	2.4	2.472	2.46747	3.9784235	9515.32264

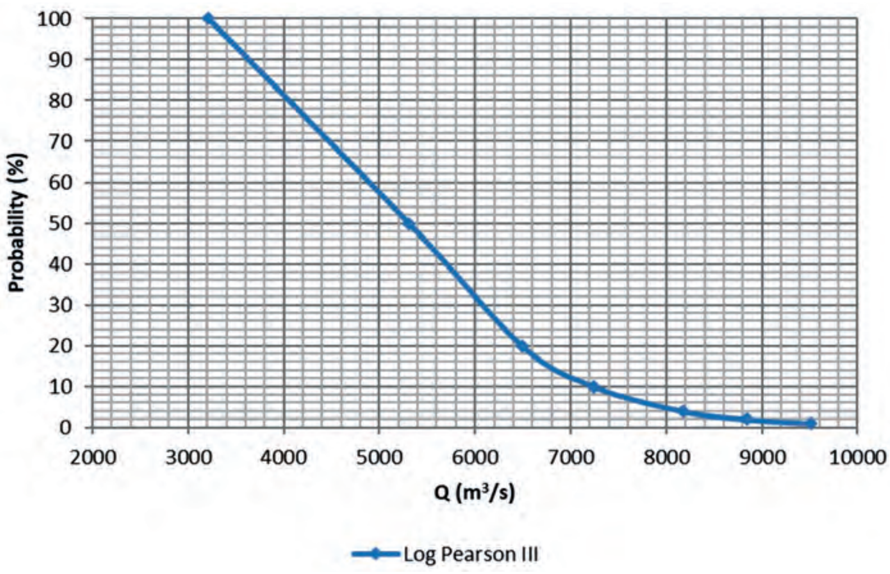

Figure 17. Log-Pearson III distribution (compiled by the authors)

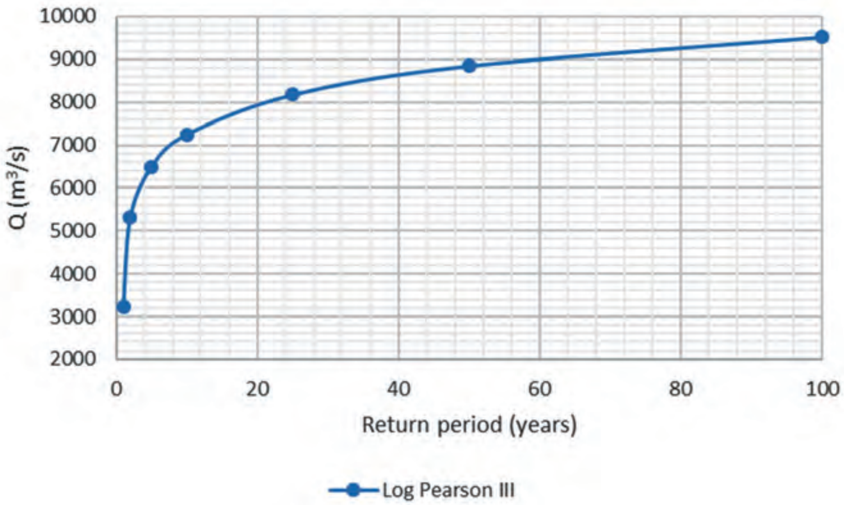

Figure 18. Discharges for different return periods according to Log-Pearson III distribution (compiled by the authors)

Normal distribution is calculated using Excel function NORM.DIST. To use this function, it is necessary to calculate the first mean value and standard deviation of the observed time series. This is already done (Table 3), so below in Figure 19 are the results.

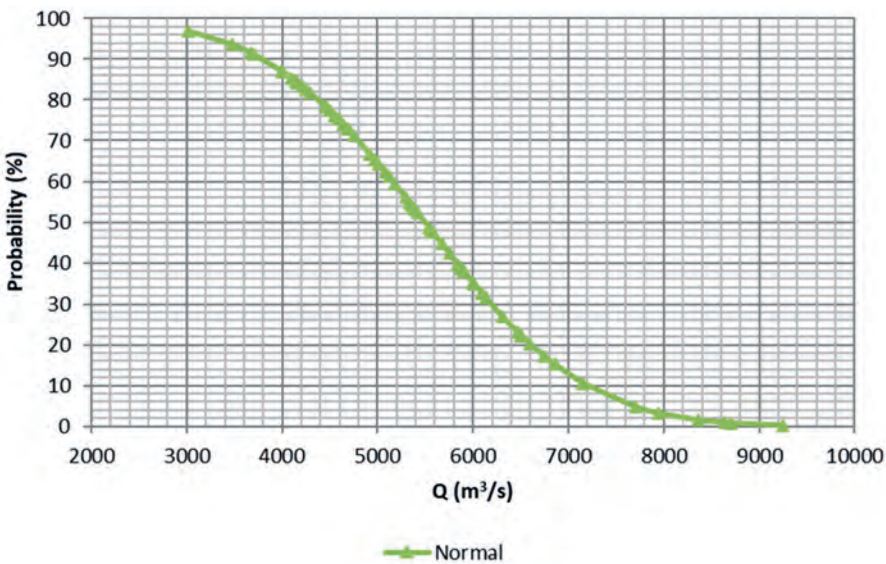

Figure 19. Normal distribution (compiled by the authors)

Gamma distribution is also determined with Excel function for this distribution, but first it is necessary to calculate the α and β parameters:

$$\alpha = E(x)2/Var \quad (24)$$

$$\beta = Var/\ E(x) \quad (25)$$

where

E(x) – is the expected value (mean),

Var – is the variance.

Results of the Gamma distribution are shown in Figure 20.

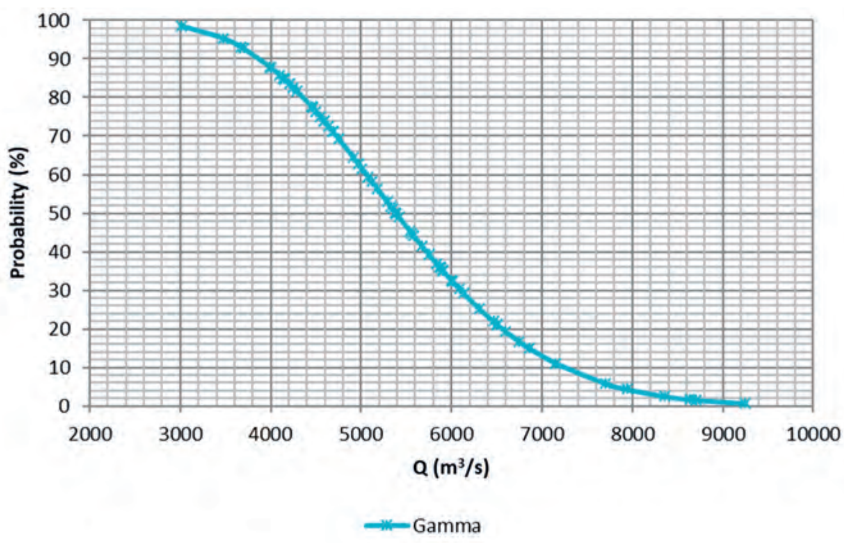

Figure 20. Gamma distributions (compiled by the authors)

All theoretical distributions are compared with empirical ones with goodness-of-fit Chi-squared test. When using this test as an Excel function, two sets must be selected: actual and expected. In this case, the actual range is empirical distribution and the expected range is a theoretical one. Obtained results showed that all distributions are a good fit to both empirical ones. When all distributions are plotted together (Figure 21), this good fit can be seen also.

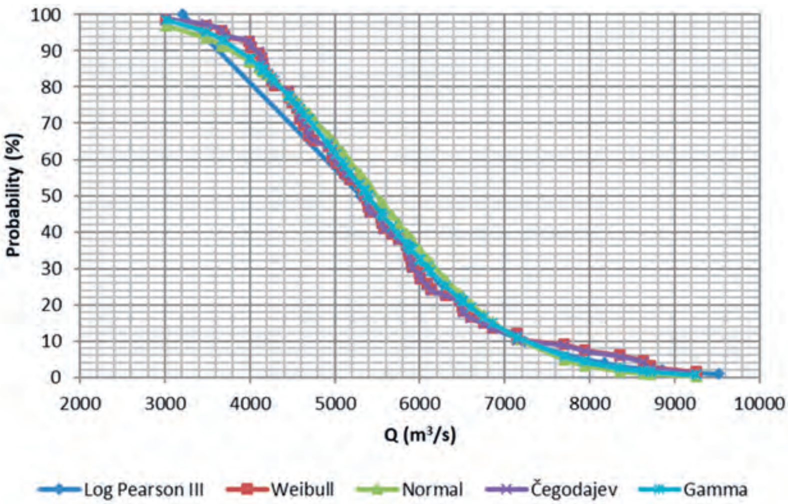

Figure 21. All distributions plotted together (compiled by the authors)

Final remarks

Flood frequency analysis briefly described in the previous chapters presumes the availability of reliable and sufficiently long hydrological data records of maximum discharges or water levels.

Very often, especially on small rivers (catchments) there are no measured data with such characteristics. These catchments are called ungauged and the procedure of flood frequency determination differs compared to these, because the only measured parameter is precipitation. It is necessary, therefore, to apply some of the various hydrological models, rainfall-runoff models (RFRO) or statistical models to determine the hydrological parameters of the flood waves.

Flood is a stochastic event and statistics is the only tool used to give us as much accurate probability of its occurrence as possible. There are many results of methods (empirical and theoretical) but none of them is absolutely accurate.

Besides, each catchment is changing in time, even if there are no drastic human interventions, which additionally makes our calculations and prognosis more complex.

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Annex

Table I. Log normal (Galton)distribution – $z\overline{\sigma}$ values for different return periods [8]

Return period (year)	Probability of occurrence p (%)	z	$z\overline{\sigma}$
10,000	0.01	3.715	0.6901
1,000	0.1	3.090	0.5740
100	1	2.326	0.4321
50	2	1.054	0.3816
25	4	1.752	0.3255
10	10	1.281	0.2380
5	20	0.842	0.1564
2	50	0.000	0.000
1.25	80	–0.842	–0.1564
1.1111	90	–1.281	–0.2380
1.0417	96	–1.752	–0.3255
1.0204	98	–2.054	–0.3816
1.0101	99	–2.326	–0.4321
1.0010	99.9	–3.090	–0.5740
1.0001	99.99	–3.715	–0.6901

Table II. Gumbel distribution – "z" values for different return periods [8]

Return period (year)	Probability of occurrence p (%)	P1	z	$\frac{1}{a}z$
10,000	0.01	0.9999	9.21	253.3
1,000	0.1	0.999	6.91	190
100	1	0.99	4.60	126.5
50	2	0.98	3.91	107.5
25	4	0.96	3.20	88.0
10	10	0.90	2.25	61.9
5	20	0.80	1.50	41.3
2	50	0.50	0.37	10.2
1.25	80	0.20	–0.48	–13.2
1.1111	90	0.10	–0.83	–22.8
1.0417	96	0.04	–1.15	–31.6
1.0204	98	0.02	–1.35	–37.1
1.0101	99	0.01	–1.53	–42.1
1.0010	99.9	0.001	–1.94	–53.4
1.0001	99.99	0.0001	–2.20	–60.5

Table III. Pearson III distribution – Probability of occurrence p (%), according to Foster-Ribkin [8]

czM	Probability of occurrence p (%)														
	0.01	**0.1**	**1**	**2**	**4**	**10**	**20**	**50**	**80**	**90**	**96**	**98**	**99**	**99.9**	**99.99**
–2.0		1.00	0.99	0.96	0.96	0.90	0.78	0.31	–0.6	–1.3	–2.3				
–1.5		1.31	1.26	1.23	1.15	1.02	0.82	0.24	–0.7	–1.3	–2.0				
–1.0		1.79	1.59	1.37	1.22	1.13	0.85	0.16	–0.8	–1.3	–2.0				
–0.8		2,02	1.74	1.65	1.42	1.17	0.85	0.13	–0.8	–1.3	–1.9				
–0.6		2.27	1.88	1.76	1.51	1.20	0.85	0.10	–0.8	–1.3	–1.9				
–0.4		2.54	2.03	1.90	1.60	1.23	0.85	0.07	–0.8	–1.3	–1.8				
–0.2		2.81	2.18	1.98	1.67	1.26	0.85	0.03	–0.8	–1.3	–1.8				
0	3.72	3.09	2.33	2.04	1.75	1.28	0.84	0.00	–0.9	–1.3	–1.7				
0.2	4.16	3.38	2.47	2.16	1.81	1.30	0.83	0.00	–0.9	–1.3	–1.6				
0.4	4.61	3.66	2.61	2.26	1.87	1.32	0.82	–0.1	–0.9	–1.2	–1.5				
0.6	5.05	3.96	2.76	2.35	1.94	1.33	0.80	–0.1	–0.9	–1.2	–1.5				
0.8	5.50	4.24	2.89	2.45	2.00	1.34	0.78	–0.1	–0.9	–1.2	–1.4				
1.0	5.96	4.53	3.02	2.54	2.05	1.34	0.76	–0.2	–0.9	–1.1	–1.3				
1.2	6.41	4.81	3.15	2.62	2.09	1.34	0.73	–0.2	–0.8	–1.1	–1.3				
1.5	7.09	5.26	3.33	2.74	2.15	1.33	0.69	–0.2	–0.8	–1.0	–1.1				
2.0	8.21	5.91	3.60	2.91	2.23	1.30	0.61	–0.3	–0.8	–0.9	–1.0				
2.5	9.30	6.60	3.83	3.04	2.28	1.24	0.53	–0.4	–0.7	–0.8	–0.8				
3.0	10.4	7.25	4.02	3.16	2.30	0.42	0.42	–0.4	–0.6	–0.7	–0.7	–0.7	–0.7	–0.7	–0.7

Table IV. Values of ϕ function in Pearson III distribution [8]

Return period (year)	Probability of occurrence p (%)	ϕ
10,000	0.01	5.50
1,000	0.1	4.24
100	1	2.89
50	2	2.45
25	4	2.00
10	10	1.34
5	20	0.78
2	50	–0.13
1.25	80	–0.86
1.1111	90	–1.17
1.0417	96	–1.47
1.0204	98	–1.60
1.0101	99	–1.74
1.0010	99.9	–2.02
1.0001	99.99	–2.18

Table V. Frequency factors K for Gamma and Log-Pearson type III distributions [24]

	Recurrence interval in years							
	1.0101	2	5	10	25	50	100	200
Skew coefficient	Percent chance (> =) = 1-F							
Cs	99	50	20	10	4	2	1	0.5
3	–0.667	–0.396	0.42	1.18	2.278	3.152	4.051	4.97
2.9	–0.69	–0.39	0.44	1.195	2.277	3.134	4.013	4.904
2.8	–0.714	–0.384	0.46	1.21	2.275	3.114	3.973	4.847
2.7	–0.74	–0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	–0.769	–0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	–0.799	–0.36	0.518	1.25	2.262	3.048	3.845	4.652
2.4	–0.832	–0.351	0.537	1.262	2.256	3.023	3.8	4.584
2.3	–0.867	–0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	–0.905	–0.33	0.574	1.284	2.24	2.97	3.705	4.444
2.1	–0.946	–0.319	0.592	1.294	2.23	2.942	3.656	4.372
2	–0.99	–0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	–1.037	–0.294	0.627	1.31	2.207	2.881	3.553	4.223
1.8	–1.087	–0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	–1.14	–0.268	0.66	1.324	2.179	2.815	3.444	4.069
1.6	–1.197	–0.254	0.675	1.329	2.163	2.78	3.388	3.99
1.5	–1.256	–0.24	0.69	1.333	2.146	2.743	3.33	3.91
1.4	–1.318	–0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	–1.383	–0.21	0.719	1.339	2.108	2.666	3.211	3.745
1.2	–1.449	–0.195	0.732	1.34	2.087	2.626	3.149	3.661
1.1	–1.518	–0.18	0.745	1.341	2.066	2.585	3.087	3.575
1	–1.588	–0.164	0.758	1.34	2.043	2.542	3.022	3.489
0.9	–1.66	–0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	–1.733	–0.132	0.78	1.336	1.993	2.453	2.891	3.312
0.7	–1.806	–0.116	0.79	1.333	1.967	2.407	2.824	3.223
0.6	–1.88	–0.099	0.8	1.328	1.939	2.359	2.755	3.132
0.5	–1.955	–0.083	0.808	1.323	1.91	2.311	2.686	3.041
0.4	–2.029	–0.066	0.816	1.317	1.88	2.261	2.615	2.949
0.3	–2.104	–0.05	0.824	1.309	1.849	2.211	2.544	2.856
0.2	–2.178	–0.033	0.83	1.301	1.818	2.159	2.472	2.763
0.1	–2.252	–0.017	0.836	1.292	1.785	2.107	2.4	2.67
0	–2.326	0	0.842	1.282	1.751	2.054	2.326	2.576
–0.1	–2.4	0.017	0.846	1.27	1.716	2	2.252	2.482
–0.2	–2.472	0.033	0.85	1.258	1.68	1.945	2.178	2.388
–0.3	–2.544	0.05	0.853	1.245	1.643	1.89	2.104	2.294
–0.4	–2.615	0.066	0.855	1.231	1.606	1.834	2.029	2.201
–0.5	–2.686	0.083	0.856	1.216	1.567	1.777	1.955	2.108
–0.6	–2.755	0.099	0.857	1.2	1.528	1.72	1.88	2.016
–0.7	–2.824	0.116	0.857	1.183	1.488	1.663	1.806	1.926

	Recurrence interval in years							
	1.0101	2	5	10	25	50	100	200
Skew coefficient								Percent chance (> =) = 1-F
Cs	99	50	20	10	4	2	1	0.5
–0.8	–2.891	0.132	0.856	1.166	1.448	1.606	1.733	1.837
–0.9	–2.957	0.148	0.854	1.147	1.407	1.549	1.66	1.749
–1	–3.022	0.164	0.852	1.128	1.366	1.492	1.588	1.664
–1.1	–3.087	0.18	0.848	1.107	1.324	1.435	1.518	1.581
–1.2	–3.149	0.195	0.844	1.086	1.282	1.379	1.449	1.501
–1.3	–3.211	0.21	0.838	1.064	1.24	1.324	1.383	1.424
–1.4	–3.271	0.225	0.832	1.041	1.198	1.27	1.318	1.351
–1.5	–3.33	0.24	0.825	1.018	1.157	1.217	1.256	1.282
–1.6	–3.88	0.254	0.817	0.994	1.116	1.166	1.197	1.216
–1.7	–3.444	0.268	0.808	0.97	1.075	1.116	1.14	1.155
–1.8	–3.499	0.282	0.799	0.945	1.035	1.069	1.087	1.097
–1.9	–3.553	0.294	0.788	0.92	0.996	1.023	1.037	1.044
–2	–3.605	0.307	0.777	0.895	0.959	0.98	0.99	0.995
–2.1	–3.656	0.319	0.765	0.869	0.923	0.939	0.946	0.949
–2.2	–3.705	0.33	0.752	0.844	0.888	0.9	0.905	0.907
–2.3	–3.753	0.341	0.739	0.819	0.855	0.864	0.867	0.869
–2.4	–3.8	0.351	0.725	0.795	0.823	0.83	0.832	0.833
–2.5	–3.845	0.36	0.711	0.711	0.793	0.798	0.799	0.8
–2.6	–3.899	0.368	0.696	0.747	0.764	0.768	0.769	0.769
–2.7	–3.932	0.376	0.681	0.724	0.738	0.74	0.74	0.741
–2.8	–3.973	0.384	0.666	0.702	0.712	0.714	0.714	0.714
–2.9	–4.013	0.39	0.651	0.681	0.683	0.689	0.69	0.69
–3	–4.051	0.396	0.636	0.66	0.666	0.666	0.667	0.667

Dejana Đorđević

Hydraulic Modelling

The quality of flood risk assessment highly depends on the proper definition of flood hazard maps. The level of confidence of these maps is governed by the quality of the hydrologic and hydraulic models. This course is focused on the hydraulic modelling of floods, i.e. flow in two-stage or compound channels. The course shall provide a state-of-the-art in 1D hydraulic modelling of floods. Models, which take into account different modes of momentum transfer between the main channel and floodplains, lead to better estimation of discharge and, consequently, better design of flood protection measures.

Characteristics of flow structure in compound channels

Overbank flow in a compound channel (CCh), which starts when the conveyance of the main channel is exceeded, is more complex than the inbank flow. The complexity originates from:

1. A sudden expansion of the channel;
2. The presence of vegetation on floodplains as a source of increased roughness when compared to the main channel roughness;
3. A random distribution of vegetation patches; and
4. Meandering of the main channel.

A special working group concerned with flow and sediment transport in compound channels was founded under the auspices of IAHR in the early 1990s aiming at:

1. Studying the characteristics of the overbank (compound channel) flow;
2. Checking a validity of the existing resistance laws that were originally proposed for the inbank flow;
3. Checking a validity of traditional methods for estimation of a stage-discharge curve in a compound channel, such as the single channel method (SCM), which is based on the Chézy-Manning equation that makes use of equivalent roughness coefficient, i.e. weighted roughness over the wetted perimeter, or the divided channel method (DCM) in which the total discharge in the cross-section is estimated as a sum of discharges in subsections with different roughness, again calculated using Chézy-Manning equation;
4. Proposing new methods for stage-discharge curve estimation in a compound channel if needed;
5. Proposing mathematical models of uniform and non-uniform flow in compound channels; and

6. Proposing 2D models for description of flow in a cross-section of a compound channel to define velocity and shear-stress distributions across the channel width that are important for the estimation of the transport capacity of the flow and conditions for sedimentation on the floodplains. The research is based on studies of compound channel flow in laboratory flumes, mainly in straight ones of prismatic and non-prismatic type with simple rectangular or trapezoidal subsections (Figure 1). Studies of the overbank flow in the case of meandering main channel are still scarce.

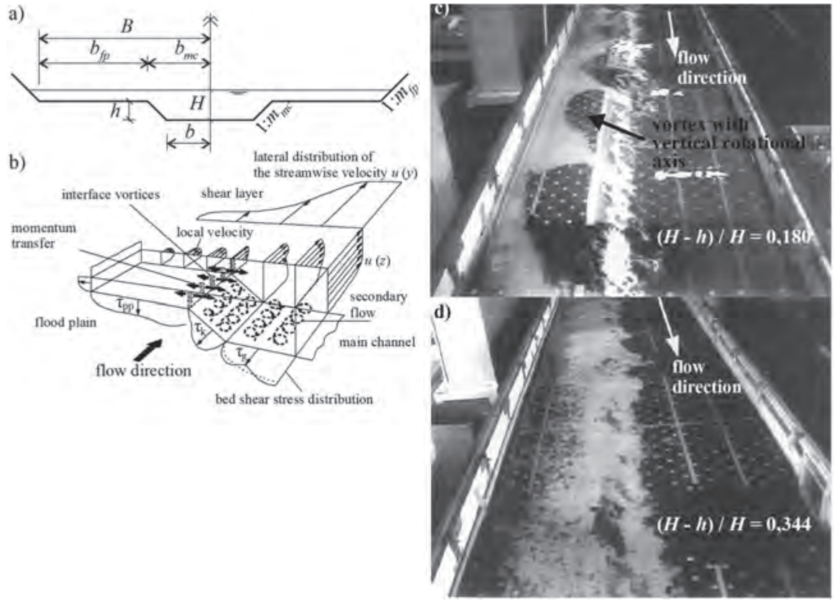

Figure 1. a) Elements of the compound channel geometry; b) flow structure in a compound channel [8]; c) longitudinal vortices with the vertical rotational axis on the floodplain when (H – h) / H = 0.180; and d) shorter longitudinal vortices with the vertical rotational axis on either side of the imaginary boundary between the main channel and the floodplain when (H – h) / H = 0.344 [7]

Experiments in straight prismatic CChs with smooth floodplains have shown that there is an inflection point in the streamwise velocity distribution across the channel width u(y) when the relative depth on the floodplain is low, i.e. when $(H-h)/H < 0.25$ (where H is the flow depth in the main channel and h is the depth of the main channel, Figure 1 a). This type of flow is also called a shallow floodplain flow. Large velocity gradients caused by the difference between the fast flow in the main channel and the slow flow over the floodplain result in increased shear between the two flows and the so-called Kelvin-Helmholz instability. This further gives rise to the development of large clockwise rotating horizontal (planform) vortices along the interface between the main channel and the floodplain on the floodplain side (Figure 1 c). These vortices are responsible for the momentum exchange between the main channel and floodplain flows and additional head losses. The exchange of momentum is accomplished by the turbulence diffusion

$\overline{u'v'}$ on the horizontal and $\overline{u'w'}$ vertical planes (Figure 1 b). With the increase in relative depth on the floodplain [$(H - h) / H > 0.25$], i.e. when the flow on the floodplain turns to a deep floodplain flow, the u-velocity becomes more evenly distributed across the channel width and the inflection point turns to a velocity dip. Thus, there are velocity gradients on both sides of the interface between the main channel and the floodplain. They give rise to the development of two counter rotating planform vortices on each side of the interface. These vortices are much smaller than those that develop in shallow floodplain flow (Figure 1 d), because of the strong secondary flow at the junction of the main channel and the floodplain, which now governs the 3D flow structure in CCh, as found by Nezu et al. and Ikeda et al. [8].

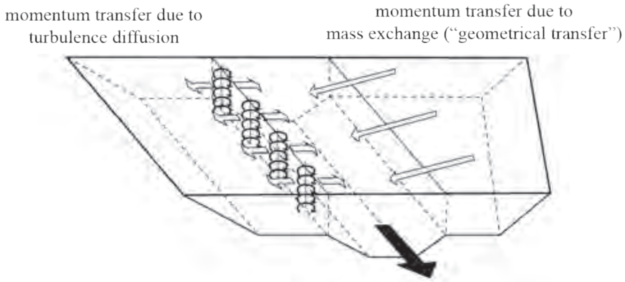

Figure 2. Types of momentum transfer between the main channel and floodplains [5]

The 3D flow in straight non-prismatic channels is further enhanced due to increased overflow from the main channel to the floodplains in case of a CCh with the diverging floodplains or due to inflow of water from the floodplains to the main channel in case of converging floodplains.

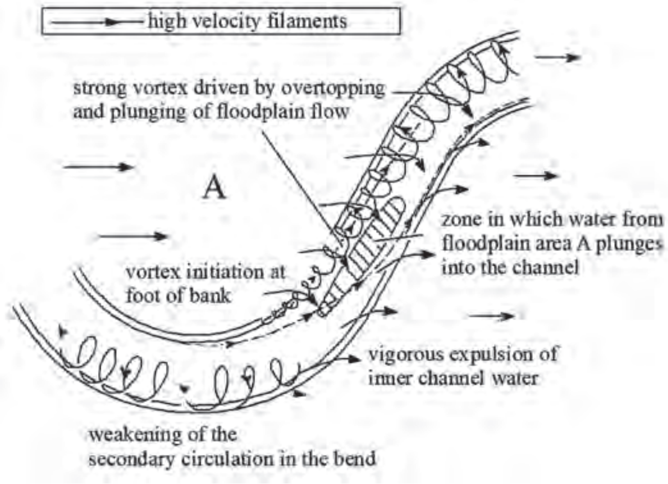

Figure 3. Interaction between the flow down the valley and the flow in the main channel in case of meandering channels [8]

This gives rise to additional momentum exchange between the two subsections of the CCh due to mass exchange, the so-called "geometrical transfer" (Figure 2). The total momentum transfer is thus the sum of the two components. Both components are lateral momentum exchanges between the two alongside flows. When the main channel meanders, there is also an interaction between the flow down the valley and that in the main channel (Figure 3). However, this type of flow has not yet been sufficiently investigated.

Despite of the fact that Prof. Miodrag Radojković from the Faculty of Civil Engineering in Belgrade suggested first improvements of the traditional procedure for calculation of 1D uniform and non-uniform flows in two-stage (CCh) channels in the mid-1980s, the DCM is still used by the vast majority of hydraulic engineering community. The suggested improvement was based on the analysis of forces that act on the three main subsections of the CCh, when they are observed independently, i.e. on the main channel and the two floodplains. The essence of Radojković's approach rests in the inclusion of the momentum transfer between the main channel and the floodplains via so-called ϕ-index, which is the ratio of the shear force and the component of the gravity force that acts in the flow direction in each of the three CCh subsections [16]. The ϕ-index method was successfully used by the working group members in processing of the data from the main flood channel facility at HR Wallingford. Moreover, Wormleaton and Merrett have shown that the method is equally applicable to different types of CCh division into subsections as presented in Figure 4, and that the best fit with measurements is achieved for rough floodplains when the interaction between the main channel and floodplain flows is pronounced.

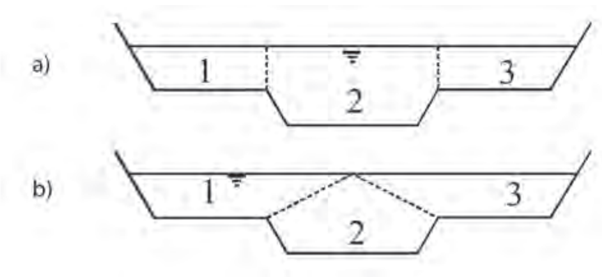

Figure 4. Possible divisions of the compound channel into subsections in the DCM; a) division using vertical; b) diagonal plains [16]

Ackers [1] proposed an empirical procedure for the improvement of the DCM results based on a large amount of experimental data a few years later. The essence of this approach is in calculation of the coherence, i.e. the ratio of the conveyance of the cross-section as a single unit and the sum of segment conveyances. Thus, coherence is a non-dimensional parameter and an indicator of the hydraulic homogeneity of the cross-section of the CCh. The discharge values calculated using the coherence method are less than those obtained by the DCM, but larger than values calculated using the SCM [8] [4].

Bousmar and Zech [2] proposed a physically based 1D mathematical model of uniform/non-uniform flow in a CCh in the late 1990s – an exchange discharge model (EDM).

Apart from losses due to friction, the energy losses in this method also include those originating from the momentum exchange between the main channel and floodplains. As already mentioned, there are two principal sources of the exchange of momentum. These are turbulent diffusion and mass exchange or "geometrical transfer". The EDM model, as shown by [1] and [4] provides much better agreement with measured stage-discharge curves than Ackers's method and the two traditional methods – SCM and DCM. Later on, in the early 2000s Proust et al. [12] improved the 1D mathematical model of non-uniform flow in a CCh proposed by Yen et al., and named it "independent subsections method" (ISM). Both the mass and momentum conservation equations are written for each subsection of the CCh as in the EDM. However, the main difference is in the numerical procedure used to solve these equations. While all equations in the EDM are combined in a single non-linear equation with one unknown variable, which is solved using the Newton-Raphson method, the system of equations is kept together in the ISM and solved iteratively using the finite difference method. Energy losses due to turbulent and mass exchanges are calculated in a similar fashion as in the EDM.

The role of vegetation, its effect on flow structure and its environmental effect

Until recently, vegetation was considered only a source of flow resistance. Therefore, it was frequently removed from channels and floodplains to enhance flow conveyance and reduce flooding. However, it was gradually recognised (during the last twenty years) that vegetation also provides a wide range of ecosystem services, such as: 1. The uptake of nutrients (nitrogen and phosphorous); 2. The production of oxygen; 3. The promotion of biodiversity by creation of spatial heterogeneity in stream velocity; 4. The attenuation of waves on the water surface; 5. The enhancement of bank stability; and 6. Trapping of sediment particles. This wide range of ecosystem services results from the fact that the vegetation alters the velocity field across different scales, which range from individual blades and branches of a single plant, to a community of plants in a meadow or a patch. Having this in mind, the proper description of the physical role of vegetation in the environment requires identification of the spatial scale relevant to a particular process. Thus, a brief review of flow structures starting from the blade scale, via patch scale to the reach scale will be highlighted in this section.

Vegetation can be emergent, when the flow depth is below its crest, or submerged, when there is a layer of water above its crest. Either one can be rigid or flexible.

Blade and individual stem scale

Flow around individual blades and leaves is modelled using the flat plate boundary-layer (Figure 5). The thickness of a viscous boundary layer, that forms at the leading edge ($x = 0$) of a plate, gradually grows in the streamwise direction $\delta(x) = 5\sqrt{\nu x / U}$. The viscous layer becomes sensitive to perturbations in the outer flow with the increasing thickness. When Reynolds number $R_x = U x / \nu$ approaches the value of 10^5, a transition to the turbulent boundary layer with the viscous sub-layer δ_s close to the blade surface occurs. Two possible cases are distinguished: one, in which the blade length is less than the transition length, and the other, in which the boundary layer becomes turbulent over a considerable portion of the blade length. In the first case, the boundary layer is laminar over the entire blade. In the latter one, the viscous sub-layer will have a constant thickness set by the friction velocity on the blade u_{b_*}.

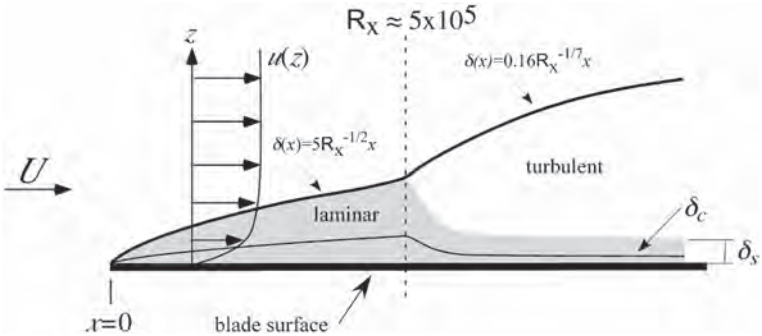

Figure 5. A flat plate boundary-layer model. The momentum boundary layer, δ, grows with distance from the leading edge (x = 0). Initially, the boundary layer is laminar (shaded grey). At distance x, corresponding to $R_x = xU/\nu \approx 5 \times 10^5$, the boundary layer becomes turbulent, except for a thin layer near the surface that remains laminar, called the viscous (or laminar) sub-layer, δ_s. In water, the diffusive sub-layer, δ_c, is much smaller than the viscous sub-layer, with $\delta_c = \delta_s S^{-1/3}$, where $S = \nu/D_m$ is the Schmidt number. The vertical coordinate is exaggerated in this figure [10]

The viscous sub-layer thickness is between $\delta_s = 5\nu/u_{b_*}$ and $10\nu/u_{b_*}$. Within this layer, the flow is essentially laminar. In addition to the viscous sub-layer, there is the concentration boundary layer δ_c. The thickness of this layer is smaller than δ_s ($\delta_c = 0.1\delta_s$), because of the difference in magnitude between the molecular diffusivity (D_m), whose order is 10^{-9} m²/s, and molecular viscosity, i.e. kinematic viscosity of water ν, which is of the order 10^{-6} m²/s.

Plants can have rigid and flexible stems. Flexible plants can be pushed over by currents, resulting in a change in morphology called reconfiguration [10]. Reconfiguration reduces flow resistance via two mechanisms: the reduction of the frontal area and the streamline adjustment. The drag on the deflected stem increases more slowly with velocity than that predicted by the quadratic law [10]. Recent studies have shown that reconfiguration depends on two dimensionless parameters, namely the Cauchy number and the buoyancy parameter. The Cauchy number (C) is the ratio of drag to the restoring force due to rigidity, while the buoyancy parameter (B), is the ratio of the restoring forces due to buoyancy and stiffness. For a stem of height *h*, width *w*, thickness *t*, and density, ρv, the two parameters are defined in a uniform flow of horizontal velocity *U* as:

$$\mathrm{C} = \frac{1}{2}\frac{\rho C_D w U^2 h^3}{EI} \tag{1}$$

$$\mathrm{B} = \frac{(\rho - \rho_v) g w t h^3}{EI} \tag{2}$$

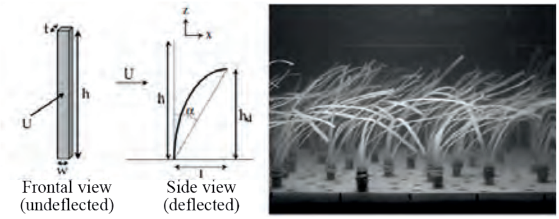

Figure 6. Geometric characteristics of individual undeflected and deflected stems (left) and a photograph from the experiments of Ghisalberti and Nepf [6]

In these expressions, E is the elastic modulus for the stem, I (= wt^3 /12) is the second moment of area, ρ is the density of water and g is the acceleration due to gravity.

The impact of reconfiguration on drag can be described by an effective blade height (h_e) which is defined as the height of a rigid, vertical stem that generates the same horizontal drag as a flexible one of total height h [10]. Based on this definition, the horizontal drag force is $F_x = (1/2)\rho C_D w h e U^2$, where the drag coefficient C_D is identical to that of rigid, vertical stems. The following relationships for effective height (h_e) and meadow height (h_m), are based on the model described in Luhar and Nepf [10]:

$$\frac{h_e}{h} = 1 - \frac{1 - 0.9\mathrm{C}^{-1/3}}{1 + C^{-3/2}(8 + \mathrm{B}^{3/2})} \tag{3}$$

$$\frac{h_m}{h} = 1 - \frac{1 - 0.9\mathrm{C}^{-1/4}}{1 + C^{-3/5}(4 + \mathrm{B}^{3/5}) + \mathrm{C}^{-2}(8 + \mathrm{B}^2)} \tag{4}$$

When rigidity is the dominant restoring force (C>>B), Eq. 4 reduces to $h_m / h \sim \mathrm{C}^{-1/4} \sim (EI/U^2)^{1/4}$ [10].

Patch scale

Uniform meadows of submerged vegetation are communities of individual plants of different densities. The meadow geometry is defined by the size of individual stems and their number per bed area. The meadow density can be defined in three different ways: 1. as the frontal area per volume $a = d / \Delta S^2$ (where d is a characteristic diameter or width and ΔS is an average spacing between stems); 2. as the solid volume fraction occupied by the canopy elements ϕ; or 3. as the frontal area per bed area $\lambda = ah_m$, which is known as a roughness density. Due to spatial heterogeneity of the velocity field, the flow in submerged meadows is described using the double-averaging concept proposed by Nikora et al. [11]. The length scale over which both mean and turbulent velocity components adjust to canopy drag is known as the canopy-drag length scale. This length scale is defined as:

$$L_c = \frac{2\,(1-\phi)}{C_D\,a} \tag{5}$$

and the spatially-averaged meadow drag as:

$$D_x = \frac{1}{2}\,\frac{C_D\,a}{(1-\phi)}\,\langle \bar{u} \rangle \left| \langle \bar{u} \rangle \right| \tag{6}$$

Here operator$\langle\;\rangle$stands for spatial averaging explained in Nikora et al. [11]. "The effect of meadow density, expressed via roughness density, on the velocity profiles and turbulence scales is presented in Figure 7. Two limits of flow behaviour are distinguished depending on the relative importance of the bed shear and meadow drag. If the meadow drag is smaller than the bed drag, then the velocity follows a turbulent boundary-layer profile, with the vegetation contributing to the bed roughness. This is the sparse canopy limit (Figure 7 a). In this limit, the turbulence near the bed will increase as stem density increases. Alternatively, in the dense canopy limit, the canopy drag is larger than the bed stress, and the discontinuity in drag at the top of the canopy generates a region of shear resembling a free shear layer, including an inflection point near the top of the canopy (Figure 7 b, c). From scaling arguments, the transition between sparse and dense limits occurs at $\lambda = ah_m = 0.1$. From measured velocity profiles, a boundary-layer form with no inflection point is observed for $C_D ah_m < 0.04$, and a pronounced inflection point appears for $C_D ah_m > 0.1$" [10].

"If the velocity profile contains an inflection point, it is unstable to the generation of Kelvin–Helmholtz (KH). These structures dominate the vertical transport at the canopy interface. These vortices are called canopy-scale turbulence, to distinguish them from the much larger boundary-layer turbulence, which may form above a deeply submerged or unconfined canopy, and the much smaller stem-scale turbulence. Over a deeply submerged (or terrestrial) canopy ($H / h > 10$), the canopy-scale vortices are highly three dimensional due to their interaction with boundary-layer turbulence, which stretches the canopy-scale vortices, enhancing secondary instabilities. However, with shallow submergence ($H / h \leq 5$), which is common in aquatic systems, large-scale boundary-layer turbulence is not developed, and the canopy-scale vortices dominate the turbulence both within and above the meadow" [10].

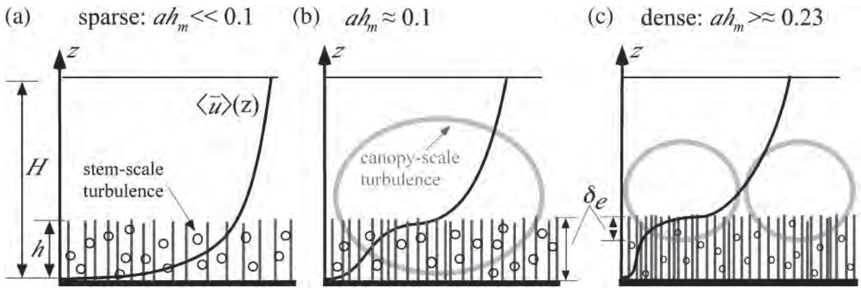

Figure 7. The mean velocity profiles through submerged meadows of increasing roughness density (ah_m). The meadow height is hm. Water depth is H. a) For ah_m < 0.1 (sparse regime), the velocity follows a rough boundary-layer profile; b) for $ah_m \geq 0.1$, a region of strong shear at the top of the canopy generates canopy-scale turbulence. The canopy-scale turbulence penetrates a distance δ_e = [0.23 ± 0.06]($C_D a$) −1 into the canopy; c) for ah_m > 0.23 (dense regime), δ_e < h, and the bed is shielded from the canopy-scale turbulence. Stem-scale turbulence is generated throughout the meadow [10]

"Within a distance of about $10h_m$ from the canopy's leading edge, the canopy-scale vortices reach a fixed scale and a fixed penetration into the canopy. The final vortex and shear-layer scale is reached when the shear production that feeds energy into the canopy-scale vortices is balanced by the dissipation by the canopy drag. This balance predicts the following scaling, which has been verified with observations" [10]:

$$\delta_e = \frac{0.23 \pm 0.6}{C_D a} \tag{7}$$

"This equation only applies to canopies that form a shear layer (i.e. $C_D a h_m \geq 0.1$). For $C_D a h_m$ = 0.1–0.23, the canopy-scale turbulence penetrates to the bed, $\delta_e = h_m$, creating a highly turbulent condition over the entire canopy height (Figure 7 b). At higher values of $C_D a h_m$, the canopy-scale turbulence does not penetrate to the bed, $\delta_e < h_m$ (Figure 7 c). If the submergence ratio $H / h_m < 2$, E_q. 7 for δ_e is not applicable, as the interaction with the water surface diminishes the strength and size of the canopy-scale vortices. Canopies for which $\delta_e / h_m < 1$ (Figure 7 c) shield the bed from strong turbulence and turbulent stress. Because turbulence near the bed plays a role in resuspension, these dense canopies are expected to reduce re-suspension and erosion" [10].

Long patches of emergent canopies of finite width can grow either along the bank (Figure 8) or may exist at the centre of a channel (Figure 9). The width of alongside canopies is denoted by *b*. In case of centreline canopies, *b* is half the width of the canopy strip. The approaching flow deflects upstream of the patch due to high drag exerted by vegetation. The upstream distance over which deflection starts is set by the scale *b* and it continues a distance x_D into the vegetation. The shear layer with KH vortices develops along the lateral edge of vegetation only after the deflection is complete ($x > x_D$). The initial growth, the final scale of the horizontal shear layer vortices and their lateral penetration into the patch δ_L, are depicted in Figure 8. The vortices extend into the open channel over length $\delta_0 \sim H/C_f$, where C_f is the bed friction [10]. There is no direct relation between δ_L and δ_0. The penetration depth is defined as:

$$\delta_L = \frac{0.5 \pm 0.1}{C_D a} \quad (8)$$

"If the patch width, b, is greater than the penetration distance, δ_L ($C_D ab > 0.5$, according to Eq. 8), turbulent stress does not penetrate to the centreline of the patch and the velocity within the patch (U_1, Figure 8) is set by a balance of potential gradient (bed and/or water surface slope) and vegetation drag. In contrast, for $C_D ab < 0.5$, turbulence stress can reach the patch centreline, and U_1 is set by the balance of turbulence stress and vegetation drag.

The centre of each vortex is a point of low pressure, which, for shallow flows, induces a wave response across the entire patch and specifically beyond δ_L from the edge. The wave response within the vegetation has been shown to enhance the lateral (y) transport of suspended particles, above that predicted from stem turbulence alone. For in-channel patches, shear layers develop along both flow parallel edges, and the vortices along each edge interact across the canopy width (Figure 9 a). The low-pressure core associated with each vortex produces a local depression in the water surface, such that the passage of individual vortices can be recorded by a surface displacement gage. A time record of surface displacement measured on opposite sides of a patch (A1 and A2 in Figure 9 b) shows that there is a half-cycle phase shift (π radians) between the vortex streets that form on either side of the patch. Because the vortices are a half cycle out of phase, when the pressure (surface elevation) is at a minimum on side A1, it is at a maximum at side A2. The resulting cross-canopy pressure gradient induces a transverse velocity within the canopy (Figure 9 b) that lags the lateral pressure gradient by π/2, that is, a quarter cycle. The synchronisation of the vortex streets occurs even when the vortex penetration is less than the patch width, $\delta_L / b < 1$, and it significantly enhances the vortex strength and the turbulence momentum exchange between the open channel and vegetation. More importantly, the vortex interaction introduces significant lateral transport across the patch" [10].

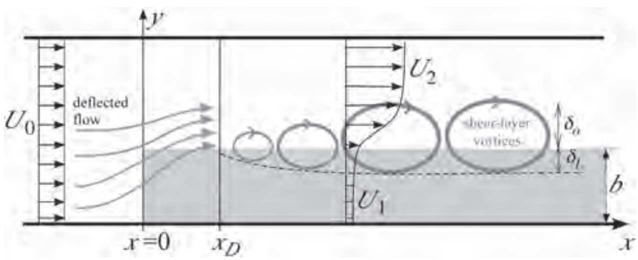

Figure 8. Top view of a channel with a long patch of emergent vegetation along the right bank (grey shading). The width of the vegetated zone is b. The flow approaching from upstream has uniform velocity U_0. The flow begins to deflect away from the patch at a distance b upstream and continues to decelerate and deflect until distance x_D. After this point, a shear layer forms on the flow-parallel edge and shear-layer vortices form by KH instability. These vortices grow downstream, but subsequently reach a fixed width and fixed penetration distance into the vegetation, δ_L [10].

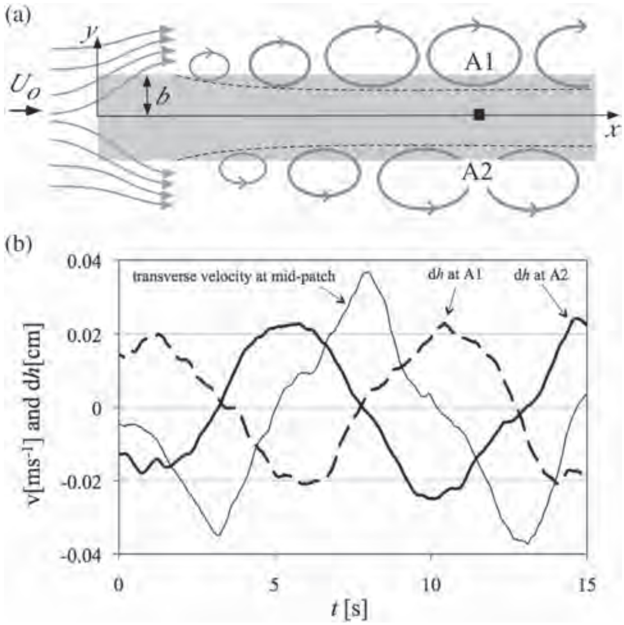

Figure 9. a) Top view of emergent vegetation with two flow-parallel edges. The patch width is 2b. The coherent structures on either side of the patch are out of phase. The passage of each vortex core is associated with a depression in surface elevation, which is measured at the patch edges (A1 and A2). The velocity is measured mid-patch (square). b) Data measured for a patch of width b = 10 cm in a channel with flow velocity U_0 = 10 cm s^{-1}. The patch centreline velocity is U_1 = 0.5 cm s^{-1}. The surface displacements measured at A1 (heavy dashed line) and at A2 (heavy solid line) are a half cycle (π radians) out of phase. The resulting transverse pressure gradient imposed across the patch generates transverse velocity within the patch (thin line), which, as in a progressive wave, lags the lateral pressure gradient by a quarter cycle (π/2 radians) [10].

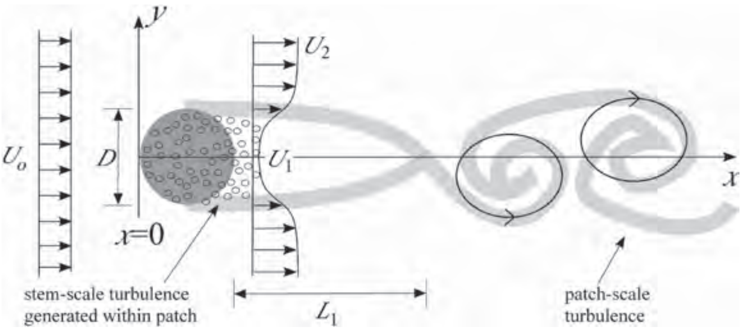

Figure 10. Top view of a circular patch of emergent vegetation with patch diameter D. The upstream, open-channel velocity is U_0. Stem-scale turbulence is generated within the patch, but dies out quickly behind the patch. The flow coming through the patch (U_1) blocks interaction between the shear layers at the two edges of the patch, which delays the onset of the patch-scale vortex street by a distance L_1. Tracer (grey line) released from the outermost edges of the patch comes together at a distance L_1 downstream from the patch and reveals the von Karman vortex street [10]

Circular patch of emergent vegetation

A circular patch with diameter D (Figure 10) is used as a model. “Because the patch is porous, the flow passes through it, and this alters the wake structure relative to that of a solid body. Directly behind a solid body, there is a region of recirculation, followed by a von Karman vortex street. The wake scale mixing provided by the von Karman vortices allows the velocity in the wake to quickly return (within a few diameters) to a velocity comparable to the upstream velocity (U_0). In contrast, the wake behind a porous obstruction (patch of vegetation) is much longer, because the flow entering the wake through the patch (called the bleed flow) delays the onset of the von Karman vortex street until a distance L_1 behind the patch. As a result, the velocity at the centreline of the wake, U_1, remains nearly constant over distance L_1. Within this region, both the velocity and turbulence are reduced, relative to the adjacent bare bed, so that it is a region where deposition is likely to be enhanced.

Both U_1 and L_1 depend on the patch diameter, D, and the drag length scale, $L_c \sim (C_D a)^{-1}$, which together form a dimensionless parameter, $C_D aD$, called the flow blockage. For low flow blockage (small $C_D aD$), U_1/U_0 decreases linearly with $C_D aD$. Using $C_D = 1$, a reasonable linear fit is:

$$\frac{U_1}{U_0} = 1 - [0.33 + 0.08] C_D \, a D \tag{9}$$

For high flow blockage, U_1 is negligibly small ($U_1 / U_\infty \approx 0.03$), but not zero. However, at some point around $C_D aD = 10$, U_1 does become zero, and the flow field around the porous patch becomes identical to that around a solid obstruction. This transition is also seen in the length scale, L_1, discussed below. Zong and Nepf suggested that L_1 may be predicted from the linear growth of the shear layers located on either side of the near-wake region, from which they derived:

$$\frac{L_1}{D} = \frac{1}{4S_1} \frac{(1 + U_1/U_0)}{(1 - U_1/U_0)} \tag{10}$$

Where S_1 is a constant (0.10 ± 0.02) across a wide range of D and ϕ. Drag is produced at two distinct scales: the leaf and stem scale and the patch scale. For low flow blockage patches, there is sufficient flow through the patch that the stem and leaf-scale drag dominates the flow resistance, that is, the flow resistance can be represented by the integral of $C_D au^2$ over the patch interior, with u being the velocity within the patch. However, for high flow-blockage patches, there is negligible flow through the patch, and the integral of $C_D au^2$ over the patch interior is irrelevant. The flow response to a high flow-blockage patch is essentially identical to the flow response to a solid obstruction of the same patch frontal area, A_p. Thus, the flow resistance provided by the patch should be represented by the patch-scale geometry, that is, $C_D A_p U^2$, with U being the channel velocity. This idea is supported by measurements of flow resistance produced by sparsely distributed bushes. A bush consists of a distribution of stems and leaves and so is a form of vegetation patch. The flow resistance generated by the bushes fit the quadratic model, ρ $C_D A_p U^2$, and notably C_D was $o(1)$, similar to a solid body. Thus, although porous, the bush generated drag that was comparable to that of a solid object of the same size (A_p). It is worth noting that C_D decreased somewhat (from 1.2 to 0.8) as the channel velocity increased. This shift is most likely due to the reconfiguration of stems and leaves that reduced A_p" [10].

Reach scale

"At the scale of the channel reach, flow resistance due to vegetation is determined primarily by the blockage factor (B_x) which is the fraction of the channel cross-section blocked by vegetation. For a patch of height h_m and width w in a channel of width W and depth H, $B_x = wh / WH$. Different studies show strong correlations between B_x and Manning's roughness coefficient n_M, noting that the relationship is nonlinear. For vegetation that fills the channel width, $B_x = h_m / H$. A few studies suggest that the vegetation distribution may also influence the resistance and specifically that greater resistance is produced by distributions with a greater interfacial area between vegetated and non-vegetated regions" [10]. Some authors have "quantified the impact of interfacial area by considering channels with the same blockage factor (B_x), but a different number (N) of patches. They showed that for realistic values of N, the resistance is increased by at most 20%, so that $N = 1$ is a reasonable simplifying assumption. For $N = 1$, the momentum balance leads to the following equations for Manning's roughness" [10]:

$$\text{For } B_x = 1: \qquad n_M \left(\frac{g^{1/2}}{K H^{1/6}} \right) = \left(\frac{C_D \, aH}{2} \right)^{1/2} \tag{11}$$

$$\text{For } B_x < 1: \qquad n_M \left(\frac{g^{1/2}}{K H^{1/6}} \right) = \left(\frac{C_*}{2} \right)^{1/2} (1 - B_x)^{-3/2} \tag{12}$$

"The constant $K = 1$ m$^{1/3}$ s^{-1} is required to make the equations dimensionally correct. Note that Eq. 11 is valid when $B_x = 1$, which indicates that vegetation covers the entire cross-section, width and depth. The coefficient C_* parameterises the shear stress at the interface between vegetated and non-vegetated regions, and $C_* = 0.05–0.13$, based on fits to field data, as shown by Luhar and Nepf. For the case of submerged vegetation that fills the channel width, the resistance is a function only of the submergence depth (H / h_M). Here, an expression for Manning's coefficient, proposed by Luhar and Nepf is presented:

$$\text{For } H/h > 1: \qquad n_M \left(\frac{g^{1/2}}{K H^{1/6}} \right) = \left[\left(\frac{2}{C_*} \right)^{1/2} \left(1 - \frac{h}{H} \right)^{3/2} + \left(\frac{2}{C_D \, ah_M} \right)^{1/2} \frac{h_M}{H} \right]^{-1} \tag{13}$$

If $C_D ah_M > C_*$, a common field condition, the second term drops out and Eq. 13 reverts to Eq. 12, because $B_x = h_M / H$ for vegetation covering the full channel width [10].

Floodplain processes

Floodplain processes are associated with the overbank deposition of sediment from river channels and overbank flow. As far as the deposition of sediments is concerned, two major mechanisms are distinguished: 1. The deposition due to interaction with the main channel; and 2. The deposition around vegetation. The deposition has important implications for floodplain development, agriculture and environment due to accumulation of contaminants that are adsorbed to sediment particles and for the creation of future sediment sources for the river channel [13]. The transfer of suspended sediment to, and its deposition on the floodplain are affected by the interaction of channel and overbank flows. This interaction, as it was shown in the second section varies with the channel planform. Thus, it may similarly be expected that the deposition pattern varies with the planform. This further means that the deposition pattern may be altered by channel engineering, for example, through channel straightening or through returning previously straightened channels to a more natural meandering state [15].

Floodplain deposition due to interaction with main channel

Intensive turbulent mixing in a lateral direction, which results from the interaction between the main channel and floodplain flows, or between the free flow and that in the vegetation zone, causes lateral net transport of suspended sediment from the main channel flow to the floodplain or the vegetated zone. Consequently, sediment ridges are developed on the floodplain or around the vegetated zone even in straight CChs. The entrainment and longitudinal transport of sediments from the bed are intensified in the main channel during floods, while the transport over the floodplain and through the vegetation is comparably low. This gives rise to the difference in sediment concentration between the main channel and the floodplain or vegetated zone and affects lateral diffusion of sediments.

Although the interaction between the main channel flow and the floodplain results in complex, three-dimensional flow structure (Figure 1 a), the presence of emergent vegetation makes flow horizontally two-dimensional. Such a flow is accompanied with organised fluctuations of low frequency that are caused by KH instability of the horizontal shear flow. They are felt throughout the flow depth and cause fluctuations in bed-load direction. These fluctuations are responsible for the net lateral transport of bed-load from the main channel, where the bed-load concentration is higher, towards the vegetated zone, where the concentration is lower. Thus, a longitudinal ridge is formed on the shoreline near the vegetated zone.

Floodplain deposition around vegetation

When the floodplain is dry, vegetation often forms colonies (Figure 11). It was shown in the previous section that the free flow (the overbank flow in this case) is retarded through and around an isolated vegetated area and that it accelerates again downstream of the vegetation patch to recover velocity. The resulting effect is arrestment of fine sediments and their deposition inside and downstream of the vegetation patch. The accumulated fertile soil facilitates invasion of new vegetation after the flood is retarded. Consequently, the vegetated area is enlarged and spread downstream. Transfer of bed-load from the main channel with the intense overbank flow during major flood events causes development of bed load deposits upstream of the vegetated area (with some local scour just in front of the vegetation) and erosion of its sides. As a result, the vegetated area resembles a mound [15]. The vegetation then becomes more firmly established and the vegetated mound is enlarged in the longitudinal direction, thus changing the morphology of the floodplain (Figure 11).

Figure 11. Vegetation colony on a floodplain [15]

Additionally, a smaller bed shear stress at the floodplains induces deposition of fine sediment on the floodplains.

Floodplain processes by overbank flow

A compound channel consists of a main channel and floodplains between the channel and levees. Floodplains contain many interesting micro-morphological features such as: secondary channels, side pools, dead zones, and so on. They are sometimes associated with vegetation, and vegetation influences fluvial processes related to these morphological features [15]. Furthermore, the variety of micro-morphological features provides favourable habitats for many organisms which contribute to the fluvial ecosystem. As with the original floodplain which existed before construction of levees, these morphologies are exposed to cyclical wetting and drying and cyclical development and destruction.

A typical morphological process which takes place when the overbank flow returns to the main channel is gully head-cutting. Gully head-cutting is characteristic for meandering compound channels or straight channels with alternate bars. This is a retrogressive erosion process, i.e. it migrates upstream. The flow from the gully is concentrated and forms an impinging jet which makes a scour hole (Figure 12). When the scour hole is deep enough, the upstream slope falls down into the scour hole, and the head-cut head migrates upstream.

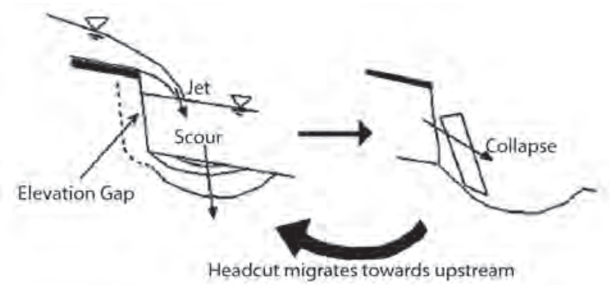

Figure 12. Head-cut erosion [15]

Overview of 1D models for compound channel flow modelling

Exchange discharge model (EDM)

The derivation of the governing equations in the EDM is based on the division of the compound channel cross-section into subsections with uniform hydraulic roughness using vertical planes (Figure 4 a). Generally, there are three subsections: the main channel and two floodplains. With this division, each subsection acts as a channel submitted to lateral flow per unit length of the interface between adjacent subsections q_l (Figure 13). This lateral flow has two components – an inflow q_{in} and an outflow q_{out}. With this decomposition, the mass conservation equation for each subsection can be written as follows:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial t} = q_l = q_{in} - q_{out} \tag{14}$$

Here the subscript *i*, indicating the subsection number ($i = 1, 2, 3$) is omitted for brevity. The space coordinate in the flow direction x and the time t are independent variables, while the cross-sectional area A, the flow discharge Q and the lateral discharge q_l are dependent variables. The two lateral flow components (q_{in} and q_{out}) are mutually exclusive only in prismatic channels. However, this is not the case with the momentum transfer due to turbulence diffusion, as will be shown shortly.

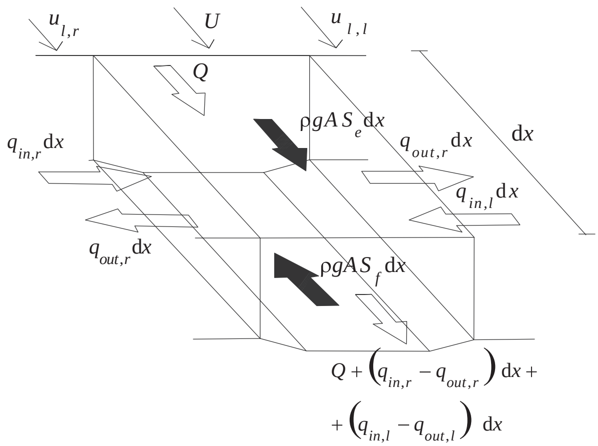

Figure 13. Momentum equilibrium for the control volume in the main channel [2] [4]

The change in the rate of momentum flux through the boundary of a control volume, caused by the action of forces, leads, according to the principle of conservation of momentum to the change in the rate of accumulation of momentum within this volume. Thus, the momentum conservation equation for the control volume of infinitesimal length dx (Figure 13) reads:

$$\frac{\partial}{\partial t}(\rho AU) + \frac{\partial}{\partial x}\left(\rho AU^2\right) + \rho gA\frac{\partial Z}{\partial x} + \rho gA\,S_f - \rho q_{in} u_l + \rho q_{out} U = 0 \tag{15}$$

The subscript i, indicating the subsection number (i = 1, 2, 3) is omitted here for brevity, again. In the previous equation r is the density of water, $U = Q / A$ is the mean velocity in the considered subsection, Z is the water level in the compound channel cross-section, g is acceleration due to gravity, S_f is the slope of the energy grade line, and u_l is the velocity of the lateral inflow in the direction of the main flow. As can be seen, the difference in mean velocities in adjacent subsections of the compound channel cross-section leads to different conveyances of momentum by the inflow and outflow lateral discharges. After the division of Eq. 15 with ρ the application of the product derivative rule, and utilisation of the mass conservation equation 1, the previous equation is simplified to:

$$A\frac{\partial U}{\partial t} + gA\frac{\partial}{\partial x}\left(Z + \frac{U^2}{2g}\right) = q_{in}\left(u_l - U\right) - gAS_f \tag{16}$$

The equation shows that only lateral inflow (q_{in}) affects momentum transfer, while the effect of the outflow is implicitly included in the variation of the kinetic energy head (the second term on the left hand side) [2]. An important consequence of this imbalance in the inflow and the outflow is the transfer of momentum due to turbulence diffusion, even when the average mass transfer through the interface between adjacent subsections is equal to zero (which is the case in prismatic compound channels).

The first term on the left hand side vanishes when the flow is steady. Thus, Eq. 16 simplifies to:

$$S_e = -\frac{\partial}{\partial x}\left(Z + \frac{U^2}{2g}\right) = S_f + \frac{q_{in}\left(U - u_l\right)}{gA} \tag{17}$$

which is the energy conservation equation for steady flow. The S_e is the total head loss per unit length. It is readily noticeable form this equation that the mechanical energy of the compound channel flow is extracted both by the friction and the exchange of discharges at the interface between adjacent subsections. The second term on the right hand side defines additional head loss per unit length due to exchange in discharges, and it will be denoted as S_{mot}. Generally, there are two adjacent subsections. Thus, the lateral inflow can be presented as a sum inflow from the right and the left subsections. Eq. 17 can now be written as:

$$S_e = S_f + \frac{q_{in,r}\,(U - u_{l,r}) + q_{in,l}\,(U - u_{l,l})}{gA} = S_f + S_{mot} \tag{18}$$

To facilitate further derivation of the model, a ratio between the additional loss due to momentum transfer and the friction loss $\chi = S_{mot} / S_f$ is introduced, and the previous equation simplifies to:

$$S_e = S_f\,(1 + \chi) \tag{19}$$

It is very important here to note that the total energy slope S_e is unique for the cross-section of the compound channel as a whole, while slopes due to friction S_f and momentum transfer S_{mot} may differ in each subsection because of the difference in roughness in the main channel and on the floodplains. Therefore, these slopes will be defined for each subsection i: $S_{f,i}$ and $S_{mot,i}$, as well as their ratio χ_i, $i = 1, 2, 3$.

The total lateral flow q_l, or exchange discharge, can be divided into two parts – one that is related to the turbulent momentum flux (q^t_{in}) and the other, which is associated to the mass exchange caused by non-prismatic shape of the compound channel (q^g_{in}). The two components should be modelled to close the problem.

Turbulence momentum flux modelling

This term is modelled by using the mixing length model on a horizontal plane. Bousmar and Zech have chosen this model as it allows for relatively simple computational procedure for the estimation of the stage-discharge curve and the definition of the relationship between the discharge and the slope of the energy grade line [2].

The lateral outflow from the main channel to the floodplain q^t_{mfp} and the lateral inflow from the floodplain to the main channel q^t_{fpm} are calculated by multiplying the absolute value of the depth-averaged fluctuation of the lateral velocity component $\left|\overline{v'}\right|$ with the interface area per unit length ($H - h_i$), where H is the flow depth in the main channel and a h_i is the depth of the main channel on the side of the floodplain i (Figure 1 a). It is assumed that the $\left|\overline{v'}\right|$ is proportional to the absolute value of the difference in streamwise velocities between two adjacent subsections $\left|U_{mc} - U_{fp}\right|$ [2]. Thus, the expression for the lateral turbulent momentum flux reads:

$$q^t_{mfp} = q^t_{fpm} = \left|\overline{v'}\right|(H - h_i) = \psi^t \left|U_{mc} - U_{fp}\right|(H - h_i) \tag{20}$$

where ψ^t is the proportionality factor. Since the turbulence momentum flux oscillates, Bousmar and Zech assume that it is equal to its doubled value through the interface between the two subsections [2].

Modelling of the exchange discharge due to change in geometry

One of the main parameters that affect floodplain conveyance is the width of the floodplain. Thus, the conveyance of the floodplain changes with the change in its width. It increases when it is widening and it reduces when it is narrowing. The change in conveyance forces a "geometrical transfer" discharge through the interface and results in the change in the discharge distribution between the main channel and floodplains along the course of the CCh. The "geometrical transfer" discharge from the main channel to the floodplain due to its widening is denoted by q^g_{mfp}, and that from the floodplain to the main channel due to its narrowing, by q^g_{fpm}. The possible layouts of the CCh and the corresponding directions of the "geometrical transfer" are presented in Figure 14.

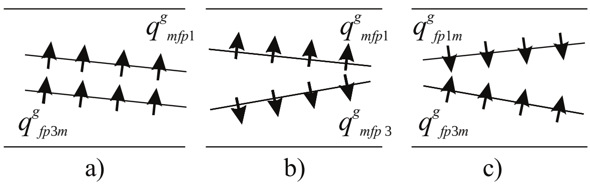

Figure 14. Possible layouts of the non-prismatic CCh: a) simultaneous widening of one, and narrowing of the other floodplain, with no change in the main channel width; b) simultaneous widening of both floodplains at the expense of narrowing of the main channel; and c) widening of the main channel at the expense of simultaneous narrowing of both floodplains [4]

For the case of increasing floodplain conveyance, the two "geometrical transfer" discharges are defined as:

$$q^g_{fpm} = 0 \quad \wedge \quad q^g_{mfp} = \frac{\mathrm{d}Q_{fp}}{\mathrm{d}x} = \frac{\mathrm{d}K_{fp}}{\mathrm{d}x} S^{1/2}_{f,fp} \tag{21}$$

and for the case of the decreasing floodplain conveyance, as:

$$q_{fpm}^{g} = -\frac{\mathrm{d}Q_{fp}}{\mathrm{d}x} = \frac{\mathrm{d}K_{fp}}{\mathrm{d}x} S_{f,fp}^{1/2} \quad \wedge \quad q_{mfp}^{g} = 0 \tag{22}$$

It is noted that the variation in the friction slope on the floodplain $S_{f,fp}$ due to change in its conveyance is neglected on the interval where the change in the conveyance is evaluated [5]. These expressions are generalised by introducing the κ parameter which indicates the flow direction with respect to the unit normal vector of the interface, and the proportionality factor ψ^g, which implicitly takes into account the aforementioned variation in the friction slope on the floodplain $S_{f,fp}$ due to change in its conveyance [2] [3] [5]:

$$q_{fpm}^{g} = \psi^{g} \kappa_{fpm} \frac{\mathrm{d}K_{fp}}{\mathrm{d}x} S_{f,fp}^{1/2} \quad \wedge \quad q_{mfp}^{g} = \psi^{g} \kappa_{mfp} \frac{\mathrm{d}K_{fp}}{\mathrm{d}x} S_{f,fp}^{1/2} \tag{23}$$

where

$$\kappa_{fpm} = \begin{cases} 0, & \dfrac{\mathrm{d}K_{fp}}{\mathrm{d}x} > 0 \\ -1, & \dfrac{\mathrm{d}K_{fp}}{\mathrm{d}x} < 0 \end{cases} \quad \wedge \quad \kappa_{mfp} = \begin{cases} 1, & \dfrac{\mathrm{d}K_{fp}}{\mathrm{d}x} > 0 \\ 0, & \dfrac{\mathrm{d}K_{fp}}{\mathrm{d}x} < 0 \end{cases} \tag{24}$$

The κ-value of 1 (κ = 1) indicates that the flow direction coincides with the unit normal vector of the interface, i.e. that the water outflows from the main channel to the floodplains. Conversely, κ = –1 shows that the flow is in the opposite direction of the unit normal vector of the interface and that the water withdraws from the floodplain to the main channel. Finally, κ = 0 implies that the considered subsection receives the water from the adjacent one.

EDM application

The EDM model is equally applicable to: 1. the estimation of the discharge in a compound channel based on the recorded flood marks for the purpose of estimation of the stage-discharge curve; and 2. the estimation of the slope of the energy grade line necessary for water level computations, when the water stage and the discharge are known. The following data are necessary for the estimation of discharge: 1. cross-sectional geometry; 2. the mean bottom slope S_0; 3. an estimation of the Manning roughness coefficient in all subsections of the CCh; and 4. the recorded flood mark(s). The estimation of the energy grade line slope, on the other hand, requires: 1. cross-sectional geometry; 2. the recorded flood mark(s); 3. the flood discharge; and 4. an estimation of the Manning roughness coefficient in all subsections of the CCh.

The two problems are solved using Manning's equation, Eq. 19, and the definition of the ratio χ_i. The discharge in the subsection i is calculated from:

$$Q_i = A_i U_i = \frac{A_i R_i^{2/3}}{n_i} S_{f,i}^{1/2} = K_i S_{f,i}^{1/2} = K_i \left(\frac{S_e}{1+\chi_i} \right)^{1/2} \tag{25}$$

and the mean velocity from:

$$V_i = \frac{R_i^{2/3}}{n_i} \left(\frac{S_e}{1+\chi_i} \right)^{1/2} \tag{26}$$

Expressions for χ_i, $i = 1, 2, 3$ can be derived from (19), (20) and (23). They read:

the left floodplain

$$\chi_1 = \frac{1}{gA_1} \left[\psi^t \left(H - h_1\right) \left(\frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_1}{1+\chi_2} \right)^{1/2} - \frac{R_1^{2/3}}{n_1} \right) + \psi^g \kappa_{21} \frac{\mathrm{d}K_1}{\mathrm{d}x} \right] \left[\frac{R_1^{2/3}}{n_1} - \frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_1}{1+\chi_2} \right)^{1/2} \right] \tag{27a}$$

the main channel

$$\begin{aligned} \chi_2 = {} & \frac{1}{gA_2} \left[\psi^t \left(H - h_1\right) \left(\frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_1}{1+\chi_2} \right)^{\frac{1}{2}} - \frac{R_1^{2/3}}{n_1} \right) + \psi^g \kappa_{12} \frac{\mathrm{d}K_1}{\mathrm{d}x} \right] \left[\frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_1}{1+\chi_2} \right)^{\frac{1}{2}} - \frac{R_1^{2/3}}{n_1} \right] \left(\frac{1+\chi_1}{1+\chi_2} \right) + \\ & + \frac{1}{gA_2} \left[\psi^t \left(H - h_3\right) \left(\frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_3}{1+\chi_2} \right)^{\frac{1}{2}} - \frac{R_3^{2/3}}{n_3} \right) + \psi^g \kappa_{32} \frac{\mathrm{d}K_3}{\mathrm{d}x} \right] \left[\frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_3}{1+\chi_2} \right)^{\frac{1}{2}} - \frac{R_3^{2/3}}{n_3} \right] \left(\frac{1+\chi_2}{1+\chi_3} \right) \end{aligned} \tag{27b}$$

the right floodplain

$$\chi_3 = \frac{1}{gA_3} \left[\psi^t \left(H - h_3\right) \left(\frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_3}{1+\chi_2} \right)^{1/2} - \frac{R_3^{2/3}}{n_3} \right) + \psi^g \kappa_{23} \frac{\mathrm{d}K_3}{\mathrm{d}x} \right] \left[\frac{R_3^{2/3}}{n_3} - \frac{R_2^{2/3}}{n_2} \left(\frac{1+\chi_3}{1+\chi_2} \right)^{1/2} \right] \tag{27c}$$

After introduction of three auxiliary variables:

$$X_i = (1+\chi_i)^{1/2} \tag{28}$$

the system of equations becomes:

$$X_1^2 - 1 = \frac{1}{gA_1} \left[\psi^t \left(H - h_1\right) \left(\frac{R_2^{2/3}}{n_2} \frac{X_1}{X_2} - \frac{R_1^{2/3}}{n_1} \right) + \psi^g \kappa_{21} \frac{\mathrm{d}K_1}{\mathrm{d}x} \right] \left[\frac{R_1^{2/3}}{n_1} - \frac{R_2^{2/3}}{n_2} \frac{X_1}{X_2} \right] \tag{29a}$$

$$X_2^2-1=\frac{1}{gA_2}\left[\psi^t\left(H-h_1\right)\left(\frac{R_2^{2/3}}{n_2}\frac{X_1}{X_2}-\frac{R_1^{2/3}}{n_1}\right)+\psi^g\kappa_{12}\frac{\mathrm{d}K_1}{\mathrm{d}x}\right]\left[\frac{R_2^{2/3}}{n_2}\frac{X_1}{X_2}-\frac{R_1^{2/3}}{n_1}\right]\left(\frac{X_1}{X_2}\right)^2+$$
$$+\frac{1}{gA_2}\left[\psi^t\left(H-h_3\right)\left(\frac{R_2^{2/3}}{n_2}\frac{X_3}{X_2}-\frac{R_3^{2/3}}{n_3}\right)+\psi^g\kappa_{32}\frac{\mathrm{d}K_3}{\mathrm{d}x}\right]\left[\frac{R_2^{2/3}}{n_2}\frac{X_3}{X_2}-\frac{R_3^{2/3}}{n_3}\right]\left(\frac{X_3}{X_2}\right)^2 \qquad (29b)$$

$$X_3^2-1=\frac{1}{gA_3}\left[\psi^t\left(H-h_3\right)\left(\frac{R_2^{2/3}}{n_2}\frac{X_3}{X_2}-\frac{R_3^{2/3}}{n_3}\right)+\psi^g\kappa_{23}\frac{\mathrm{d}K_3}{\mathrm{d}x}\right]\left[\frac{R_3^{2/3}}{n_3}-\frac{R_2^{2/3}}{n_2}\frac{X_3}{X_2}\right] \qquad (29c)$$

Knowing that the velocity in the main channel is greater than that on the floodplains, the system (29) must satisfy the following conditions:

$$0<X_1\le 1 \;\wedge\; 1\le X_2 \;\wedge\; 0<X_3\le 1 \qquad (30a)$$

$$\frac{1}{X_1}\frac{R_1^{2/3}}{n_1}\le\frac{1}{X_2}\frac{R_2^{2/3}}{n_2} \qquad \wedge \qquad \frac{1}{X_3}\frac{R_3^{2/3}}{n_3}\le\frac{1}{X_2}\frac{R_2^{2/3}}{n_2} \qquad (30b)$$

With these limitations, (29a) and (29c) can be considered quadratic equations in X_1 and X_3. For practical evaluation, only positive roots, which satisfy (30b) are taken:

$$\frac{X_1}{X_2}=\frac{1}{2}\left\{\left(2\frac{\psi^t\left(H-h_1\right)}{gA_1}\frac{R_1^{2/3}}{n_1}\frac{R_2^{2/3}}{n_2}-\frac{\psi^g\kappa_{21}\frac{\mathrm{d}K_1}{\mathrm{d}x}}{gA_1}\frac{R_2^{2/3}}{n_2}\right)+\left[\left(4\frac{\psi^t\left(H-h_1\right)}{gA_1}+\left(\frac{\psi^g\kappa_{21}\frac{\mathrm{d}K_1}{\mathrm{d}x}}{gA_1}\right)^2\right)\left(\frac{R_2^{2/3}}{n_2}\right)^2+\right.\right.$$
$$\left.\left.+4X_2^2\left(1-\frac{\psi^t\left(H-h_1\right)}{gA_1}\left(\frac{R_1^{2/3}}{n_1}\right)^2+\frac{\psi^g\kappa_{21}\frac{\mathrm{d}K_1}{\mathrm{d}x}}{gA_1}\frac{R_1^{2/3}}{n_1}\right)\right]^{\frac{1}{2}}\right\}\left[X_2^2+\frac{\psi^t\left(H-h_1\right)}{gA_1}\left(\frac{R_2^{2/3}}{n_2}\right)^2\right]^{-1} \qquad (31a)$$

$$\frac{X_3}{X_2}=\frac{1}{2}\left\{\left(2\frac{\psi^t\left(H-h_3\right)}{gA_3}\frac{R_3^{2/3}}{n_3}\frac{R_2^{2/3}}{n_2}-\frac{\psi^g\kappa_{23}\frac{\mathrm{d}K_3}{\mathrm{d}x}}{gA_3}\frac{R_2^{2/3}}{n_2}\right)+\left[\left(4\frac{\psi^t\left(H-h_3\right)}{gA_3}+\left(\frac{\psi^g\kappa_{23}\frac{\mathrm{d}K_3}{\mathrm{d}x}}{gA_3}\right)^2\right)\left(\frac{R_2^{2/3}}{n_2}\right)^2+\right.\right.$$
$$\left.\left.+4X_2^2\left(1-\frac{\psi^t\left(H-h_3\right)}{gA_3}\left(\frac{R_3^{2/3}}{n_3}\right)^2+\frac{\psi^g\kappa_{23}\frac{\mathrm{d}K_3}{\mathrm{d}x}}{gA_3}\frac{R_3^{2/3}}{n_3}\right)\right]^{\frac{1}{2}}\right\}\left[X_2^2+\frac{\psi^t\left(H-h_3\right)}{gA_3}\left(\frac{R_2^{2/3}}{n_2}\right)^2\right]^{-1} \qquad (31b)$$

After substitution of (31a) and (31b) into (29b) a single, non-linear equation in X_2 is obtained:

$$F\left(\frac{X_1}{X_2},\frac{X_3}{X_2},X_2\right)=F\left(X_2\right)=0 \qquad (32)$$

The equation is solved using the Newton-Raphson method. The remaining unknowns X_1 and X_3 are then calculated from (31a) and (31b). Consequently, three χ_i-ratio values are found from (28) and the discharge distribution between the subsections or the energy grade line slope can be determined. It is worth mentioning that the ratio χ_i is exclusively a function of the channel geometry and the roughness of subsections, which makes this method attractive for solving these practical engineering problems related to floods.

Total flood discharge is estimated from:

$$Q = \sum_i Q_i = \sum_i K_i \left(\frac{S_e}{1+\chi_i} \right)^{1/2} = \sum_i \left(\frac{K_i}{\left(1+\chi_i\right)^{1/2}} \right) S_e^{\ 1/2} \tag{33}$$

Since field measurements during floods are difficult and dangerous, the estimation is still based on an unrealistic and simplified assumption that the flow is uniform, i.e. that $S_e = S_0$. As it can be noticed, the EDM method makes use of corrected subsection conveyances:

$$K_i^* = \frac{K_i}{(1+\chi_i)^{1/2}},\ i = 1, 2, 3 \tag{34}$$

If, on the other hand, an energy grade line slope is needed for the water profile computations, it should be estimated based on the known water stage and total discharge values. In this case, the global ratio $\chi = S_{mot} / S_f$ (for the whole cross-section) is calculated based on the subsection ratios χ_i and conveyances K_i:

$$\chi = \left(\frac{\sum_i K_i}{\sum_i \left(K_i / \left(1+\chi_i\right)^{1/2} \right)} \right)^2 - 1 \tag{35}$$

and the S_e is then calculated from:

$$S_e = S_f + S_{mot} = S_f \left(1+\chi\right) = \left(\frac{Q}{\sum_i K_i} \right)^2 \left(1+\chi\right) \tag{36}$$

Here, S_{mot} is a global momentum transfer for the cross-section.

Independent subsections method (ISM)

In contrast to EDM, where a single water level value for the cross-section is calculated, the water surface profile in the ISM is estimated within each subsection. Moreover, the additional loss due to momentum transfer between adjacent subsections is explicitly divided into two terms – one that refers to the apparent shear stress (τ_{ij}) acting on the interface between subsections (which is responsible for the momentum transfer due to turbulence diffusion) and the other, which refers to the lateral mass exchange by the lateral discharge per unit length (q_{in} and/or q_{out}). In ISM, Eq. 15 for the steady flow transforms to:

$$S_{f,i} = S_0 - \frac{\mathrm{d}h_i}{\mathrm{d}x} \pm \frac{\tau_{ij} h_{if}}{\rho g A_i} - \frac{1}{gA_i}\frac{\mathrm{d}}{\mathrm{d}x}\left(A_i U_i^2\right) + \frac{\left(U_{in} q_{in} - U_{out} q_{out}\right)}{gA_i}, \ i = 1, 2, 3 \tag{37}$$

where h_i is the flow depth in the subsection i, h_{if} is the flow depth at the interface between adjacent subsections, U_{in} and U_{out} are subsection streamwise velocities with which the lateral mass discharge enters and leaves the subsection, respectively. Other variables are the same as in (15).

The mass conservation equation for each subsection reads:

$$\frac{\mathrm{d}A_i U_i}{\mathrm{d}x} = q_{in} - q_{out}, \ i = 1, 2, 3 \tag{38}$$

In the CCh with three subsections (Figure 1 a), there are only two lateral mass discharges: the one between the left floodplain and the main channel q_{lm} and the other, between the right floodplain and the main channel q_{rm}. The lateral discharge is positive when mass leaves the floodplain, and negative when it enters the floodplain. Thus, the following is valid: 1. for the left floodplain $q_{out} = q_{lm}$ and $q_{in} = 0$; 2. for the right floodplain $q_{out} = q_{rm}$ and $q_{in} = 0$ and for the main channel $q_{out} = 0$ and $q_{in} = q_{lm} + q_{rm}$. Eq. 38 can be written now for each subsection:

the left floodplain

$$\frac{\mathrm{d}Q_l}{\mathrm{d}x} = -q_{lm} \tag{39}$$

the right floodplain

$$\frac{\mathrm{d}Q_r}{\mathrm{d}x} = -q_{rm} \tag{40}$$

the main channel

$$\frac{\mathrm{d}Q_m}{\mathrm{d}x} = q_{lm} + q_{rm} \tag{41}$$

The mass conservation equation for the CCh cross-section as a whole is reached by combining (39)–(41) into a single one:

$$\frac{\mathrm{d}Q_m}{\mathrm{d}x} + \frac{\mathrm{d}Q_l}{\mathrm{d}x} + \frac{\mathrm{d}Q_r}{\mathrm{d}x} = 0 \tag{42}$$

Momentum equations for the three subsections are derived from (37) and (38) in the form similar to that for simple-channel non-uniform flow:

the left floodplain

$$\left(1 - \frac{U_l^2}{gh_l}\right)\frac{\mathrm{d}h_l}{\mathrm{d}x} = S_0 - S_{fl} + \frac{U_l^2}{gB_l}\frac{\mathrm{d}B_l}{\mathrm{d}x} + \frac{\tau_{lm} h_l}{\rho g A_l} + \frac{q_{lm}\left(2U_l - U_{if,l}\right)}{gA_l} \tag{43}$$

the right floodplain

$$\left(1-\frac{U_r^2}{gh_r}\right)\frac{\mathrm{d}h_r}{\mathrm{d}x} = S_0 - S_{fr} + \frac{U_r^2}{gB_r}\frac{\mathrm{d}B_r}{\mathrm{d}x} + \frac{\tau_{rm}h_r}{\rho g A_r} + \frac{q_{rm}\left(2U_r - U_{if,r}\right)}{gA_r} \tag{44}$$

the main channel

$$\begin{aligned}\left(1-\frac{U_m^2}{gh_m}\right)\frac{\mathrm{d}h_m}{\mathrm{d}x} = S_0 - S_{fm} + \frac{U_m^2}{gB_m}\frac{\mathrm{d}B_m}{\mathrm{d}x} - \frac{\tau_{lm}h_l}{\rho g A_m} - \frac{\tau_{rm}h_r}{\rho g A_m} - \\ - \frac{q_{lm}\left(2U_m - U_{if,l}\right)}{gA_m} - \frac{q_{rm}\left(2U_m - U_{if,r}\right)}{gA_m}\end{aligned} \tag{45}$$

The width of the subsection is denoted by B_i. It is assumed that all subsections are rectangular. Shear stresses at interfaces τ_{lm} and τ_{rm} are modelled by:

$$\left|\tau_{lm}\right| = \rho\psi^t (U_{mc} - U_l)^2 \text{ and } \left|\tau_{rm}\right| = \rho\psi^t (U_{mc} - U_r)^2 \tag{46}$$

where ψ^t is, again, the model parameter. Streamwise velocities at the interface between the main channel and the left and right floodplains are denoted by $U_{if,l}$ and $U_{if,r}$, respectively. Proust et al. [12] distinguished three possible cases for defining interface velocities:

the prismatic CCh and transfer of mass which occurs from subsection i towards subsection j:

$$U_{if} = U_i \tag{47}$$

the non-prismatic CCh and constant total channel width:

$$U_{if} = U_i \quad \text{if} \quad \mathrm{d}B_i/\mathrm{d}x < 0 \tag{48}$$

$$U_{if} = U_j \quad \text{if} \quad \mathrm{d}B_i/\mathrm{d}x > 0 \tag{49}$$

the non-prismatic CCh with variable total channel width:

$$U_{if,l} = \varphi_l U_l + (1-\varphi_l)U_m \quad \text{and} \quad U_{if,r} = \varphi_r U_r + (1-\varphi_r)U_m \tag{50}$$

Knowing that $E_i = Z_i + U_i^2/2g$ is the subsection total head, the Z_i being the water level in subsection i, Eq. 37 can be written as follows:

$$\begin{aligned} S_{e,i} &= -\frac{\mathrm{d}E_i}{\mathrm{d}x} = S_{f,i} \pm \frac{\tau_{ij}h_{if}}{\rho g A_i} + \frac{q_{in}(U_i - U_{in}) + q_{out}(U_{out} - U_i)}{gA_i} = \\ &= S_{f,i} + S_{td,i} + S_{m,i}\end{aligned} \tag{51}$$

With this notation, knowing that the second term in the bracket on the left hand side is Froude number, the analogy with equations for the simple-channel non-uniform flow becomes more obvious:

$$\left(1-\underbrace{\frac{U_l^2}{gh_l}}_{\mathrm{Fr}_i}\right)\frac{\mathrm{d}h_l}{\mathrm{d}x} = S_0 - \underbrace{S_{fl} + \frac{U_l^2}{gB_l}\frac{\mathrm{d}B_l}{\mathrm{d}x} + \frac{\tau_{lm}h_l}{\rho g A_l} + \frac{q_{lm}\left(2U_l - U_{if,l}\right)}{gA_l}}_{S_{e,i}} \tag{52}$$

$$\left(1-\mathrm{Fr}_i\right)\frac{\mathrm{d}h_l}{\mathrm{d}x} = S_0 - S_{e,i}$$

The system of three mass conservation and three momentum conservation equations together with the four closure equations is solved iteratively using finite differences.

Comparative analysis of traditional and new, improved models and the assessment of their performance

The two models from the previous section were tested and compared against the data from the FCF (Flood Channel Facility) made in HR Wallingford (Figure 15). This is a straight two-stage channel of trapezoidal cross-section in both the main channel and floodplains. The channel is 56 m long and 10 m wide.

The longitudinal slope of the channel is S_0 = 1.027%. The main channel is made of concrete, and floodplains are made of Plexiglas. Channel dimensions from Figure 1 a, including bank slopes are given in Table 1. Experimental series with smooth and rough floodplains were used for calibration and comparison of the two models (Table 1). The first two cases (series no. 2 and 3) with smooth floodplains were used to study sensitivity of models to changes in CCh width.

The first and the third case (series no. 2 and 6) were used to test the model in the absence or complete exclusion of one floodplain, i.e. symmetrical vs. asymmetrical CCh results were compared. Finally, the first and the fourth case (series no. 2 and 7) were used to assess the models' ability to estimate the stage-discharge curve in a real case, i.e. when floodplains are rough and when it increases with the flow depth (this would correspond to the case of emergent vegetation on the floodplains).

In cases with smooth floodplains, the estimated value of Manning's coefficient of n = 0.01 $\mathrm{m}^{-1/3}\mathrm{s}$ was used, while the variation of the Manning's coefficient value with the depth for the rough floodplains was defined based on the experimental data [1]. The flow in all experimental series was uniform. Eight overbank depths were considered in each series.

Figure 15. Flood channel Facility in HR Wallingford [8]

Table 1. Geometry of FCF for different compound channel layouts [9]

Series No. (layout)	B [m]	b [m]	B/b [/]	m_{mc} [/]	Floodplain roughness
2	6.3	1.5	4.20	1	no
3	3.3	1.5	2.20	1	no
6	6.3	1.5	4.20	1	no
7	6.3	1.5	4.20	1	yes

Stage-discharge curves for different compound channel geometries

Symmetrical compound channels

Stage-discharge curves calculated using two presented methods (EDM and ISM) and two traditional methods (SCM and DCM) are compared to measured ones in Figure 16 a. Values of the ψ^t parameter were adjusted to achieve the best agreement with measurements. The value of the parameter $\psi^g = 0$, since the channel is prismatic. The optimal value of ψ^t parameter in EDM depends on the overall width of the CCh. In narrower channels ($B / b = 2.20$) it is greater ($\psi^t = 0.10$) than in wider channels ($\psi^t = 0.05$). In both cases, discrepancies from the measured values are within the measurement error – they are less than 5% (Table 2). On the other hand, the optimal value of the ψ^t parameter does not depend on the overall CCh width – it is 0.065. However, the maximal discrepancy exceeds 5% at low floodplain depths when $B / b = 2.20$ and at high floodplain depths when $B / b = 4.20$. The total discharge is over predicted by 8% in the former case, while in the latter case, the percentage is even greater – 13.6% (Table 2). In the remaining part of the stage-discharge curve, discharges predicted by ISM are slightly greater than those predicted by EDM (the differences amount to 4%). Traditional methods produce much greater discrepancies from the measured discharge values – SCM at low floodplain depths, when there is pronounced transfer of momentum between the main channel and the floodplain, under estimates discharge values up to 46%, while the DCM overestimate discharges by approximately 10%.

It is interesting to note that the two traditional methods produce much lower discrepancies at high relative floodplain depths $(H – h) / h > 0.31$ – for DCM they are below 7% and for SCM, they are below 5%. This amelioration of traditional methods' performance can be explained by the fact that the hydraulic conditions in the CCh cross section gradually tend to become uniform again at high floodplain depths. This justifies the application of traditional methods in discharge estimation only at very high relative floodplain depths.

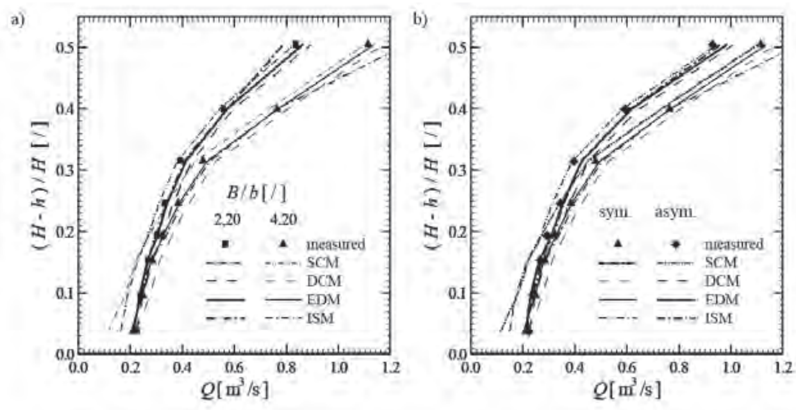

Figure 16. Comparison of calculated and measured stage-discharge curves for the entire cross-section: a) effect of floodplain width; b) effect of floodplain asymmetry (B / b = 4.20) [9]

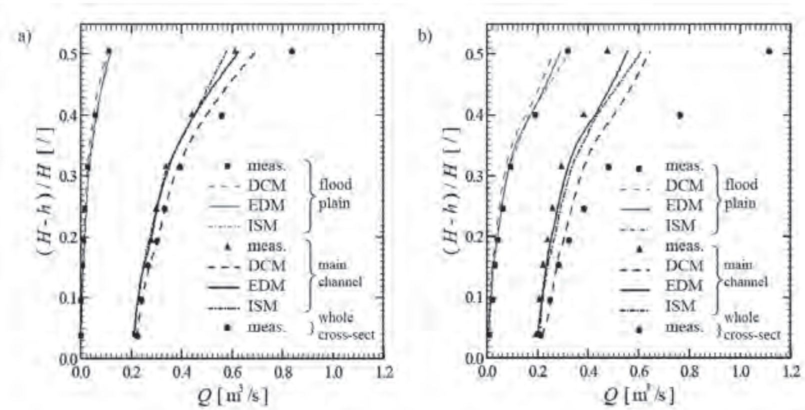

Figure 17. Comparison of calculated and measured discharge distributions between main channel and floodplain. Effect of floodplain width: a) B / b = 2.20; b) B / b = 4.20 [9]

Advantages of new methods in stage-discharge curve estimation become even more obvious when the discharge distribution between the main channel and floodplains is considered (Figure 17). This is particularly highlighted when it comes to the analysis of sediment transport and related processes on floodplains.

The estimation by the EDM and ISM is much better than that by the DCM. In narrower CChs, where the momentum transfer is more pronounced, the EDM performs slightly better than the ISM both in the main channel and floodplains (Table 2). Discrepancies for the DCM are 2.0–2.5 greater than those for the EDM.

Asymmetrical compound channels

Stage-discharge curves for symmetrical and asymmetrical CChs are compared in Figure 16 b. Both the plotted stage-discharge curves and the data from Table 2 confirm that both the EDM and the ISM satisfactorily estimate total discharge in the asymmetrical CCh – discrepancies range from 3 to 8%.

However, discrepancies for the DCM are 1.5 to 4.0 times greater than those for EDM and ISM. Results for the SCM show similar behaviour as for the symmetrical CChs – for $(H - h) / h < 0.31$ values of the discharge are considerably underestimated (20–30%), whereas at high relative floodplain depths they reduce to only 2%.

As far as the discharge distribution between the main channel and floodplains is concerned, new methods perform well, with the note the ISM now gives slightly better results than the EDM (Table 2). In this case, discrepancies for the DCM are 2.5–5.0 times greater than those for EDM.

Table 2. Ranges of relative discrepancies between calculated and measured discharges for the whole cross-section, main channel and floodplain [9]

	Series No.			
	2	3	6	7
Method	**Whole cross-section**			
SCM	–46.0 ÷ –1.0	–27.8 ÷ –0.7	–33.6 ÷ 1.8	–49.5 ÷ –25.5
DCM	3.5 ÷ 11.1	–0.5 ÷ 11.0	0.4 ÷ 14.2	2.4 ÷ 59.0
EDM	–4.4 ÷ 2.6	–5.1 ÷ 5.3	–5.5 ÷ 7.4	–4.5 ÷ 3.7
ISM	–48 ÷ 13,6	-7,9 ÷ 3.5	–3.3 ÷ 7.9	–4.6 ÷ 5.6
	Main channel			
DCM	13.0 ÷ 35.6	–0.3 ÷ 14.5	4.0 ÷ 20.4	8.0 ÷ 105.9
EDM	3.1 ÷ 16.8	–5.4 ÷ 4.1	–5.4 ÷ 8.8	0.8 ÷ 7.2
ISM	4.8 ÷ 28.6	–7.5 ÷ 1.4	–2.3 ÷ 7.6	1.5 ÷ 9.5
	Floodplain			
DCM	–47.6 ÷ –17.5	–24.5 ÷ –9.7	–11.0 ÷ 0.4	–79.4 ÷ –20.9
EDM	–28.6 ÷ –7.5	3.7 ÷ 33.3	0.4 ÷ 54.8	–69.7 ÷ –2.0
ISM	–52.4 ÷ 2.2	–6.8 ÷ 57.2	–3.7 ÷ 34.9	–71.1 ÷ –1.6

Stage-discharge curves for different floodplain roughness

The estimation of the stage-discharge curve with new methods for the case with rough floodplains (Figure 18) required adjustment of the ψ^t parameter values for a second time. It was found that the best agreement with measurements in the EDM was achieved for ψ^t parameter values between 0.05 and 0.10, while the optimal value for the ISM was the same as for CChs with smooth floodplains, i.e. $\psi^t = 0.065$. When the floodplain is rough, velocity gradients between the main channel and the floodplain is greater, and the advantages of new methods become even more obvious. Discrepancies do not exceed 6% (Table 2). The SCM underestimates total discharge by 25–50%, while the overestimate by DCM increases with the floodplain flow depth from 2.4 to 60%. If one neglects high discrepancies at very low floodplain depths (around 70%), when the measurement error is comparably high, it can be said that both the EDM and the ISM successfully assess the distribution between the main channel and floodplains – discrepancies for the main channel are less than 7.5% for the EDM and less than 9.5% for the ISM. On floodplains where the floodplain discharge does not exceed 20% of the total discharge, disagreement is greater, but it can be attributed to higher uncertainties in measured variables that result from difficulties in velocity measurements at the interface between the main channel and the floodplain.

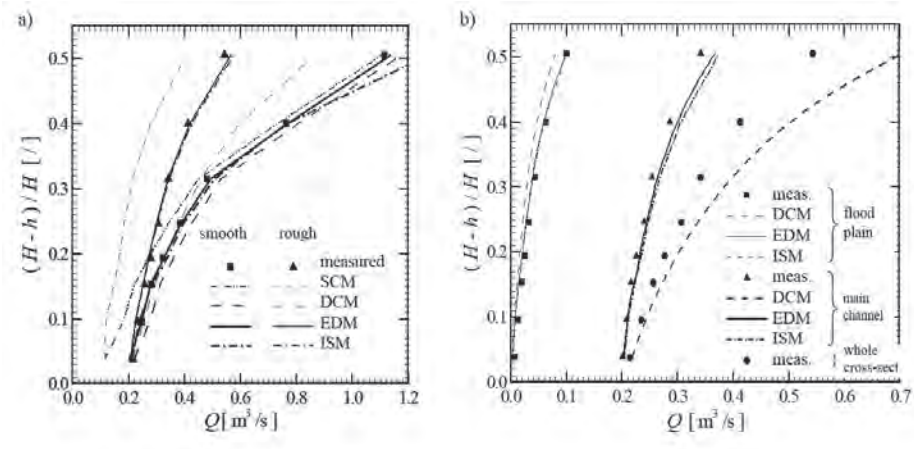

Figure 18. Comparison of calculated and measured stage-discharge curves. Effect of floodplain roughness (B / b = 4.20): a) total discharge; b) discharge distributions between main channel and floodplain [9]

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Radu Drobot

Water Retention and Localisation

Theoretical background: Continuity equation in a reservoir

The attenuation represents the reduction in peak flow following the storage of water in a reservoir. It may be represented by the following differential equation, which is the equation of continuity applied to a reservoir:

$$dQ = Q(t) - q(t) = \frac{dV}{dt} \tag{1}$$

It states that the inflow *Q(t)* in the reservoir during an interval of time minus the outflow *q(t)* during the same interval is equal to the change in storage.

In the following, the time horizon is discretised into intervals of duration Δt, indexed by *i* The discretisation of the equation (1) means the replacement of differentials by finite differences:

$$\Delta Q = \frac{\Delta V}{\Delta t} \tag{2}$$

Basic data

To undertake the flood wave attenuation in a reservoir the following data are necessary:

a) The flood discharges ***Q(t)***

The input can be a registered flood or a synthetic flood characterised by the probability of exceedance of the maximum discharge. The registered floods are processed in order to derive synthetic hydrographs, used for different hydraulic computations: deriving the flooded areas for medium or low probabilities of exceedance, designing the outlets of the dam and or establishing operation rules during exceptional floods.

A synthetic flood is usually defined by the following parameters (Figure 1):

$Q_{p\%}$ – maximum discharge corresponding to the probability of exceedance P%; it is obtained by statistical processing of the partial series of annual maximum discharges

T_{incr} – the increasing time of the flood hydrograph, from the beginning to the flood peak (as the average value of the most significative registered floods)

T_{tot} – total duration of the flood hydrograph (again as the average value of the most significative registered floods)

γ – the compactness (the shape) coefficient, computed also as the average value of the most significative registered floods:

$$\gamma = \frac{W}{Q_{P\%}^{max}\ T_{tot}} \tag{3}$$

where W is the flood volume:

$$W = \int_0^{T_{tot}} Q\ (t)\ dt \ \approx \sum_{i=1}^{N} Q_i\ \Delta t \tag{4}$$

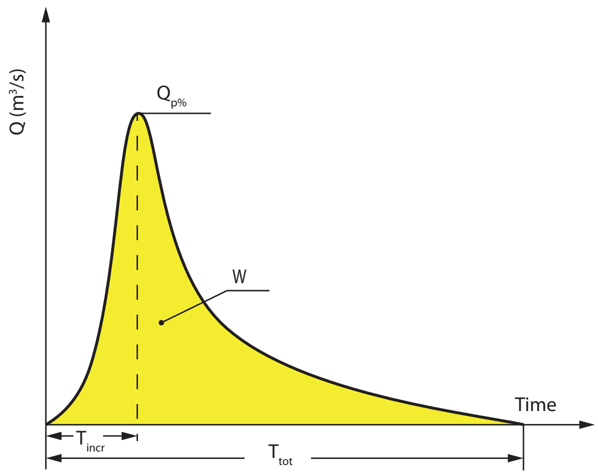


Figure 1. Parameters of the synthetic floods [5; 10]

Q_i represents discretised values of the discharges and Δt is the time step.

b) The elevation-storage **V(H)** relationship

The relationship $V(H)$ is derived by planimetering a topographic map at a convenient scale. If A_i and A_{i+1} are the areas delineated by the contour lines H_i and H_{i+1} of equal elevation above the mean sea level, the corresponding volume of water between these contour lines is:

$$\Delta V_{i,i+1} = \frac{1}{3}\ \left(A_i + \ A_{i+1} + \sqrt{A_i\ A_{i+1}}\right)(H_{i+1} - \ H_i) \tag{5}$$

The volume in the reservoir at the elevation H_{i+1} is obtained with the relationship:

$$V_{i+1} \ = \ V_i \ + \Delta V_{i,i+1} \tag{6}$$

A graphical representation of the elevation-storage curve is presented in Figure 2.

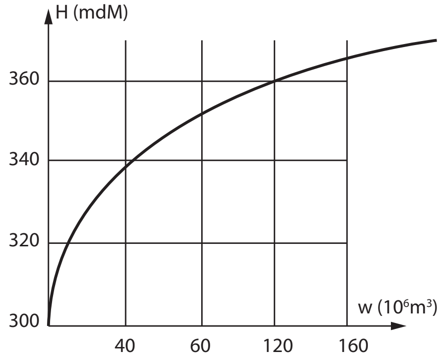


Figure 2. Elevation-storage curve [11]

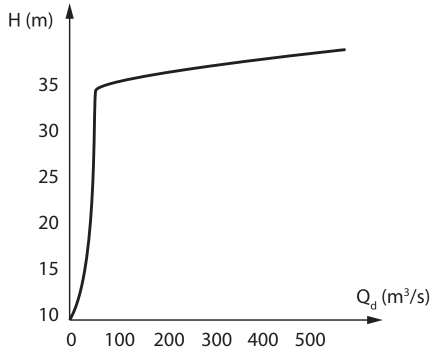


Figure 3. Elevation-outflow curve [11]

c) The elevation-outflow $\boldsymbol{q(H)}$ relationship

The elevation-outflow relationship (Figure 3) is obtained from hydraulic equations relating head and discharge. The discharge value for the bottom gates is differently computed according to the type of flow: free surface flow or pressure flow.

For the free surface flow through the bottom gates, the discharge computation is based on the well-known Chézy formula:

$$v = C\sqrt{RI} \tag{7}$$

$$Q = A\,C\sqrt{RI}$$

Where $C = \frac{1}{n} R^{1/6}$ – is the Chézy coefficient;

R – hydraulic radius
I – friction slope or hydraulic gradient
n – Manning's coefficient of roughness
A – area of the bottom gates

In case of pressure flow, the relationships used for velocity and discharge computation are:

$$v = \varphi\sqrt{2g\,H} \tag{8}$$

$$Q = A\,\varphi\sqrt{2g\,H}$$

Where H is the distance between the water level in the reservoir and the geometric centre of the gate. The increase of the discharge versus H is not very important due to the relatively low influence of $H^{1/2}$.

The flow from the reservoir above the spillway crest is computed with the following relationship:

$$Q = m\, b_c\, \sqrt{2g}\, h^{\frac{3}{2}} \tag{9}$$

Where m is a variable coefficient of discharge;
b_c – effective length of the crest (fluid contraction being considered)
h – total head on the crest including velocity of approach head

As opposed to the bottom gates, the increase of the water level in the reservoir above the spillway crest has a strong influence on the output discharges due to the power 3/2 affecting the head h.

Simulation models of flood wave attenuation in a reservoir

Using the values of the discharge at the beginning of each step of computation:

a) Equation (2) can be written in the following way:

$$Q_i - q_i = \frac{V_{i+1} - V_i}{\Delta t} \tag{10}$$

Where Q_i is the input flow in the reservoir at the time i while q_i is the output at the same moment; V_{i+1} and H_i are the water volumes in the reservoir at the time $i + 1$ and i respectively.

Consequently, the volume in the reservoir at the time i is:

$$V_{i+1} = V_i + (Q_i - q_i)\,\Delta t \tag{11}$$

The discharge $q_i = q(H_i)$, where H_i is the water level in the reservoir at the time i.

This approach can be used only when the time step Δt is small, being of the order of minutes. Otherwise, the errors of approximation of the flood volume are unacceptable.

b) Using the values of the discharge at the beginning and the end of each step of computation:

$$\frac{Q_i + Q_{i+1}}{2} - \frac{q_i + q_{i+1}}{2} = \frac{V_{i+1} - V_i}{\Delta t} \tag{12}$$

In this case, the volume V_{i+1} is:

$$V_{i+1} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right)\Delta t - \frac{q_i}{2}\,\Delta t - \frac{q_{i+1}}{2}\Delta t \tag{13}$$

Which can be written:

$$V_{i+1} = \widetilde{V_i} - \frac{q_{i+1}}{2}\Delta t \tag{14}$$

Where

$$\widetilde{V}_i = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \frac{q_i}{2} \Delta t \tag{15}$$

The left term $\widetilde{V}_i$ is a constant for each computation step and can be obtained based on the known values of the input Q_i and Q_{i+1} (prespecified values) and the output q_i (calculated in the previous step). Thus, in the equation (14) there are 2 unknowns: V_{i+1} and q_{i+1} An iterative procedure can be used to evaluate the outflow q_{i+1} based on elevation-storage $V(H)$ and elevation-outflow $q(H)$ relationships. The iterative procedure for computing the outflow q_{i+1} consists of the following steps:

First iteration:

$$q_{i+1}^{(0)} = q_i \tag{16}$$

$$V_{i+1}^{(0)} = \widetilde{V}_i - \frac{q_{i+1}^{(0)}}{2} \Delta t$$

$$H_{i+1}^{(0)} = H^{-1}\left(V_{i+1}^{(0)}\right)$$

$$q_{i+1}^{(1)} = q\left(H_{i+1}^{(0)}\right)$$

Second iteration:

$$V_{i+1}^{(1)} = \widetilde{V}_i - \frac{q_{i+1}^{(1)}}{2} \Delta t \tag{16'}$$

$$H_{i+1}^{(1)} = H^{-1}\left(V_{i+1}^{(1)}\right)$$

$$q_{i+1}^{(2)} = q\left(H_{i+1}^{(1)}\right)$$

The iterations continue until one or all criteria of stop are verified:

$$\left|V_{i+1}^{(k)} - V_{i+1}^{(k-1)}\right| \leq \varepsilon_V \tag{17}$$

$$\left|H_{i+1}^{(k)} - H_{i+1}^{(k-1)}\right| \leq \varepsilon_H$$

$$\left|q_{i+1}^{(k)} - q_{i+1}^{(k-1)}\right| \leq \varepsilon_q$$

Where ε_V, ε_H and ε_q represent the desired precision of computation. The values obtained during the last iteration represent the values of the output q_{i+1}, volume V_{i+1} and water level in the reservoir H_{i+1}:

$$q_{i+1} = q_{i+1}^{(k)}\ ;\ V_{i+1} = V_{i+1}^{(k)};\ \ H_{i+1} = H_{i+1}^{(k)} \tag{18}$$

This approach allows large time steps, as opposed to the first approach based on the approximation of the water volume and input values at the time *i*.

c) Deriving a storage-outflow function ***E(H)***.

For this purpose, equation (13) is written in the following form:

$$V_{i+1} + \frac{q_{i+1}}{2}\Delta t = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right)\Delta t - \frac{q_i}{2}\Delta t \tag{19}$$

The left side of the relation (19) is denoted by $E(H)$ and can be derived based on elevation-storage $V(H)$ and elevation-outflow $q(H)$ relationships. Thus, for different values of H the corresponding values of $E(H)$ are computed (Figure 4):

$$E(H) = V(H) + \frac{q(H)}{2}\Delta t \tag{20}$$

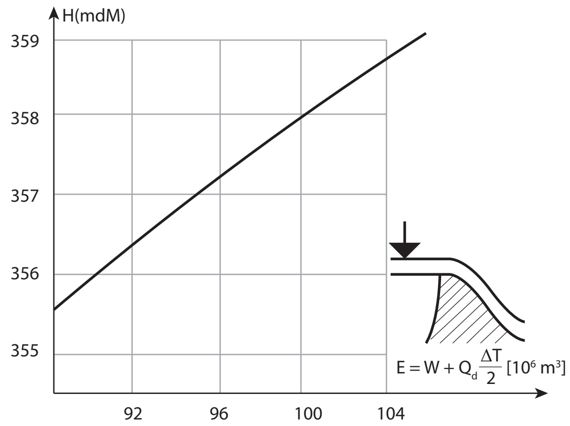


Figure 4. Elevation-outflow curve [11]

It has to be mentioned that the function elevation-storage $E(H)$ depends on the hypothesis of outlets operation rules. Consequently, corresponding to a proposed operation scenario of the outlets a specific graph $E(H)$ is obtained. The outflow q_{i+1} is derived as follows.

Based on known values of V_i and q_i (the volume in the reservoir and the outflow respectively at the beginning of the computation step) and on the inflow Q_i and Q_{i+1} at the time *i* and *i* + 1 the right term of the relation (10) is computed; the result represents in fact the value of $E(H_{i+1})$:

$$E(H_{i+1}) = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right)\Delta t - \frac{q_i}{2}\Delta t \tag{21}$$

Using the function $E(H)$, the value H_{i+1} representing the water level in the reservoir at the end of the computation step (time *i* +1) is obtained either graphically or numerically.

Based on the elevation-storage $V(H)$ and elevation-outflow $q(H)$ relationships, the values V_{i+1} and q_{i+1}, representing initial values for the next step are derived.

The results of the flood attenuation in a reservoir are the time series: $\{V_i\}$, $\{H_i\}$ and $\{q_i\}$which are graphically represented for visualisation purposes.

Considering individual operation rules for the outlets

Till now it was supposed that the output q_i is obtained considering all outlets open, which is not the case in the current practice. During the reservoir exploitation, depending on the water level in the reservoir some outlets (like the bottom gates) can be open, then closed and open again. Others will become active only when the water level exceeds the crest of the spillways. Consequently, the outlets are grouped into classes of identical operation, all outlets in a class having the same operation rules. The output q_i can be expressed as:

$$q_i = \sum_{j=1}^{n} s_j\ (H_i)\ q_j(H_i) \tag{22}$$

Where: n is the number of the classes of identical operation of the outlets. The outlets are differentiated not only by their structural but also by functional (operational) characteristics, outlets of the same structural type belonging to different classes. For example, if a dam has 4 bottom gates, 2 of them can belong to a class according to the operation rules while the other 2 can belong to another class.

H_i – is the water level at time i

$q_i(H_i)$– the outflow delivered by only one outlet from the class j at the time i

$s_j(H_i)$ – a state variable indicating how many outlets are in operation at the level Q_i

$s_i(H_i) = 0$ – if no outlet from the class Q_i is in operation

$s_j(H_i) = 1$ – if one outlet from the class Q_i is in operation, etc.

Considering the relation (22) for the output q_i, the equation (12) becomes:

$$\frac{Q_i + Q_{i+1}}{2} - \sum_{j=1}^{n} \frac{s_{i,j}\, q_{i,j} + s_{i+1,j}\, q_{i+1,j}}{2} = \frac{V_{i+1} - V_i}{\Delta t} \tag{23}$$

In this case, the volume V_{i+1} is:

$$V_{i+1} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \sum_{j=1}^{n} \frac{s_{i,j}\, q_{i,j}}{2}\, \Delta t - \sum_{j=1}^{n} \frac{s_{i+1,j}\, q_{i+1,j}}{2} \Delta t \tag{24}$$

Which can be written:

$$V_{i+1} = \widetilde{V_i} - \sum_{j=1}^{n} \frac{s_{i+1,j}\, q_{i+1,j}}{2} \Delta t \tag{25}$$

Where

$$\widetilde{V_i} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \sum_{j=1}^{n} \frac{s_{i,j}\, q_{i,j}}{2}\, \Delta t \tag{26}$$

In this simulation model, having the operation rules of the outlets previously defined, at each time *i* one obtains the state variables of the system: the volumes $\{V_i\}$ and the water levels $\{H_i\}$ in the reservoir, as well as the outlets $\{q_{ij}\}$ of each class and the total output discharges $\{q_i\}$.

Simulation-optimisation model

A simulation-optimisation model contains:

a) The objective function Z to be optimised.

The maximum value of the outflow should be limited at the discharge capacity of the river bed q_{adm}, without inundating the floodplain. At the same time, the flood wave should transit the reservoir as quickly as possible to allow the attenuation of successive floods. Consequently, the output discharges should be as close as possible to the value of q_{adm} and the objective function of the model is:

$$\text{(min) } Z = \sum_{i=1}^{N} \left(\sum_{j=1}^{n} s_{i,j}\, q_{i,j} - q_{adm} \right)^2 \tag{27}$$

b) Equation (24) of the dynamic of the attenuation process, which during the optimisation allows the computation of the term $s_{i,j}\, q_{i,j}$.

c) Constraints concerning the state variables or the output variables, as follows:

- limitation of the water level in the reservoir to a maximum value H_{max}:

$$H_i < H_{max} \tag{28}$$

- limitation of the output fluctuations:

$$|q_i - q_{i-1}| < \varepsilon_q \tag{29}$$

where ε_q is the maximum allowed difference between two successive values of the outflow

- limitation of the hydraulic gradient of water level decrease or increase in the reservoir in order to prevent slope landslides:

$$\frac{\Delta H}{\Delta t} < G_{max} \tag{30}$$

here G_{max} is the maximum allowed gradient established during the design phase

- prescribing a target volume or level in the reservoir at the end of attenuation process:

$$\left|V_N - V_f\right| < \varepsilon_V \tag{31}$$

$$|H_N - H_f| < \varepsilon_H \tag{32}$$

where N is the number of computation steps, V_N and H_N are the volume and the water level at the end of the computation, while V_f and H_f are the target values in order to satisfy both the water users and to assure the flood protection in case of successive floods.

The decision variables are represented by the water levels between which the outlets are active. For instance, in Figure 5 a possible scenario of outlets operation is presented in the case of a dam with uncontrolled spillways. The bottom gates become active being open at the level Ht1, which can be the Normal Retention Level or a lower level. These gates are closed at the level Ht2, in order to protect the downstream area from flooding by avoiding the superposition of the outflows discharged simultaneously by the bottom gates and the spillways. Finally, the bottom gates are open again at the level Ht3, despite the large outflows downstream the dam in order to protect the dam of overflowing.

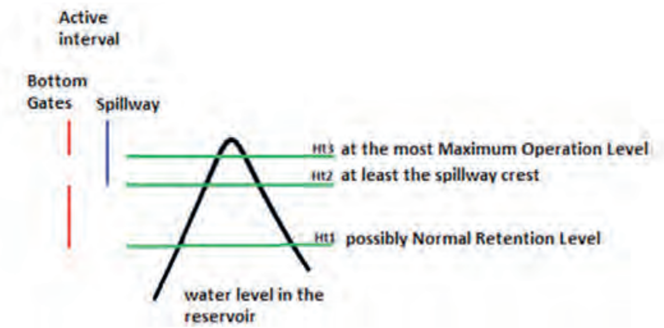


Figure 5. Outlets operation rules [8]

The decision variables are obtained by optimisation. Because the objective function (27) cannot be expressed analytically, the optimisation process is based on algorithms like Nelder-Mead method (downhill method) or on genetic algorithms which do not need the computation of the derivatives. Although fast, Nelder-Mead can stop the search in a local optimum. On the contrary, Genetic algorithms need more computational time but they find the global optimum.

Case studies of flood attenuation

Operation of Dridu Dam outlets

The Dridu dam is located in a low area of the Ialomiţa river basin, 800 m upstream the confluence with Prahova River. The Dridu retaining wall is represented by a concrete spillway dam and a front earth dam that continues with lateral dams called inadequately dykes. The height of the dam is 20 m, and the volume of the water reservoir is of 45 hm^3.

During the 24 years of operation some atypical phenomena and incidents were observed. Consequently, the Normal Pool Level was decreased from 69.20 m to 68 m above the Black Sea level. The minimum operation level to guarantee water supply to the population is 63 m, while the minimum retention level to produce hydropower is 65 m. The spillways are controlled by radial gates. The gates are partially lifted in order to regulate the rate of flow. A family of rating curves is derived depending on the opening *e* between the crest of the spillway and the lower part of the gate (Figure 6).

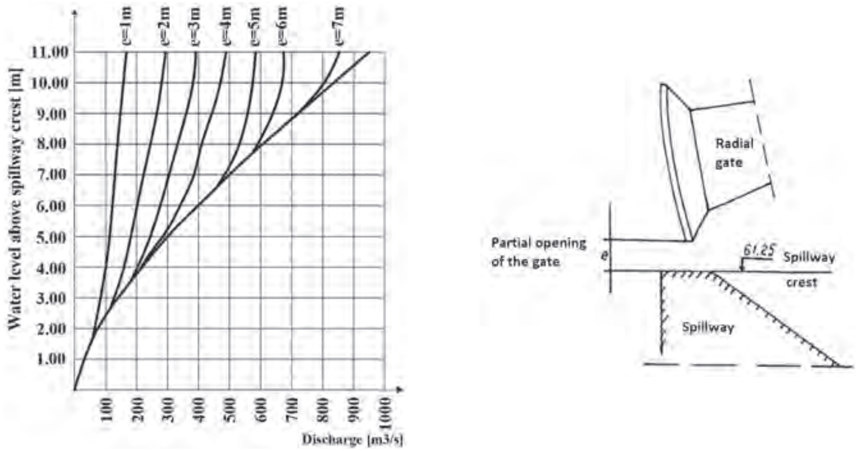
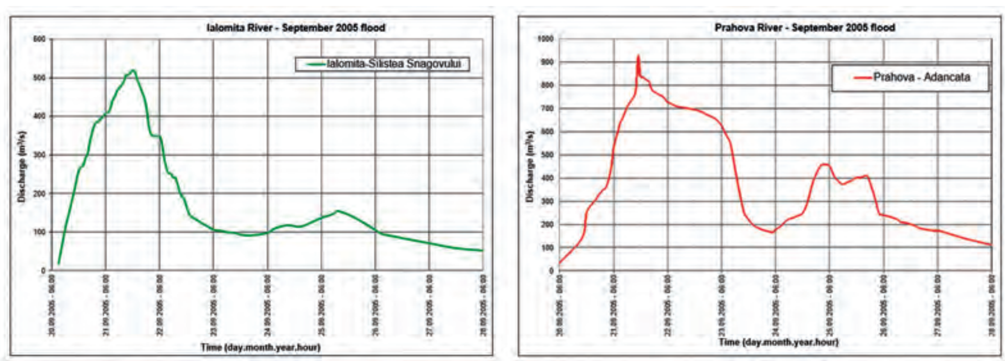


Figure 6. Rating curve for a radial gate [13]

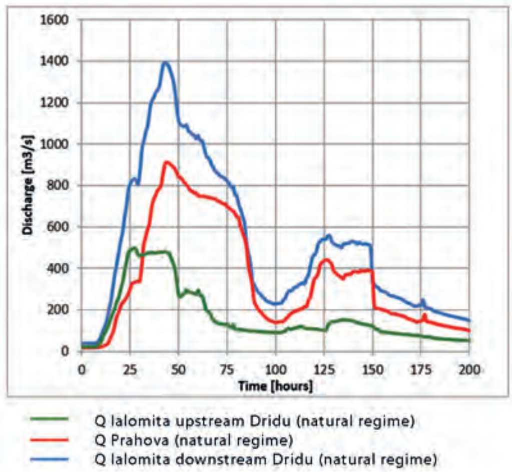
In 2005, floods occurred in the whole country. Between 20–30 of September, the flood produced on the Ialomița River reached a maximum discharge of about 500 m^3/s, while the flood on its tributary had a maximum value of about 900 m^3/s (Figure 7). As previously mentioned, the dam is located on the Ialomița River, while the flood on the Prahova River cannot be controlled.



Ialomița River – Siliștea Snagovului gauge station *Prahova River – Adâncata gauge station*

Figure 7. Floods on 20–30 September 2005 on Ialomița and Prahova Rivers [13]

In natural regime (no dam), the flood hydrograph downstream the confluence of the Prahova River with the Ialomița River, would look like in Figure 8, and the maximum discharge would have been almost 1,400 m^3/s. The operation of the dam outlets should be done in such a way to avoid the superposition of the maximum discharges. The solution is to put into operation the bottom gates at the very beginning of the flood on the Ialomița River. In this way, the additional storage volume created in the reservoir will be used to retain the flood peak on the Ialomița River, when the discharges on the Prahova River are at maximum. For this purpose, all the radial gates are gradually elevated to 1.5 m, then to 3 m. During the peak on the Prahova River the gates are let down to 1.75 m and finally are closed for 15 hours (Figure 9). In the following, only one gate is open at 1.75 m on the recession limb of the flood. This manoeuvre is made to prevent the increase of the water level in the reservoir over 68.00 m, representing the safe level in operation. At the end of the attenuation process, the water level in the reservoir is brought at 68.00 m in order to assure the necessary reserve for water users. The maximum discharge downstream the confluence is about 980 m^3/s (Figure 10).



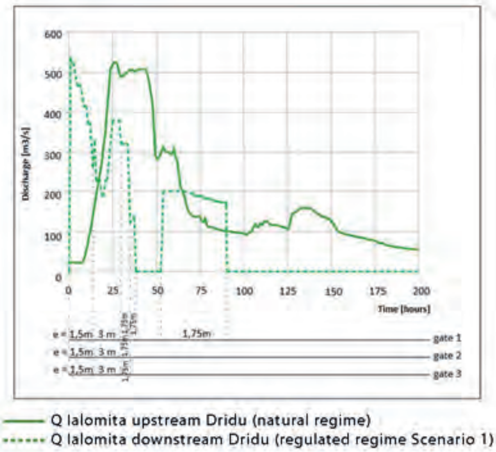


Figure 8. Superposition of the floods downstream the confluence in natural regime (no Dridu dam) [13]

Figure 9. Gates operation during the flood on the Ialomița River (Scenario 1) [13]

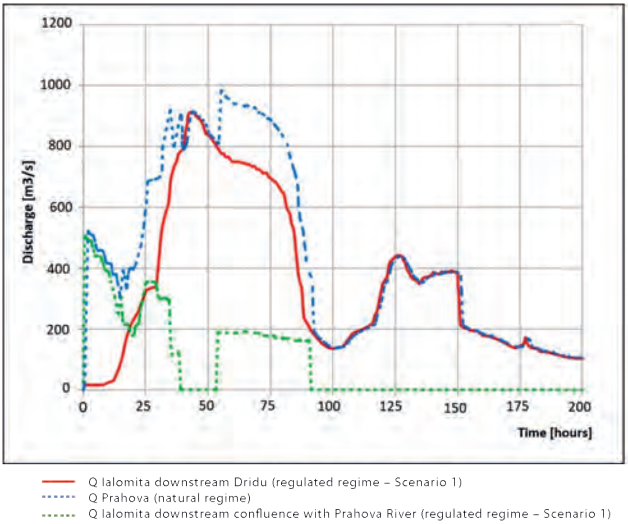


Figure 10. Superposition of the floods downstream the confluence in regulated regime (Scenario 1 of operation) [13]

A more sophisticated operation of the gates is needed to obtain a flat shape of the maximum discharges downstream the confluence (Figures 11 and 12). In this Scenario the safe water level in the reservoir is exceeded for a short period, which is however acceptable.

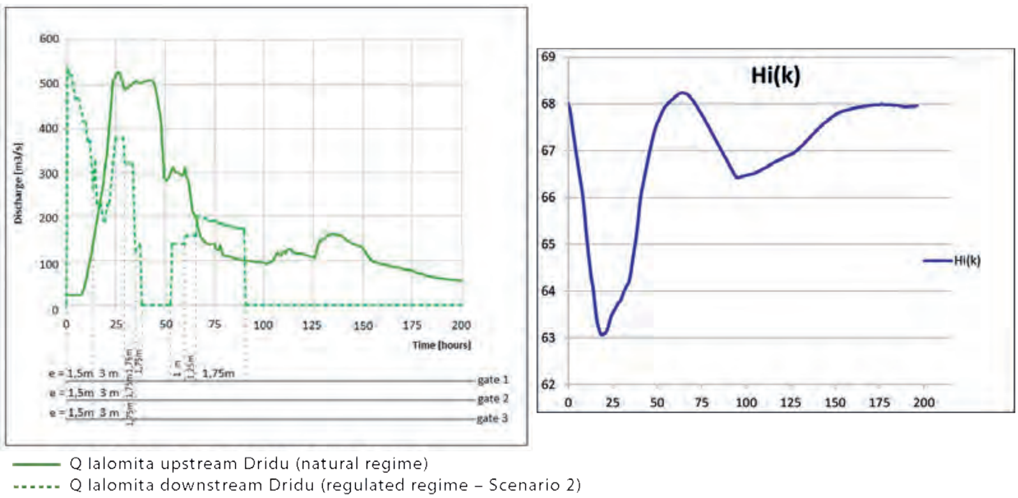


Figure 11. Gates operation during the flood on the Ialomița River (Scenario 2) [13]

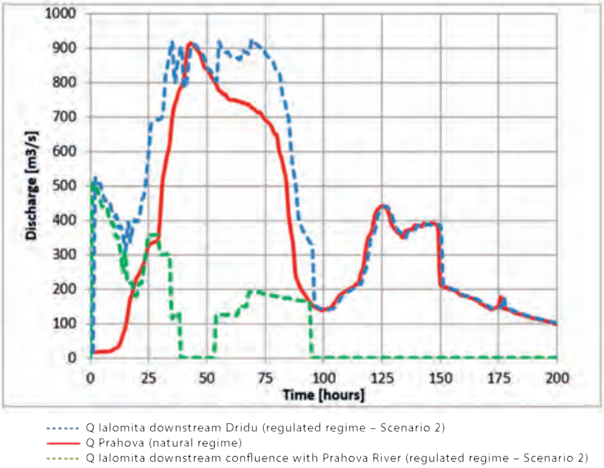


Figure 12. Superposition of the floods downstream the confluence in regulated regime (Scenario 2 of operation) [13]

The maximum discharge downstream the confluence in Scenario 2 is practically flat, being reduced from 1,400 m^3/s to 920 m^3/s (Figure 13).

The small height of the dam as well as the constraints imposed to the maximum water level by safety reasons limits an advanced attenuation of the flood downstream the confluence.

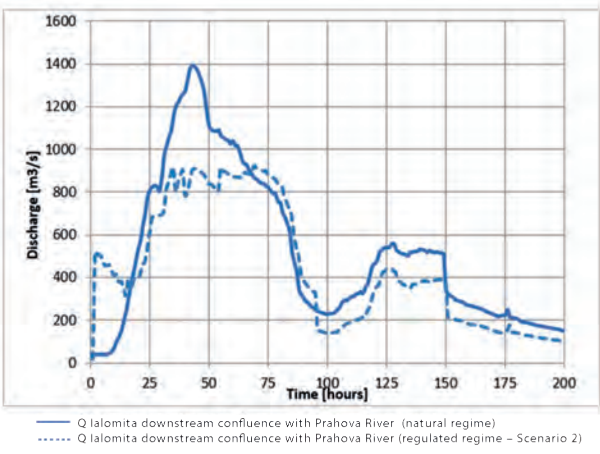


Figure 13. Superposition of the floods downstream the confluence in natural regime (Scenario 2) versus superposition in natural regime [13]

Flood management in the Jijia River basin

Joint operation of the reservoirs

The hydrotechnical development of the Jijia River (Figure 14) includes 4 permanent frontal reservoirs, from which the Ezer reservoir is situated on the Jijia River, Cătămărăști and Drăcșani reservoirs on the Sitna River, and Hălceni reservoir on the Miletin River. Câmpeni reservoir, located on the Miletin River upstream Hălceni reservoir is a frontal non-permanent reservoir. The Hălceni polder is just upstream Hălceni reservoir. At the same time, in the lower part of the Jijia River 6 polders at Țigănași were designed as the ultimate control structures for flood attenuation. The polders are symmetrically located, 3 polders being placed on the left side of the Jijia River and the other 3 polders in mirror on the right side of the river.

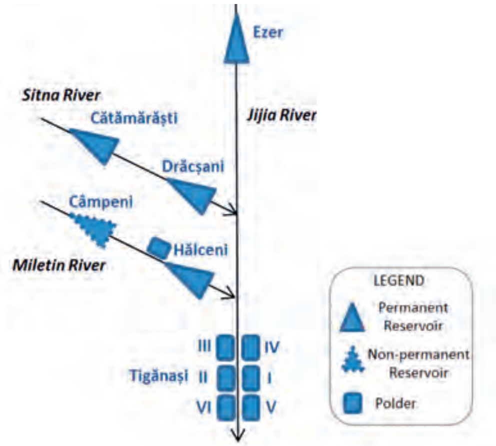


Figure 14. Jijia hydrotechnical development [12]

The maximum flood control volume of the reservoirs (4 x frontal permanent, 1 x frontal non-permanent, 1 x lateral non-permanent) is of about 77.35 mil. m^3, while the volume of the 6 polders at Țigănași is approximately 80.3 mil. m^3.

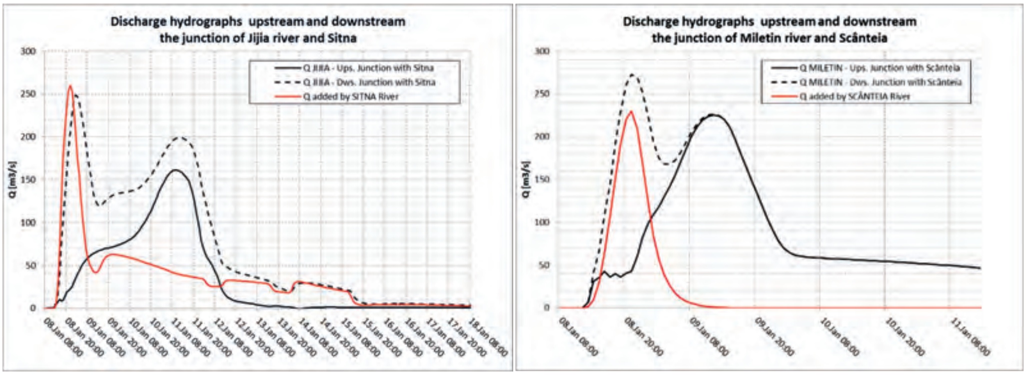
The software used for numerical simulations is Mike 11 by DHI. The specific numerical model is able to consider as input:

– either the precipitation on the entire river basin followed by a coupled hydrological-hydraulic modelling – in this case the main modules used are: Rainfall-Runoff module (RR), Hydrodynamic module (HD) and Structural Operation module (SO)
– or directly the input hydrographs due to small tributaries of the Jijia River and its main tributaries: Sitna River and Miletin River as upper boundary conditions – in this case only the following main modules are used: Hydrodynamic module (HD) and Structural Operation module (SO)

The main steps in mathematical modelling are:

- schematisation of the river network, by creating the topologic model of the network
- integration of the Digital Terrain Model (DTM), of both the floodplain area and the river bed
- database creation, which should include detailed information related to the river basin, hydraulic network, hydraulic structures and their operation, meteorological and hydrological data
- setting up the hydraulic model and calibration of the hydraulic parameters of the river bed using steady state simulations
- choosing the most significant floods
- analysing the meteorological data which generated significant floods
- calibration of the hydrological parameters, based on physiographical characteristics of the river basin and previous values of hydraulic parameters for the river bed – the hydraulic parameters for the floodplain were proposed according to Landcover information concerning the land use
- validation of the hydrological and hydraulic parameters using other registered floods
- statistical processing of the maximum annual precipitation for different duration obtained from the meteorological and/or hydrometric stations
- evaluation of the synthetic floods components, by keeping the same probability of exceedance along the river stretch between two successive hydrometric stations
- flood propagation in the modified hydraulic regime due to existing hydraulic structures
- assessment of the efficiency of the existing operation rules of the reservoirs using synthetic floods corresponding to the following probabilities of exceedance of the maximum discharge: 10%, 1% and 0.1%
- improvement of the coordinated operating rules by a trial and error procedure

The concentration time of the tributaries is smaller than the concentration time of the main river basin, leading to a different moment of the peak discharges. As a result, the flood downstream the confluence has two peaks and the maximum discharge is smaller than in the case of superposition of the maximum discharges (Figure 15). In other cases, due to the spatial variability of the precipitation it is not possible to avoid the superposition of the flood downstream the reservoirs with the floods due to tributaries.



a) Jijia River and Sitna tributary *b) Miletin River and Scânteia tributary*

Figure 15. Discharge hydrographs upstream and downstream the confluence of the main river with a tributary [8]

The simulations showed that the effect of the reservoirs is important, due to large flood protection volumes. Still, the bottom gates are able to evacuate relative low discharges and the reservoirs spillways have no gates. Under these conditions the possibility to improve the operation rules of the reservoirs is quite limited, the only chance to increase the role of the reservoirs being the release of the water through the bottom gates immediately after the early warning of an imminent flood. In Figure 16, some examples of the flood waves upstream the Cătămărăști reservoir and the flood hydrographs downstream reservoirs for different timing of bottom gates opening are presented. The maximum discharge is decreased from 176 m^3/s to 25–80 m^3/s depending on when the bottom gates are open.

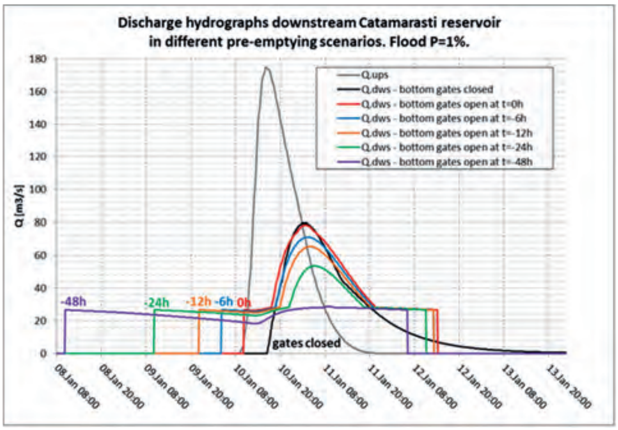


Figure 16. Effect of bottom gates opening based on early warning of a flood [8]

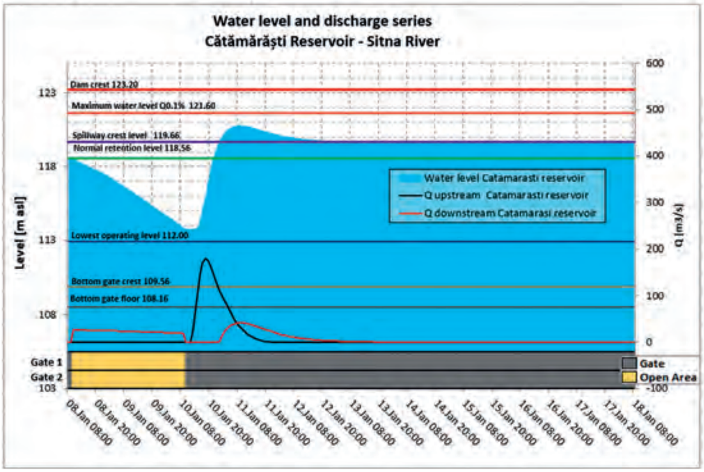
In order to understand how important it is to jointly operate both reservoirs on the Sitna River, other scenarios were imagined, taking into account all the possible combinations of operating each reservoir. The following scenarios of operating the bottom gates of both reservoirs, Cătămărăşti and Drăcşani, were examined: 1. Keeping permanently the gates closed; 2. Opening the gates at the time of arrival of the flood into the reservoir; 3. Opening the gates 24 hours before the arrival of the flood; and 4. Opening the gates 48 hours before the arrival of the flood.

Some tributaries (Morişca, Dresleuca, Burla) of the Sitna River bring important contributions during the flood period. However, the attenuation on the Sitna River due to the significant retention capacity of the Cătămărăşti and Drăcşani reservoirs is important (Table 1), with a reduction for the flood 1% at the confluence with Jijia River from 380 m^3/s (natural regime) to 152–182 m^3/s (regulated regime). Still, the potential to optimise the operation rules in order to minimise those values is quite limited due to the absence of flap gates at spillways.

The operation rules of the Cătămărăşti and Drăcşani reservoirs are presented in the lower part of Figure 17, where the yellow strip means the gates are open, while the grey strip corresponds to closed gates. On the same figure, the input and output hydrographs from the reservoirs are shown, as well as the water level evolution in the reservoirs.

Table 1. Maximum discharges along the Sitna River – Flood 1% (compiled by the author)

No.	Cross-section	Q Max [m^3/s]
1	Q upstream Catamarasti reservoir	181
2	Q downstream Catamarasi reservoir	42
3	Q gauge station Catamarasti	54
4	Q upstream Morisca tributary	58
5	Q downstream Morisca tributary	123
6	Q upstream Dresleuca tributary	125
7	Q downstream Dresleuca tributary	181
8	Q upstream Dracsani reservoir	180
9	Q downstream Dracsani reservoir	100
10	Q upstream Burla tributary	100
11	Q downstream Burla tributary	150
12	Q gauge station Dracsani	150
13	Q gauge stationTodireni	182



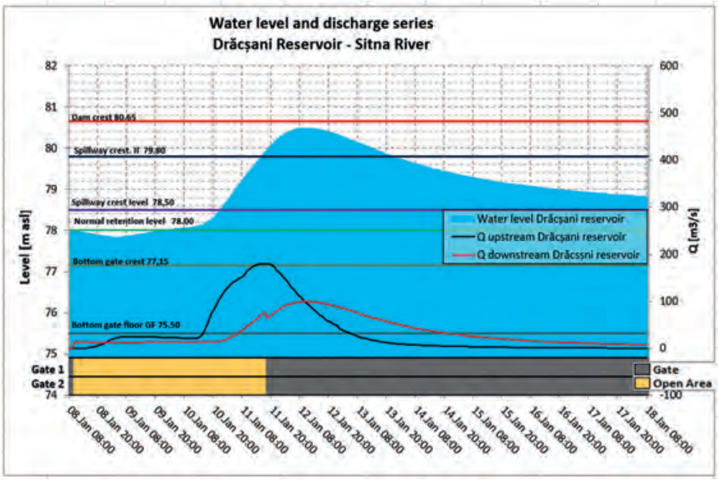


Figure 17. Flood attenuation in the Cătămărăşti and Drăcşani reservoirs [8]

As it can be seen in Figure 18, the best joint operation rule corresponds to the situation when the bottom gates are kept closed at Cătămărăşti, while decreasing the water level in the Drăcşani reservoir starts 48 hours before the flood arrival by opening the bottom gates. A common pattern identified in the simulations is: the best efficiency in terms of maximum discharge at the Sitna River upstream the confluence with the Jijia River (values between 152–155 m^3/s) is obtained when the Drăcşani reservoir water level starts to be lowered 48 hours earlier of the flood occurrence in the reservoir, whatever rules are chosen for Cătămărăşti.

The worst solution is to open the bottom gates at the Cătămărăşti reservoir 48 hours before the flood arrival, opening the Drăcşani reservoir only at the arrival time of the flood and keeping the bottom gates open during the entire flood event. The maximum discharge in this case is 182 m^3/s at the confluence of the Sitna River with the Jijia River.

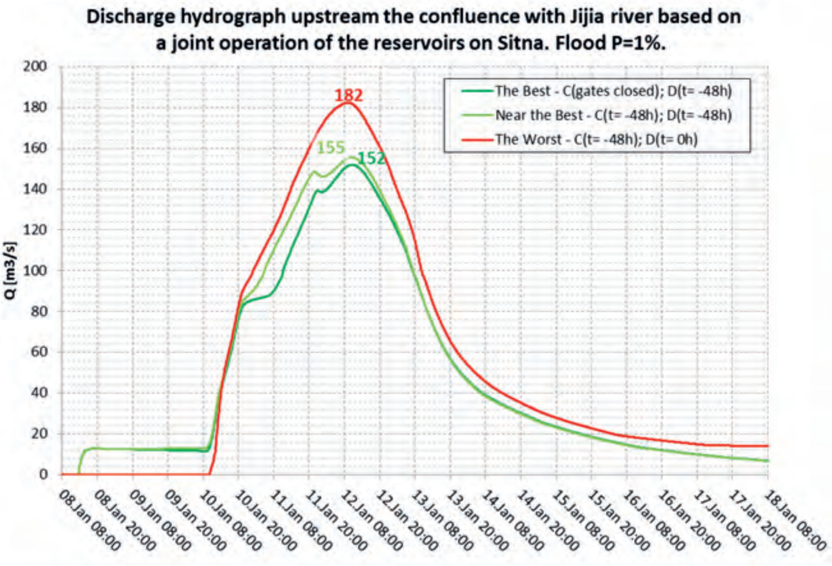
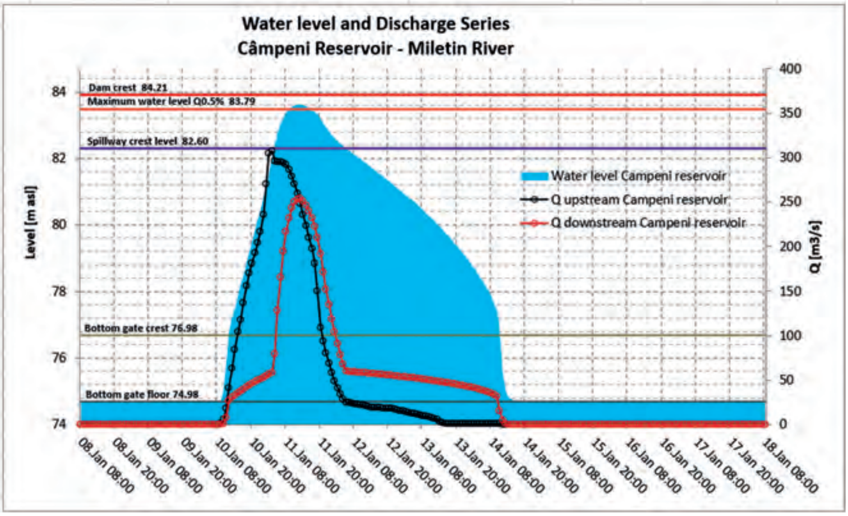


Figure 18. Discharge hydrograph upstream the confluence with the Jijia River for different operation scenario of the bottom gates [8]

A similar approach was used for the operation of the reservoirs on the Miletin River. The evolution of the maximum discharges along the Miletin River, considering the attenuation in the reservoirs as well as the influence of the floods on the tributaries is presented in Table 2. On this important tributary of the Jijia, there are two reservoirs: Câmpeni and Hălceni. The last one is, in fact, a system of two reservoirs, compound by a permanent reservoir and a polder, situated immediately upstream the reservoir, on the left bank of the river. Câmpeni reservoir is non-permanent, and therefore there is not possible to operate the bottom gates. In this case, a joint operation on the Miletin River is not possible. Hălceni is situated only at 6 km upstream the confluence with the Jijia River, thus, the discharge immediately downstream this reservoir can be considered the contribution of the Miletin River to the Jijia River.

Table 2. Maximum discharges along the Miletin River – Flood 1% (compiled by the author)

No.	Cross-section	Q Max [m^3/s]
1	Q upstream Campeni reservoir	307.7
2	Q downstream Campeni resevoir	253.3
3	Q upstream Scanteia tributary	246.9
4	Q downsteam Scanteia tributary	269.9
5	Q upstream Recea tributary	308.6
6	Q downstream Recea tributary	327.4
7	Q upstream Halceni reservoir	179.6
8	Q downstream Halceni reservoir	119.8
9	Q upstream Confluence, Ui@. River	119.5



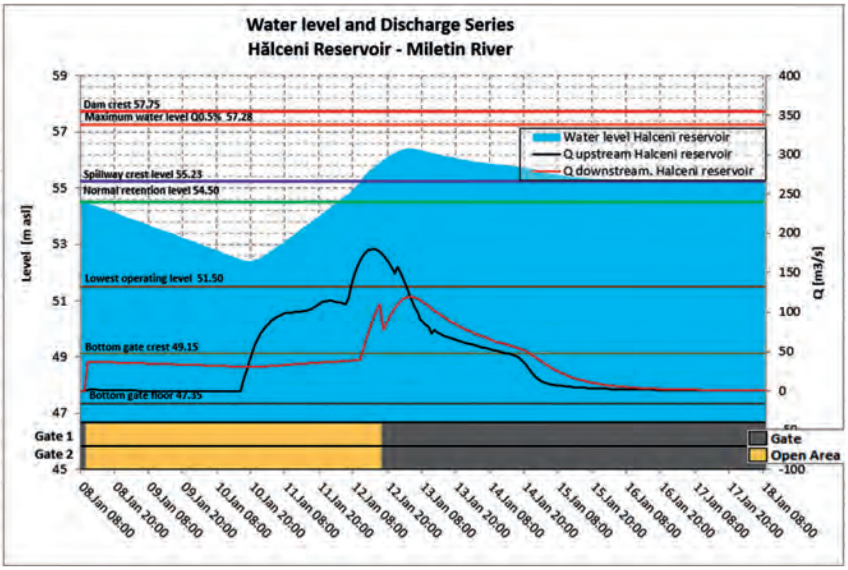


Figure 19. Flood attenuation in the Câmpeni and Hălceni reservoirs [8]

The operation rules of the Hălceni reservoir are presented in the lower part of Figure 19, with the same meaning: the yellow strip means the gates are open, while the grey strip corresponds to closed gates. On the same figure, the input and output hydrographs from the reservoirs are shown, as well as the water level evolution in the reservoirs. Câmpeni reservoir is a non-permanent storage reservoir and the bottom gates are always open.

Hălceni polder and Hălceni reservoir (see Figure 14) have an important role in flood attenuation: the polder diminishes the maximum discharge from 194 m^3/s to 108 m^3/s, and depending on how efficient the bottom gates of the frontal dam are operated, the downstream maximum discharge can vary from 54 m^3/s to 95 m^3/s (Figure 20). The difference between the best operation rule of the bottom gates and the worst one is only 40 m^3/s in this case.

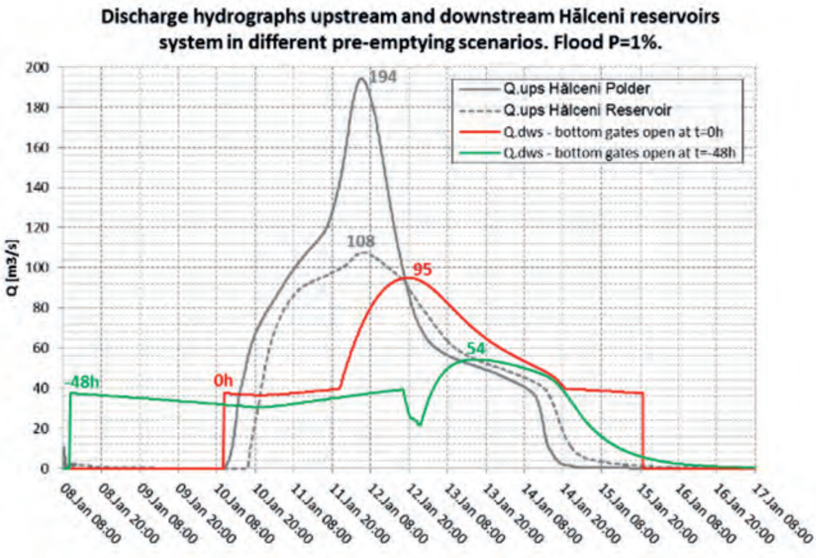


Figure 20. Discharge hydrograph upstream and downstream of the Hălceni reservoirs system [8]

The estimated maximum discharge in natural regime at the confluence of the Miletin River with the Jijia River for the flood 1% is 360 m^3/s. The attenuation due to the Câmpeni and Hălceni reservoirs is significant, the maximum discharge being reduced from 360 m^3/s to 54–95 m^3/s, depending on the bottom gates operation of the Hălceni reservoir.

Modelling polders effect

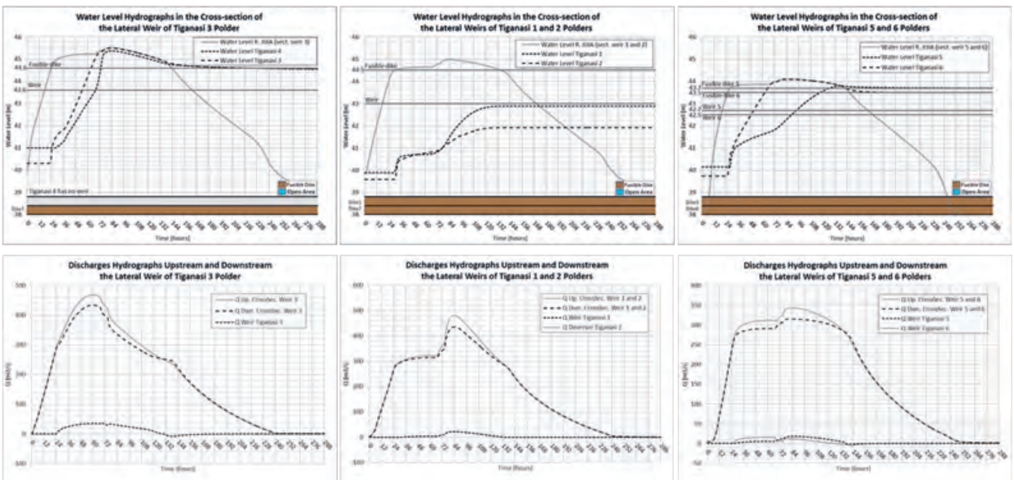
According to the common agreements signed between water authorities in Romania and the Republic of Moldova, the maximum discharge on the Jijia River upstream the confluence with the Prut River should be limited to a maximum of 220 m^3/s. Consequently the maximum discharge on the Jijia River upstream the confluence with the Bahlui River is limited to a maximum of 220 m^3/s. It has to be mentioned that this discharge is a little bit higher than the maximum discharge corresponding to 5% probability of exceedance at the Victoria gauge station, which is the last station upstream the confluence with the Bahlui river.

The catchment area at the Victoria gauge station is 3,643 km^2, the peak time of the floods in natural regime is 60 hours, the total flood duration is 240 hours, while the shape coefficient is 0.45. The maximum discharge corresponding to 1% probability of exceedance in natural regime is 350 m^3/s, and for 0.1% probability of exceedance is 575 m^3/s. According to the above-mentioned data, the threshold discharge of 220 m^3/s can only be achieved by the attenuation of the flood waves in reservoirs or polders. In the lower part of the Jijia River 6 polders at Țigănași were designed as the ultimate control structures to maintain the maximum discharge under the threshold value of 220 m^3/s. The polders are symmetrically located, 3 polders being placed on the left side of the Jijia River and the other 3 polders in mirror on the right side of the river.

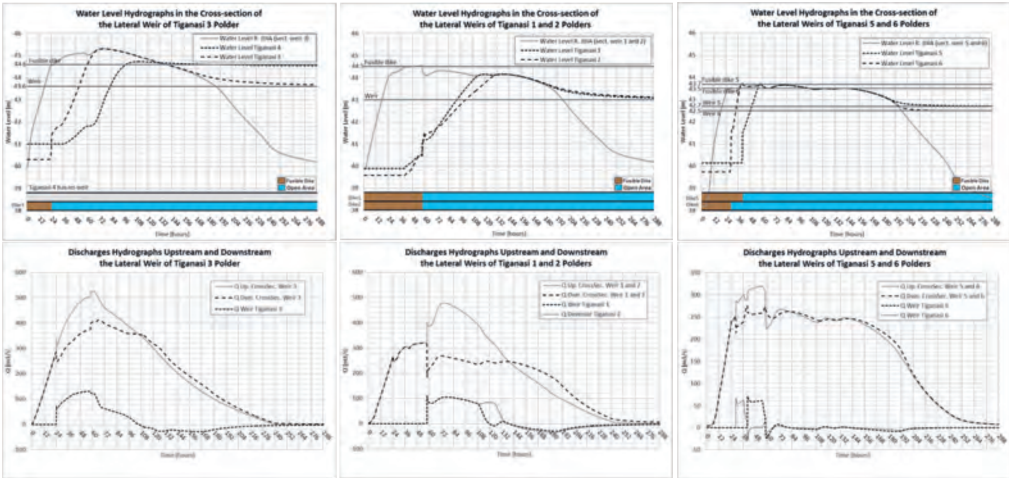
The maximum flood control volume of the reservoirs (4 x frontal permanent, 1 x frontal non-permanent, 1 x lateral non-permanent) is of about 77.35 mil. m^3, while the volume of the 6 polders is approximately 80.3 mil. m^3. The software used for simulations is Mike 11 by DHI.

The effect of the operation rules of the reservoirs is important for flood control mainly on the tributaries (Sitna and Miletin). After the confluence with the Jijia River, their effect is counterbalanced by the floods produced on the Jijia River or its tributaries. The floods from upper Jijia could be important due to the limited effect of the Ezer reservoir (which is very upstream) and the significant input of the tributaries and inter-basins downstream this reservoir. It means that the permanent reservoirs on the Sitna and Miletin rivers cannot mitigate the floods at the level of the whole basin in such a way to maintain the maximum discharge at Victoria station under the threshold value.

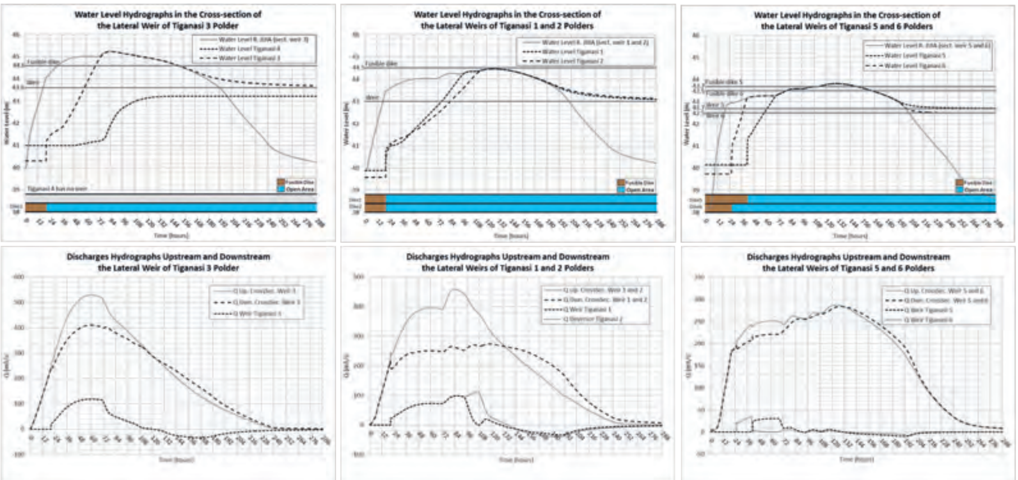
Under these conditions, the Țigănași polders are the most downstream hydrotechnical works able to modify the flood waves coming from upstream and to observe the threshold value imposed by the international convention between Romania and the Republic of Moldova.



a)



b)



c)

Figure 21. Water levels and discharges according to the analysed scenarios [8]

a) Scenario no. 1; b) Scenario no. 2; c) Scenario no. 3

The simulations reproduced the possible behaviour of the polders, which are protected by fuse dikes. The fuse dike is an erodible layer of 1 m for the polders 3, 5 and 6, and 1.5 m for the polders 1 and 2 respectively. The following scenarios were analysed:

Scenario no. 1 – The fuse dikes do not breach, even overtopped.

Scenario no. 2 – The fuse dikes breach after being overtopped by a water depth of min. 5 cm.

Scenario no. 3 – The fuse dikes breach by internal erosion when the water level in the river is 50 cm higher than the spillway crest.

The obtained results are presented only for the flood corresponding to the maximum discharge 0.1% (Figure 21). This flood is characterised by a maximum discharge of 575 m^3/s and a volume of 223 mil. m^3 upstream the polders in natural regime.

By representing on the same graph the discharge hydrographs upstream and downstream Țigănași polders one can notice that the most significant attenuation is produced in Scenario no. 2 when the fuse dike breaches by external erosion, after overtopping. In this case, the maximum discharge (271 m^3/s) is reached on the increasing limb of the flood wave (Figure 22). In Scenarios no. 1 and 3, the maximum discharge is reached on the recession limb (284 m^3/s in Scenario no. 3, and 325 m^3/s in Scenario no. 1).

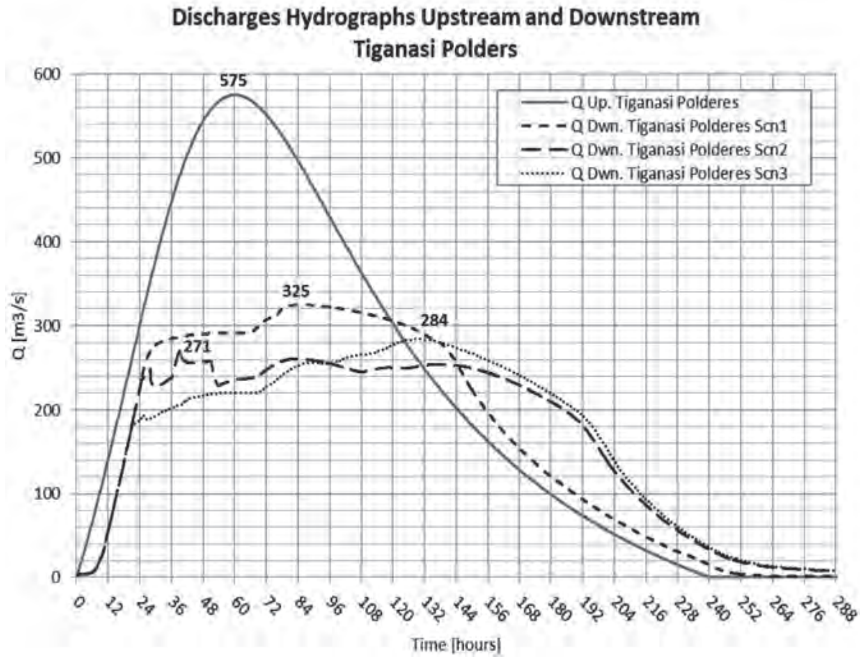


Figure 22. Discharge hydrographs upstream and downstream polders [8]

These simulations put into evidence the major role played by the polders. By the storage of about 51 mil. m^3, the maximum discharge upstream the polders is reduced from 575 m^3/s to 271–325 m^3/s downstream the polders, depending on the breach scenario.

Polder no. 4 (Țigănași 4) has no fusible dike because its flooding should be avoided as much as possible. Still, in the case of the flood 0.1% even this polder is flooded no matter the breach scenario.

Since the increasing time of the flood is about 60 hours, representing the minimum anticipation time, the water management authority has enough time to examine the behaviour of the whole hydraulic system, to run mathematical simulations and to adapt, if it is the case, the framework operation rules to the real time evolution of the flood.

DDS for operation of the lateral reservoirs during flood periods

At the end of the 19th and beginning of the 20th century, land reclamation works started in the Danube floodplain and especially in the Danube Delta. The first embankments were realised at Mahmudia (467 ha) in 1895 and Chirnogi (1,058 ha) in 1904. The latest reclamation works date from 1985, the total protected area reaching almost 395,000 ha. On the lower Danube (downstream Iron Gates) the river banks on the Romanian side are protected by dykes on a length of 1,100 km. The floodplain is divided by transverse dykes into agricultural zones which could be used as polders for storing water during high floods. A number of 34 enclosures were realised all along the Danube (Figure 23).

In natural regime, the floodplain had a retention capacity evaluated at 20.3 billion m^3. After the embankment's realisation, an increase of more than 1 m of the water level compared to the natural regime occurs in the towns of Brăila and Galaţi during floods close to 1% probability of exceedance. Besides Brăila and Galaţi, there are other towns along the Danube River, like Giurgiu, Olteniţa and Călăraşi which are threatened by floods close to or higher than 1% probability of exceedance.

Figure 23. Enclosures along the Danube River on the Romanian territory [9]

Galaţi area has a special situation, being located between the confluences of two important tributaries: Siret River (upstream Galaţi town) and Prut River (downstream). The maximum discharges on the Prut River are controlled by the Stânca-Costeşti reservoir, being lower than 600–700 m^3/s in the section Oancea. On the contrary, the maximum discharges on the Siret River can reach 4,000–4,200 m^3/s in the Lungoci section, 65 km upstream the confluence with the Danube.

The flooding in Galaţi town occurs due to the upstream floods as happened in 2006 (15,800 m^3/s on the Danube River at Brăila gauging station upstream Galaţi town, 1,375 m^3/s on the Siret River and 627 m^3/s on the Prut River) or can be aggravated because of the high floods on both tributaries as happened in 2010 (15,480 m^3/s at Brăila, 2,460 m³/s on the Siret River and 700 m³/s on the Prut River). In 2006, the maximum discharge at Grindu on the Danube River, just downstream Galaţi was 16,200 m³/s, which is quite close to the discharge corresponding to 100 years return period, while in 2010, the maximum discharge at Grindu was 16,780 m³/s.

The flood occurred in April–May 2006 had at Baziaş, at the entrance of the Danube in Romania, a maximum discharge of about 15,800 m³/s, being the highest registered discharge in the interval 1840–2006. The maximum water levels of this flood were up to 60 cm higher than the highest maximum levels of the floods occurred after the Danube embankment. At the same time, the design levels were exceeded up to 127 cm (Figure 24), for more than 20–35 days.

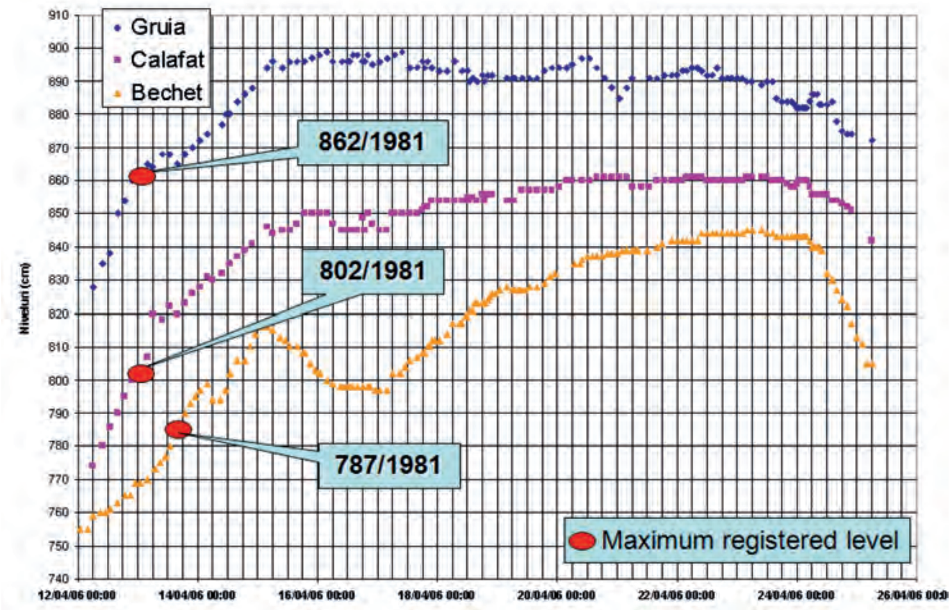


Figure 24. Water levels during the April 2006 flood and the maximum registered levels in the past [3]

The historic flood on April–May 2006 on the Danube River was at the origin of the accidental or voluntary breaches in the dykes. A surface of 73,144 ha mainly used for agricultural purposes, in 10 enclosures protected by dykes, was flooded. Some settlements were also flooded, 16,530 inhabitants being evacuated. Important towns on the river bank were also threatened by floods. Despite the damages registered in the upstream part of the river, the enclosures flooding had also beneficial effects on the downstream water levels.

Depending on the breaches' location and the failure time, the maximum level drawdown was about 28 cm. Thus, a possibility to lower the water levels during floods and to save from inundation important assets is to flood deliberately less important areas, with lower damages. In order to cut the peak and to mitigate the flood effects during the April 2006 event, 3 controlled breaches were set up at Rast, Călăraşi-Răul and Făcăieni-Vlădeni.

Table 3. Water volumes accumulated in the flooded enclosures during the April–May 2006 flood [3]

No.	Breaches	Area (ha)	Volume (mil. m^3)
	Uncontrolled breaches		
1	Ghidici–Rast–Bistreț	11,120	350
2	Bistreț–Nedeia–Jiu	15,000	285
3	Jiu–Bechet–Dăbuleni	6,000	120
4	Potelu–Corabia	11,500	230
5	Oltenița–Surlari–Dorobanțu	8,000	213
6	Oltina	2,890	94
7	Ostrov–Pecineaga	1,491	10
8	Rasova	1,500	66
	Controlled breaches		
1	Călărași	10,748	195
2	Făcăieni–Vlădeni	4,895	53
	Total	73,144	1,616

The hydraulic simulations can be used to investigate the possibility of decreasing the water levels in the downstream sections by inundation of the upstream enclosures from the Danube floodplain. If the decision is taken to flood the upstream enclosures, other downstream towns will also benefit from this effect. In fact, the enclosures' inundation operates at least partially like the natural attenuation in the floodplain. The decrease of water level depends on the volume stored in the enclosures. Of course, the inundation should be controlled in such a way to diminish the total damages on the whole stretch of the Danube River on the Romanian sector. The flood risk management thus involves a good knowledge of the economic damages for different scenarios of flooding.

A number of 13 enclosures with individual volumes in the range 40–780 million m^3 were selected if necessary for deliberate flooding by the Danube Delta National Research Institute (Figure 25). Nevertheless, the total volume which could be stored in these areas is less than 4.5 billion m^3.

Figure 25. Proposed enclosures (in blue) for flooding [9]

Proposed enclosures for flooding [9]

No.	Enclosure name	Dike crest level (m asl)	Enclosure volume (mil. m^3)
1	Seaca_Vanatori_Suhaia_Zimnicea	24.00	496
2	Bujoru_Pietrosani	19.5	41
3	Vedea_Slobozia	20.00	170
4	Gostinu_Greaca_Arges	16	723
5	Oltenita_Surlari_Manastirea	15	213
6	Boianu_Sticleanu_Calarasi	14	777
7	Borcea de Sus 1	11	218
8	Borcea_de_Jos_III	8	228
9	Borcea_de_Jos_I_II	6	45
10	Macin_Zaclau	5	346
11	Zaclau_Isaccea	5	702
12	Ciobanu_Garliciu	7	79
13	Peceneaga_Turcoaia	7	122

The effect of flooding the enclosures should not be, however, over evaluated. If considering the total volume of about 4.5 billion m^3 which could be stored into the above-mentioned enclosures, the total drawdown in the most favourable conditions would be less than 62 cm at Brăila for the flood corresponding to 1% probability of exceedance (Figure 26).

The problem which should be solved is what enclosures have to be flooded in order to obtain the necessary water level decrease to avoid significant damages at the most important towns along the Danube River. At the same time, it is obvious that flooding some enclosures has economic and social consequences on the affected areas.

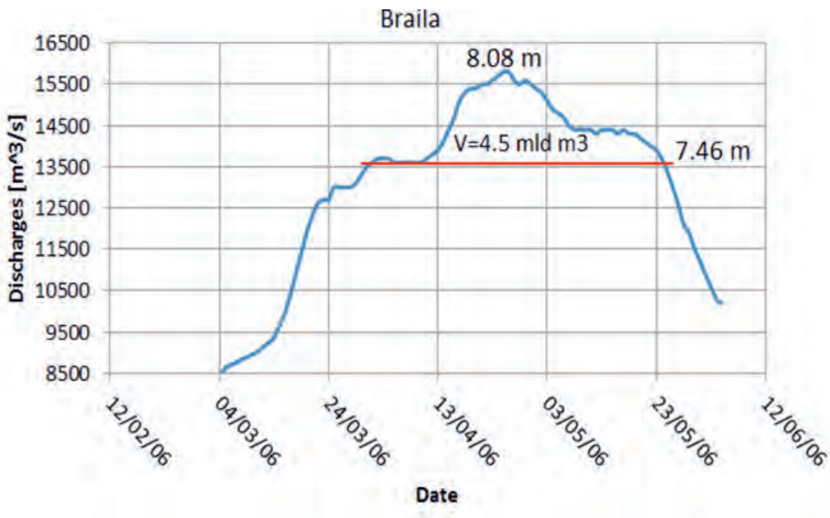


Figure 26. Maximum effect at Brăila after flooding the selected enclosures [6]

Choosing the enclosures that should be flooded involves decisions which cannot be taken under pressure during the event. A Decision Support Tool (DST) was developed by UTCB in order to evaluate the hydraulic consequences (drawdown of the water level and of the maximum discharge) for different scenarios of polders accidental or deliberate flooding. At the same time, a special sub-model for breach development (evolution in time of breach elevation and length) was set up.

Different scenarios of polders inundation were proposed and the corresponding hydraulic consequences were evaluated.

The background of the DSS tool is presented in Figure 27. On the left side of the picture one can notice the following elements:

- the Danube River and its main tributaries (marked with a blue line)
- the gauge stations on the Danube River and the most downstream gauge station of the tributaries upstream the junction with the Danube River (marked with red triangles)
- the location of all enclosures which presents favourable conditions for flooding (marked with a yellow circle with a blue arrow suggesting the water entering the enclosures)

The Danube stretch between Olteniţa and Călăraşi gauge station was analysed in the light of flood management. Two important enclosures are along this river stretch: Oltenița–Surlari–Mănăstirea (213 mil. m^3) and Boianu–Sticleanu–Călărași (777 mil. m^3).

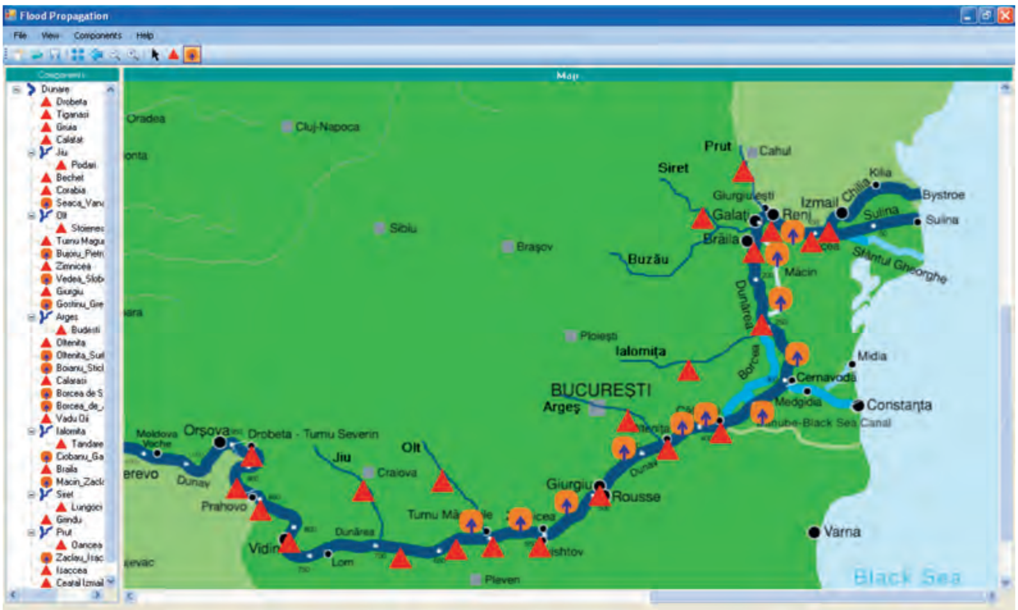
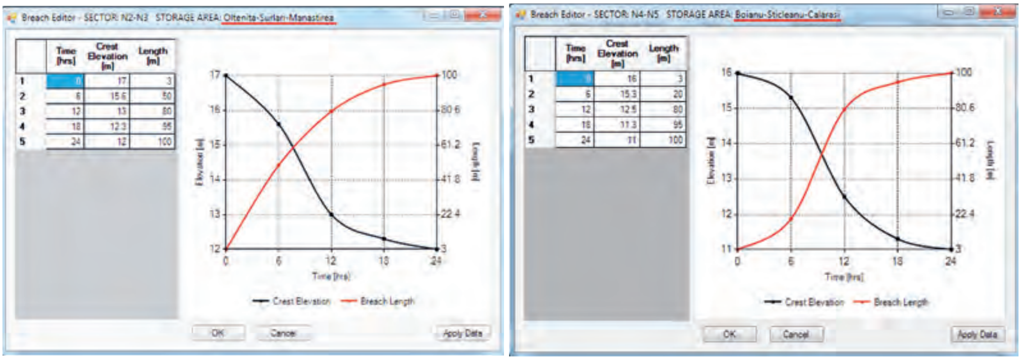


Figure 27. Background of the DSS tool [6]

The breach development involves the quantitative description of the breach crest elevation and breach length evolution. Thus, the crest elevation decreases from 17 m asl to 12 m asl in the case of Oltenița–Surlari–Mănăstirea enclosure, and from 16 to 11 m asl for Boianu–Sticleanu–Călărași enclosure (Figure 28). In both cases, the maximum breach length was supposed to be 100 m.



Oltenița–Surlari–Mănăstirea enclosure

Boianu–Sticleanu–Călărași enclosure

Figure 28. The breach development [6]

The necessary data for the DSS tool is the following:

- the river configuration: gauging stations with their attributes (Figure 29)
- the rating curves for each gauge station (Figure 30)
- the enclosures and their elevation-storage curves (Figure 31)
- discharge series along the Danube River (Figure 32)

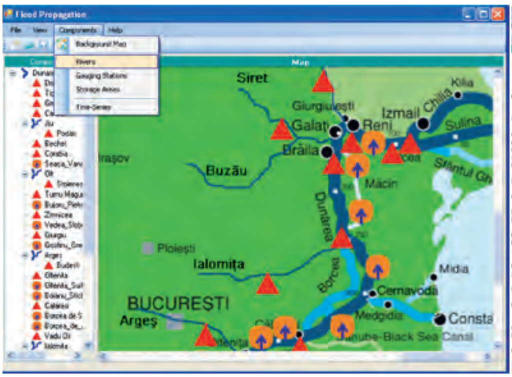


Figure 29. The river network [6]

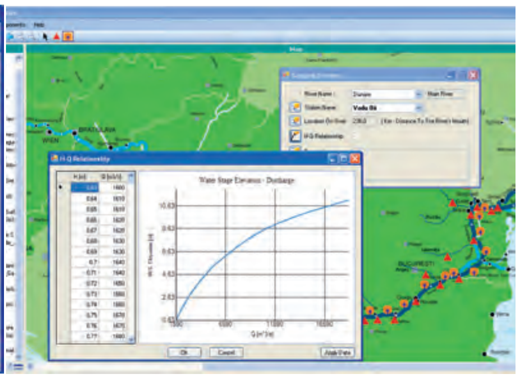


Figure 30. The rating curves [6]

Figure 31. Enclosures [6]

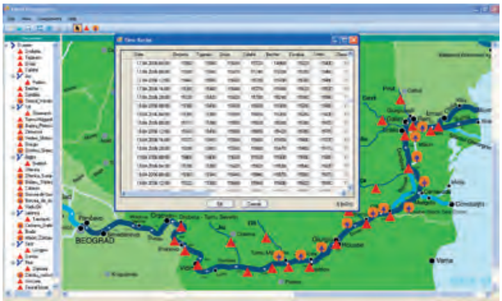


Figure 32. Discharge series [6]

Different scenarios of enclosures flooding were tested. For the beginning (Simulation no. 1) only the enclosure Oltenița–Surlari–Mănăstirea was flooded. The flooding effect is not significant (Figure 33): although a maximum discharge of about 1,000 m^3/s is entering the enclosure, due to its small retention capacity (213 mil. m^3), a temporary decrease of the Danube discharges can be noticed, but very soon the flood reaches the initial discharges. Anyway, the maximum discharges were not at all affected by the enclosure flooding.

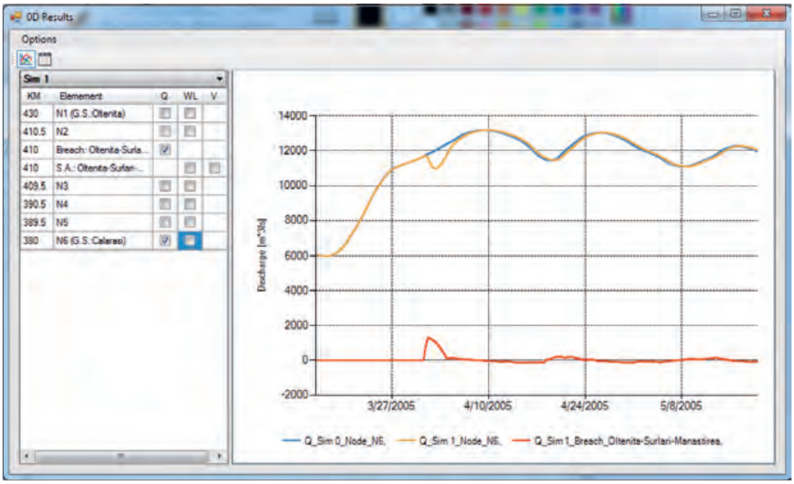
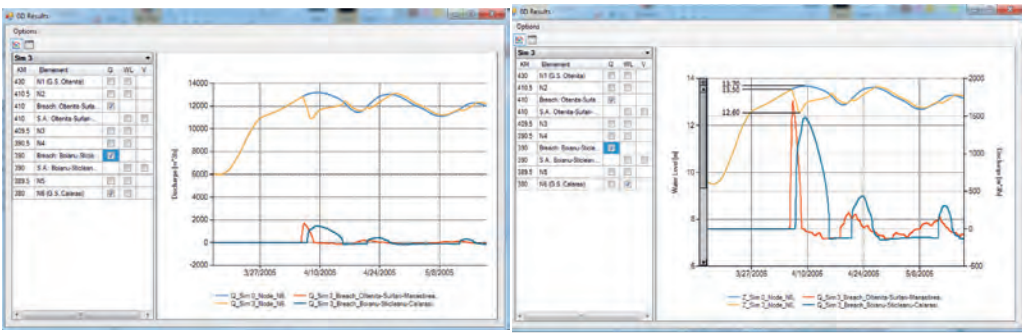


Figure 33. Hydraulic consequences of the Oltenița–Surlari–Mănăstirea enclosure flooding [6]

In order to increase the flooding effect, not only the Oltenița–Surlari–Mănăstirea enclosure, but also the Boianu–Sticleanu–Călărași enclosure (777 mil. m^3) were flooded. It can be noticed that together, these two enclosures have a total volume of about 1 billion m^3. This storage capacity is remarkable. However, the effect on downstream discharges and water level is again not significant. In Simulation no. 3, the Oltenița–Surlari–Mănăstirea enclosure was flooded first, followed by the flooding of the Boianu–Sticleanu–Călărași enclosure (Figure 34), while in Simulation no. 5, the order of flooding was reversed (Figure 35). The purpose was to find out if the order of enclosures flooding is important concerning both downstream discharges and water levels.

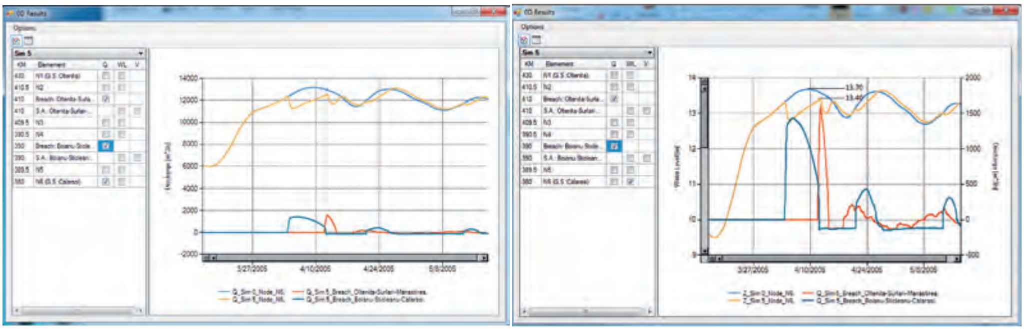


a) Discharges

b) Discharges through the breaches and water levels downstream

Figure 34. Results of Simulation no. 3 [6]

In Figure 34 a and 35 a, the discharges before and after the flooding are represented, while in Figure 34 b and 35 b, the discharges flowing into the two enclosures and the water levels on the Danube are put into evidence.



a) Discharges

b) Discharges through the breaches and water levels downstream

Figure 35. Results of Simulation no. 5 [6]

Apparently, the results are similar. However, comparing the water levels computed at Călăraşi in both cases (Figure 36), one notices that while the water level decrease in Simulation no. 3 is of 20 cm (from 13.70 m asl to 13.50 m asl), in Simulation no. 5, the effect is 50% higher, meaning 30 cm decrease (from 13.70 m asl to 13.40 m asl).

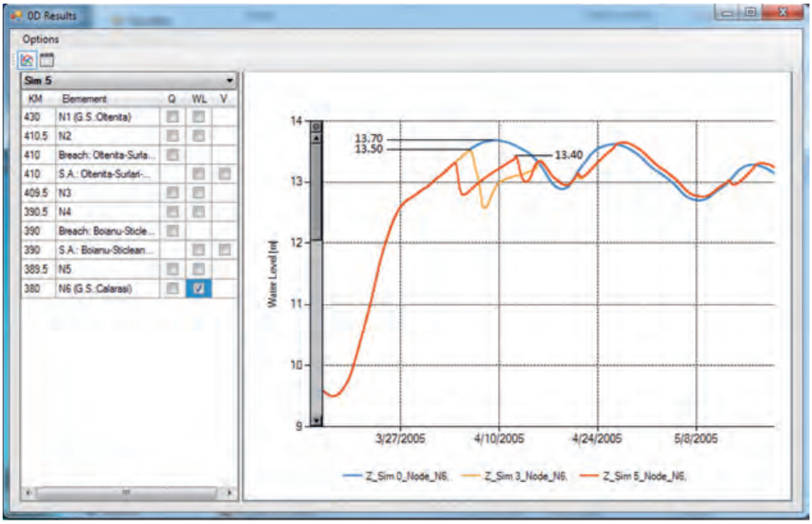


Figure 36. Comparison of water levels: Simulation no. 5 versus Simulation no. 3 [6]

However, this drawdown is not necessarily kept along the Danube River till Galaţi. The decrease of the water level will allow a supplementary volume from the tributaries to enter into the Danube River, by reducing the backwater effect of the high levels of the Danube upon the tributaries. Thus, the water levels in the Danube will continue to increase, especially due to the input of the Siret River.

By considering these facts, it can be concluded that local measures (like using mobile dykes) to protect the vulnerable areas of the towns along the Danube River are more preferable than flooding upstream enclosures.

The main conclusions of the model simulations are the following:

- The inundation of small or medium volume enclosures (less than 200 million m^3) has small effects on the water level decrease.
- The large enclosures should be inundated during floods peak, not before, in order to obtain the maximum effect downstream. Thus, a forecast of at least 7 days (including the forecast on the main tributaries: Olt and Siret) is necessary. Anyway, the water level decrease is quite small even when flooding large enclosures. If flooding all selected enclosures, with a total storage volume of 4.5 billion of m^3, the water level decrease at Galaţi in the most favourable conditions will be maximum 62 cm.
- The enclosures inundation is efficient only for large floods (more than 15,000 m^3/s) in order to use the enclosures storage volumes at the maximum extent.
- Local protection measures in Galaţi area should be initiated instead of expecting the decrease of water level by flooding the upstream enclosures.

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Andrijana Todorović

Flood Risk Assessment

Accurate flood risk assessment is necessary to detect the most endangered zones in an area of interest. Hence, accurate flood risk assessment provides a foundation for effective flood risk management.

However, flood risk assessment is quite challenging. To start with, there is no generally accepted definition of flood risk. According to some definitions, flood risk is equivalent to the probability of flooding. Other definitions include, in addition to flood probability, taking flood consequences into account. Regardless of the definition, assessment of flood probability and (especially) quantification of consequences are complex tasks accompanied by considerable uncertainties.

Definition of flood risk

There are various definitions of the term flood risk. In Japan, for example, flood risk is related to the probability of flooding of certain area inferred from the flow forecasts [1]. According to some definitions accepted in the U.K., flood risk is related to the probability of flooding and four flood risk levels are recognised [2]. These four levels include the following: high, medium, low and very low risk, with probability of flooding from a river or the sea greater than 3.3%, between 1% and 3.3%, between 0.1% and 1%, and less than 0.1%, respectively [2]. Flood risk assessed in such manner is communicated to the public via online accessible maps [2]. Some insurance companies in Germany follow the ZÜRS system for flood risk assessment [3]. According to this system, high, medium and low flood risk levels are recognised. An area is categorised as high, medium and low flood risk if affected by floods of 10-year return period or below, by floods of 10- through 50-year return period, and floods of 50-year or greater return period, respectively (see Figure 1). Flood risk can also be considered with respect to associated health risks, i.e. the potential to threaten people's health and lives [4].

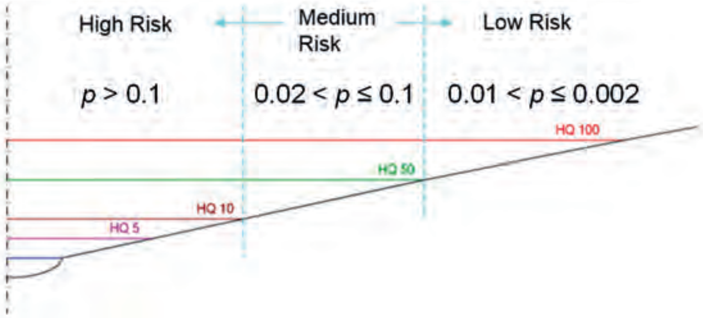

Figure 1. Flood risk classification in Germany according to the ZÜRS system [3]
Note: The probability of exceedance is given dimensionless and denoted with p

Flood risk assessment should not be related to the probability of flood occurrence only since potential consequences of floods are neglected this way. Hence, flood risk is defined as "a combination of the probability and the potential consequences of flooding from all sources – including from rivers and the sea, directly from rainfall on the ground surface and rising groundwater, overwhelmed sewers and drainage systems, and from reservoirs, canals and lakes and other artificial sources" [5]. The probability of flooding is referred to as hazard, while flood risk represents the combination of the probability of hazardous event (flood) and its consequences. In other words, flood risk corresponds to the probability of damages due to a flood event [6]. Flood consequences depend on vulnerability and exposure [7]. The former represents susceptibility to flood impacts, while the latter generally signifies the presence of people and assets in flood-prone areas [8]. Exposure can be mitigated to certain extend by implementing flood protection measures. Figure 1 shows the three components of flood risk: flood hazard, vulnerability and exposure (the latter two comprise flood consequences).

For the purpose of flood risk assessment, flood consequences are commonly represented by damages/losses quantified in monetary terms [3]. Such an approach to flood risk assessment is adopted and applied in many European countries (e.g. Germany, the Netherlands), and it is the basis of this course.

Figure 2. Components of flood risk [8]

To illustrate the outlined definition of flood risk, two hypothetical areas are considered: a flood prone area A, which is covered by forest and flooded on average once in two years, and area B, which is heavily populated with an industrial zone and heritage buildings. Area B is protected by dykes designed according to flow of 100-year return period. Regardless of being frequently flooded, damages in area A are negligible, whereas flood damages in area B would be enormous with potential casualties and impact to the historical/cultural heritage. According to the definition above, flood risk in area A can be considered low, and high in area B because of potential far-reaching and devastating consequences. However, flood hazard is considerable in area A, which is frequently flooded, and minor in area B (because of the dykes and low probability of flooding).

To assess flood risk, both flood probability and consequences represented by monetary value, have to be combined, as shown in Figure 3. The combination of these terms results in the expected annual damages (EAD) in the considered area. Imposing threshold values of the EAD, various flood risk levels are defined and assigned to distinct zones within the area of interest (e.g. high, medium and low). Flood risk can be fairly represented by flood risk maps, which are straightforward and can be easily understood by the public.

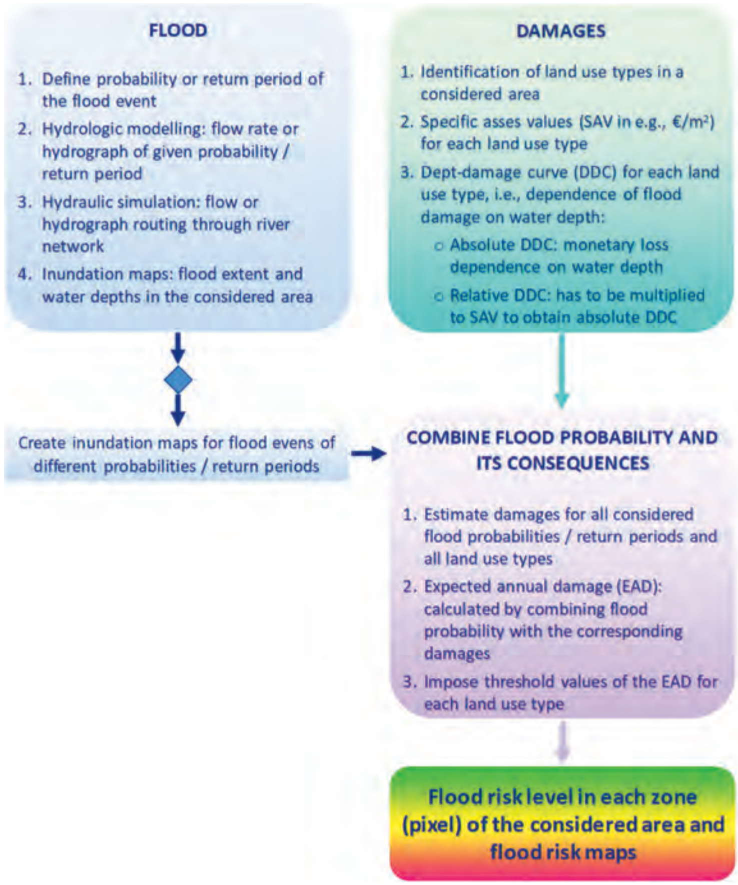

Figure 3. Methodology for flood risk assessment (compiled by the author)

Consequences of floods

Floods are by far the most frequent and disastrous events (see Figure 4). Flood consequences generally depend on the number of people in an area affected by the flood and asset values [6]. Floods make negative impacts on buildings, including public buildings, such as hospitals and heritage buildings, then industry, traffic, crops and livestock, and, most importantly, human health and lives. Some consequences are arisen immediately and are caused by a direct contact to the flooding water, while others are accompanying (e.g. costs of traffic disruptions). Some flood consequences can be easily quantified in terms of money, such as damages to buildings or infrastructure, while others are rather difficult to represent in monetary terms (e.g. casualties).

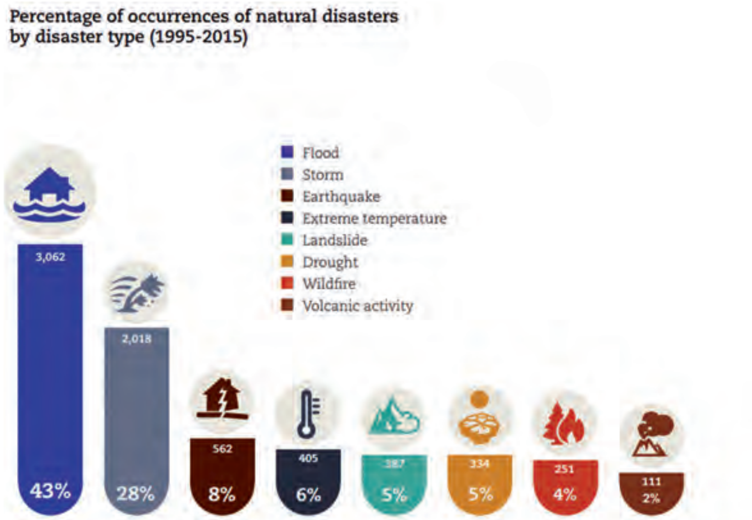

Figure 4. Frequency of natural disasters [30]

Hence, flood damages are generally categorised as follows [9]:

Direct and indirect: direct damages refer to the harmful effects due to contacts with flooding water. Indirect, secondary damages are not caused by flooding water itself, but by accompanying effects, such as loss of production because of flooding or costs due to traffic disruptions and power cuts. A thorough review of methods for assessment of indirect damages can be found in the literature [10].

Tangible and intangible damages generally can/cannot be easily quantified in terms of money, respectively. For example, the intangible damages include casualties, harmful health effects, damages to historical and cultural heritage, negative impacts on the environment. There are various methods intended to quantify intangible damages (for review see e.g. [9]), but these are generally not considered in flood risk assessment studies and, thus, they are beyond the scope of this course.

An overview of flood damages is given in Table 1.

Table 1. Classification of flood damages with examples (adopted from [9])

	Tangible	Intangible
Direct	Physical damages to assets: Buildings (structural damages) Infrastructure (e.g. roads, railways, etc.) Belongings and inventory Vehicles Losses of livestock Losses of crops	Loss of lives Negative health effects Harmful effects on the environment Damaging effects to historical and cultural heritage
Indirect	Loss in production (e.g. industrial, agricultural) Loss of incomes Costs of traffic disruption Loss of value added Evacuation and emergency costs Costs of post-flood clean-up Damages due to consequent landslides	Difficulties with post-flood recovery Increased vulnerability of survivors

To illustrate this classification of flood damages, a family dwelling is considered: direct damages refer to the building structure, furniture and assets, while indirect damages include e.g. clean-up and rehousing [11]. Figure 5 shows some of the consequences of the severe flood event in the Kolubara catchment in Western Serbia in May 2014. During this flood event, twenty-four casualties were reported. Many cities in this catchment suffered enormous damages to the buildings and infrastructure, including bridges and roads (direct, tangible damages). The agriculture of the area also suffered great damages (indirect, tangible). The thermal plant "Nikola Tesla" and local open pits of the coal mine that supplies the thermal plat were also flooded (direct, tangible damages), which resulted in the reduced supply of electric currency (indirect damages). Subsequent landslides caused additional (indirect) damages [12].

In addition to the asset values and number of people in a considered area, flood consequences also depend of flood characteristics, such as flood extent, water depth and velocity, flood duration, rate of the rise of water level, debris flow and waves [3] [8]. Damages also depend on the time of flood occurrence: for example, damages to agriculture, i.e. losses in crop production are the greatest due to floods during harvesting seasons [9]. Since flooding water is usually heavily polluted, damages also include far-reaching chemical and biological impacts on health and environment [11].

The larger flood extent, i.e. the greater area affected, the greater is the number of affected people and assets. Also, deeper water causes greater damages. For example, rather shallow water can be prevented from entering a building (e.g. by putting up shields), while deeper water reaches higher floors of buildings, augmenting damages manifold.

Figure 5. Devastating flood in the Kolubara catchment (Serbia) in May 2014: the city of Obrenovac [34] and losses in crop production ([35], top panels), power plant "Nikola Tesla" [36] and open pits of the "Tamnava" coal mine ([37], bottom panels)

These two flood characteristics are identified as the most important for flood damage assessment, i.e. a significant part of total flood damages can be explained by the flooding extent and water depth [9]. High water velocities can put people in danger, and can wash away moveable assets (e.g. vehicles), develop and enhance erosion. Table 2 shows impacts of different combinations of water depth and velocity on people. A combined impact of water depth and velocity is also illustrated in Figure 6. Longer flood duration implies longer exposure to water, which augments damages. For example, building fabric is particularly sensitive to long exposure to water [9]. Impact of flood duration on damages is shown in Figure 7. High rate of water level rise means shorter warning lead times, which further leads to higher damages [10].

This statement is supported by the figures in Table 3, which clearly indicate that increase in warning lead time can considerably mitigate flood damages. Longer warning lead times are necessary as they enable evacuation of more people and allow people to relocate/elevate their moveable belongings. High debris load can enhance mechanical impacts of flowing water, and, thus, deteriorate flood damages to buildings and infrastructure. Furthermore, debris deposits have to be cleaned-up after the event, increasing indirect damages. Most importantly, debris are one of the major causes of injuries and casualties due to floods [9]. Waves make mechanical impact on the building fabric, and also can cause injuries and jeopardise human lives. Strong wave impacts are generally associated to coastal floodings [9]. Various flood impacts on health and environment are elaborated by the U.S. EPA [13]. Some common consequences to health include diarrheal diseases and wound infections. Frequent animal and insect bites accompany floods, considering that animals are also being displaced. Health can also be threatened by exposure to various chemicals and pollutants, such as fuels, pesticides, paints, cleaning supplies, silica from construction materials, hazardous chemicals from the industry, etc. Figure 9 shows major impacts of polluted flooding water on health.

Table 2. Risk to human lives assessed from water depth and velocity [4]

Depth velocity (m2/sec)	Description	Risk to human lives
< 0.75	Shallow flood water or deep standing water. Caution is needed.	Low
0.75 < 1.5	Deep water or high velocity. Dangerous to vulnerable groups.	Moderate
1.5 < 2.5	Deep water or high velocity. Fatalities are mainly due to exposure to the flooding water. Dangerous to most people.	High
2.5 > 7.0	Extreme danger from deep, fast flowing water. Fatalities due to exposure to the flooding water. Dangerous for all.	Extreme
> 7.0	Extreme danger from deep, fast flowing water and risk of building collapse. Dangerous for all.	Extreme

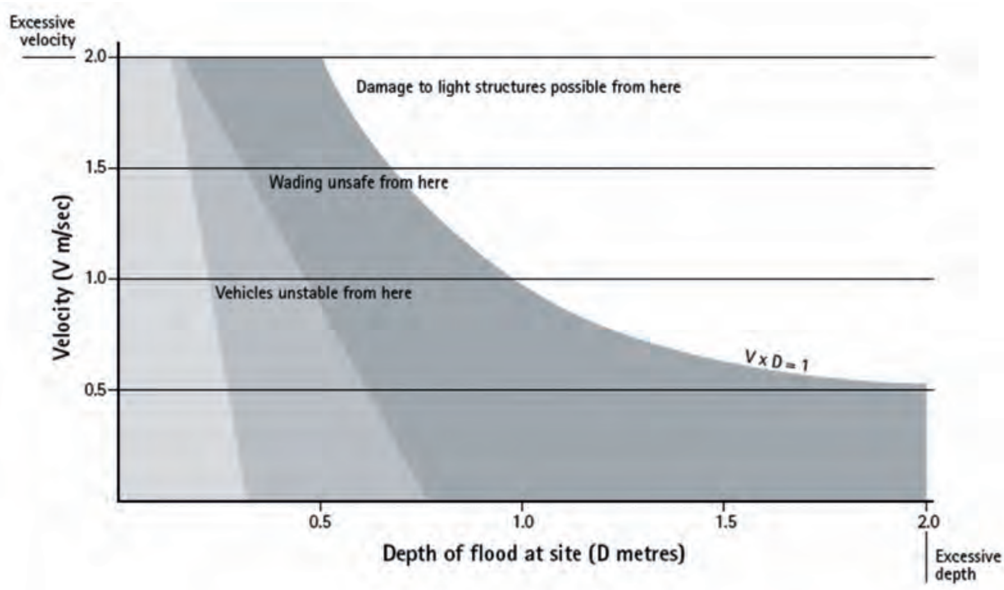

Figure 6. Combined impact of water depth and velocity [11]

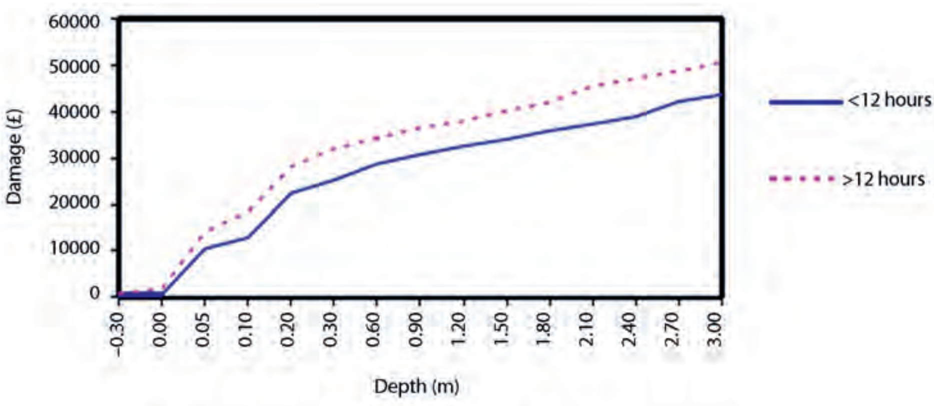

Figure 7. Impact of flood duration on damages [9]

Table 3. Impact of flood warning lead times on damages on residential properties [9]

Depth of flooding (m)	Flood warning lead times			
	Up to 2 hours	2–4 hours	6 hours	8 hours
	Damages avoided (% of the total damage)			
1.2	25.3	35.7	38.7	40.7
0.9	26.4	37.6	40.6	42.6
0.6	25.5	37.2	40.2	42.2
0.3	30.0	42.1	45.1	47.1
0.1	24.5	32.8	35.8	37.8

Figure 8. Impact of debris load during floods [31] [32]

Outline of the course

This course explains a methodology for flood risk assessment by combining flood hazard and flood consequences, assessed in monetary terms.

The following section explains in detail the assessment of both components: namely, how to obtain inundation maps due to flood event of a given probability, and corresponding direct, tangible damages, quantified in terms of monetary loss. Assessment of indirect and/or intangible damages is beyond the scope of this course. Damages are obtained from assessed asset values and water depth only. Based on flood probabilities and corresponding damages, computation of expected annual damages that combine these terms, is thoroughly elaborated. Finally, identification of different flood risk levels based on the expected annual damages is described.

Information on flood risk can be effectively visualised and communicated to the public and decision-makers by flood risk maps. The final section concisely explains how to obtain flood risk maps by employing GIS tools.

For the sake of clarity, a glossary explaining the key terms is provided at the end of this course material.

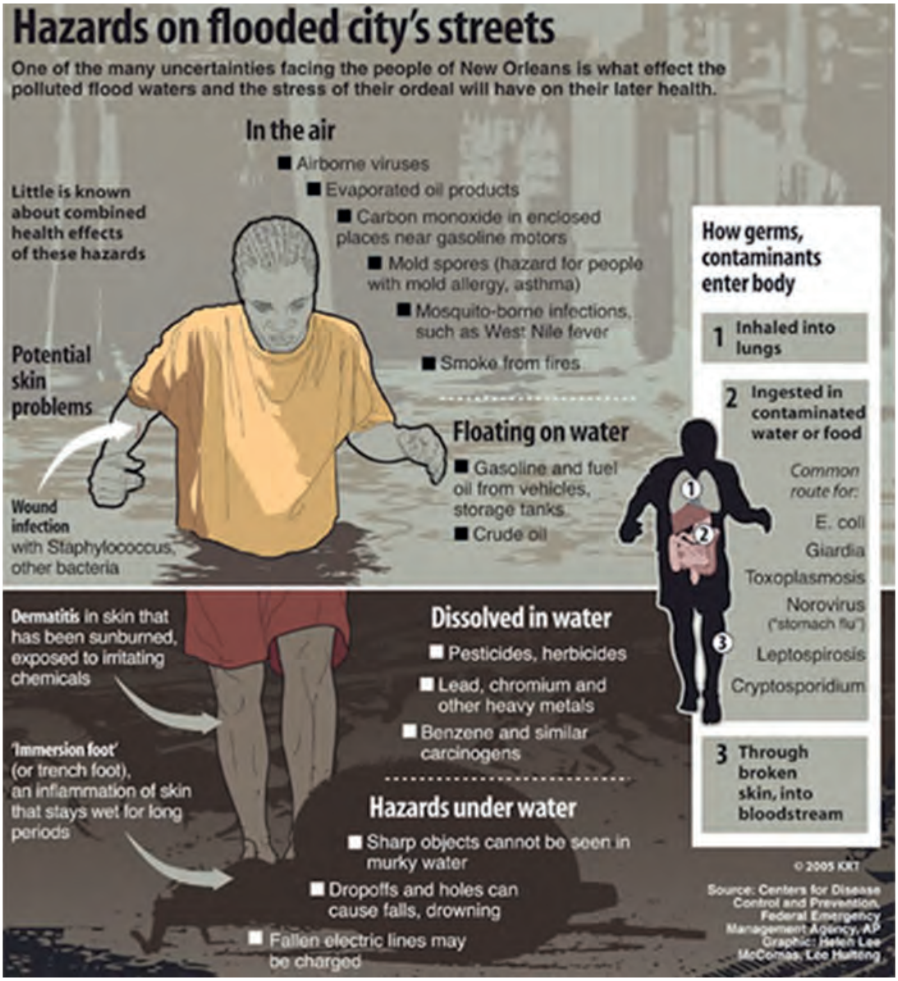

Figure 9. Health risks due to polluted flooding water [33]

Methodology for flood risk assessment

As explained in the previous sections, flood risk assessment comprises estimation of flood hazard and the consequent damages. Flood hazard (potential harm) is related to the probability of flooding: for example, great flood hazard implies flood prone areas. Section *Probability of flood – Hazard* explains how to obtain inundation maps for a flood event of a given probability of exceedance/return period. Assessment of monetary equivalent of direct, tangible flood damages is elaborated in section *Flood damages,* as well as how to combine flood probability with its consequent damages, resulting in expected annual damages (EAD, subsection *Expected Annual Damage [EAD]*). Identification of flood risk level from the assessed EAD is explained in section *Flood risk assessment.*

Probability of flood – Hazard

Flood hazard indicates the probability of flooding of a considered area. Methodology for flood probability assessment, which is the first part of the flood risk assessment methodology, includes the following steps (Figure 10):

Estimation of the flow rate or the entire hydrograph for the given flood probability or return period (e.g. 100 years).

Flow or hydrograph routing through a river network by using a hydraulic model.

Use of the hydraulic simulation results for inundation maps, which show flooding extent and hydraulic variables (e.g. water depth) within the flooded area.

Create inundation maps for various characteristic probabilities/return periods.

Figure 10. Inundation maps for the purpose of flood risk assessment (compiled by the author)

Flood probability

There are two approaches to flood flow estimation: namely, the statistical approach and the deterministic one (see Figure 11). The former is based on the application of statistical methods and requires observed flow series, whereas the latter approach relies on rainfall–runoff models and design rainfall [14].

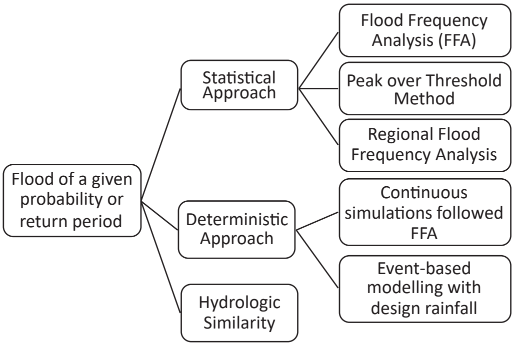

Figure 11. Approaches to flood estimation (adopted from [14])

Flows of a given probability/return period are commonly obtained by applying the Flood Frequency Analysis (FFA [15]) or Peak over Threshold method (PoT [16]). Flood frequency analysis is based on the series of annual flow maxima, and it is aimed to obtain the fittest model (i.e. theoretical distribution) to the empirical distribution of the observed annual maxima. However, application of this method requires that the flows are independent and identically distributed. These requirements are examined by applying statistical tests prior to the FFA. Steps of the FFA are outlined in Table 4. The PoT method comprises of probabilistic modelling of: 1. Flood occurrence, i.e. expected number of events exceeding the set threshold per year; and 2. Flood magnitude (i.e. peak flow rate). Discrete distributions are used to estimate the probability of flood occurrence, while flow magnitude is described with a continuous probability distribution. Unlike FFA, the PoT method is based on the flood exceeding a given threshold, meaning that several floods in one year can be taken into analysis, whereas maximum flows from some other years that do not exceed the threshold are discarded. For simplicity, the threshold is usually set to provide statistically independent flows. The mathematical model of the PoT method is more complex than the FFA, but, considering that the largest observed flows are taken into account regardless of the year of their occurrence, PoT is expected to yield more reliable quantile estimates than FFA. Additionally, there are quite complex PoT model versions that do not require independent data [16].

Application of statistical methods requires long-term and reliable flow observations. For example, for a reliable flood frequency analysis at least 25 years of flow observations are required [15]. Also, an empirical rule states that the Gumbel distribution can be applied for reliable flow quantile estimation for return periods of twice the length of the observed period or shorter [17]. If only a short data is available at a considered stream gauge, a regional flood frequency analysis can be applied. Specifically, annual maxima from several gauges within the same region are normalised (e.g. with respect to the mean flow at the gauge), and concatenated into single, long series, followed by the FFA. In this way, dimensionless flow quantiles are obtained. The quantiles at a gauge are calculated by multiplying the dimensionless quantiles with the mean flow at that particular gauge [14].

Table 4. Flood frequency analysis: an outline (compiled by the author)

Step	Procedure
0	Obtain annual maxima series from the available flow record.
1	Calculation of the sample statistics: mean value, standard deviation, skewness, etc.
2	Test for homogeneity and independence. The former is tested with e.g. z-test, Student, Man-Whitney or Mann-Kendall tests, and the latter with e.g. Bartlett test.
3	Calculation of empirical distribution by using e.g. Weibull probability plots.
4	Distribution fitting, i.e. calculation of parameters of probability distributions: (log-)normal, Gumbel, (log-)Pearson III, Generalised Extreme Value (GEV), etc. Various methods can be used for parameter estimation: method of moments, L-moments, weighted L-moments, maximum likelihood method.
5	Goodness-of-fit tests: Kolmogorov-Smirnov, Anderson-Darling, etc. Selection of the most suitable probability distribution.
6	Compute flow quantiles, i.e. flows of different probabilities/return periods by using the selected probability distribution.

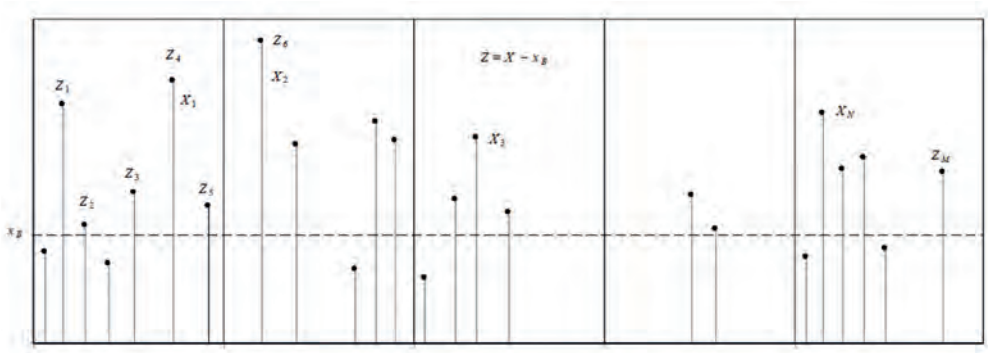

Figure 12. Peak over Threshold method: x_B – threshold, X – flows, Z – peaks over threshold [16]

Table 5. Peak over Threshold method: most frequently used models [16]

Number of occurrences distribution	Peak height distribution		
	Exponential	Weibull	Generalised Pareto
Poisson	P + E	P + W	P + GP
Binomial	B + E	B + W	B + GP
Negative binomial	NB + E	NB + W	NB + GP

Rainfall–runoff models enable flow simulations from input meteorological data (precipitation, temperature, potential evapotranspiration) and data on the considered catchment (e.g. area, hypsometric curve, land use types).

Rainfall–runoff (hydrologic) models simulate flows at a catchment outlet for given rainfall data and initial conditions in the catchment. Presently, there are numerous rainfall–runoff models that vary in complexity, spatial discretisation and data demands. Hydrologic models are generally classified as event-based or continuous. The former simulate catchment response to a single rain event (i.e. they simulate single flood hydrograph at a catchment outlet). The latter are used for long-term, continuous simulations that include periods during and in-between rainfall events [18].

For the purpose of flood frequency estimation, event-based models are usually applied. Runoff simulations with event-based models include: 1. Calculation of runoff volume; and 2. Runoff routing to the catchment outlet. Runoff volume (i.e. rainfall–runoff partitioning) is usually calculated by applying the SCS CN method [19], while runoff routing models include the rational method and various models based on the unit hydrograph (UH) theory [20]. The rational method is the simplest event-based model, and its application is limited to small, urbanised catchments [20]. Some of the commonly used UHs are Clark or Snyder [21]. These models consist of equations that comprise parameters, which have to be adjusted to provide the best possible fit to the observed flows, i.e. the models have to be calibrated before their application. The calibration is performed to achieve the best possible fit between simulated and observed hydrographs in terms of: peak flow magnitude and timing, runoff volume, rinsing limb slope and timing and recession limb. For model calibration, models are forced with the observed meteorological series. The event-based models can also be used for ungauged catchments that lack long-term, reliable flow observation necessary for model calibration. Synthetic unit hydrograph (SUH) models are employed for this purpose [22]. The SUH models are derived from topographic data on the catchment. One of the most frequently used is the dimensionless SCS SUH [21].

To obtain flows of a given return period, UH and SUH models are forced with design rainfall, derived from depth–duration–frequency curves and assumed hydrograph shape, i.e. change in rainfall intensity throughout the design event. Since uniform rainfall intensities result in lower peak flows, it is recommended to force the models with time-varying rainfall for the purpose of flood flow modelling. Such design rain event can be obtained by using e.g. the Chicago method. An underlying assumption in this approach to flood flow estimation is that the return period of the simulated peak flow is equal to the return period of design rainfall used for the model run.

Continuous hydrologic models can also be used for estimation of flood flows. This approach includes two steps: 1. Calibration (and evaluation) of a continuous model; and 2. Application of Flood Frequency Analysis over the series of simulated annual maxima [14]. The application of this approach can be justified by the fact that meteorological record series are usually considerably longer than flow observation. Hence, forcing the model with (longer) meteorological input series can provide longer flow series, which are expected to increase reliability of the flow quantiles. However, this approach to flood estimation is rarely applied, primarily because common calibration of continuous models leads to poorly simulated flow peaks (mainly their underestimation) [23].

Floods of a given probability/return period at an ungauged river cross-section can be estimated by applying the principle of hydrologic similarity [17]. The flood of return period T is calculated as follows:

$$\frac{Q(T)}{Q^*(T)}=\left(\frac{A}{A^*}\right)^{\alpha} \quad (1)$$

where Q*(T) represents the flow of a given return period T at the point of the river with drainage area A*, while A is the area of the ungauged catchment. The recommended value of the parameter α is 1/3, although its value can be adjusted to fit available data [17].

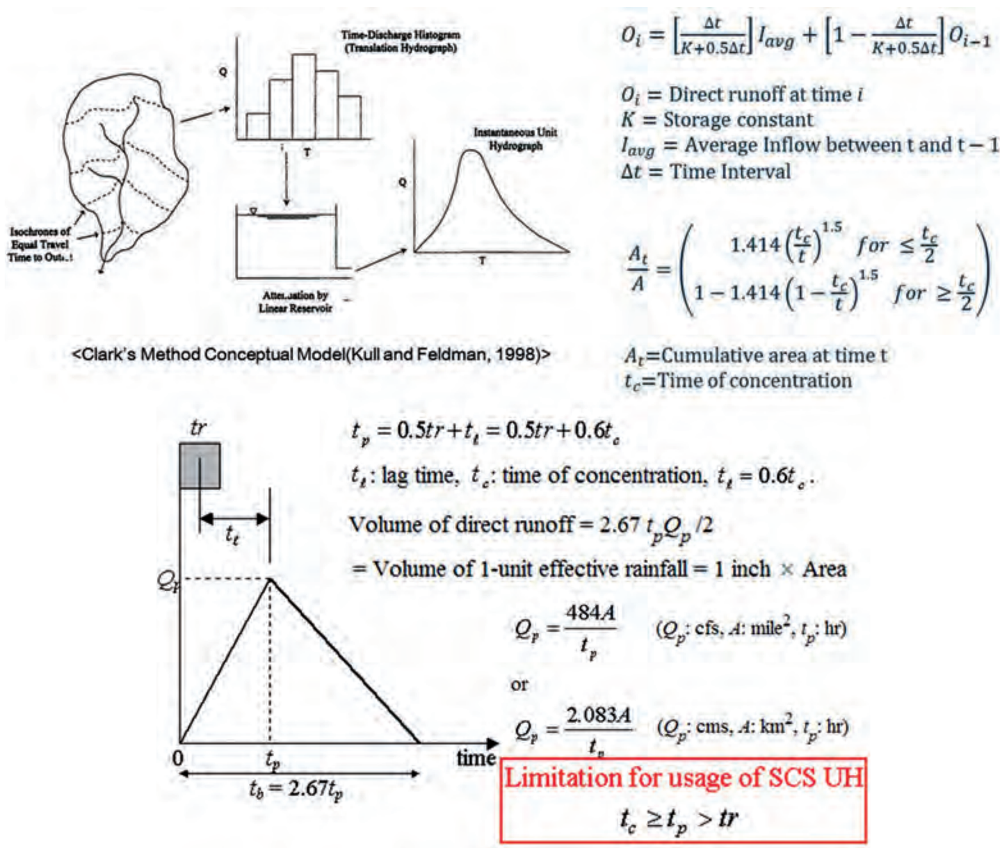

Figure 13. The Snyder UH (top panel) and the SCS SUH (bottom panel) [38]

Flood routing and inundation maps

Hydraulic simulations, i.e. flood routing enables the computation of flow rate and other hydraulic variables at any point of the river network and at any time, given the input flood rate or hydrograph, network geometry, initial and boundary conditions [20]. Water levels across the river network and in the inundation/flooded area of primary interest to flood risk assessment.

Flood routing can be steady or unsteady: the former implies routing of constant flow rate in time, while the latter denotes routing of an entire flood hydrograph (i.e. time-variable flow). Unsteady routing results in higher water level than steady flow routing [24], thus it is preferred for flood risk assessment.

There are parsimonious flood routing methods, such as the linear reservoir equation or the Muskingum method. However, accurate flood routing requires distributed routing models, which are based on the partial differential equations describing mass, momentum and energy conservation laws [20]. Some of the distributed flow routing models are e.g. kinematic, diffusion or dynamic wave models. To apply the distributed models, the geometry of the river network has to be specified, as well as initial and boundary conditions. Additionally, the partial differential equations embedded in these models can seldom be solved analytically. Specifically, analytical solutions are possible if strong assumptions on geometry are made and some terms in the equations omitted. Therefore, various numerical methods are applied for flood routing. Routing of flash flood poses a great challenge for numerical modelling due to sudden change in water depth that cannot be captured by commonly applied methods [20].

The routing models also vary according to flow direction that they can simulate. One-dimensional models are frequently used, while application of 2D models, which provide detailed description of flow, is constrained by considerable computational time [9]. Computational requirements are even higher for three-dimensional models, which are mostly applied to simulate complex flow in junctions and their immediate vicinity.

One of the most frequently used software for flood routing is HEC-RAS by the US Army Corps of Engineers [25]. This software is user-friendly (see Figure 14) and can be freely downloaded from the US Army Corps of Engineers HEC webpage (HEC).

Based on the simulated water level, inundation maps can be obtained. As shown in the bottom panel of Figure 14, inundation maps indicate flood extent, i.e. flooded areas, and show water depth within the flooded area. An example of inundation map is shown in the bottom panel of Figure 14. For the purpose of flood risk assessment, inundation maps are required for various probabilities/return periods (see subsection *Expected Annual Damage [EAD]*). Besides water depth, other hydraulic variables, such as water velocity, pressure force, shear stress can be shown as well.

Inundation maps are usually obtained by importing water levels into a GIS environment. For example, hydraulic simulation results obtained with HEC-RAS can be easily imported by applying the HEC-GeoRAS plug in. The version for ArcMap can be readily obtained from the HEC webpage. A similar tool is available as a plug-in for QGIS.

The application of hydraulic models for flood routing is a subject of the specialised course of this postgraduate programme, which explains hydraulic modelling in detail.

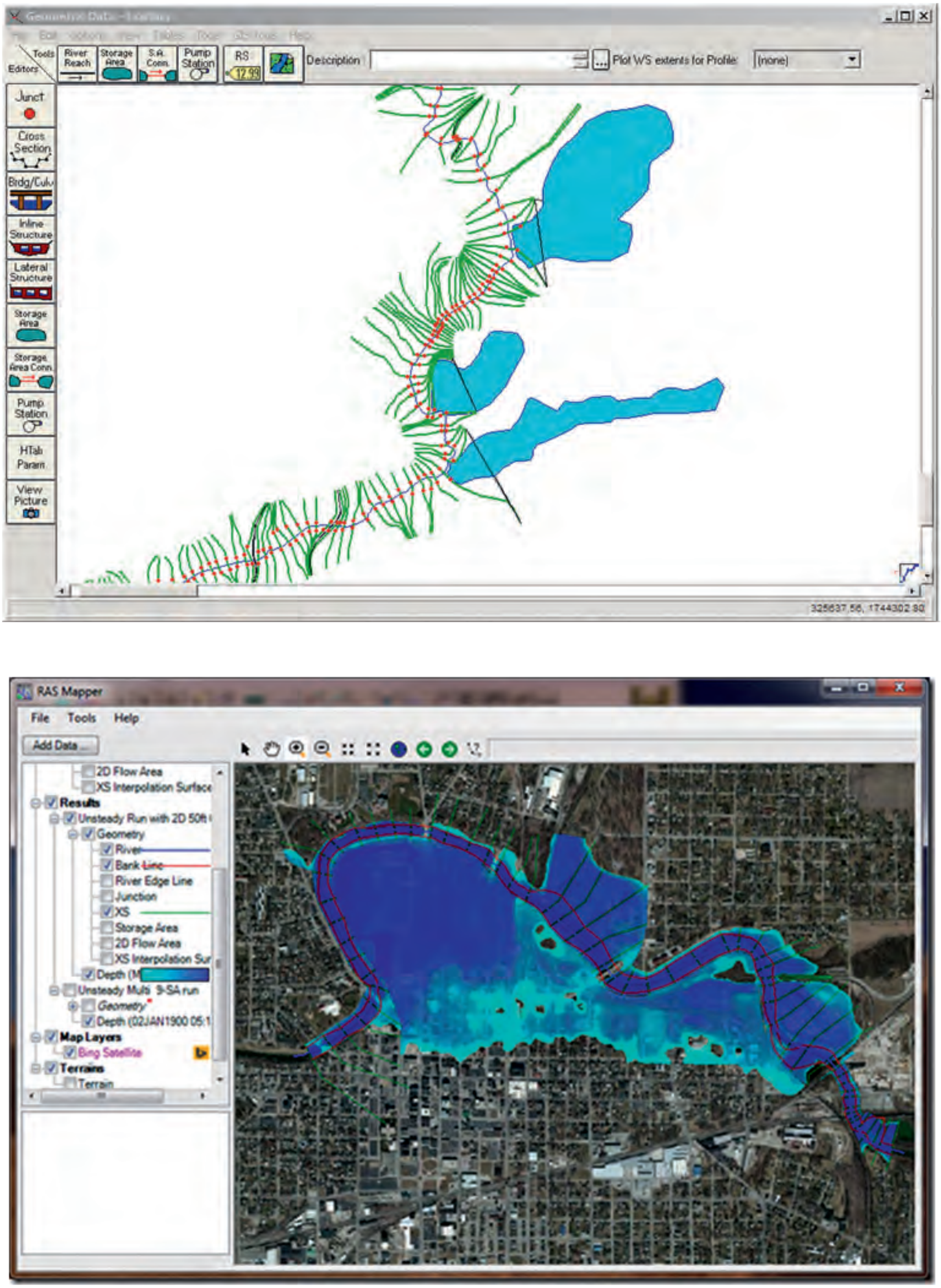

Figure 14. Hydraulic simulations with HEC-RAS: cross-sections in the HEC-RAS window (top panel, [39]) and inundation map (bottom panel, [40])

Flood damages

For the purpose of flood risk assessment, flood consequences are quantified in monetary terms [3]. This methodology for flood risk assessment recognises only direct, tangible damage (e.g. damages to buildings or infrastructure), whereas indirect and/or intangible damages are not considered here (see section *Consequences of floods*). Additionally, only water depth is considered, whereas other hydraulic variables, such as water velocity or flood duration, are neglected.

Estimation of flood damages includes the following (Figure 15):

- identification of land use types in the area of interest
- assess specific asset values for each LUT
- obtain depth–damage curves (DDC) for each LUT

Compute damage (in monetary terms) by combining water depth due to flood event of a given return period, degree of damage given the water depth, and information of asset value.

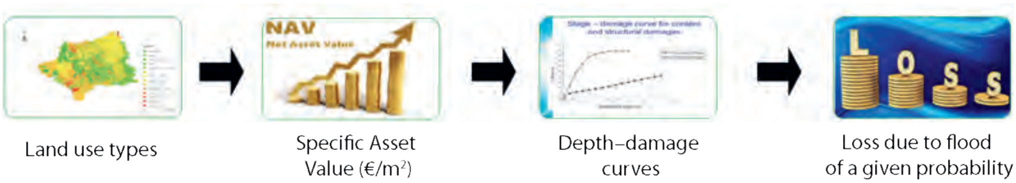

Figure 15. Assessment of direct, tangible damages due to a flood event of a given probability (compiled by the author)

Identification of land use types

For assessment of direct, tangible damages, land use types (LUTs) in a considered area have to be identified. Depending on the area and scope of the flood risk assessment study, i.e. required accuracy and spatial resolution, the number of identified LUTs varies from a few to several hundreds. For example, high spatial resolution and micro-scale studies imply a large number of LUTs. Categorisation of LUTs in the considered area should result in 1. Minimum variance within one LUT category; 2. Maximum variance among different LUTs. Additionally, LUT categorisation should be constrained by available depth–damage curves and data on asset values [9]. For example, setting an abundant number of residential LUTs with only one available DDC or with data on property values does not increases accuracy of the damage assessment. Generally, buildings within a residential area are all different: for example, they differ according to the number of storeys, presence of cellar, elevation of the ground flood, etc. Additionally, different buildings are differently furnished in terms of luxury, implying large variations in asset values. Bearing in mind these differences, several residential categories may be defined, depending on the scope of the study. Categorisation of LUT in the Rhine basin is shown in Figure 16.

Information on LUTs in the considered area can be obtained from primary and secondary data sources. The primary data sources are essentially field surveys. The secondary source includes LUT databases, cadastral maps or real estate market data, all of which provide aggregated data on LUTs [9]. Field surveys do provide most accurate information on LUT, however, they can be carried out only for small areas, i.e. for the purpose of micro-scale studies. Concerning secondary sources, there are some geo-databases with information on LUTs for each country. For example, LUT data can be obtained from ATKIS-DLM in Germany, or the GeoSrbija portal for Serbia. The Corine Land Cover [10] database that covers the entire Danube basin can be accessed via the link https://land.copernicus.eu/pan-european/corine-land-cover/clc-2012?tab=mapview.

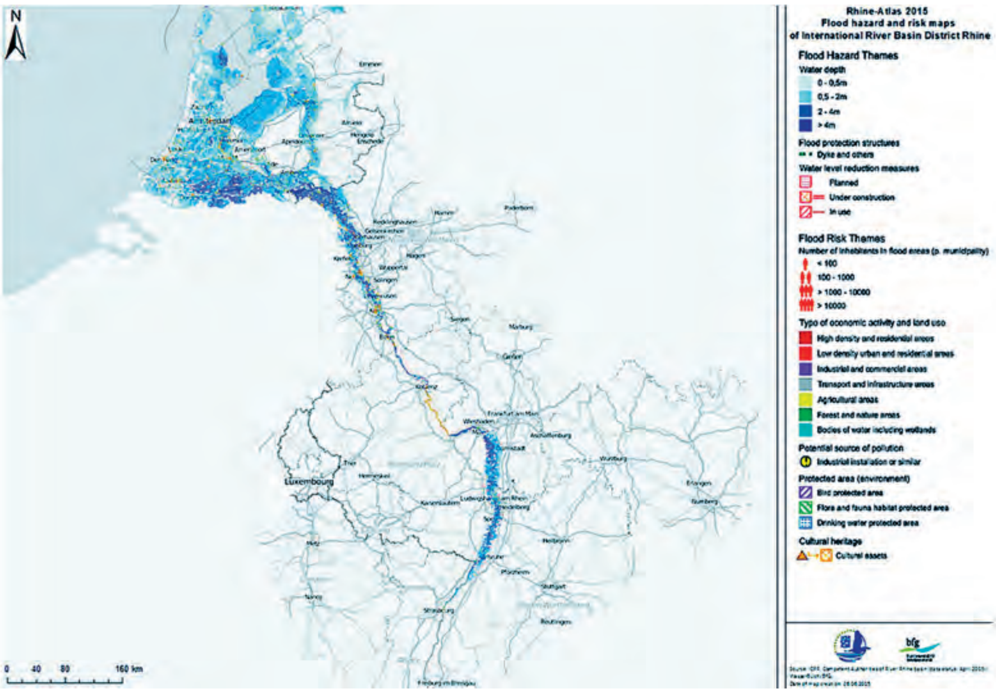

Figure 16. Rhine Atlas [26]

Asset value estimation

There are two approaches to asset value assessment: namely, assessment of value at purchase price and at the actual price [3]. The former concept does not take depreciation into account, and, basically provides full replacement value. As such, it provides overestimated asset values and, hence, should be avoided [9]. Additionally, even severe flood events do not necessarily cause total damage of buildings, particularly in case of reinforced concrete structures. The latter approach, which takes depreciation into account and provides realistic estimates of asset values, is preferred. Only one approach can be used for a flood risk assessment study: the combination of both approaches is strongly discouraged [9].

Assessment of values of buildings in residential areas should mandatorily include vehicle values. Value of infrastructure assets (e.g. road, railways, water supply or sewer systems, etc.) can be obtained from the construction costs that are usually made available in publications. Values of crops are equal to the investments to produce the crops, while value of livestock can be inferred from market price. A detailed guide on the asset value estimation can be found in the literature [9].

Flood damages assessments should refer to large areas (e.g. entire regions). On the other hand, asset values are often inferred over much smaller, “sample” areas: for example, value of dwellings is assessed based on a sample of individual dwelling within a smaller zone, assuming that this zone is representative for the entire considered area. To enable extrapolation to wider areas, asset values are given per unit area, i.e. as specific asset values (e.g. in €/m^2). For example, once a representative zone is selected, the number of buildings within such a zone is determined, and then multiplied by the mean estimated value of the building to yield the total asset value of the zone. The total value is then divided by the zone area, resulting in specific asset value. A similar calculation can be done for road and rail network: although values of these assets are regularly given per unit length, values per unit area can be calculated taking into account e.g. the road/rail track width [9].

Specific asset values greatly facilitate flood risk estimation, as explained in the sequel.

An example of specific asset values, assessed for the North Rhine region, are given in Table 6.

Table 6. Specific asset values for North Rhine-Westphalia [9]

Land-use category	Value of fixed assets (EUR/m^2)	Value of mobile assets (EUR/m^2)	Total (EUR/m^2)
Settlement	231	59	289
Industry	231	80	311
Traffic	263	2	265
Agricultural area	No differentiation	No differentiation	9
Forest	No differentiation	No differentiation	1
Other	No differentiation	No differentiation	0

Damage-Depth Curves (DDC)

Depth–damage curves (DDCs) represent dependence of damages on flooding water depth. As such, these curves provide a link between flood characteristics and flood consequences, i.e. damages that are essential for flood risk assessment.

There are two types of these curves: absolute and relative [9]. Absolute DDCs show damages in monetary terms versus water depth (at the abscissa, see e.g. the bottom panel of Figure 17). Relative DDCs show damages as the share of total asset value at the given water depth. Specifically, loss takes value between 0 (no damage) and 1 or 100% (total loss of the asset) [3]. Absolute DDCs can easily be converted into relative ones by dividing ordinate values by the estimated asset value, and vice-versa. Relative DDCs are convenient, since they can be easily transferred across different regions, and applied with site-specific asset values.

DDCs are derived for each land-use type in the considered area. The curves for forests or agricultural land are generally obtained from less detailed data than DDCs for residential or industrial areas, or road and rail networks [3]. There are two approaches to DDC derivation: from real survey data (ex-post) and from synthetic data (ex-ante, "what if" approach), i.e. expected values based on the assumptions on damage magnitudes [9]. The latter approach generally leads to overestimated damages, as even severe floods do not always cause total damages, i.e. total loss of assets.

Derivation of DDC is a quite challenging task. The following explanations about DDC creation are based on the example of DDC for a residential building. Figure 17 shows ex-ante derivation of the absolute DDC for a single two-storey residential building, considering the relative share of damages to the individual dwelling components into the total damage (Figure 18). Abrupt jumps in the DDCs in Figure 17 are noticed when the water stage reaches a level at which power sockets are put up, or the second floor of the building.

Similarly to the asset values, the DDCs have to be extrapolated to much wider areas (e.g. a region). In other words, DDCs have to be representative of many buildings of varying characteristics within the considered area. To this end, DDCs are created either by averaging data for numerous buildings, or by analysing data after flood events. For example, the flood damage data for the U.K. are available from the Flood Hazard Research Centre (FHRC), and for Germany from the HOWAS database [9].

Averaging data from numerous flood events and/or on numerous buildings results in smooth curves, without any abrupt jumps that are apparent in DDCs derived from synthetic data (ex-ante approach). Regional DDCs obtained for the Rhine and Elbe basins are shown in Figure 19.

Considerable variations in elevation, robustness of the building structure, presence of cellar and furnishing luxury [27] result in an extensive scatter in damage data, as shown in Figure 20. This means that, although DDCs derived either by averaging over numerous buildings or from the post-event survey data are considered representative of the entire considered region, they are accompanied by enormous uncertainties (see Figure 20). To take differences among the buildings into account, different DDCs are obtained for few distinctive residential building types in the U.K. based on the data from the FHRC (Figure 21).

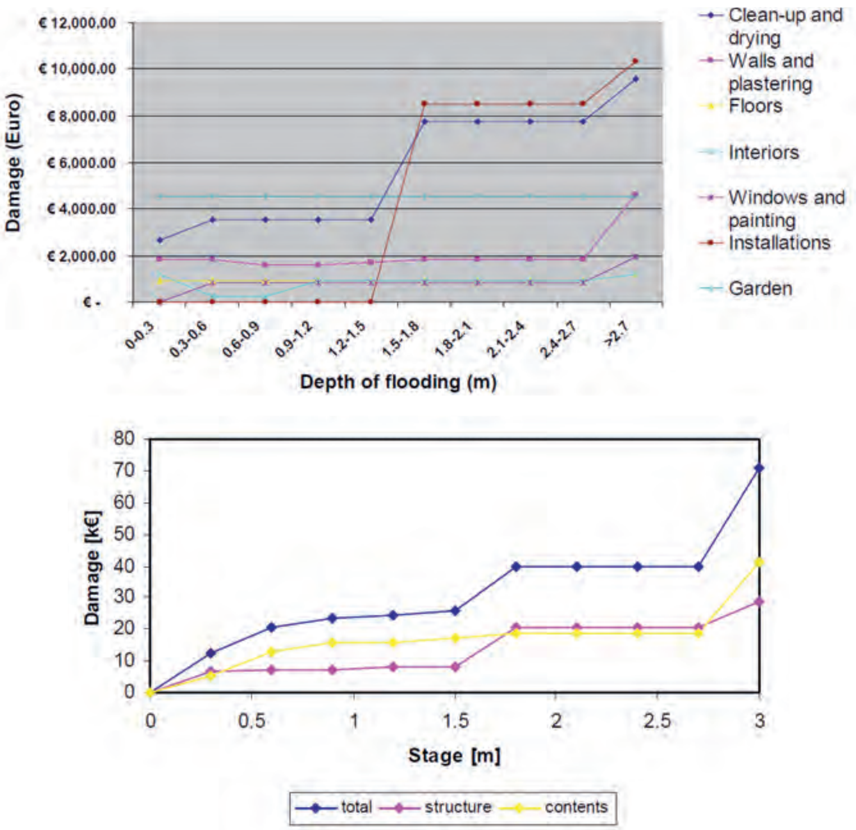

Figure 17. DDC for a two-storey house: development of absolute DDCs (top panel), individual damage components of the DDC (bottom panel) [11]

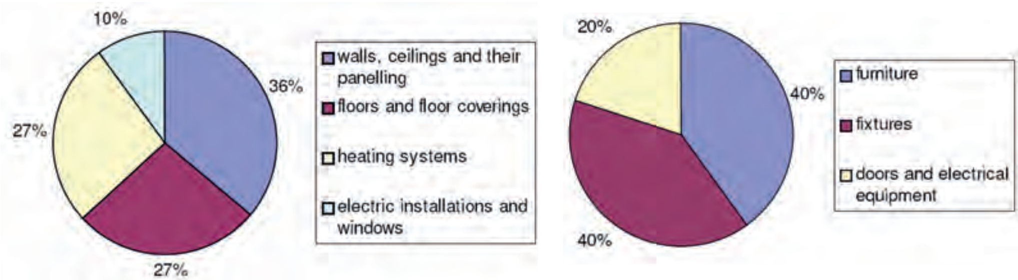

Figure 18. Damages to dwellings: structure (left panel) and properties (right panel) [11]

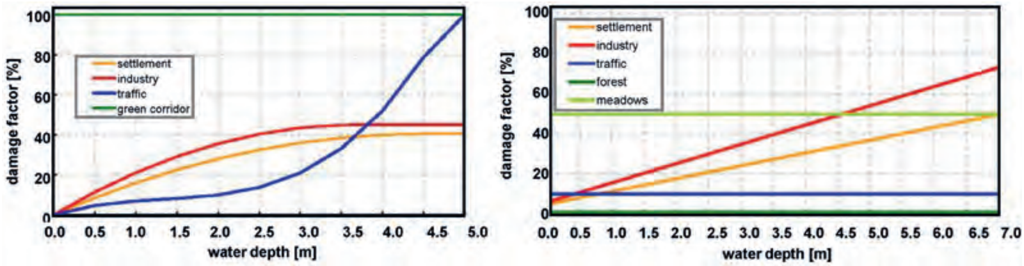

Figure 19. The IKSE (the Elbe River, left panel) and IKSR (the Rhine River, right panel) relative DDCs [3]

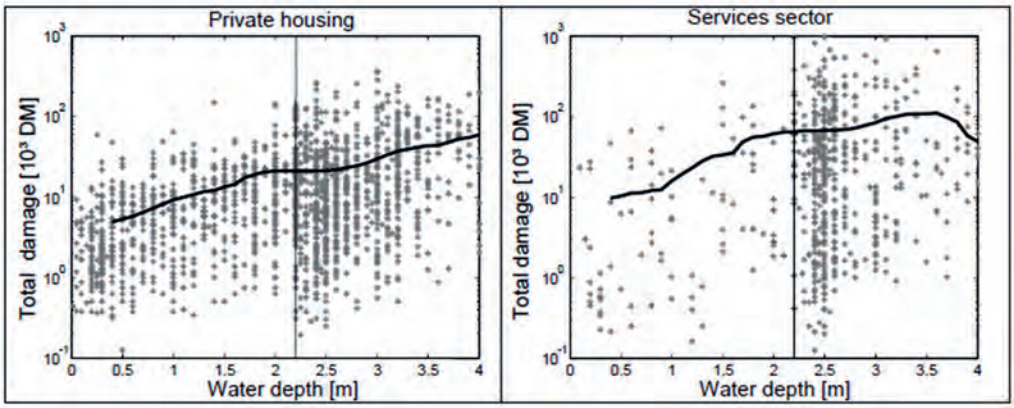

Figure 20. Scatter plots of depth–damage data [28]

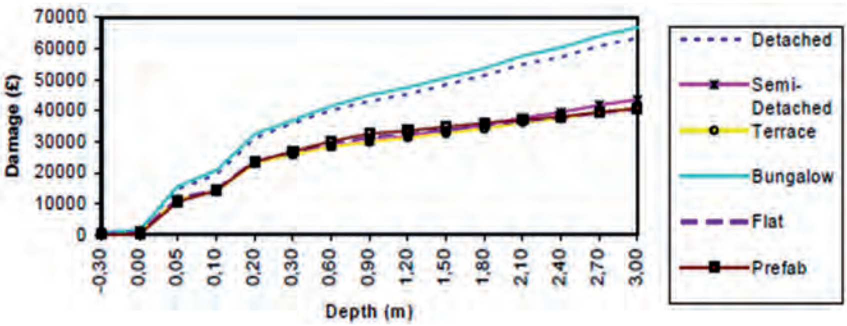

Figure 21. Absolute DDCs for different residential building types in the U.K. [9]

Expected Annual Damage (EAD)

One area can be affected by flood events of various probabilities/return periods (e.g. 50-year or greater). Each of these floods causes damages of different magnitudes. Therefore, consequences of floods of various probabilities should be taken into account for the purpose of flood risk assessment. To this end, expected annual damages (EADs), which combine the probability of flood events that affect the considered area, and the corresponding flood damages are computed [9]. This is in line with the definition of flood risk, which is a product of flood hazard, i.e. the probability of occurrence and flood consequences, represented in monetary terms (see section *Definition of flood risk* and Figure 3).

To calculate EAD, the product of flood probability P and consequent damages D is represented in an integral form as follows [6]:

$$\mathrm{EAD} = \int_{P_{crit}}^{P_{max}} P(h) \cdot D(h)\, dP \qquad (2)$$

where h stands for the water depth, and P(h) and D(h) denote the probability of a given water depth (i.e. flood event that results in a given water depth), and consequent specific damage quantified in monetary terms, respectively. Specific damage dependence on the flood probability is illustrated in Figure 22. The value of the integral in equation (2), i.e. the blue area below the function in Figure 22 represents EAD [5] [6].

The lower limit of integration Pcrit is the probability of the critical flood event that causes flooding of the considered area and triggers damages [3]. In other words, flood events of higher probability of exceedance (shorter return period), do not trigger flooding of the area, and, hence, do not cause any damage. The probability of the critical flood event should be defined bearing in mind that some minor flood events do not cause any measurable damage [9]. The upper limit of integration is set arbitrarily to a quite small probability of exceedance (e.g. 0.001 or 10,000-year return period). Flood damage at lower probabilities of exceedance cannot be calculated, and it represents residual risk [3]. The integration of equation (2) is performed by applying the trapezium rule, assuming a linear increase in damages in between two characteristic flood probabilities [3].

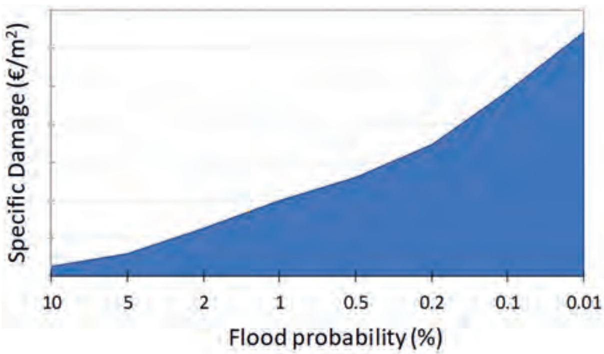

Figure 22. Specific damage as a function of the flood exceedance probability (compiled by the author)

To obtain the curve shown in Figure 22, specific damages have to be estimated for several floods of characteristic probabilities/return periods. For example, in Figure 22 damages are obtained for the following return periods: 10-, 20-, 50-, 100-, 200-, 500-, 1,000- and

10,000-year return periods. These specific damages should be computed for each LUT in the considered area, following the methodology described in the previous sections.

It should be noted that damages due to flood event do not represent actual damages, but rather rough assessments of the expected damages [9]. As stated previously, there are considerable uncertainties in DDCs, as well in the asset value assessment. Also, measures that can be taken prior to and during a flood event, such as relocation or elevation of movable assets, or putting up shields around a building or a property, are not accounted for in this approach although these measures can mitigate flood damages. Therefore, the EADs also represent approximations of average flood damages in a given year.

Flood risk assessment

According to this methodology, a level of flood risk is inferred from EAD, taking into account LUT. Additionally, for assessment of flood risk levels two things have to be defined: namely, risk levels (e.g. high, medium and low) and EAD threshold values that allow differentiation among different risk levels. For most LUTs, following three levels of flood risk are recognised: high, medium and low [3].

The threshold EAD values depend on the LUT, but also on the geographical region. Specifically, flood risk levels are related to costs of flood insurance per year, which, on the other hand, depend on EAD in a considered area. For example, the threshold for high flood risk should be EAD equal to the unacceptably high flood insurance costs. Considering that insurance costs and "unacceptable costs" considerably vary, there are no unique, generally accepted EAD threshold values.

For example, for residential areas, the EAD value of 0.1 €/m^2/year can be used to discriminate between low and medium flood risk, and EAD value of 1 €/m^2/year can be used to identify high risk areas (i.e. annual insurance costs of 1 €/m^2 can be considered prohibitive and unaffordably high by most citizens) [3]. The same principle is applied for flood risk assessment in agricultural areas. In these areas, only low and moderate levels of flood risk are recognised with EAD value of 0.012 €/m^2/year as the threshold EAD value [3]. Note that these EAD threshold values can vary with the region, depending on the economy of the region and insurance policies.

As stated in the previous sections, this methodology for flood risk assessment does not consider risks to human health and lives. However, flood risk to people can be readily estimated from the product of water depth and velocity, and by imposing threshold values given in Table 2.

Flood risk maps

Information on flood risk levels are easily obtained and communicated to the public and decision-makers via flood risk maps, which are obtained by using a GIS tool.

In addition to effective visualisation, GIS tools greatly facilitate the manipulation of different data (e.g. inundation maps, LUT and asset values) and they are essential for flood risk assessment. Flood risk maps are obtained following the methodology elaborated in the previous sections. The process of creating flood risk maps by using QGIS is described and illustrated with examples in this section. Being freely-available, QGIS is preferred over e.g. ArcMap.

The latter is certainly more user-friendly, and offers more features, however, it is not freeware.

Inundation maps. Flood flow rates or hydrographs are computed and routed externally, by employing appropriate hydrologic and hydraulic models. Simulated water levels during flood events of different return periods are exported from the hydraulic model to a GIS environment. For example, water levels simulated with HEC-RAS can be easily imported to ArcMap or QGIS by using appropriate plug-ins (as explained in section *Flood routing and inundation maps*), resulting in inundation maps.

Land use types. LUT layers have to be either imported in the GIS environment or created based on the e.g. orthophoto maps. Specific asset values can be appointed to each LUT as by adding a column in the attribute table and entering estimated values, as shown in Figure 23. LUT data are commonly made/available as vector shapefiles. For the purpose of flood damage computations, a raster version of LUT layer is required. A LUT layer can readily be rasterised e.g. with the Rasterise function (under Raster drop-down menu, command Conversion in QGIS).

Since DDCs differ across LUTs, pixels with one LUT have to be extracted, as shown in Figure 24. In this way, pixels with industrial LUT are assigned value 1, and the remaining value 0.

Damages due to flood event of a given return period. These damages (in monetary terms) are easily calculated by applying the Raster calculator, as shown in Figure 25. In this example, the IKSR DDC is applied to estimate damages due to 1,000-year flood in industrial zones. Note that the auxiliary raster layer enables that the damages are estimated for pixels with industrial LUT only. Damage computation should be repeated for all LUT and all return periods considered.

Expected Annual Damage (EAD). EAD is easily calculated from the estimated damages, by applying the trapezium rule (see subsection *Expected Annual Damage [EAD]*) with the raster calculator, as shown in Figure 26. Note that values in “Raster calculator expression” in Figure 26 correspond to differences between the flood probabilities (e.g. 0.005 is the difference between 1/100 and 1/200).

Flood risk maps. Based on the EAD and adopted threshold values for every LUT, flood risk maps are obtained. Initially, flood risk can be derived for individual LUTs, considering different threshold values or risk categorisation, and merged into a single raster file with the command Merge raster layers in QGIS environment.

The maps created for the purpose of flood risk assessment in the Resava catchment are shown in Figure 27.

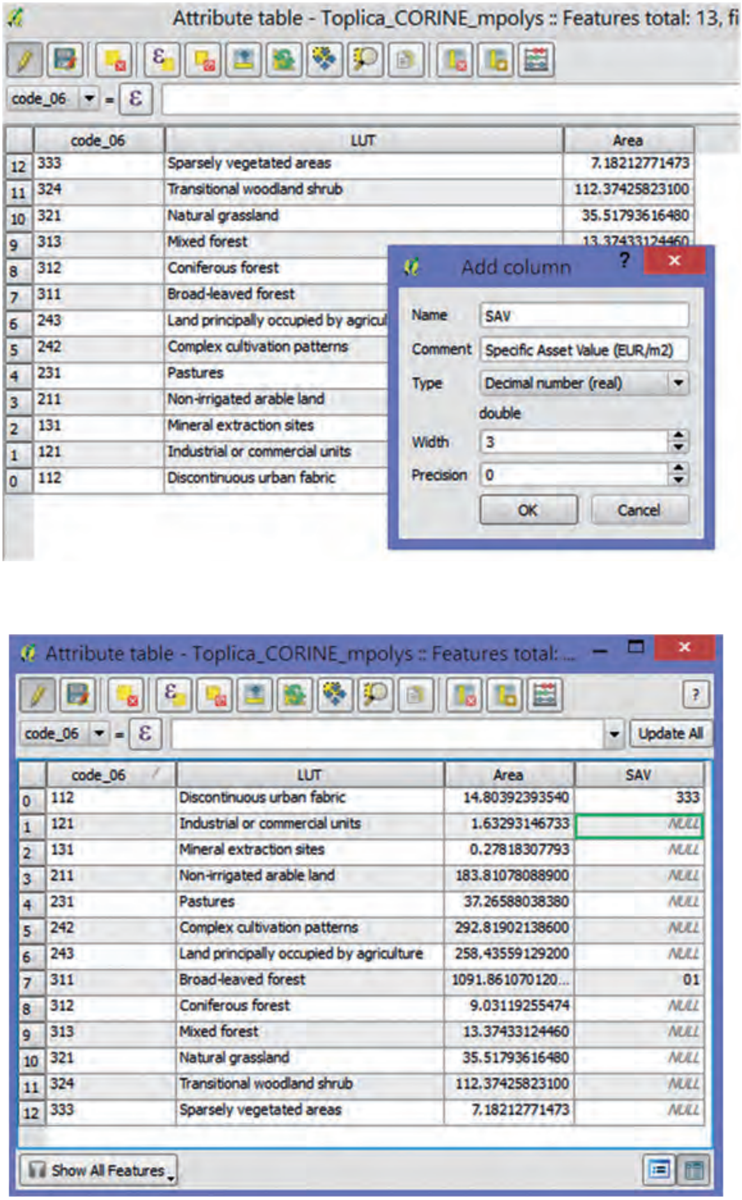

Figure 23. Appointing specific asset values in GIS environment to each LUT (compiled by the author)

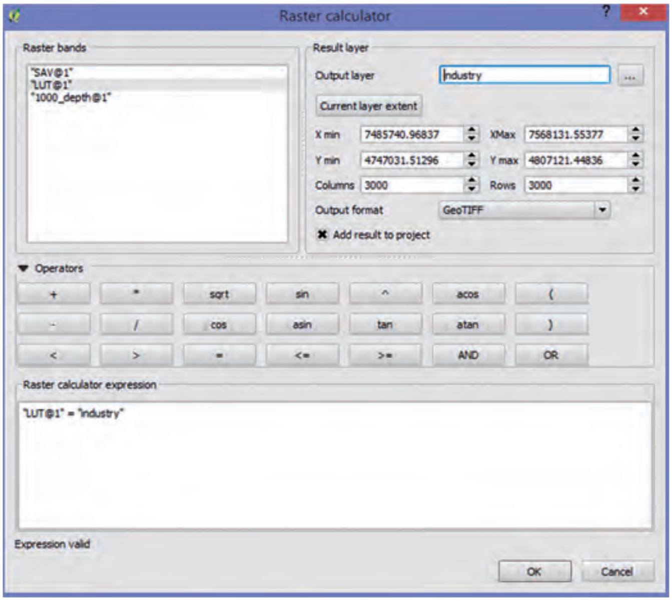

Figure 24. Identification of pixels with industry (compiled by the author)

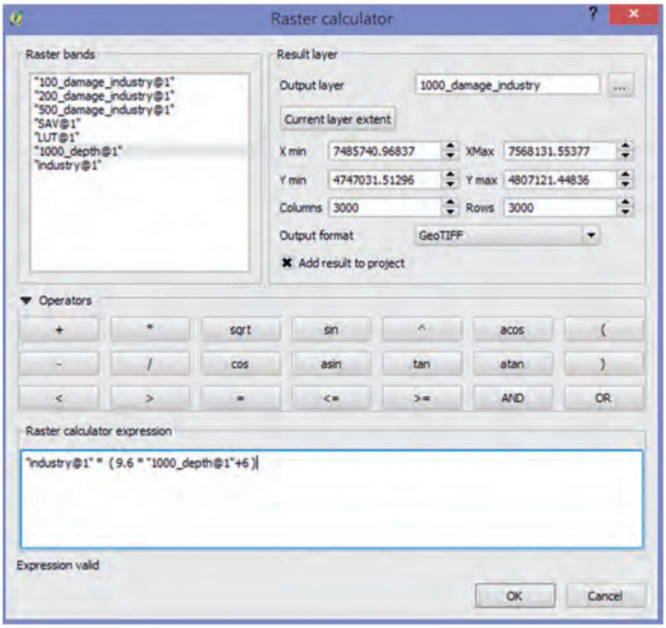

Figure 25. Calculation of damages due to 1,000-year event in industrial zones by applying IKSR DDC (compiled by the author)

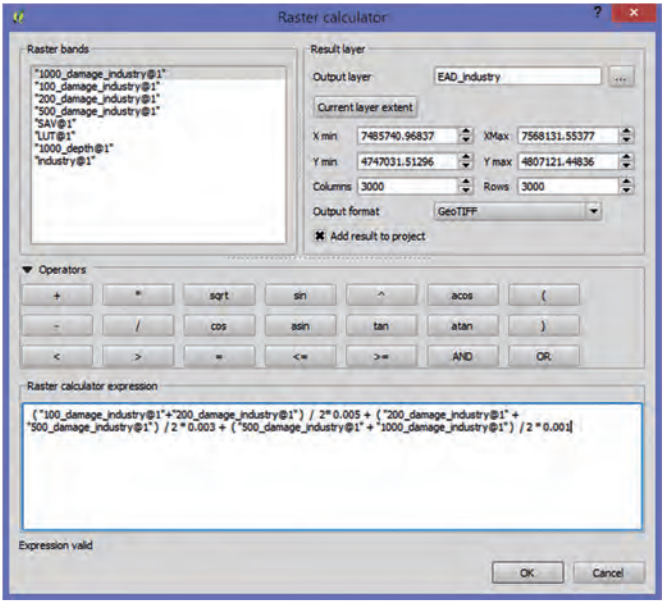

Figure 26. Calculation of expected annual damages by applying the trapezium rule to 100-, 200-, 500- and 1,000-year flood events in industrial zones (compiled by the author)

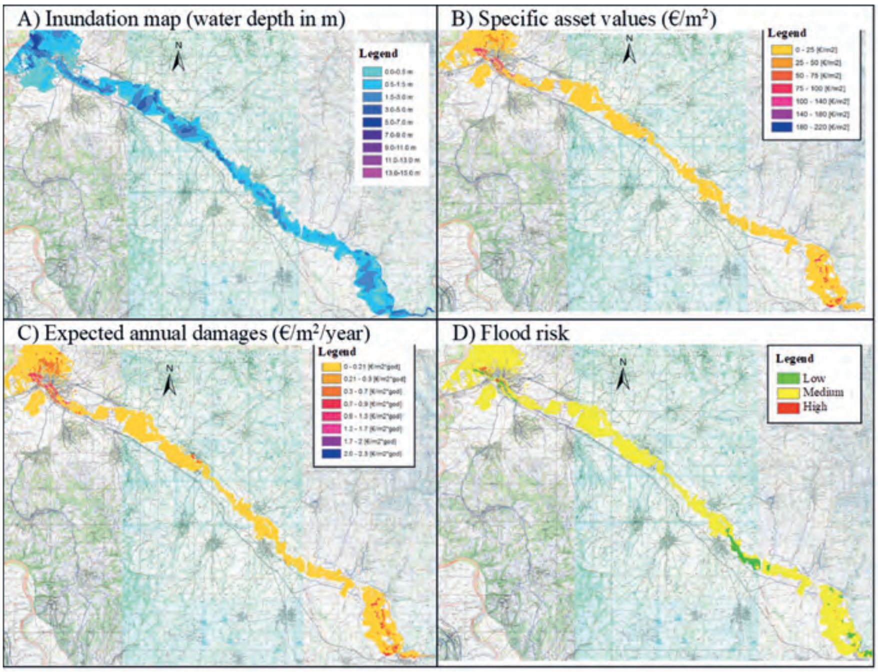

Figure 27. Flood Risk Mapping in the Resava catchment: A) inundation map (1,000-year return period); B) specific asset values; C) expected annual damages; and D) flood risk [29]

Glossary

Assets – represent entities of value in a considered area, such as houses, vehicles, inventories, infrastructural systems (e.g. traffic, water supply and sewer systems), etc.

Consequence – negative flood impacts, which generally include social, economic and environmental impacts. Consequences can be represented in terms of monetary loss or qualitatively (e.g. high, low). Only economic consequences are considered in this course.

Damages – losses due to flood expressed in monetary value. Here, only direct, tangible damages are considered.

Depth–damage curves (DDC) – functions showing dependence of damages on water depth for a considered land use type (e.g. residential or industrial areas, agricultural land, etc.).

Absolute DDC – show damages expressed in monetary value versus water depth.

Relative DDC – show damages in relative terms, as share of total asset value.

Direct damages – damages caused by direct contact with flooding water (e.g. damages to structures, industrial facilities, infrastructural systems, crops, etc.).

Expected Annual Damages (EAD) – specific damages in a considered area calculated by combining various probabilities of flooding (e.g. 50- through 1000-year return periods) and the corresponding damages, represented by their monetary value. EAD is given in €/m^2/year.

Exposure – the situation that people, infrastructure, housing, production capacities and other tangible human assets are being situated in a flood-prone area [8].

Flood – An overflow of a large amount of water beyond its normal limits, especially over what is normally dry land (source: https://en.oxforddictionaries.com/definition/flood).

Flood hazard – related to frequency of flooding. Flood hazard is great in flood prone areas.

Flood risk – risk is a function of probability, exposure and vulnerability. Here, it is calculated by multiplying of flood probability by its consequences, which are quantified in terms of monetary loss.

Flood risk assessment – procedure of estimation of flood risk in a certain area, according to the methodology adopted, including thresholds used to differentiate among different risk levels.

Flood risk maps – maps showing different degrees of flood risk across a region of interest. These maps clearly indicate high risk areas, and can be easily used by decision-makers and citizens.

Flow quantile – flow rate of a given probability of exceedance or return period. This value is estimated by applying statistical methods (e.g. Flood Frequency Analysis).

Hazard – a potential source of danger or a harmful event.

Flood hazard – probability of flooding of certain area.

Indirect damages – damages accompanying direct ones. Indirect damages are not caused by direct contact with flooding water. These damages include e.g. loss in production and income, loss due to traffic disruption, etc.

Intangible damages – damages that are difficult to represent in terms of monetary value [9]. These include e.g. loss of human lives, negative effects on health and environment, damages to cultural heritage, etc.

Inundation – flooded area that are otherwise dry.

Inundation maps – maps showing the flood extent and water depth in each pixel of the inundated (flooded) area for a flood of a given return period. These are obtained by overlying common terrain maps by the results of hydraulic simulations.

Inventories – household contents or, for businesses, stocks of outputs that are still held by the units that produced them [9].

Land use types – include e.g. residential zones, industrial zones, agricultural land, forest (deciduous or coniferous), shrubs, traffic, etc.

Probability – here, the term “probability” is used to represent the probability of exceedance, i.e. the reciprocal value of the return period.

Return period – mean time interval between exceedances of a specified flow. It is calculated as a reciprocal value of the flow probability of exceedance and is expressed in years [15].

Specific asset value – asset value per unit area (e.g. €/m^2). It is obtained by estimating the total asset value within a considered area, and dividing this estimate by the total area.

Tangible damages – damages that can be readily quantified in monetary terms. Tangible damages include e.g. damages to buildings or infrastructure.

Vulnerability – potential of a system to be harmed. According to UN, vulnerability is defined as: “the conditions determined by physical, social, economic and environmental factors or processes which increase the susceptibility of an individual, a community, assets or systems to the impacts of hazards” [8].

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Enikő Anna Tamás

River Regulation and Sediment Transport

Introduction

Sediment monitoring on the river Danube started as early as the end of the 19th century, with scattered measurements carried out. Regular sediment sampling was developed in the first half of the 20th century all along the river, with different station density and monitoring frequencies in different countries. After the first few decades of regular sampling, the concept of (mainly industrial) development changed along the river and data needs changed as well, furthermore the complicated and inexact methods of sampling bed load on the alluvial reach of the river were not developed further.

River regulation works were started in the 19th century and are still going on along the Danube. The aim of the early activities was flood protection and to improve conditions for navigation. In the course of high-flow regulation, lowland areas along the river were protected from inundations. The first dikes had a local role, later they were followed by higher levees, stretching for a long distance along the river.

Both sediment transport processes and river regulation activities have been playing main roles in the morphological development of our rivers, and, as they are very much interconnected, we shall try to understand them in order to be able to draw consequences as regards to the flood conveyance capacity of our rivers.

Basics of sediment transport

The importance of the monitoring of sediment processes is unquestionable: sediment balance of regulated rivers suffered substantial alterations in the past century, affecting navigation, energy production, fish habitats and floodplain ecosystems alike; infiltration times to our drinking water wells have shortened, exposing them to an eventual pollution event and making them vulnerable; and sediment-attached contaminants accumulate in floodplains and reservoirs, threatening our healthy environment. The changes in flood characteristics and rating curves of our rivers are regularly being researched and described, involving state-of-the-art measurement methods, modelling tools and traditional statistics. Sediment processes, however, are much less known. Unlike the investigation of flow processes, sediment-related research is scarce, which is partly due to the outdated methodology and poor database background in the specific field.

Sediment-related data, information and analyses form an important and integral part of civil engineering in relation to rivers all over the world. In relation to the second largest river of Europe, the Danube, it is widely known in expert community and for long discussed at different expert forums that the sediment balance of the river Danube has changed drastically over the past century.

The most important parameters describing fluvial sediment transport are sediment load, Q_s, meaning the amount of sediment (volume or mass) passing through a given cross-section during a specified time; sediment yield, G_s, which is the mass of sediment passing by during a specified period of time; and, for suspended sediments, sediment concentration, cs, which is the ratio of the mass of sediment and the volume of the water in which it is contained.

The measurement of fluvial sediment is based on sampling procedures, as a result of which, based on protocols, sediment load and concentration can be calculated. Based on regular sediment measurement, it is essential to estimate the correlations between flow characteristics and sediment parameters, for different water regime conditions. The best way to achieve this is to draw up sediment rating curves.

The frequency of suspended sediment sampling is very low along the river, it is best organised in the upstream countries, where also on tributaries like the Drau/Drava monitoring stations are in operation. Sampling frequency of suspended load is 3 to 7 per year in Hungary, and even lower downstream.

Sediment management is a major challenge, as most methods developed until now are unsustainable, require continuous intervention and are also expensive. However, there is a new focus on the subject in the 21st century, which still lacks uniform methodological recommendations for measurements and analyses, and the number of engineers with sediment expertise and experience is alarmingly low. Data related to sediment quantity are unreliable and often contradictory. It is difficult to produce high quality long-term databases that could support and enable the mathematical calibration of sediment transport models. Furthermore, global changes in river basins due to climate change or changes in land use practices, as well as erosion conditions, make sediment monitoring and quantitative sediment management a very important task for the near future, especially under the recent impacted conditions and in light of the Water Framework Directive of the EU.

Definitions

Sediment in hydro-engineering constitutes of materials transported or deposited by the river (not including floating debris and organic matters).

Sediment transport is a term which collects processes connected to the erosion of the banks and the riverbed, the transport of sediments and the deposition.

Sediment can be characterised quantitatively and qualitatively. In river management, we focus mainly on the quantitative determination of the sediment. Thus, we can speak of sediment load, sediment yield, sediment concentration, etc.

Sediment load means the mass (or the volume) of the transported sediment in a time unit (second) in a given cross-section of a river:

$Q_s = m_s/t$ [g/s]

Sediment concentration means the ratio of the sediment load and the discharge in a given section:

$c_s = Q_s/Q$ [g/m^3]

Sediment transport is not a uniform phenomenon. The transportation of solid matters along the rivers is generally distinguished by the type of movement of the sediment particles, which can migrate to each other. Basically, there are three types of sediments to be distinguished.

Bed material is the material forming the riverbed (characteristically cobbles, gravels, sand, silt and clay). Bed material is not necessarily sediment, as on upper reaches of the rivers it can also be a solid rock which is naturally not transported.

Bed sediment (-load) consists of relatively coarser sediment particles moving on the top of the riverbed, relatively close to it, by jumping, rolling, sometimes rising and settling back down. It usually consists of relatively larger particles which often settle to the bed material and vice versa, bed material is sometimes mobilised and transported as bed load.

Bed load particles are not moving "individually", because they interact with each to a large extent.

Suspended sediment (load) constitutes of relatively smaller size sediment particles, which are kept in suspension by the turbulent flow. Suspended sediment is moving with almost the same velocities as the water transporting them. Suspended sediment particles have almost no effect on each other.

Wash (or dissolved) load includes the sediment particles which are so small that they are continually kept in suspension by the flow. We sometimes distinguish this type in theory, but in practice it is very hard to separate these from suspended load particles, so when measured and/or calculated we consider these together with suspended load.

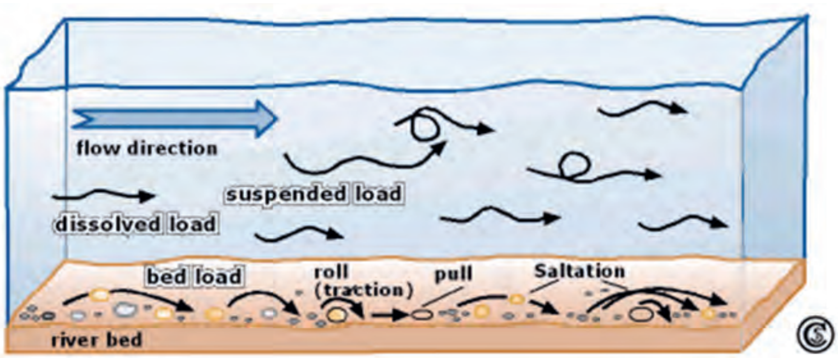

Figure 1. Types of sediment transport (Anette Stumptner after Christopherson 1994, 424[1])

[1] Online: www.geo.fu-berlin.de/en/v/iwm-network/learning_content/environmental-background/fluvial_processes/

According to the above, we can group sediments into the following types:
wash (or dissolved) load, Q_{sw}
suspended load, Q_{ss}
bed load, Q_{sb}
The total sediment load thus can be calculated by adding up the different types:
$Q_s = Q_{sb} + (Q_{sw} + Q_{ss}) \sim Q_{sb} + Q_{ss}$

Physical characteristics of the particulate matter, such as weight and size, determine when the grain is suspended and when it sinks to the bottom. During periods of high discharge, the turbulence can lift up larger grain sizes, which are then transported temporarily as suspended load. The threshold of the minimum grain size for suspension strongly correlates with the flow velocity and turbulence.

Transport is further characterised by grain sorting, dispersion and mixing processes. Most of the particles do not move exclusively on the top or the bottom but are continuously exchanged (large particles come in and displace smaller particles as the bottom layer is continuously rotating).

Therefore, it is important to know the particle or grain size distribution (PSD) of the sediment, which is the ratio of different particle sizes in a given sample or in a vertical/section.

Why sediments are moving

For a fluid to begin transporting sediment that is currently at rest, the bed shear stress exerted by the fluid must exceed the critical shear stress for the initiation of motion of grains at the bed.

$t_b > t_c$, or, dimensionless $t_b* > t_c*$

The critical shear stress is a function of the Reynolds number related to the particle

$t_b* = f(Re_p*)$

Particle Reynolds number

$Re_p = u_p D/n$,

where u_p is characteristic particle velocity, D is the grain diameter and n is the kinematic viscosity.

The specific particle Reynolds number is formed by replacing the velocity term in the Particle Reynolds number by the shear velocity

$u* = SQRT(t_b/\square f)$
$Re_p* = u* D/n$

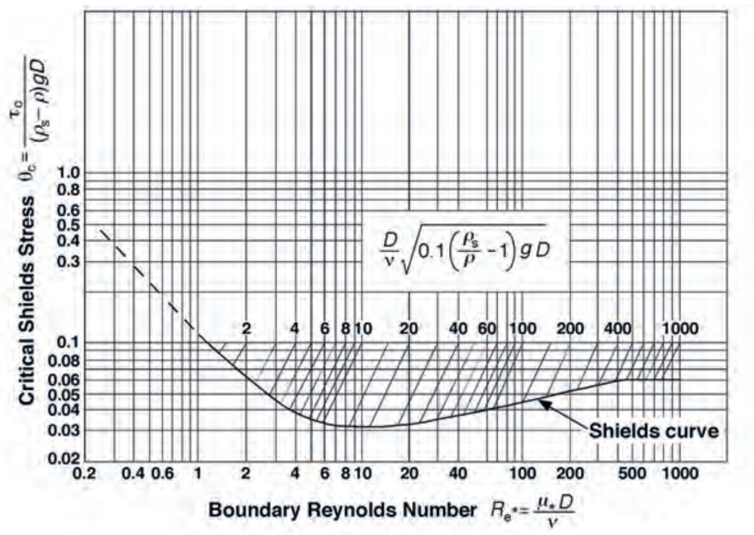

Figure 2. The Shields diagram (Shields, 1936) Anwendung der Aehnlichkeitsmechanik und der Turbulenz-forschung Auf Die Geschiebebewegung, Zugl.: Berlin, TeH., Diss., 1936

The settling velocity

w_s

is a function of the particle Reynolds number. Generally, for small particles (laminar approximation), it can be calculated with Stokes's Law. Sometimes it is also called "terminal velocity" or "fall velocity". We can use it in PSD determination.

Stokes's Law

The force of viscosity on a small sphere moving through a viscous fluid is given by:

$F_d = 6phRv$,

where:
F_d is the frictional force (Stokes's drag) acting on the interface between the fluid and the particle
η is the dynamic viscosity
R is the radius of the spherical object
v is the flow velocity relative to the object
Fd [N = kgms^{-2}], η [Pa = kgm^{-1}s^{-1}], R [m] v [m/s]

Why to know sediment processes

There are several concerns related to sediments in aquatic systems. The problems related to high amounts e.g. include reduced capacity in reservoirs, and increased dredging requirements in shipping channels. Another concern with eroded sediments is that they can transport other pollutants into receiving waters. The plant nutrient phosphorus, for example, is most often transported from the fields where it was applied as fertilizer by chemically bonding to clay minerals. Many agricultural pesticides also bond to eroded clays and organic matter. Once these chemicals have entered the aquatic ecosystem, many processes occur that can result in the release of the pollutants from their sediment carriers.

Sediment size	Environmental issues	Associated engineering issues
Silts and clays	Erosion, especially loss of topsoil in agricultural areas; gullying	
	High sediment loads to reservoirs	Reservoir siltation
	Chemical transport of nutrients, metals, and chlorinated organic compounds	Drinking-water supply
	Accumulation of contaminants in organisms at the bottom of the food chain (particulate feeders)	
	Silting of fish spawning beds and disturbance of habitats (by erosion or siltation) for benthic organisms	
Sand	River bed and bank erosion	River channel deposition: navigation problems Instability of river cross-sections
	River bed and bank erosion	Sedimentation in reservoirs
	Habitat disturbance	
Gravel	Channel instability when dredged for aggregate	Instability of river channel leads to problems of navigation and flood-control
	Habitat disturbance	

Figure 3. Typical issues associated with sediment transport (compiled by the author)

The determination of sediment transport can be carried out in various ways. The most commonly used grouping of sediment determination is detailed here.

Data on riverine sediment transport rates can be derived typically from direct measurement techniques, but because these are very resource demanding, a common approach is to use transport equations, hydraulic models, transport curves or other estimating techniques instead. Yet, a reliable set of measured field data would still be required to calibrate these methods and to find the correlations which ensure their exactness.

Thus, the most reliable and useful fluvial–sediment–discharge data are derived from direct measurements of suspended sediment (along with water discharge) and/or bed load transport.

Sediment transport measurements

We generally need to measure sediment transport of streams in the field in order to quantify transport rates, which usually means separate measurements for suspended sediment load and bed load, based on which we can give the amount of total load. The results of the measurements can later be used in the calibration and/or verification of sediment transport rates derived from equations or hydraulic models in the calibration of sediment surrogate technologies.

During sediment measurements we obtain samples, which are also suitable for analyses of PSDs, concentrations of chemical constituents, and to quantify other selected characteristics of the entrained material.

Measurement technologies can be discrete or continuous.

The traditional type of sediment measurements are discrete measurements. Due to modern technologies, the measurement of time series, or continuous measurement is also possible, but in order to do this we use sediment surrogate measurements.

We have to mention though that suspended phase surrogate measurements tend to be spatially limited (i.e. not adaptable for bigger streams or long stretches), and although bed load surrogate technologies have been developed and field tested but none has yet been fully integrated into operational monitoring.

It is also very important that all the surrogate technologies require calibrations with data produced by reliable physical samplers and sampling methods. That is why direct field measurements still have an unquestionable importance in getting to know sediment-related phenomena.

Sediment sampling

Suspended load measurement

Direct measurement of SSL is possible either with a pump, or with a bottle. In general, we have to try to ensure that our sampling method is isokinetic.

In the following, the methods of sediment sampling in Hungary are described. However, we have to mention that in other countries sampling methods are not always comparable, and thus the data obtained from the different measurement methodologies must be very carefully used because of possible inhomogeneity issues.

When sampling suspended load in the whole cross-section, in Hungary it is usually three verticals and two samples per vertical. Before analyses, the samples taken from different depths are joined, thus the results in the grain size distribution give a vertical average only instead of a 2D distribution in the cross-section.

The most effective way of sampling suspended load is with a pump. An advantage is that it is not needed to regain the sampler onboard between the points. Thus, this method is the fastest, which is an issue, particularly at high velocities and when sampling is done in the navigation route. During sampling, it is very important to ensure that the sampling nozzle faces the flow, the pipe is not bent and to let enough time before taking samples to flush the pipe.

Sampling needs to be carried out in an isokinetic way as mentioned before, so with care to adjust the revolutions per minute value (RPM) or the discharge of the pump for the velocity through the nozzle V_{in} should not differ much from the velocity of the flow v at the given point:

$$0.8v \leq V_{in} \leq 1.5v$$

In case the velocities are outside this range, the RPM of the pump should be accordingly adjusted, or a tap should be installed at the end of the pipe to ensure that intake velocities match. In order to determine intake velocity, the discharge of the pump (qp) has to be divided by the cross-section area of the nozzle (fn):

$$V_{in} = qp / fn$$

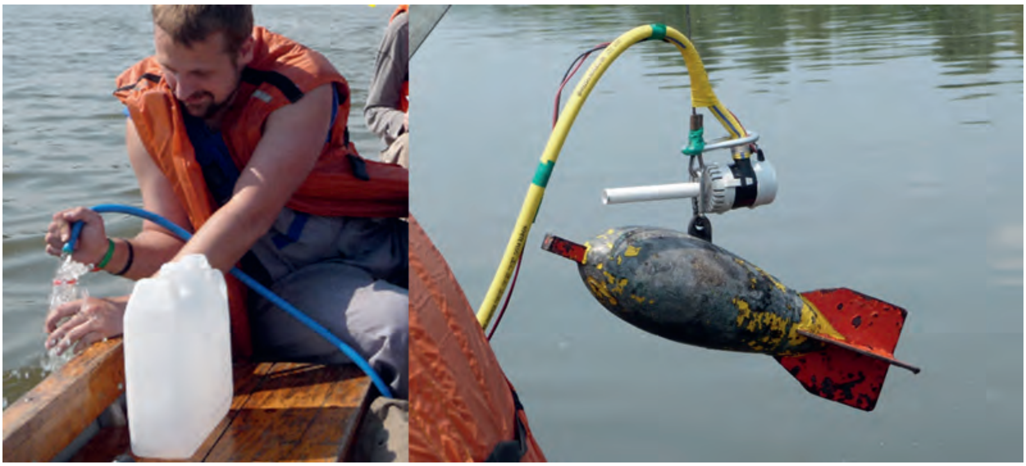

Figure 4. Sampling suspended load with a pump (photo taken by J. Sziebert)

In practice, we perform sampling with a constant pumping discharge, assigning a fixed intake velocity to different velocity ranges of the flow, keeping the hydraulic coefficient between the values 0.8 and 2.0. This ensures a maximum 20% difference in concentrations, which is acceptable.

Sampling with a wide range of different bottle samplers is also common worldwide. We mention here the two types of sampling bottles which are typically used in the Danube River Basin.

The first is the so-called Delft bottle sampler which was developed in the Netherlands. It operates on a flow-through principle. A reduction in flow velocity in the sample bottle between the nozzle and outlet results in collection of sand particles larger than about 0.1 mm for subsequent analysis.

Figure 5. The Delft bottle[2]

The second typical bottle sampler is the fish-shaped isokinetic sampler developed by the USGS. It shall also be deployed in several verticals. The apparatus can be lowered to the desired measurement point by a winch. It contains a bottle that can be removed and transported to the laboratory for analyses.

Figure 6. The isokinetic bottle sampler[3]

Surrogate measurements for suspended load determination include light or sound or turbidity sensors of different types. In these types of measurements, the determination of the sediment load would in every case require a reliable calibration.

If we already have a large enough time series of sediment data, it would be possible to determine Qs in form of the correlation between suspended sediment transport and water discharge, that is usually called the sediment rating curve.

The sediment rating curve can be obtained as the function of the discharge:

$$Q_s = a\,Q^b$$

[2] Online: www.royaleijkelkamp.com/products/augers-samplers/sludge-sediment-samplers/suspended-sediment-sampler-sets/

[3] A Guide to the Proper Selection and Use of Federally Approved Sediment and Water-Quality Samplers, https://pubs.usgs.gov/of/2005/1087/pdf/OFR_2005-1087.pdf

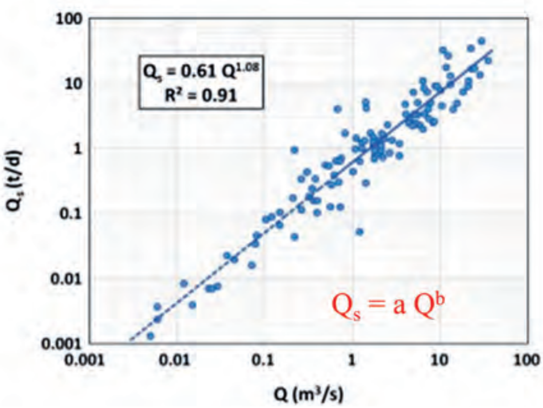

Figure 7. The sediment rating curve[4]

Based on the obtained sediment data for a certain cross-section, we shall draw up the sediment concentration distribution.

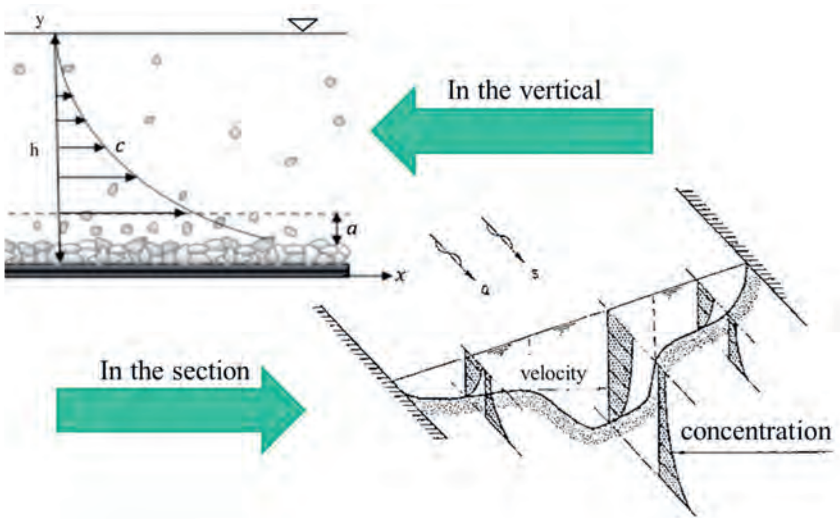

Figure 8. Typical sediment concentration distribution of a stream (compiled by the author based on [3])

4 Atieh, M, Mehltretter, SL, Gharabaghi, B, Rudra, R (2015): Integrative neural networks model for prediction of sediment rating curve parameters for ungauged basins,Journal of Hydrology, Volume 531, Part 3, pp 1095–1107, https://doi.org/10.1016/j.jhydrol.2015.11.008.

Bed load measurement

The sampler types which can be used to directly measure bed load can be:
- box or basket samplers
- pan or tray samplers
- pressure difference samplers
- trough or pit samplers

Direct sampling is difficult, as bed load transport is usually highly variable in space and time across the river, thus empirical equations based on sampling are questionable, and the results of empirically-based calculations or laboratory experiments are very difficult, or impossible to calibrate with field data.

The bed load of rivers is moving intermittently over the surface of the riverbed. As bed load samplers disturb the current, they have an effect on the transported bed load. When choosing the appropriate sampler, we have to minimalise disturbance. Sampling of the bed load usually happens with the Helley-Smith sampler (in fine sediment, e.g. sand) or the Károlyi sampler (in coarser sediment). Both samplers have different sizes and gaps for different river types, depending on the grain size and the mass flow of the sediment.

The samplers are lowered to the bottom of the river, and, depending on the sampling time (usually 10–15 minutes), based on the mass of the sample taken, bed load transport can be calculated:

$$q_b = \frac{G}{b \cdot T \cdot \rho_s},$$

where G is the dry mass of the sample
b is the width of the gap of the sampler
T is the sampling time and
ρ_s is the density of the bed load sample

In our experience, it is very useful to equip these samplers with an underwater camera, in order to be able to see the clogging of the sampler or anything blocking it; furthermore to determine the exact sampling time needed to collect a reasonable amount of sediment in the apparatus.

On the reaches where bed load is mainly sand, the Helley-Smith sampler is used, fitted with a camera. Depending on the expected coarseness of bed material, different size Helley-Smith samplers can be used. For example on the Drava River (right bank tributary to the Danube) for fine sand, a smaller sampler (length: 960 mm, height: 200 mm, wingspan: 400 mm, gap size: 152 × 152 mm, weight: 17 kg), and for coarse sand a bigger one (length: 1,550 mm, height: 240 mm, wingspan: 510 mm, gap size: 150 × 150 mm, weight: 35 kg).

Figure 9. The Helley-Smith sampler with onboard camera mounted above the inlet (photo taken by the author)

Indirect bed load measurement is theoretically also possible, though not widely used yet. For this, we have to define the virtual velocity of the bed material, which is the total distance travelled (possibly incorporating multiple steps) by individual grains divided by the measurement interval.

The ability of the ADCP to determine virtual velocities of bed material sediments for potential bed load transport determination has been recognised for some time. A few investigators have made good progress in making measurements of virtual velocity using the ADCP.

Bed material measurement

The sampling of the bed material is usually achieved with a bucket sampler. Because armouring of the riverbed is to be expected on the river Drava, the edge of the sampler was sharpened to facilitate its penetration into the riverbed (length: 535 mm, diameter: 180 mm, weight: 9 kg).

Processing of the samples

The PSD of bed load and bed material have to be determined.

According to the traditional method, these are determined using Taylor sieves – separating fractions (0.063; 0.125; 0.25; 1.0; 2.0; 4.0; 8.0; 12.0; 16.0; 24.0; 32.0; 48.0; 63.0; 96.0; 125.0 mm); or by settling velocity method – separating fractions (> 0.10 mm; 0.05 – 0.10 mm; 0.02 – 0.05 mm; 0.01 – 0.02 mm; 0.005 – 0.01 mm; < 0.005 mm).

Drying of the samples is carried out at 105°C. After that, dry matter content measurement (analytical precision) is done. In a case when the ratio of fractions with diameters less than 0.15 (0.1) mm is higher than 10%, the hydrometry method must be used to establish the particle size distribution.

For suspended load, which is usually contained in about 5l water samples, the first important step is to measure the exact amount of the sample, to know from how much water we will measure sediment concentrations. The samples are then left to settle. When the sediment settles in the bottom of the containers, the excess water is carefully sucked from the containers and approximately 0.5l is left. The amount of clean water removed is precisely measured and recorded in the protocol. The samples are dried in an electronic oven for 24 hours at 105°C temperature. Then, we measure the weight of each sample and its dry matter content on a precision scale. Concentration is calculated as

$C_{ss} = \frac{m_d}{V_s}$, where

m_d is the dry matter weight and

V_s is the total volume of the sample.

In case of suspended load, when the grain size is too small for screening, the grain size determination is done with a special Atterberg-type settling device, which is operating on the principle of the Stokes equation. The main part of the settling velocity meter is a cylindrical glass tube with an inner diameter of 35–40 mm. There are six level markings on the tube. On the bottom of the tube, there is a stricture and a tap. It ends in a ca. 4 mm diameter rider. On the top of the tube, there is a funnel with a throttle. With the throttle open, the tube is filled up with distilled water, and the sample is also poured into the tube through it. There is a vent in the axis of the throttle, connected with a 0.1–0.2 mm diameter nozzle, to secure that outflow velocity does not exceed 0.2–1 cm s^{-1}. The tube has to be mounted on a stand with its axis vertical.

Before filling the samples in the tubes, we leave them dissolve in aqueous solution of sodium metasilicate to avoid coagulation. Then we fill the tubes with sodium metalsilicate solution and we pour the samples into it. The grain size fractions are < 0.10 mm; 0.10–0.05 mm; 0.05–0.02 mm; 0.02–0.01 mm; 0.01–0.005 mm and > 0.005 mm.

Figure 10. Atterberg-type settling device for PSD determination of fine sediments (photo taken by the author)

The particle size distribution (PSD) of the different samples can be drawn up as percentages from the data as a distribution curve. The results of sediment analyses are summarised by PSD curves. The measured precise weight of the fractions can be drawn up as percentages from the data as a distribution curve in e.g. MS Excel.

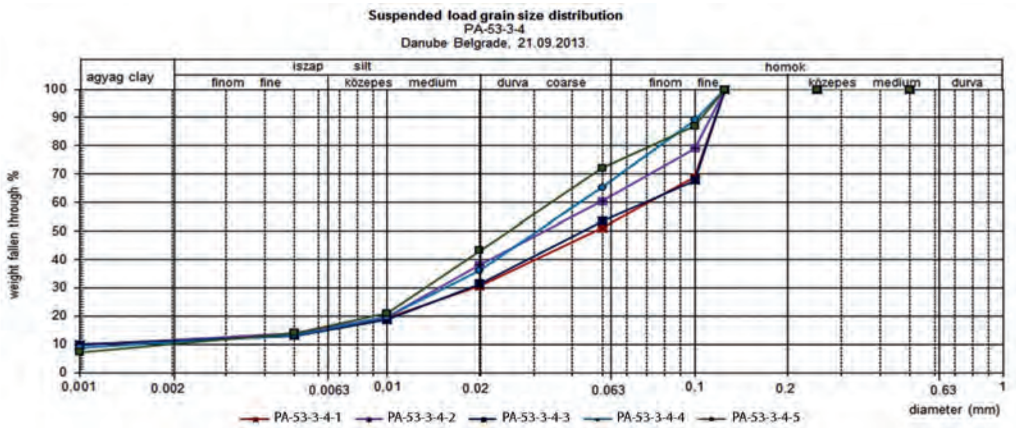

Figure 11. Typical PSD of the suspended load on the alluvial reach of the Danube River (compiled by the author)

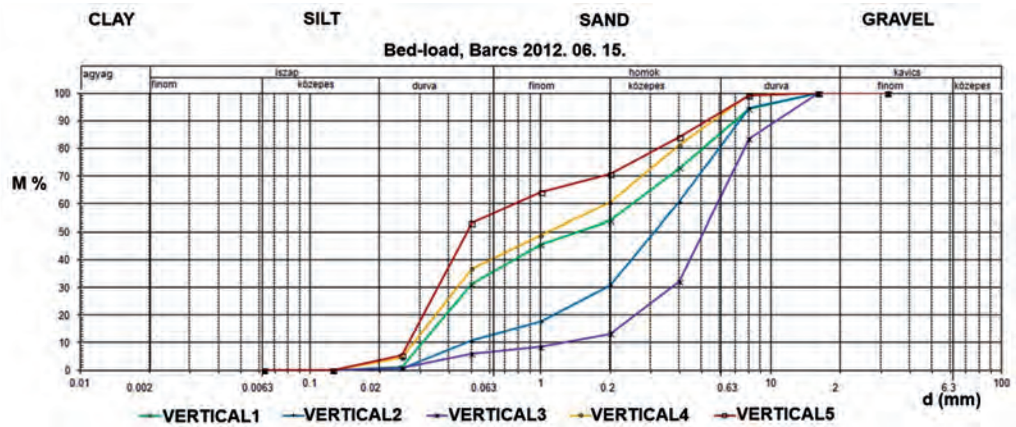

Figure 12. Typical PSD of the bed load on the middle reach of the Drava River (compiled by the author)

From the PSD we read the values of d_m, d_{eff}, d_g, d_{10} and d_{60} from the diagram and we calculate U unevenness factor ($U = d_{60}/d_{10}$), according to the Hungarian measurement standard.

Analyses based on sediment measurements

Concentrations of suspended load and grain size distribution curves of suspended sediment, bed load and bed material can be determined from each sample. Total yields can be also estimated if a simultaneous discharge measurement is available.

For a better description of sediment transport processes, we determine the correlation between discharge and suspended sediment concentration.

For the bed load and the suspended load, it is generally true for the whole Danube River (and its tributaries as well) that insufficient monitoring activities have been carried out, and long-term series are not available to assess the overall sediment balance on many rivers. Effects of important events or circumstances are often not captured. During floods, the amount of transported sediment can exceed the long-term average annual values. Also, the input of the sub-basins responsible for a flood in the main river strongly influences the concentration of the sediment load. Furthermore, in case of heavily eroded surface layers, it is possible that the river incises into softer sediment layers.

Data related to sediment quantity are unreliable and often contradictory. Even in countries where sediment sampling is well-organised and frequent, like Germany and Austria, it is difficult to produce high quality long-term databases that could support and enable the mathematical calibration of sediment transport models. Furthermore, global changes in river basins due to climate change or changes in land use practices, as well as erosion conditions, make sediment monitoring and quantitative sediment management a very important task for the near future, especially under the recent impacted conditions and in light of the WFD.

Basics of river regulation

River regulation (training or channelisation) includes those methods of engineering (resectioning, straightening, construction of levees, diversions, etc.) that modify existing river channels or create new channels, often changing the relationship between river channels and floodplains. Here we summarise these activities based on the Encyclopedia of Water Science [15].

Channelisation is carried out both on very large rivers and small streams; it is widespread in lowland rivers, but also many upland (mountain) rivers have experienced this type of human intervention. Human impact on rivers has a long history. Most alluvial rivers in Europe have been channelised during the last 2000 years and in the United States, the Federal Government has been regulating the rivers since the 1870s. Early regulation activities appeared in the Danube River Basin with the start of economic and industrial development; however, major interventions have been carried out in the 19th century.

Goals of river regulation

The most common purposes of river regulation are flood control, land drainage improvement, creation of new spaces for urbanisation or agriculture, maintenance or improvement of navigation and reduction of bank erosion.

However, along the Danube River itself, the most important goals tend to be the safe conveyance of floods and ice apart from the improvement of the navigation corridor.

Types of river regulation

Resectioning by widening and deepening

Widening and deepening increase the channel cross section; therefore, channel capacity to contain flows is increased and floodplain is inundated less frequently (flood control and agricultural purposes). In some cases this type of intervention is adopted to lower the water table for the improvement of agriculture. Channels are commonly designed with trapezoidal cross sections, but rectangular sections can be used where banks are stable (e.g. concrete banks) and triangular sections in small ditches.

Straightening

Straightening implies the cut of river bends (meander cutoff in case of a meandering river); it produces shortening of the river channel, increasing of the gradient and increasing of the flow velocity. The purpose is to reduce flood heights.

This type of activity was very commonly applied to many alluvial rivers worldwide, including the Danube River, over the 19th and the beginning of the 20th century.

Levees (or embankments)

The aim of levees is to increase channel capacity so that flood flows are confined and do not inundate the areas adjacent to the channels (floodplains), which would be inundated under normal conditions. Levees generally have a trapezoidal section and can be built close to channel banks (in this case levees must be quite high) or more far apart (for instance including the "shifting belt" or the "erodible corridor" of the river). This type of intervention, which is used both in rural and urban areas for flood control, requires extensive maintenance of the structure itself (geotechnical properties of materials may decay through time) as well as of the river channel.

It is very typical in the lowland part of the Danube River Basin. In Hungary alone, there are more than 4,000 kilometres of levees established, which is the second longest in Europe (after the Netherlands).

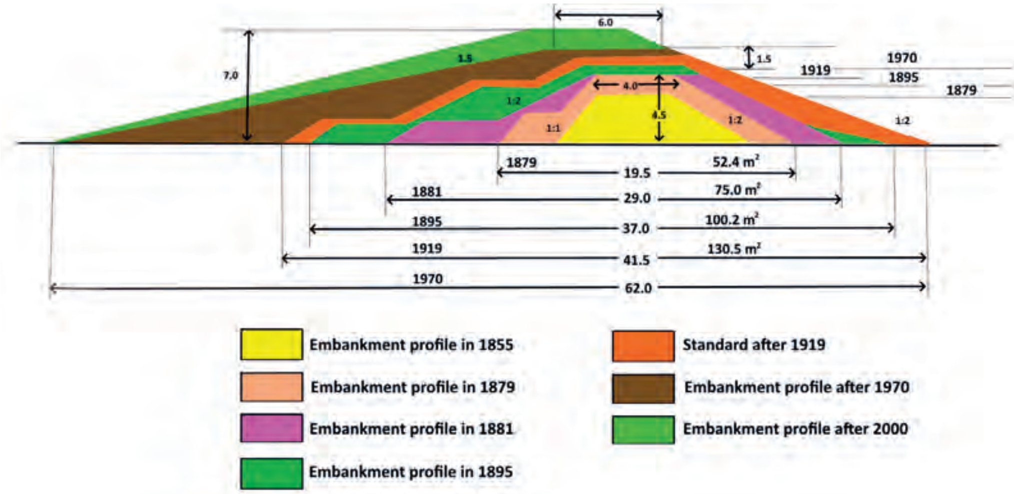

Figure 13. The changes in the Tisza river embankment as flood levels increased [11]

Flood walls and lined channels

This type of method is commonly used in urban areas where other kinds of channelisation are limited or where access for maintenance is restricted. Lined channels generally have a rectangular cross section with vertical sides made of concrete. This type of channelisation produces a remarkable decrease of channel roughness, an increase of flow velocity, and, consequently, a decrease of water level for a given discharge.

Figure 14. Flood wall in Budapest, Hungary[5]

[5] Online: https://fromhungarywithlove.wordpress.com/2018/07/27/the-great-floods-of-danube-in-budapest/#jp-carousel-1737

Bank protection structures

The use of revetments is a technique adopted to prevent bank erosion. Different materials (concrete, gabions, synthetic materials, live or dead vegetation) are used for revetments.

Groynes

Groynes are structures built transverse to the river flow and extending from the banks into the channel. The aim of these structures which deflect the direction of the flow, is mainly to induce sediment deposition behind the structures and to protect the banks from erosion processes (groynes can be either impermeable or permeable).

Diversion channels

New channels can be constructed to divert flows out of the existing channel (e.g. the Danube River in Vienna). Diversion channels are usually aimed at flood control (for instance where the river channel cannot be resectioned or where levees cannot be built or built higher) and agriculture improvement.

Culverts

This type of channelisation has often been used for urban streams, but also for small rural/mountain streams. In the latter case large-diameter concrete pipes are used. Culverting of a stream is most likely the "hardest" type of channelisation since it implies the disappearance of the stream below ground surface for short or longer reaches.

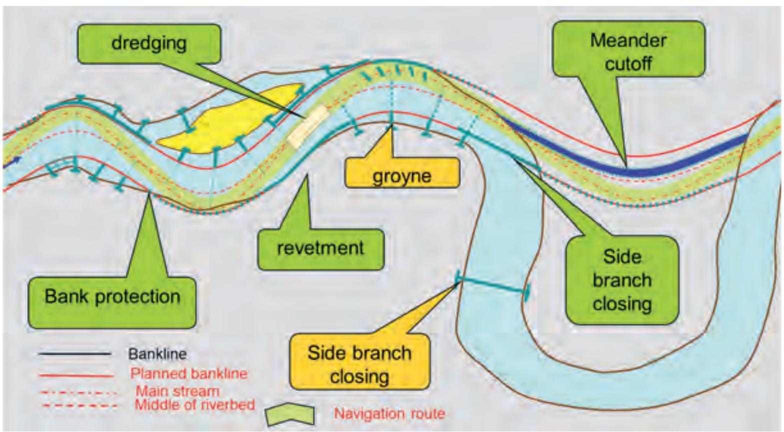

Figure 15. Summary of the most common river regulation structures for alluvial rivers (compiled by the author)

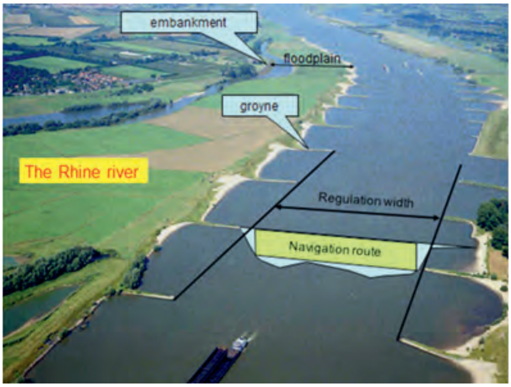

Figure 16. A typical channelised river (Rhine in the Netherlands)[6] *(compiled by the author)*

Consequences of river regulation and mitigation measures

It is widely known and for long discussed at different expert forums that the sediment balance of regulated rivers has changed drastically over the past century. The biggest problem for the experts along the Danube River to face with, which has to be addressed all along the river is sediment deficit.

Several studies have documented that channelisation may have different effects on channel morphology, riparian and floodplain ecology, human infrastructures, etc. Such effects regard not only the channelised reaches of a river but, quite often, also the upstream and downstream reaches (e.g. increased flood discharges in the downstream reaches). In some cases, the effects of channelisation have been really dramatic since early channelisation projects were designed with little or no consideration of sediment transport and river dynamics.

The effects of channelisation may be grouped into the following categories: river morphology and dynamics; hydrology; ecology; human structures and activities. Since in many situations channelisation is not the only human impact on rivers and their drainage basins, it is worth noting that most of these effects are often the results of a combination of such impacts (channelisation, dams, sediment mining, land use changes). We now go through these impacts referring to the Encyclopedia of Water Science, and, at the end we present the case study of the Danube River in Hungary.

River morphology and dynamics

River morphology and dynamics can be significantly affected by channelisation. Since the different types of channelisation imply changes in the morphological and hydraulic characteristics of a river (width, depth, slope, bed and banks roughness), morphological

6 Photo: https://www.dutchwatersector.com/

adjustments are likely to take place to attain a new equilibrium condition. As regards bed-level adjustments, river incision, due to increased stream power, is a common phenomenon, but also bed aggradation is not infrequent. Other remarkable effects are those produced by the construction of levees (or by incision induced by other types of channelisation): such construction dramatically changes sediment fluxes, reducing sediment deposition in the floodplain.

Hydrology

Channelisation works affect river and floodplain hydrology. As for floods, channelisation generally produces higher velocity in the channelised reach (therefore lower water stage) but can induce increased flood discharges in the downstream reaches due to reduction (or elimination) of floodplain storage. Deepening of the channel or incision induced by channelisation may strongly affect relationships between the river and its floodplain. In the case of unconfined aquifer, the lowering of the water table is likely to occur, whereas in the case of confined aquifer an increase of stream flows may take place. In coastal reaches, changes of water table levels can produce soil salinisation due to variations in salt wedge position. In very low gradient rivers, overbank flows, which under natural conditions are due to backwater effects and are fundamental from an ecological point of view, can be significantly reduced or eliminated. In addition, there are several examples of the effects of channelisation on water quality.

Ecology

River channelisation frequently has serious effects on aquatic and riparian ecosystems, but may have far reaching effects extending into the floodplain. Both flora and fauna along the river are affected by changes induced by channelisation, such as morphological, sedimentological and hydrological change. Floodplain ecosystems may be affected since hydrological and sedimentary connectivity between the river and its floodplain may dramatically change. Wetland environments, which are often drained for agriculture, are frequently affected by channelisation.

The example of the Danube River in Hungary

The sediment regime of the Danube has been altered by two major types of human interventions: river regulation (meander cuts, building of groynes, dredging), and hydropower dam construction. Sediment deficit effects can be detected everywhere, but the most severe impact is affecting the free-flowing alluvial reaches downstream of impoundments. The gravel bed border section of the Danube between Slovakia and Hungary has experienced an extra large channel incision between 1965 and 1990 due to intensive sediment dredging for commercial purposes. The total volume of extraction was estimated about 64 million m^3. This could not have been supplemented even by the original bed load transport of the Danube (before barrage constructions). The reach downstream of Budapest was not so heavily affected by industrial dredging, because the demand for the finer (mostly sand) bed material was less than for gravel.

Several investigations of recent decades have revealed a significant lowering of the river bed along the alluvial reach of the river Danube. We are now assessing this phenomenon based on the published research [13].

The deepening of the channel is a result of erosion processes that not only affect discrete sections but also long reaches of the river. This study is devoted to the Hungarian section of the Danube but, considering the reasons, it can be presumed that similar symptoms could be experienced in other parts of the Danube. Classifying the Hungarian Danube, it belongs mainly to the middle course, partly to the lower course with a lowland type. The river bed is alluvial. In accordance with the lowland character, the slope is mild (10–15 cm/km). The river bed is wide, with a typical width of 400–500 m. The average depth for the mean discharge is 5–6 m. A middle course type river usually has a balanced sediment transport and a balanced channel. The fact that this is not so nowadays for the Danube indicates that there were, or are, external effects disturbing the early balance.

Regular hydrological observations started in the second half of the 19th century in Hungary. It started first with observation of the water level at many stations along the Danube. Somewhat later, at the beginning of the 20th century, regular discharge measurements also began. Nowadays, there are water level records available for more than 100 years, and discharge records for 80–90 years for many stations on the Danube.

In the 1980s, a drying process of the floodplains on the lower Hungarian Danube reach became evident. The statistical analysis of the water levels took place, and showed that the riverbed of the Danube was lowering, the reason for which was thought to be the river regulation that took place in the 19th century. The maximum level decrease until 2005 is 216 cm, at the Dunaföldvár station (rkm 1,560).

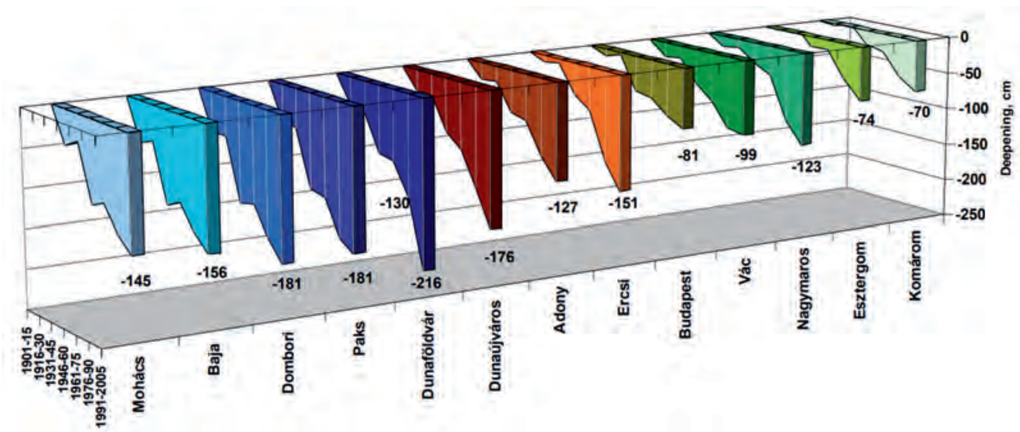

Figure 17. Lowering of the Danube stages along the Hungarian reach [2]

If we look at the dredging activities on the reach, it seems that apart from major river training works carried out all along the reach, dredging plays an important role in this extensive and fast deepening in that section.

Major hydraulic structures are built on the river on the reach in Germany and Austria, where slope permits for an effective exploitation of hydropower, furthermore there is one HPP in Slovakia (Gabcikovo) and two on the border of Serbia and Romania (Iron Gate I–II). The reservoirs of these HPPs are trapping a very large proportion of suspended sediment, apart from preventing the flow of bed load by blocking the river. On the Austrian reach, the "shallow" run of the river dams release much of the accumulated fine material during floods. Gabcikovo and in particular the Iron Gate dams, as are larger and partially deep impoundments, have much less potential for remobilisation (at least 35% of suspended load is trapped there). This phenomenon is not only a problem of the downstream reach – causing permanent erosion – but the accumulation in the reservoirs themselves pose a major daily challenge in the operation of HPPs. The observed steady deepening of the sand bed reach of the Hungarian Danube is rather due to this suspended sediment deficit than to the occasional dredging.

The morphological survey of the river bed (bathymetry) is a more direct and exact method to follow the changes of the bottom. However, adequate and sufficient results are available only from the latest decades, following the appearance of GPS, echograph and GIS applications. Nowadays, bathymetry is done every 5–6 years. Comparing the result of more surveys, done in different periods for the same spot, the changing of the river bed can be directly analysed. These comparing results of a typical section (near Dunaföldvár) are illustrated here.

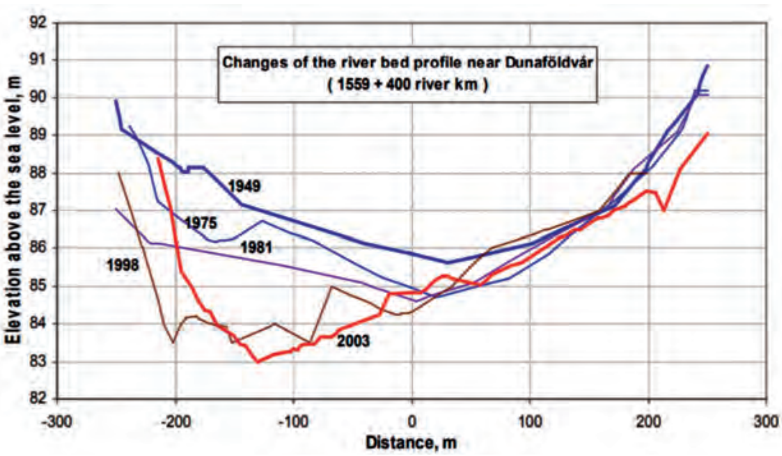

Figure 18. Changes in the Dunaföldvár section over time [2]

It is proven that the increased slope after the regulation (shortening of the river course) causes an increased sediment transport capacity and induces riverbed erosion.

Sediment transport capacity C (m^2 $time^{-1}$) can be calculated as the function of discharge and slope

$$C = \alpha \cdot Q_m \cdot \Lambda_n$$

where Λ is the slope gradient ($\partial z/\partial x$); m and n are constants giving an indication of the system. Sediment transport rate S (m^2 $time^{-1}$) is calculated based on the integrated continuity equation for sediment movement. The rate of sediment already in transport is S_0 (m^2 $time^{-1}$). If $S_0 < C$ there is erosion, while when $S_0 > C$ there is sedimentation. If S_0 decreases while the slope increases, erosion accelerates.

For a substantiated planning of sediment management measures, we need to know as much of the sediment transport of the river as possible. Not only the quantity, but the quality of sediment (grain size distribution) has to be examined, and it is vital that we have reliable field data.

Sediment management is a major challenge, as most methods developed until now are unsustainable, require continuous intervention and are expensive as well. Almost exclusively, artificial sediment supply is used as a compensation to the extensive sediment deficit problems along some European rivers. This is a symptomatic treatment, and we have to admit that it cannot be the long-term solution. It works for local–regional sections. No doubt, it should be reduced, maybe partially substitutes by granulometric improvements and by increased lateral erosion and channel shift on rivers with limited navigation usage and space such as the Drava.

Artificial supply of bed load, corresponding to the natural materials and grain sizes is carried out on the Rhine river since 1978, it amounts to approximately 2.1*105 m^3/y and costs around 8*106 EUR on an annual estimate. For case studies on waterways as shown, lateral erosion is almost no option, it works long term as long as there is good bed load supply feasible, granulometric improvements (size enlargement) is not yet proven in practice on the Danube, partly on the Elbe but only with shift of impact section.

For the prevention of further incision and in order to improve navigation a project is planned in Austria, east of Vienna. The granulometric improvement of the riverbed shall be carried out, in order to facilitate armouring, by artificially supplying coarse gravel 40 mm < d < 70 mm which will not start moving in the riverbed even in case of Q = 5,000 m^3/s, which has a return time of approximately 1 year. A relatively high amount of 3–4*105 m^3/s (2*106 m^3/y) sediment material has to be continuously fed in the river, in order to substitute sediments captured by several barrages upstream of the affected Danube section downstream Vienna.

Another measure in order to restore a part of the natural morphological processes is the removal of embankments from along the river (till now only behind point bars and transition bars only and with partial protection below the low water level). This kind of trials are currently going on along the Austrian reach of the Danube River, and short-term monitoring results show that no drawbacks from the viewpoint of navigation and flood safety are arising.

Often channelisation has induced severe effects on the environment (e.g. channel dynamics, groundwater resources, aquatic and riparian ecology, etc.) as well as on human structures (e.g. bridges, roads). For this reason, in some countries, especially in those strongly affected by channelisation, there have been some changes in the attitude about stream management through a more careful use of traditional methods, through the use of different approaches (in particular geomorphological and ecological ones) and the restoration (or rehabilitation) of existing channelised rivers.

As for floods, it should not be forgotten that channelisation itself induces an increase of human occupation and activities in floodplains and therefore an increase of flood risk.

Driven between the levees, floods are not able to spread – as they did in the natural, original state – and have a stronger effect on the bed forming processes.

During medium-flow regulation, strongly meandering bends were cut, the horizontal alignment of the river was fixed with bank protection structures and groynes, utilising also the energy of the flow to form the bed. The aim of this work was to speed up the travelling of floods and to help the movement of drifting ice. The improved shape of the channel was favourable for navigation as well because the narrowed river bed resulted in deeper water for longer duration. However, this transformation resulted in shortening of the river, increasing the slope and therefore increasing the sediment transportation capacity. Consequently, the original balance has been shifted towards sediment erosion. Narrowing of the river bed also increased flow velocities, involving the increase of the sediment transportation capacity again.

Low-flow regulation is an indispensable auxiliary element of medium-flow regulation. It concentrates on the stretches of the river where medium-flow regulation was not enough to develop the required profile, water is shallow and fords obstruct navigation. Additional regulation structures and direct dredging is applied on these stretches to get the required measures of the navigation profile. However, dredging helps the deepening processes, in more respects.

Deepening of the river bed has unfavourable effects on the natural environment, on navigation and also on operation of man-made structures in the river.

To improve the conditions of navigation is an important aim of river regulation work. Medium-flow regulation activities have been more or less finished by nowadays. Taking into consideration that the horizontal alignment of the Danube is practically stabilised – by cutting many bends, building bank protection structures and groynes – this work has been accomplished. However, the bed of the river is still continuously developing and forming. These changes are – at least partly – consequences of earlier interventions. The most frequent navigation obstructions are fords, shallows and contractions of the navigation channel during low water periods. Fords are continuously building and forming in the changing, unstable channel – calling for a permanent control of the responsible authorities. More studies, dealing with the analysis of low water periods and the efficiency of low water regulation prove that, despite of the extensive regulation dredging, the conditions of navigation did not improve.

The Gemenc Forest (part of the Danube–Drava National Park, Hungary) is one of the substantial floodplain forests of Europe. Gemenc is an irrecoverable natural value. The water demand of the forest and the wetlands is supplied by the Danube, partly by subsurface feeding, partly through the net of oxbows and channels, sometimes inundating the whole area during floods. Forestry experts observed certain drying processes of the forests in the second half of the 1980s. They noticed changes in the development of trees and also the replacement of autochthonous plant species. It was revealed by the detailed hydrological analysis of the water levels that the flood events reaching the oxbow lakes become rarer and their duration also decreased. Besides, the low water periods with longer duration are also unfavourable regarding the subsurface feeding. Some projects started after the investigations aiming at the improvement of water supply and water retention in the oxbow lakes; however, the effects of these interventions extend only to small areas and, regarding the whole of the forest, they cannot be appreciated as overall and accomplished solutions.

The Danube has an essential role in the agricultural water supply along the river. Large areas are supplied by artificial, gravitationally operated channels. When the water level is very low in the river, pumping is necessary to lift the water into the channels. In this system, developed primarily for gravitational operation, pumping entails a high economic burden. This problem appeared once or two times annually in the recent years.

The lowering of the river bed of the Danube is a complex problem as it has been outlined above. It has far-reaching effects which require complex solutions, taking into consideration not just economic but also ecological and social aspects, as well.

In recent years, there have been several studies dealing with the deepening of the river bed, the problems arising from this and the possible solutions. Even nowadays, there are several such projects underway searching for solutions. An essential question is whether traditional river regulation means are suitable to reach the above mentioned aims. Building barrages, that is to say the channelisation of the problematic sections of the Danube, could be an evident alternative for regulation structures and dredging. For this, as has already been mentioned, there are a number of examples on the Austrian and German stretch of the Danube.

Despite the problems, we have to state that knowledge of the sediment transport is essential to understand the morphological processes forming the river bed.

Consequently, the improvement of the methodology seems to be necessary both for the suspended and also for the bed load. Not just measuring techniques but also data processing methods should be improved. International cooperation would be highly desirable.

Concluding remarks

Today the goal of the sediment sampling is not only to describe sediment transport in the flow, but further to provide calibration and validation data for numeric modelling. Sediment measurements are different in the different countries in Europe. Methodologies and samplers vary, both for field and laboratory analyses. Even in Hungary, sampling and laboratory techniques have been modified several times in the past. Also, sediment sampling was never really systematic, and the sampling campaigns did not follow the hydrological processes. That is why sediment data can hardly be compared. The data series are inhomogeneous and cannot be statistically analysed. Sampling has to be carried out as to be able to obtain a true picture about the changes of sediment transport across the flow, along the flow and with respect to variability with depth. The sampling points have to be determined based on morphological and flow conditions. Discharge measurement has to be executed in parallel to sediment sampling. For a few years, water authorities in many countries have been using Acoustic Doppler Current Profilers (ADCP) for the measurement of the discharge. This opens up new possibilities for future analyses. Despite this fact, we still have to emphasise that the availability of hydromorphological data is extremely important for assessments under the Water Framework Directive, also to support ecological status evaluation. However, the lack of information on some large rivers is evident. The changes in the hydrological and sediment regime of river systems induced by hydromorphological alterations are not well understood, so in the near future there is an urgent need for a harmonised database. To this end, the first step is to intensify and reorganise hydromorphological monitoring, including sediment sampling and data management.

The hydrometry services of the different Danube countries do not have enough resources and they might also lack expert support. ICPDR expert groups are currently not dealing with sediment issues as a priority, though Monitoring and Assessment EG has produced an overview of the situation in the river basin, which more or less covers the subject of morphological monitoring as well.

The most urgent and highest importance action would be the introduction of a unified, regular, exact and thorough sediment monitoring system in the whole river basin.

When planning sediment management actions for the Hungarian reach, it is very important to take into account that the German and Austrian reaches are much more heavily regulated and flow conditions are much different, this way the environmental and nature protection aspect interventions can also differ a lot. Before applying the methods developed for the upstream reaches, there is a need to thoroughly investigate all potential impacts and effects from this point of view as well.

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Flood Forecasting

"Everything should be made as simple as possible, but no simpler."
Albert Einstein

One of the EUSDR PA5 targets[1] is to provide and enhance continuous support to the implementation of the Danube Flood Risk Management Plan – adopted in 2015 in line with the EU Floods Directive – aiming to achieve significant reductions of flood risk events, while also taking into account potential impacts of climate change and adaption strategies. The target is based on the assessment that there is no uniform specialised training on flood management in the Danube region (DR) and that the lack of knowledge in this domain constrains in many ways the implementation of both the EU Flood Directive and the Danube Flood Risk Management Plan (DFRMP). Noting that there is a need to harmonise flood risk management methodologies in the region, it has also been established that, in general, the Civil engineering curricula at various universities in the region does not put adequate emphasis on flood management issues.

Based on the above premise, the main objective of the project – *International Postgraduate Course on Flood Management* funded by the Danube Strategic Project Fund (DSPF) – is to develop a harmonised international postgraduate course on flood and flood risk management. The course is intended to integrate knowledge and expertise of the participating partners and professionals in the Danube region; the whole project is expected to result in a comprehensive flood management curriculum that offers the possibility for professional development of practicing civil engineers and flood managers throughout the region. The main goal is to offer a possibility for uniform education in this domain in the Danube region based on the EU directives and the EUSDR PA5 targets as well as on the DFRMP needs, state-of-the-art research results and extensive operative experience of the selected lecturers.

Flood forecasting – Scope of the course

Within the context of the overall curriculum, the course material on flood forecasting presented in the text below provides an overview of the following main issues and topics in this field:

– flood phenomena and flooding
– flood forecasting
– the role flood forecasting plays in flood management
– flood forecasting and warning systems (FFWS)

[1] For more information see www.danubeenvironmentalrisks.eu/eusdr-pa5-targets

– key components of a dependable real time FFWS
– concepts and methods used for modelling of river flow and runoff processes in a river basin and real time flood forecasting
– latest trends and developments in establishing FFWS

In preparing the teaching material the authors, apart from using their own experience, relied on recognised sources and publications in the field of hydrology and hydrological forecasting, including the technical manuals, guides and specialised publications of the World Meteorological Organization (WMO) and UNESCO. Whenever deemed necessary and of interest for better understanding of the lecture material, it will be illustrated with real world examples, primarily focusing on the examples from Europe and the Danube region.

Floods and flooding

Floods are without doubt the most devastating natural disasters, striking numerous regions in the world each year. During the last decades, the trend in flood damages has been growing exponentially. This is a consequence of the increasing frequency of heavy rain, changes in upstream land use and a continuously increasing concentration of population and assets in flood prone areas. In general, all countries in the world are vulnerable to floods, causing damages that significantly affect the national GDP. In the Danube River Basin, important initiatives have and are being devoted to implementing appropriate countermeasures, both structural and non-structural, to alleviate the threats of water-related disasters. The worldwide impact of flooding is alarming and the UNESCO World Water Assessment Programme provides a clear evidence to this effect. Figure 1 shows the significance of flooding in the context of all water-based natural hazards.

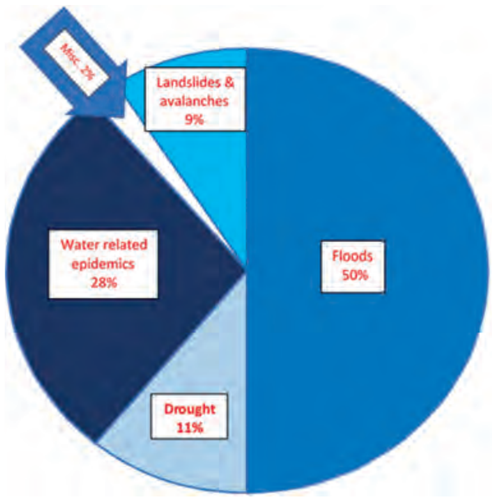

Figure 1. Types of water-related natural disasters, 1990–2001 (adapted from [24])

Floods account for more than 15% of all deaths related to natural disasters; for example, between 1987 and 1997, 44% of all flood disasters affected Asia, claiming 228,000 lives (roughly 93% of all flood-related deaths worldwide). Economic losses for this region totalled US$ 136 billion.

Many deaths have been caused by floods in the European countries as well. The 2002 floods in Europe claimed 100 lives and caused circa US$ 20 billion in damage. Over 12% of the population of the United Kingdom of Great Britain and Northern Ireland lives in fluvial flood plains or areas identified as being subject to the risk of coastal flooding while about half the population of the Netherlands lives below mean sea level.

With respect to the Danube Region, it has been estimated that close to 15% of all the population in the region lives on the flood plains of the River Danube and its tributaries. Memories are still fresh of the disastrous floods that hit the Sava River Basin in May 2014. By volume, the Sava River is the largest Danube tributary, with an average discharge of about 1,700 m^3/s, which accounts for almost 30% of the Danube's total discharge at the confluence of the two rivers in Belgrade.

Figure 2. Areas in Croatia (HR) Bosnia and Hercegovina (BIH) and Serbia (SRB) affected by the May 2014 floods (2014 Southeast Europe floods; https://en.wikipedia.org)

The May 2014 floods in the Sava region affected large areas in Croatia, Bosnia and Hercegovina and Serbia along the Sava river and its tributaries (Figure 2). It resulted in 79 casualties and substantial economic damage in the three countries, assessed in the range of 3.0–3.8 billion Euros.

With the frequency and variability of extreme floods changing because of urbanisation, along with population growth in flood-prone areas, land use changes, climate change and a rise in sea levels, the number of people vulnerable to devastating floods is expected to rise. Adequate flood management and flood risk reduction actions are increasingly required to build up the capacity necessary to cope with floods. Flood Forecasting forms an important tool in reducing vulnerabilities and flood risk, and forms an important ingredient of the strategy to "live with floods", thereby contributing to national sustainable development.

Types of floods

There are quite a few definitions in use for the term "floods"; for consistency and to avoid possible confusion, we will use WMO/UNESCO International Glossary of Hydrology [25]. This widely recognised Glossary represents an informal world standard, in which all terms are defined in several international languages.

The Glossary defines "flood" as follows:

1. Rise, usually brief, in the water level in a stream to a peak from which the water level recedes at a slower rate.
2. Relatively high flow as measured by stage height/water level or discharge.
3. Rising tide.

Note that, unlike in some other languages, in English the term "flooding" signifies the effects of a flood as distinct from the flood itself, i.e. "flooding" is defined as "overflowing by water, due to flood, of the normal confines of a stream or other body of water, or accumulation of water by drainage over areas that are not normally submerged".

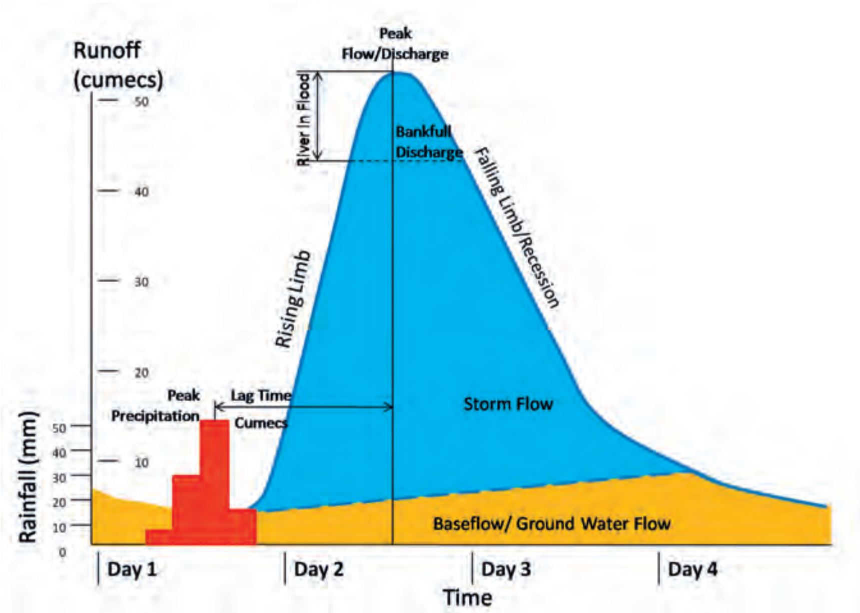

Figure 3. Illustration of a typical flood hydrograph overflowing river banks and causing flooding (https://mappedmusings.wordpress.com/tag/hydrograph/)

The Glossary gives definitions of a wide range of terms used in relation to floods and flooding; there is no need to catalogue them all here, but only to highlight the main features of different flood types.

Flash floods. These floods are frequently associated with violent convection storms or thunderstorms of a short duration falling over a small area. Flash flooding can occur in almost any area where there are steep slopes; it is common in mountainous regions subject to frequent severe thunderstorms. Flash floods are often caused by heavy rain of short duration. Flooding caused by flash floods frequently washes away houses, roads and bridges over small streams and has a critical impact on communities living in these often-remote areas.

Fluvial floods. This type of floods, causing fluvial flooding, occurs over a wide range of river basins. Floods in river valleys occur mostly on flood plains as a result of flow exceeding the capacity of the steam channels and spilling over the natural banks or erected embankments. Compared to fluvial floods, flash floods are often more damaging, occurring in narrow, steep and confined valleys, characterised as the name implies by the rapidity of formation and high flow velocities, which makes them particularly dangerous to human life.

Single event floods. This is the most common type of floods and flooding; widespread heavy rains over a drainage basin – lasting several hours to a few days – result in severe floods. Typically, these heavy rains are associated with cyclonic disturbance, depressions and storms with well-marked largescale frontal cloud systems.

Multiple event floods. These result from heavy rainfall associated with successive weather disturbances following closely after each other. On the largest scale, they often include floods in the large river plains in central Indian regions often caused by the passage of a series of low-pressure areas of depressions from the Bay of Bengal. This type of floods can also affect large basins in mid-latitude areas in winter, for example over the western Europe and also in the Danube region.

Seasonal floods. These floods occur with general regularity as a result of major seasonal rainfall activity, and mainly affects the areas with a monsoonal type of climate. As such they do not affect Europe and the Danube region (mentioned just for completeness).

Coastal floods. Storm surges and high winds coinciding with high tides are the most frequent case of coastal flooding. The surge itself is the result of the rising sea levels due to low atmospheric pressure. In a particular configuration, such as large estuaries or confined sea areas, the rising water level is amplified by a combination of factors (such as shallowing off the seabed and retarding of return flow). This type of floods affects major deltas such as the Mississippi, Ganges, Irrawaddy and also the Danube delta.

Tsunamis, resulting from sub-seabed earthquakes are a very specific cause of occasionally severe coastal flooding.

Urban floods. Urban flooding occurs when intense rainfall within towns and cities creates rapid runoff from paved and built-up areas, exceeding the capacity of storm drainage systems. In low-lying areas within cities, the formation of ponds from runoff occurs not only because of high rainfall rates but also due to drainage obstructions caused by debris blocking drainage culverts and outlets, often because of lack of maintenance.

A number of major cities situated in delta areas, such as for instance New Orleans, Dhaka and Bangkok, are protected by embankments and pumped drainage systems. When rainfall rates exceed the pumping capacity, the rapid accumulation of storm runoff results in extensive urban flooding.

Snowmelt floods. In upland and high-latitude areas where extensive snow accumulates over winter, the spring thaw produces meltwater runoff. If temperature rises are rapid, the rate of melting may produce floods, which can extend to lower parts of river systems. This type of floods occurs regularly in the Danube region.

The severity of meltwater floods will increase if the thaw is accompanied by heavy rainfall and can further be exacerbated if the subsoil remains frozen or is already saturated to its full field capacity. Although a seasonal occurrence where major snow fields exist in headwaters, which may produce beneficial flooding in downstream areas, severe effects can occur on smaller scales, especially in areas subject to changes between cold and warmer rainy winter weather.

Ice- and debris-jam floods. In areas that experience seasonal melting, if this is rapid, ice floes can accumulate in rivers, forming constrictions and damming flows, causing river levels to rise upstream of the ice jam. A sudden release of the "ice jam" can cause a flood wave similar to that caused by a dam break to move downstream. This type of flood occurs occasionally in the Danube region as well.

Both meltwater and heavy rainfall in steep areas can cause landslips, landslides and debris flows. As these move downstream, major constrictions can build up. When these collapse or are breached, severe flooding can result. Both of these phenomena are very difficult to predict.

Types of basins, processes and flood response

There is a wide range of river basins and systems all of which react in a specific manner to heavy rain, storms, or combined effects of sea and inland meteorological and hydrological/hydraulic conditions. In general, it is possible to define six main types of basins according to their temporal and spatial response to the hydrometeorological event, as follows:

1. Urban basins, densely populated with a high proportion of impermeable surfaces (small, up to a few square km); they respond in one or two hours and can overwhelm the capacity of the urban drainage network.
2. Upper watersheds and small to medium catchments, of an area between 10 and 500 km^2, which will respond in a few hours. The catchments located in upland areas with steep slopes react quickly causing flash floods.
3. Medium-sized rivers (with an area between 500 and 10000 km^2); they are characterised by relatively long-distance flow propagation with varying contribution of tributaries. In these basins flood can take days to affect the lower reaches.
4. Large river basins with an area of over 10,000 km^2 are characterised by long distance flow propagation for which flood response is in terms of weeks and reflects major seasonal meteorological conditions.

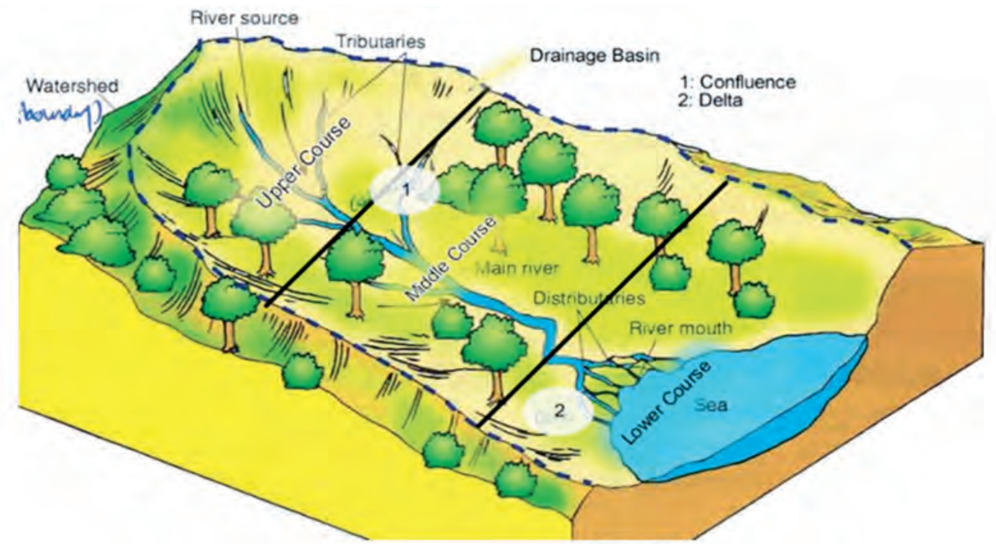

Figure 4. Illustration of a typical large river basin with its parts (adapted from https://teamgeographygcse.weebly.com/)

5. The very specific domain of river deltas and estuaries, which are under combined influence of maritime storm surge, tide effects and upstream incoming flood.
6. Groundwater-controlled river systems, subject to long periodic fluctuations of the water table.

Any forecasting service is above all dependant on the types of flood causing physical processes in the basin. Table 1 illustrates interaction between the basin size, the relevant physical processes/events and their effect on flood response.

Table 1. Interaction between basin size, physical processes/events and their effect on flood response [29]

Type of basin	**Physical process**						
	Wind	Infiltration	Rainfall intensity	Runoff	Propagation	Tide and surge	Water table
Urban		X	XXX	XXX	X		
Upper basin		XX	XX	XXX	X		
Long/large river	X		X	XX	XXX		X
Estuary	XXX				XX	XXX	
Aquifer	X			X		X	XXX

Legend: xxx – dominant effect; xx – normal effect; x – minor effect

The main processes in both the upper and urban catchment areas refer to interaction of infiltration and runoff that combine to produce high flow concentration in lower river reaches and low-lying basin areas. When it comes to long, large rivers, which by their very nature have large sub-basin areas, the combination of sub-basin floods and the way they combine along the main river – whether their flood peaks occur at different times or coincide, i.e. occur almost synchronously – will affect the way a flood propagates towards the lower reaches.

The Danube River Basin

Almost everything is known about the Danube, the largest river in the European Union; yet, it may be of interest to repeat a few basic facts and highlight issues related to floods and flood risks in the Danube River Basin.

The Danube River (Donau in German, Dunaj in Slovak, Duna in Hungarian, Dunav in Serbo-Croatian and Bulgarian, Dunărea in Romanian, Dunay in Ukrainian) with a total basin area of 801,463 km² is the second largest in Europe (after the Volga river). It rises in the Black Forest mountains of Germany and flows for some 2,850 km to its mouth at the Black Sea – with an average discharge at the mouth of 6,500 m^3/s. Along its course, it passes through 10 countries – Germany, Austria, Slovakia, Hungary, Croatia, Serbia, Bulgaria, Romania, Moldova and Ukraine – whilst more than 80 million people from 19 countries share the Danube catchment area, making it also the world's most international river basin.

It is usually divided into three major sub-regions – the Upper, Middle and Lower Basins (the latter including the Danube Delta). The Upper Basin extends from the source in Germany to Bratislava in Slovakia. The Middle Basin is the largest of the three and extends from Bratislava to the dams of the Iron Gate Gorge on the border between Serbia and Romania. The lowlands, plateaus and mountains of Romania and Bulgaria form the Lower Basin of the River Danube. Before reaching the Black Sea, the river divides into three main branches, forming the Danube Delta, which covers an area of about 6,750 km².

Flooding is the most common and most costly natural disaster in Europe; as evident from Figure 5, the Danube basin is no exception. In fact, large areas throughout the Danube basin are exposed to flood risks, and every type of flood outlined in the previous section can occur.

The increasing regularity of dangerous hydrological and meteorological phenomena in the basin has become a major cause for concern in Europe. Scenarios of the European Environmental Agency[2] predict that flood damage and the number of people affected by flooding will rise substantially by 2100 as a result of climate change, with one scenario estimating a rise in flood damage by some 40% and an increase in the number of people affected around 242,000, i.e. by about 11%.

From historical records and written chronicles, there is evidence about 78 significant floods that occurred along the Danube over the last nine centuries while 23 of them took place in the 18th century before extensive flood protection works were started.

[2] For more information see www.eea.europa.eu

Figure 5. The Danube River Basin and its major flood hazard areas – catchments > 4,000 km² (www.danubegis.org/maps)

Recent years saw an increase of flood frequency, and high-water marks have set records three times since August 2002 whilst five of the most significant floods in history have occurred in the last 10 years. Multi-annual averages of precipitation have been exceeded by 1.5 to 2.0 times recently, a maximum that was never before observed since systematic meteorological measurements have been available. Cognizant of the worsening situation, the EU eventually formalised flood management in 2007 through its Flood Directive. The International Commission for the Protection of the Danube River (ICPDR) established three years earlier, i.e. in 2004, its own long-term action programme for sustainable flood prevention in the basin, which was instigated by the disastrous floods that occurred in August 2002 in the Danube River Basin. The ICPDR also coordinates the implementation of the EU Flood Directive in the Danube Basin.

The role of flood forecasting in flood management

The need for flood forecasting and warning systems comes as a consequence of the limitations of structural flood protection systems. Due to the existence of settlements in flood-prone areas and to the need to meet community safety and protection of assets, the provision of an adequate flood forecasting and warning service is a necessity. Forecasting and warning services are, in most cases, provided by national state agencies; their main goal is to deliver reliable and timely information to the flood managers, civil protection agencies as well as to the general public. This should be accomplished with long enough lead time of forecast to allow people to take measures to protect themselves from flooding or take appropriate actions.

As no preventive or flood defence measures can ever be completely effective, flood forecasting constitutes a crucial part of flood management. The reality of economic limits to the provision of a flood defence system, together with the possibility that the capacity of defence systems may be exceeded, or may fail, require that other measures should be in place.

This is the reason why flood forecasting forms a part of flood management planning and development strategies, which recognise that there are occupied flood plain areas where non-structural measures can be effective. Flood management requires a variable degree of response from the water management agency, local or municipal authorities, transport and communications operations and emergency services. Flood forecasting has to provide information to these users both for preparation and response: at the most extreme level, flood forecasting is part of the wider disaster management capacity, which devolves from the highest level of government.

The precise role of flood forecasting varies depending on the circumstances dictated by both the hydrological and meteorological conditions and the built infrastructure, i.e. the flood risk areas in question. Cities present different problems from rural areas. Location of flood risk areas in relation to rivers, coasts or mountain ranges has a significant bearing on the types of flood forecasting required. The nature and type of floods and flooding events is also important, particularly whether floods are regular in occurrence (as for example in a highly predictable seasonal climate, such as monsoon or hurricane seasons) or highly irregular, such as violent thunderstorms.

No less important is to note the vital role that the flood forecasting and streamflow forecasting in general play in management and optimal operation of reservoirs and river flow regulation.

There is no design carved in stone for a flood forecasting system. The balance between particular components of the system, such as for example meteorological input and hydrological forecasts, scale and timing, have to be adapted to circumstances. Within a given country, a number of different flood types are usually encountered, and each will require a different forecasting approach. Headwater areas may require a system concentrating on flash floods, whereas flood plain areas may need a system to be focused on the slow build-up of flooding and inundation.

The nature of flood risks and impacts

Flood risks are related to hydrological uncertainties, which are also linked to social, economic and political uncertainties. In characterising future flood risks, the biggest and most unpredictable changes are expected to result from population growth and economic activity. This can be demonstrated by the historical development of coping with flooding, where the initial resilience of a largely rural population is lost by more complex societies. Flood risk management consists of systematic actions in a cycle of preparedness, response and recovery and should form a part of Integrated Water Resources Management (IWRM). Risk management calls for the identification, assessment and minimisation of risk, or the elimination of unacceptable risks through appropriate policies and practices.

Flood forecasting and warning activities are largely designed to deal with certain design limits of flooding. For example, depending on a range of probabilities of harmful damage, and available monitoring facilities, modelling and operational forecast systems can be set up in relation to known risks and impacts. In particular, these will focus on areas of population, key communications and infrastructure and the need to operate effective responses to flood. The magnitude of flood events and hence impacts are variable, so flood forecasting and warning has to operate over a range of event magnitudes. These vary from localised, low impact flooding, which can be countered by relatively simple measures, such as installing temporary defences, closing flood gates and barriers, to larger scale flooding, where property damage and losses occur, road and rail closures arise and evacuation of areas at risk takes place. Defence and remedial measures are designed or planned to operate up to a particular severity of flooding, which may be designated as having a particular probability. The measures are related to an economic decision relating costs against losses. Typical design criteria are 100 years (an event that is considered to have a 1% chance of occurring in any given year) for urban areas with key infrastructure, 50 years (2%) for lesser population centres and transport facilities, 20 years (5%) for rural areas and minor protection structures.

Beyond the limits of designed flood management and particularly for catastrophic events, for example dam or embankment failure, some aspects of flood forecasting and warning provisions may not be fully effective. However, it is important that in these cases monitoring facilities are sufficiently robust, as some continuing observations will be of vital assistance to emergency response and relief activities. In this respect, resilience of monitoring instruments, their structure and telemetry, are of considerable importance, especially as on-the-ground reporting may become impossible.

Flood forecasting systems: Main aspects

To avoid possible confusion, before entering into discussion about hydrological forecasting systems, there is a need to clarify the meaning of forecasting as other terms are also frequently used for the same pursuit. Quite often the term hydrological prediction is used as a synonym for hydrological forecasting even though they signify two completely different concepts [16].

What does forecasting really mean?

Classical definition of hydrological forecasting [28] reads as follows: a hydrological forecast is the estimation of future states of hydrological phenomena. In its revised and expanded version it reads [24]: a hydrological forecast is an estimate of the future state of some hydrological phenomenon, such as flow rate, cumulative volume, stage level, area of inundation or mean flow velocity, at a particular geographical location or channel section.

Both definitions are talking about point estimate without dealing with uncertainty (and hence risk!) that is unavoidable in making forecasts. The question asked in this context is: How much and when? For instance, in the case of a single event forecast one asks the question: how much will the maximum peak flow and/or peak water level be and when in time that maximum will occur? As another example, the relevant question in the case of continuous forecasting is as follows: How much will the flow and/or water level be one time step ahead from now on, two steps ahead, three steps ahead and so on? In this example, the time variable is not changing but is fixed and is incremental in an equidistant fashion and, in fact, one is seeking to forecast expected values of the entire future hydrograph at an ordered set of discrete time intervals. Therefore, one can state the following: the sequence of expected future values is a series of point estimates. These point estimates are the continuation of the past and present history of the hydrograph into the future.

The same principle applies to hydrological variables as to any other dynamic phenomena that is changing in time: the past and the present condition the future. Therefore, the best estimate of the expected value of the future outcome of a dynamic process (be it hydrological or any other) is the conditional mean (providing the related uncertainties are Gaussian).

As much as the conditional means may be smart, they say nothing about the associated uncertainties in the future that, in turn, characterise the reliability of future forecasts. Intuitively, however, one feels that as the lead time of forecast increases so does the level of uncertainty, i.e. the longer the lead time, the less reliable forecasts are.

The question is how can this uncertainty be reflected in the definition of forecast? The answer is relatively straightforward and points to the probabilistic analysis of forecast errors for there are evidently errors in our forecasts one step ahead, two steps ahead, and so on. These forecast errors scatter around the conditional means and they have their own statistical distribution; so if we are able to measure somehow what those scattering deviations are, we are equally able to estimate how reliable our forecasts are. So, providing that the forecasting errors have Gaussian (normal) distribution, which is a fairly reasonable assumption, the first and the second statistical moment – i.e. the mean and the variance (or its square root, the standard deviation) – are sufficient to characterise error distribution. Now, using the same arguments that were applied in arriving at the conditional mean as best linear unbiased estimate, the best estimate for the variance of the error sequence for a given lead time of forecast is the conditional variance. To conclude, if we need to establish the best forecast s-steps ahead, along with finding the measure of the corresponding forecasting uncertainty, we have to determine the conditional probability distribution of the hydrograph s-steps ahead, conditioned by the past and present information about that hydrograph.

In view of the above considerations, we can now revise the above classical definition of hydrological forecasting to read as follows [16]: Hydrological forecasting is the identification of the expected occurrence of a hydrological event specified with respect to its actual time of occurrence, its quantitative measure and its reliability (i.e. measure of uncertainty) as conditioned by available past and present information about the event.

So instead of asking in this context – How much and when? – our question on hydrological forecasting would in essence be this: How much, when and how certainly?

Having come this far, we can now ask what the hydrological prediction is then. The question that one asks in this context is this: How much and how often? Note that this is an entirely different question, and opens up the domain of hydrological frequency analysis, that is an entirely different concept from the one applied in real-time hydrological forecasting.

Type of forecast service

The processes occurring in the catchment can be monitored and forecasted by a combination of observations, measurements and modelling. It is necessary to design a flood forecasting and warning service based on a given type of catchment, physical processes and their effect on flood occurrence. Apparently, upland and urban catchments require different type of service and approach to monitoring and modelling than the large catchment areas. The former catchments present particular challenges in assuring flood forecasts with sufficient lead time for taking timely mitigation measures. Here, the emphasis must be on effective continuous real-time monitoring and rapid transmission and processing of data rather than modelling.

In large basins the lead time of forecast may not be so critical so that data sampling may only be necessary at intervals of several hours. In these basins more emphasis need to be placed on identifying distribution and patterns of rainfall that occur, and on observation of the hydrological response in the contributing sub-basins. To be noted that some large catchments have also problems with drainage congestion, which can arise when water level in the main river is high thus ‘congesting’ incoming floods from the smaller sub-catchments and causing flooding of the adjacent low-lying areas.

The most advanced form of forecasting and warning service that can be provided deals with forecasts of water levels and discharges together with information on the associated inundation extent and depth. However, such a high level of service is easy to offer even in the most developed countries, and is usually restricted to limited areas of high economic importance and highly populated districts. The level of service is largely dictated by the cost of instrumentation, monitoring network, high resolution mapping as well as by the complexity of modelling.

The type and level of forecasting and warning service that can be provided usually represent a balance between the technical feasibility to forecast the flood hazards and the economic justification for protecting vulnerable populations, important areas and infrastructure. Different types of forecasting services – from basic minimal levels to those of high quality and sophistication – can be summarised as follows:

Simple threshold-based flood alert: this basic service relies on real-time hydrological measurements along rivers. It does not deal with quantitative forecasting but qualitative estimation of the increase of river flow or water level. No hydrological or hydrodynamic model is required as hydrograph trends are extrapolated based on experience to estimate if and when critical threshold levels may be reached.

Flood forecasting: this higher-level service is based on the use of simulation tools and modelling. The tools can include simple methods such as statistical curves, level-to-level correlations or flood time-of-travel relationships; they allow a quantified and time-based forecast of water level to provide flood warning to an acceptable degree of reliability (i.e. the measure of uncertainty). Whether this simple approach is used, or a more sophisticated one – through models that integrate and replicate the behaviour of rivers throughout the basin – the simulation tools must be calibrated beforehand by using historical data from recorded floods. The simulation methods also need regular updating to ensure that catchment relations remain properly identified. The information delivered by a warning service is not confined to station locations, as in the flood alert, but can be focused on specified locations at risk.

Vigilance mapping: one step up service compared to the flood forecasting above. Flood forecasting and warning services produce a map-based visualisation (i.e. 'vigilance map') as an Internet service. The maps provide information on the levels of risk derived from observations and models, and are characterised with a colour code (e.g. green, yellow, orange, red) indicating the severity of the expected flood.

Inundation forecasting: This is the most sophisticated service that can be delivered to the public. It requires the combination of a hydrological or hydrodynamic level-and-flow model, that also deals with forecast uncertainty, with digital representation of the flood plain land surface. The level of detail and accuracy of the terrain model depends on the nature of the area at risk. The greatest level of complexity should be applied to sensitive areas of flood plain, where flood extent is dictated by minor relief, and to urban areas. Such models do have the ability to predict flooding to very precise locations, for example, housing areas or critical infrastructure locations such as power stations and road or rail bridges. The development of this approach also requires an in-depth knowledge of inundated areas from previous severe events.

Forecast lead time

The lead time of a hydrological forecast [24] is the period from the time of making the forecast (that is, the time origin of the forecast) to the future point in time for which the forecast applies. Definitions of categories of lead time are subjective, depending on the size and type of the catchment within a particular region or even country. In addition, it also depends on the type of flood, processes in the catchment that cause floods, real time information available, and capability of hydrological and meteorological models used.

However, the basic principle for assessing forecast lead time requirements is the minimum period of advance warning necessary for the preparatory action and for the flood mitigation measures to be taken effectively. This will depend on the needs of the target community or area. Individual householders and businesses may require from one to two hours to move vulnerable items to upper storeys or put sandbags or small barriers in place. Protection of larger infrastructure, setting up of road diversions and movement of farm animals to a place of safety may require lead times of several hours. On large rivers with a long lead time but major potential impact, the lead time for evacuating populations at risk may be in the order of days. Thus, the concept of forecast lead time has to be flexible and the minimum time may be entirely dependent on the catchment characteristics and the forecasting and warning system facilities. For small and urbanised catchments, flood response time may be so short that it is extremely difficult to provide an effective warning. If a high-risk, high impact situation exists, then the problem of short lead times has to be addressed by sophisticated automated alarm systems linked to real-time hydrological and hydraulic modelling.

The situations presented below illustrate some of the issues affecting forecast lead time:

In situations where only forecasts available are based on historical water level records, extrapolation of the water level graph over a period of a few hours is possible (hence, the forecast lead time equals a few hours), depending on the catchment characteristics and nature of the causative event.

In the case that telemetric rain gauge data or radar rainfall information are available, these can provide additional advance warning. In this case, an experienced forecaster using subjective judgement can estimate the likely flood response time. In a more sophisticated procedure, data can be used as input into a hydrological forecast model, thus extending the forecast lead time.

Further to the situation described above, forecast lead time can further be increased if a rainfall forecast is available, from a meteorological service based on numerical weather prediction (NWP) models. If this forecast can be presented as input to the catchment forecast model, several hours may be added to the flow-forecast lead time. This presumes a high level of cooperation between the meteorological service provider and the flood forecasting and warning service, i.e. if these two services are institutionally separated and independent of each other as is the case in some countries in the Danube basin and elsewhere.

Data and technical requirements

Precise details concerned with data and technical requirements depend on the particular nature of a flood forecasting and warning system and its objectives. In general, the overall technical requirements of an advanced reliable flood forecasting system are as follows:

- A real-time data collection subsystem that includes meteorological information, discharge data at the appropriate gauged sections of the river, or water levels and rating curves, and also soil moisture measurements when required. The subsystem may have manual or automatic recording gauges, data collection platforms, radars, satellites, airborne sensors and extensive use of GIS for presenting available information in a useful format.
- Access to the outputs of a numerical meteorological forecasting subsystem, i.e. numerical weather prediction (NWP) models to serve as meteorological forecasting inputs, such as the quantitative precipitation forecast (QPF) during the required lead time of the flood forecasting model.
- A subsystem to combine data from various sources and to provide a feedback mechanism for recalibration of measuring tools and techniques, and initialisation of model error correction.
- A catchment model subsystem, with a user friendly interface, to calculate discharge at the catchment outlet at required time intervals, along with a corresponding estimate of uncertainty.
- A subsystem comprising of a hydrodynamic or a hydrological channel routing model to calculate movement of the flood wave along the channel, the water levels, the effects of dyke breaches and reservoir operation, and the interaction with the flood plain and flooded areas, giving a flood inundation forecast.
- An error correction subsystem with an algorithm for improving the estimates of discharge based on the latest feedback from observed river-gauge data.
- A subsystem for tide or estuary modelling in case of backwater effects influencing the flood flow regime.
- Appropriate communications, GIS networks and decision support systems, producing forecast details at various levels and map forecasts showing flood inundation in real time.

Among all technical requirements given above, a need for adequate data represents a fundamental prerequisite for establishing and operating any flood forecasting system. A short overview of the main data types of a flood forecasting system is given further in the text.

Hydrological data essentially relate to measurement of river flow and water level, and the monitoring instruments should be able to record accurately peak values of both. A network of stream gauges is required for flood forecasting while the nature and composition of the network is determined by required lead time of forecast, forecast accuracy as well as the location of forecast profiles. The forecast profiles usually coincide with a location of hydrological station, as the most convenient solution for modelling river flow, model calibration and verification as well as for operational verification of the issued forecasts. At each such hydrological station, an accurate stage-discharge relationship (also called rating curve) should be established and maintained. Moreover, these stations should be equipped with telemetry links to the operational forecast centre.

However, forecast points can and need also to be designated to a specific reach of a river where flood impact is potentially high, for example near towns, important industrial facilities or agricultural areas.

Meteorological data. Rainfall intensity and duration, quantitative precipitation forecasts and historical precipitation data (for calibration of rainfall-runoff models) are all necessary prerequisites to develop and operate a successful flood forecasting and warning system. Meteorological data and forecasts are required in real time to maximise the lead time for flood forecasts and warnings. The principal item of meteorological data used is rainfall and this is required from a network of rain gauges or radar coverage. These data will provide a best estimate of rainfall over the area modelled, whether over a grid or to obtain a basin average.

The traditional techniques for rainfall forecasting (often referred to as nowcasting), based upon ground-based telemetric rain gauges and meteorological radars, are still widely used. This is because networks have been progressively developed from conventional and broad-based hydrometeorological networks and for this reason are deemed cost-effective. With respect to radars, the use of radar data is warranted for they are available in real time, provide a finer spatial resolution of the precipitation field and have the ability to track approaching storms even before they reach the boundary of the catchment of interest. Radar has some advantages where rain gauges are sparse and the storms are localised, but are of limited value if storms cover large areas simultaneously covering the sites of many rain gauges. In such cases the gauges tend to produce more accurate estimates of rainfall than radar data even though it will still give a better indication of the spatial distribution than that achieved by the use of classical methods, i.e. Thiessen polygons or Kriging interpolation.

With the rapid development of space technology, increased capabilities of the meteorological satellites represent nowadays another option besides radar-based information. There are available open source algorithms which can calculate precipitation from both visual (during the day) and infra-red (during the night) satellite data (e.g. Meteosat). This way useful estimates of precipitation fields/images – with a resolution of 2.5 x 2.5 km up to 1 x 1 km depending on the location – can be established. These satellite-based estimates of precipitation fields have a frequency of 15 minutes and are available within 15 minutes after observation. What is more, these data can also be mixed and matched with radar data if available to arrive at more reliable estimates.

Last but not least, as already mentioned, numerical weather prediction models may be utilised, where available, to provide single and/or ensemble meteorological forecasts (of precipitation, temperature, humidity, etc.) that can be used as inputs into flood forecasting models.

Topographic data are increasingly required for development of flood forecasting systems, as a result of increasing demand for models that can produce realistic estimates of spatial flooding. To this effect, distinction is to be made between conventional topographic information, which can be obtained from classical maps and used to delineate catchment areas, and more detailed information available from digital elevation model

(DEM) data. A new breed of the high-resolution digital maps, obtained through use of LIDAR surveying technology, is now available and being increasingly used in a wide range of applications, including hydrology, hydraulics and flood forecasting. LIDAR high resolution DEM data provide much more accurate information of flood plain and channel capacity for hydraulic models and can be linked to a GIS to provide visualisation of flood inundation extent and flood plain infrastructure.

Other information and data. It is necessary to consider how to use other available data and information as part of the flood forecasting and warning system. Physical catchment data, such as geology, soil type, vegetation and land use data are used to estimate hydrological model parameters as part of the model calibration. Other useful data and information may include:

- population and demographic data as an indication of settlements at risk from flooding
- reservoir and flood protection infrastructure associated with control rules
- inventories of properties at risk
- location of key transport, power and water supply infrastructure
- systematic post-flood damage assessments

Flood Forecasting and Warning System (FFWS): Key Components

Flood forecasting and flood warning systems are closely linked and usually considered a unique system; nonetheless, they essentially comprise of two systems, each with its own specific components, role and responsibility.

Flood Forecasting System (FFS)

As already highlighted, the FFSs can in considerable manner defer from each other; however, there are several key components that are basically the same or similar in each FFS. To establish an effective real-time flood forecasting system, at a bare minimum the following key components need to be in place and linked in an organised manner:

1. Outputs from the meteorological numerical weather-prediction (NWP) models, in particular the rainfall forecasts, including ensemble forecasts, dealing with both rainfall quantity and time of occurrence.
2. Network of manual or automatic hydrological stations, linked to a forecasting control centre by a reliable real-time telemetry.
3. Network of manual or automatic meteorological stations, linked to a forecasting control centre by a reliable real-time telemetry.
4. Availability at the forecasting control centre of a real-time hydrological information system (RT–HIS) for processing and management of a) telemetry data received from the network of hydrological stations; b) telemetry data received from the network of meteorological stations; c) products of NWP models and

rainfall fields/images, if available, based on analysis and processing of radar and/or satellite data; d) other non-real-time and GIS data (such as DEM data, geology, soil type, vegetation cover, land use, etc.).

5. An adequate flood forecasting model, calibrated and verified as appropriate, and linked to the RT–HIS and operating in real time.

Flood Warning System (FWS)

Flood warnings are based on, but distinct from flood forecasts. They are issued when an event is occurring, or is imminent, and must be issued to a range of users, with various motives. Key objectives among them are:

- to bring operational teams and emergency personnel to a state of readiness
- to warn the public of the timing and location of the flood event
- to warn as to the likely impacts on, for example, roads, dwellings and flood defence structures
- to give individuals and organisations time to take preparatory actions
- in extreme cases, to give warning to prepare for evacuation and emergency procedures

Early warning systems of a flood may save lives, livestock and property and invariably contributes to lessening of the overall negative flood impact. They motivate individuals and communities threatened by hazards to react effectively, i.e. in time and in an appropriate manner, so as to reduce the impacts and damages of the flood hazard. They are consequently essential in mitigating the effects of hazards. As an example, information-sharing for flood alerts is essential for both coastal areas and rivers. The disastrous 1953 coastal flood in Western Europe, for instance, showed the high water levels arriving in England more than six hours before they hit the French, Belgian and Dutch coasts.

Unfortunately, this information did not arrive at the other side of the North Sea coast on time. This information from the Met Office of the U.K. would have increased the sense of urgency in the Netherlands and would likely have saved lives.

To be effective and comprehensive, early flood warning systems, and hazard warning systems in general, should be composed of four inter-related elements:

1. Risk knowledge aimed at increasing knowledge about the risks that individuals and communities face.
2. Monitoring and warning service aimed at providing the necessary information. Warning services must have a sound scientific basis for predicting and forecasting, and must be reliable enough to operate continuously to ensure accurate warnings in time to allow action. Warning services for different hazards should be coordinated where possible to gain the benefit of shared institutional, procedural and communication networks.

3. Dissemination and communication aimed at informing individuals and communities about risks and actions. Warnings should contain clear, useful information leading to proper responses to reach the individuals and communities at risk. Communication channels and tools must be identified beforehand and established at regional, national and community levels.
4. Response capability aimed at ensuring that proper response and action is undertaken by the individuals and specialised emergency agencies.

Figure 6. Main inter-related steps of the flood forecasting and warning system chain [26]

Due to their very nature, flood warnings need to be understood quickly and clearly. For this reason considerable attention has to be given to how technical information, produced by forecasters within a flood forecasting system, is conveyed to non-specialists from different organisations, the public, the media and in some cases illiterate population groups.

Main components of a national FFWS

From the above considerations it can be summarised that the main components of a national flood forecasting and warning system are the following:

- collection of real-time data for the prediction of flood severity, including time of onset and extent and magnitude of flooding
- preparation of forecast information and warning messages, giving clear statements on what is happening, forecasts of what may happen and expected impact
- communication and dissemination of such messages, which can also include what action should be taken
- interpretation of the forecast and flood observations, in order to provide situation updates to determine possible impacts on communities and infrastructure
- response to the warnings by the agencies and communities involved
- review of the warning system and improvements to the system after flood events

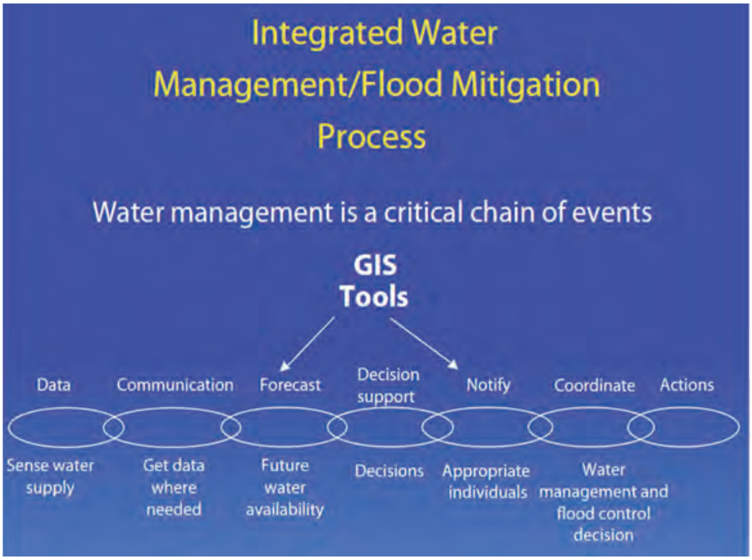

Figure 7. The linkages between the elements of FFWS and the application of GIS tools in integrated water/ flood management process [24]

There are a number of features common to all flood forecasting and warning systems, which are related to causes, impacts and risks. The following characteristics are to be well understood and are considered below in more details.

Features of different FFWS components

Meteorological phenomena. They are the prime natural causes of flooding, either as rainfall or snow and snowmelt. Clearly the ability to forecast critical events, in both time and space and also quantitatively, is of significant value in flood forecasting and warning. Meteorological knowledge associated with flood warning issues fall into two broad areas, namely the climatology behind flooding and the operational meteorology involved. The National Meteorological Service would be expected to be the best equipped to provide both, perhaps with the assistance of appropriate research organisations.

Climatology includes the understanding of rain bearing systems, their seasonality and the extremes of their behaviour. Understanding the types of weather systems from which flooding can originate will contribute largely to decisions about what sort of observational and forecast systems may be required. Thus in an arid zone, where flash floods are predominant, the observation and forecasting facilities must be geared towards rapid recognition of an event. The most effective means for this would be by satellite or radar, while broad scale, synoptic forecasting would be of limited value.

Understanding the seasonality of rain-bearing systems is very important operationally, as this will have a bearing on staff assignments and the organisation of alert and background working patterns. For areas in which the rainy season is well defined, for example Monsoon Asia, tropical Africa and Central America, attention needs to be paid to ensuring adequate staff cover to allow both regular situation updates and round-the-clock monitoring of severe conditions. In temperate and continental areas however, flood events are more random in their occurrence, so flexibility within organisations is required, so that staff can undertake flood warning duties as necessary, i.e. in states of emergency, though their routine tasks may be wider.

Hydrometeorological statistics (primarily rainfall, but also evaporation) are vital to flood forecasting and warning operations and they are usually dealt with separately from climatology data. The purpose of the data and statistics is to estimate the severity and probability of actual or predicted events and to place them in context. Long-term records are essential and this requires investment to install and maintain rain gauge networks (plus evaporation and/or climatological stations), to assure staff and facilities to process and analyse records and to maintain a flexible and accessible database.

Hydrometeorological data are also vitally required in real time for the provision of flood forecasts and warnings. To this effect, it is essential that a representative proportion of the rain gauge network is linked to the forecasting and warning control centre by telemetry. This has a three-fold purpose:

1. To allow staff to monitor the situation in general terms.
2. To give warnings against indicator or trigger levels for rainfall intensity and/or accumulations.
3. To provide inputs into hydrological forecast models, in particular into rainfall-runoff models.

Hydrological inputs/component. The requirements concerning hydrological information for a flood forecasting and warning system are similar to those for meteorology, in that it is necessary to have an understanding of the overall flood characteristics of the area as well as having real-time hydrological information for operational purposes. Key observation and data requirements are for water levels in lakes and rivers, river discharge and in some cases groundwater levels.

The network of hydrological observing stations have a dual role – to provide 1. hydrological non-real time data for long-term statistics; and 2. real time data through telemetry to a control/forecasting centre. Water level ranges at given points can be linked to various extents of flooding, so a series of thresholds can be set up to provide warning through telemetry.

The upstream–downstream relationship between water levels is also an important means of forecasting. Early flood warning systems, which are in use in some FFWS, depended on knowledge of the comparative levels from a point upstream to resulting levels at a point of interest at the flood risk site and the time taken from a peak at an upstream point to reach a lower one. These were presented as tables or graphs of level-to-level correlations and time of travel.

Developments in real-time flood modelling now provide the facility to provide more comprehensive information on forecasting of levels, discharges, timing and extent of flooding.

Dissemination of forecasts and warnings. The effective dissemination of forecasts and warnings is very important. A balance has to be struck between information to the public and information to other bodies involved with flood management. Historically, this has resulted in a dichotomy for flood warning services, which have to partition support resources between the community and government. The subject has been the focus of severe criticism in the light of past failures at service delivery. Thus, the language used and the type of information passed on has to be carefully considered and structured. There has been a gradual evolution away from confining flood forecasting and warning information to authorities, that is, government, to a more direct involvement of the public. This has been helped by the growth in telecommunications, the computer, the IT revolution and increased ownership and coverage of media, such as radio and television. It is important, however, to maintain a broad spectrum for dissemination and not to be seduced by high-tech approaches. Even in technically advanced societies it is doubtful whether Internet communication of flood warning information can be entirely effective. The elderly and poor members of the community may not have the necessary facilities at home, and it may also be doubtful whether people will consult Websites when a dangerous situation is in place. It must also be remembered that these systems are dependent on telecommunications and power links that are themselves at risk of failure during flood events.

As a counter to over-sophistication and reliance on high-tech methods some alternative facilities need to be provided. In the past, in most parts of the world, emergency services (police, fire service, civil defence) have been closely involved in flood relief activities. Their role may change with changing technology but they still need to be involved in communicating flood warnings and rescue. Other general warning systems, such as flood wardens and alarm sirens should not be abandoned without careful consideration of the consequences.

Institutional aspects. A flood forecasting and warning system needs to have clearly defined roles and responsibilities. These are wide ranging, covering, inter alia, data collection, formulation and dissemination, uncertainty of outputs and any legal or liability requirements. Whatever the functional and operational responsibilities of the separate agencies involved in flood forecasting and warning, there is a fundamental responsibility through central government for public safety and emergency management. There may not be, however, a general statutory duty of the government to protect land or property against flooding, but the government recognises the need for action to be taken to safeguard the wider social and economic well-being of the country.

Operating authorities may have permissive powers but not a statutory duty to carry out or maintain flood defence works in the public interest. However, such responsibilities may be incorporated through legislation within acts and regulations under which different government departments operate. When legislation is set up or amended, it is therefore extremely important that interfaces between the duties and obligations of affected departments are carefully considered before statutory instruments are introduced.

The institutional structure and responsibility may become complicated for the following reasons. Several ministries may carry separate responsibilities for activities related to flood forecasting and warning. Furthermore, within implementing organisations, flood forecasting and warning duties may represent only a fraction of their overall responsibilities.

Some countries have a combined hydrological and meteorological service, for example Russia, Iceland, some eastern European countries. This theoretically eases issues over data collection, use and dissemination that arise when one organisation collects atmospheric data and the other provides rainfall and river data. In many cases rainfall data are collected by both the meteorological and hydrological agencies and the type of data is influenced by historical factors, or the primary requirement for rainfall data.

Flood forecasting and warning as a focused activity in the hydrometeorological sector is a relatively recent development. This may be evidence of the growing seriousness of flood impacts, both as a result of greater financial investment and pressure of population. Previously in the United Kingdom, France and other European countries, response mainly focused on flood defence and warning through the general meteorological forecasting of severe weather. However, the occurrence of a number of severe events from 1995 to 2003 led to the setting up of national flood forecasting and warning centres/services. This has provided the opportunity to enhance the development of monitoring networks specifically designed for flood forecasting and warning purposes. Hydrological networks consist of instruments that have electronic components for data storage and transmission (rain gauges and water level recorders) and meteorological effort has focused on collection and delivery of satellite and radar data.

Legal aspects. Any flood forecasting and warning system has to deal with uncertainty. This is inherent due to the nature of the meteorological and hydrological phenomena involved. To this has to be added the uncertainties involved with equipment and human error within an operational structure. Uncertainty may be dealt with in the design and planning processes, where a decision is made on the level of uncertainty, that is, the risk of failure that is acceptable. This then becomes a balance between the cost of safe design against that of the losses caused by damage. Except where "total protection" is provided for key installations such as national security locations and nuclear plants, aspects of uncertainty can be approached through probabilistic methods. The probabilistic approach is increasingly being used as part of risk analysis, where impact and consequence in human and economic terms are linked with the causative meteorological and hydrological characteristics.

Liability in strict legal terms is difficult to apply to the various activities in flood forecasting and warning. Whereas a contractor may face liability for the failure of a flood protection structure (for example a dam or a flood wall) or a manufacturer for a product not meeting specifications as to flood resistance or proofing, most national and international legal systems and codes regard floods and the causes thereof as “Acts of God”. Liability in regard to flooding tends to operate in a “reverse” way, i.e. that compensation or redress for losses and damage may not be given if a case shows that there has been some form of negligence in design or that guidelines have been ignored.

In many countries governments or international agencies can provide compensation or assistance in rebuilding, but there is no legal obligation. Insurance is increasingly fulfilling the role of government in recovery actions, particularly in developed countries where it is a commercial arrangement. The increasing use of insurance has, however, meant that when events occur, the cost to insurance companies becomes larger, leading to rises in premiums. This situation also leads to insurance companies deciding on what is or is not a worthwhile risk, which often leads to properties in high flood risk areas being uninsurable.

Methods and models used for real-time flood forecasting

With recognition of the importance of flood forecasting and warning for flood management, the expectations from flood forecasts in terms of magnitude, reliability and forecast lead time have substantially increased, and there is no doubt this trend is only going to intensify in the future. Past methods of simple extrapolation of forecasts from gauged sites can no longer satisfy the requirements.

While the heart of any flow forecasting system is a hydrological model, it goes without saying that catchment modelling is just one of the crucial elements on which the effectiveness and efficiency of an integrated flood forecasting and warning system (FFWS) depends.

There exist a bewildering number of hydrological models in use in various parts of the world, including various procedures for their development, calibration and verification. As an example, Figure 8 illustrates typical procedures and steps needed in creating a suitable flood forecasting model [24].

As catchments respond to the hydrological cycle phenomena in a broadly similar manner, one might expect that modelling would be well focused, involving a fairly simple process of refinement to make the models more robust, versatile and sophisticated. Nonetheless, the fact is that a selected model can work quite well in some cases but poorly in others. For this reason, a modular approach to modelling is increasingly used, whereby each identifiable component of the runoff-generating process in the catchment (e.g. snowmelt, infiltration, groundwater, flood routing, etc.) can be represented as a separate module, which is then included into the overall model structure. Moreover, in a good model the forecast uncertainty – arising from data measurement error, model structural (physical) inadequacy and suboptimal parameter estimation – must be estimated and explicitly accounted for.

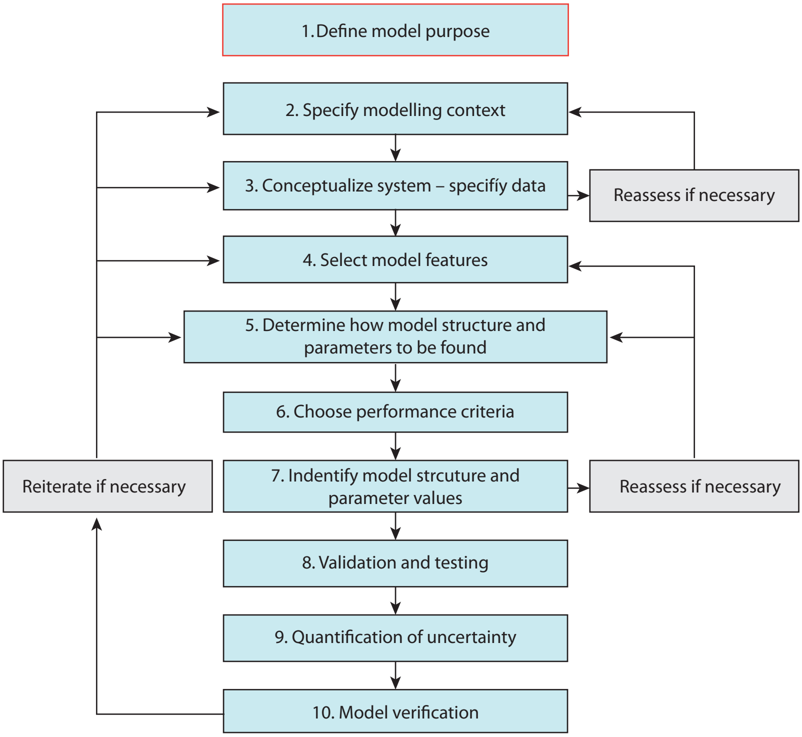

Figure 8. Typical steps and procedures required in creating a suitable flood forecasting model [24]

To be useful, the selected forecasting models must satisfy certain criteria, depending on the requirements of the stakeholders and the end-users of forecasts. On the other hand, the degree of model complexity should be consistent with the actual "information carrying capacity" [11] of the data available to calibrate and run them in operational mode. Increasing model complexity, in terms of the number of components and parameters involved, is not necessarily warranted or justifiable, and can actually be counterproductive. The best practice in developing and using FFWSs is evolving towards the use of more physically-based distributed models or at least an integrated suite of simplified models running simultaneously.

Prerequisites for a reliable real-time hydrological forecasting model

An ideal real-time, fully operational forecasting model should satisfy the following prerequisites [17], i.e. it must:

- account for the physical laws that govern the hydrological processes (i.e. rainfall-runoff in a catchment and streamflow along a main channel)
- explicitly account for forecasting uncertainties

- react, as quickly as possible, to changes that might occur in the watershed due to natural and human causes by modifying model parameters, i.e. must be adaptive while having parameters that are sensitive to such changes
- be rendered with long enough yet the most reliable forecast lead time
- specify and produce unbiased forecast errors
- be able to accommodate any changes in the observation network and the resulting additional information without changes in the model structure
- make data substitution possible through interpolation or finding analogies where there are missing measurements
- be numerically stable and without high demand of computer resources and time for computation
- express fast convergence for any numerical scheme in the model
- have a structure that makes it possible to include the model in operational systems of water management
- have recursive algorithms so that the model can be run on computers with limited memory capacity

It is safe to say that for the time being no universal, operative forecasting model exists, and most probably there will not be any in the near future. Yet, the generalisation of existing models should be accomplished, and an attempt made at the creation of new, ever more general models. The aim is to arrive at a model that would have modular structure and be as little site-specific as possible. To this effect, the MIKE SHE model[3] or Delft-FEWS platform may be considered good examples.

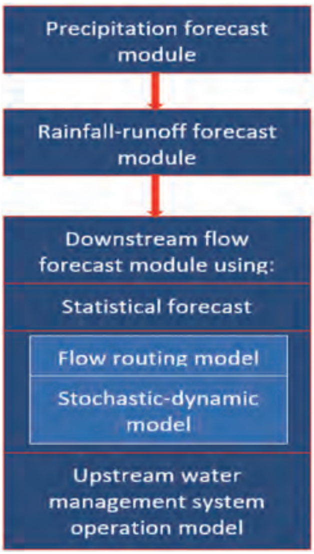

Figure 9. Modular structure of forecasting models (compiled by the authors based on [17])

[3] For more information see www.mikepoweredbydhi.com/products/mike-she

Such a modular structure is illustrated in Figure 9 above, where each module represents a sub-function within the complete task of hydrological forecasting [17].

Currently, a great number of hydrological models is in use in various parts of the world. Many models that were implemented in operational FFWSs decades ago are still in use, having undergone only occasional refinement or cosmetic interface updating, as the forecasts produced by them are still considered adequate by their end-users.

Describing such methods and models in detail, and the elaboration of their mathematical development is beyond the scope of these notes. Model development is a specialist undertaking and, regrettably, there is still a wide gap between developers and practitioners, including many specialists that are directly involved in operational flood forecasting. In the text that follows only the main categories or classes of models illustrated in the above Figure 9, and some representative examples, are presented.

Precipitation forecast module

Information about precipitation/rainfall distribution over a catchment represents a crucial forcing of any model used for modelling rainfall-runoff process in a catchment and forecasting of flow hygrogram at its outlet profile. The ability to forecast meteorological critical events quantitatively, such as rainfall and temperature in both time and space, is of significant value in flood forecasting and warning. Nowadays, such forecasts are produced by using numerical weather prediction (NWP) models and are available from National Meteorological and/or Hydrometeorological Services and, for Europe and hence the Danube basin, from the European Centre for Medium-Range Weather Forecast (ECMWF[4]). Apart from classical NWP products, the ECMWF is also producing ensemble forecasts;[5] all its real-time products are available to Member States[6] and Cooperating States[7] in real time who can use them either without modification or to prepare their own user-oriented specific forecasts for end users. The NWP products in general, in particular

4 For more information see www.ecmwf.int/

5 An 'ensemble forecast' consists of 51 separate meteorological numerical forecasts made by the same computer model, all activated from the same starting time. The starting conditions for each member of the ensemble are slightly different, and physical parameter values used also differ slightly. The differences between these ensemble members tend to grow as the forecasts progress, that is as the forecast lead time increases.

6 Austria, Belgium, Croatia, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, the Netherlands, Norway, Portugal, Serbia, Slovenia, Spain, Sweden, Switzerland, Turkey and the United Kingdom.

7 Bulgaria, the Czech Republic, Estonia, the former Yugoslav Republic of Macedonia, Hungary, Israel, Latvia, Lithuania, Montenegro, Morocco, Romania and Slovakia.

quantitative precipitation forecasts (QPF), are still fairly inaccurate and have inadequate spatial-temporal resolution in comparison with hydrological modelling requirements. Nevertheless, they are of great value and are regularly being used as input to hydrological models for flood forecasting.

The NWPs are not the only option for operational quantitative precipitation forecast. As already mentioned, for very short lead time of forecast, extrapolation of radar and/or satellite precipitation patterns, also called nowcasting by meteorologists, represent another possibility. Nowcasting methods capture the initial information very well, but as they do not include physics, the skill decreases rapidly with increase of lead time. (The NWPs, on the other hand, capture the physics of large systems very well, but lack local detail because of their limited spatial resolution). This option is especially valuable for severe weather conditions like the thunderstorms causing flash floods, as exact location and timing of these phenomena are difficult to predict without a radar/satellite nowcast.

It is important to stress at this point that the only way to improve either NWP or radar/satellite nowcasting products, QPFs in particular, is through appropriate calibration and verification of NWP models and radar/satellite images in a given catchment. There is no other way to achieve this, but through adequate spatial coverage of meteorological/precipitation telemetry network in the catchment that provides observed real-time meteorological data necessary for the purpose. At first glance it may look like a paradox, but the fact is that the evident contemporary achievements in meteorology cannot bear fruits without expanding rather than downsizing the telemetry network of meteorological stations.

Rainfall-runoff forecast module

Modelling of runoff processes in a catchment started in the first half of the 20th century, and coincides with the publication of L. K. Sherman's (1932) seminal paper on the unit hydrograph (UH). Sherman's theory described a dynamic link between effective precipitation and direct runoff by postulating a number of hypotheses that led to a linear construction (later called a dynamic linear system), which connected two quantities, input and output, changing in time. Direct runoff as output was calculated as a weighted sum of past effective precipitation inputs. The weights formed a function that was called 'unit hydrograph' (UH) that essentially was the response of a catchment to a unit volume input of effective rainfall spread uniformly over the catchment area during one time unit. The UH was not known a priori, but had to be identified from input–output records.

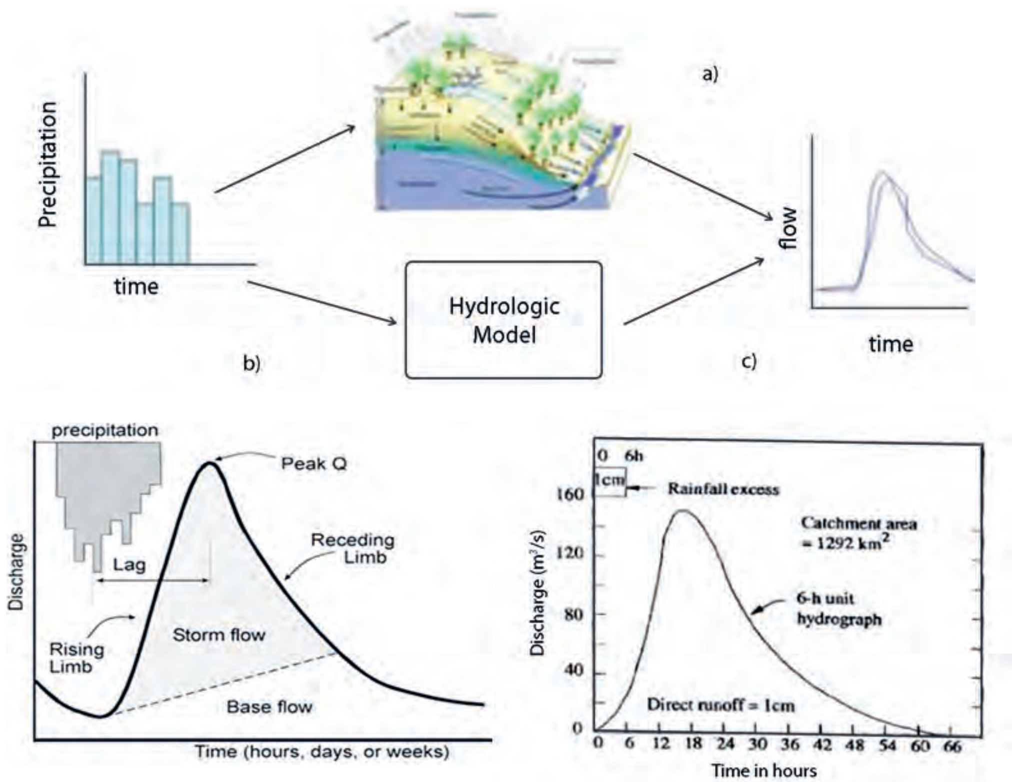

Figure 10. Illustration of a) rainfall-runoff processes in the catchment; b) typical flood hydrograph at the catchment outlet profile; and c) classical T-Unit Hydrograph (adapted by the authors based on [2])

The UH concept is still alive and well. It is being used in many models and engineering applications and thousands of papers have been published in scientific journals on the subject. Yet, the problem is that neither effective precipitation nor direct runoff exist in physical reality, and the concept epitomises vast oversimplification of the complex rainfall-runoff process. So hydrologists started using the UH concept, but also continued to rely upon the use of coaxial graphical procedures, the Antecedent Precipitation Index (API) rainfall-runoff charts and the like.

Things changed significantly with the advent of digital computing. It was recognised that the UH is in essence a black box, which cannot take into account physical processes in the whole land phase of the hydrological cycle, such as canopy storage, evapotranspiration, infiltration, soil moisture, groundwater flow, snow melt and flow routing. How can the human impacts such as land use changes, reservoirs and urbanisation be considered? How to go from external to internal modelling?

This led to the era of conceptual models – starting with the introduction of elementary linear reservoirs all the way to the complex Sacramento and Stanford-type watershed models and their numerous derivatives.

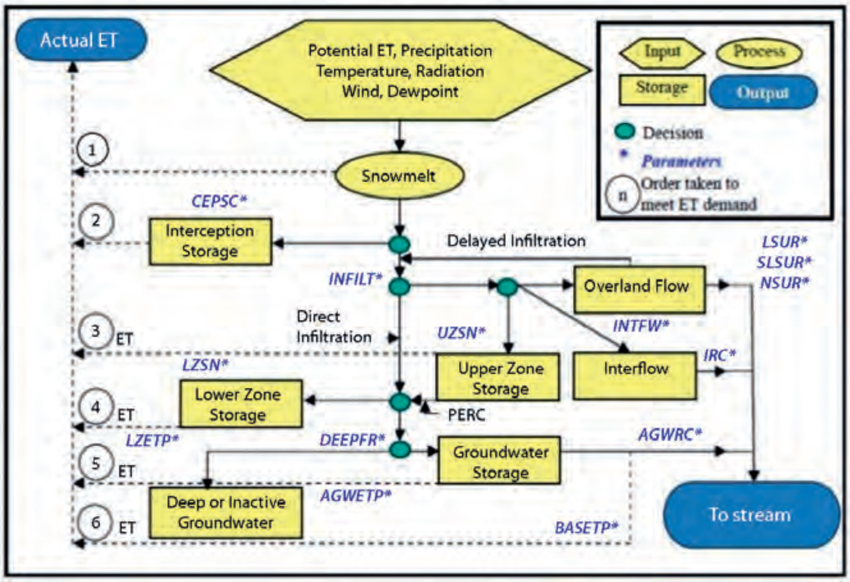

Figure 11. Stanford watershed conceptual model (adapted from [30])

Complex processes in the catchment that occur between catchment input and output have been gradually brought back to the stage in a digital framework. Between total precipitation and total river flow a deterministic structure has been conceptualised based on the physical understanding of the mechanisms and processes in the hydrological cycle.

Dynamic developments in this field led to introduction of a number of new concepts and approaches in modelling rainfall-runoff process. According to the model structure, the following type of models are being used:

– empirical (stochastic) data-driven rainfall-runoff models
– “physically inspired” lumped conceptual rainfall-runoff models
– physically or process-based distributed rainfall-runoff models
– hybrid physically-based/conceptual distributed models
– hybrid metric-conceptual models
– event-based versus continuous simulation
– simulation models versus forecasting models

Focusing on models that can be used for operational real-time flood forecasting, frequent problems in using distributed complex models – either the Sacramento/Stanford-type conceptual or the physically-based distributed models – arise from many parameters that have to be determined and, even more important, lack of adequate observed data for their calibration, verification and subsequent operational use.

Some of these parameters are measurable, such as for example the ratio of impervious areas in an urban catchment; other parameters, however, have to be calibrated from observed data, either manually or by applying some of the optimisation techniques.

As a result, they often lose their physical meaning in the process and eventually lead to disappointing model performance in spite of the model complexity.

Sometime this complexity is in the heart of the problem, in particular when it comes to using a model for operational flood forecasting. It has been argued that less complex, i.e. simplified conceptual models, consisting of elementary linear reservoirs and/or linear channels, contain very few control parameters and can easily be calibrated.

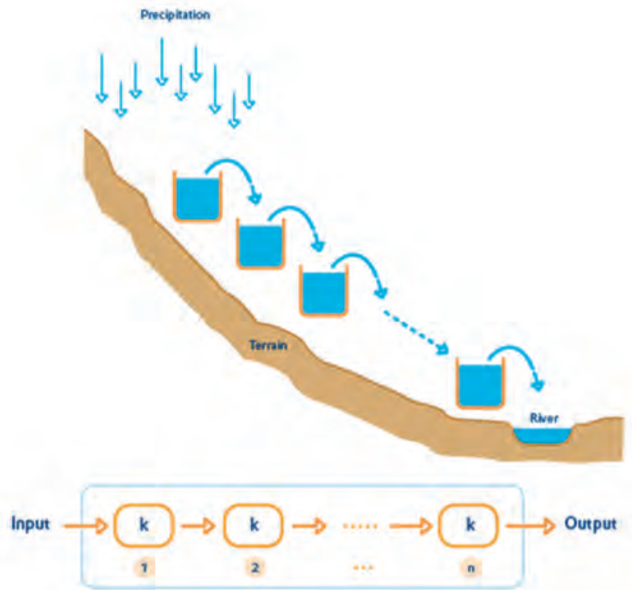

Figure 12. Kalinin–Milyukov–Nash (KMN) cascade conceptual model [16]

In spite of this simplification, they are usually capable of explaining a large portion of the catchment precipitation-runoff process. Very often simple linear reservoir cascade models could explain more than 90% of the total process variance. One such model is the Kalinin–Milyukov–Nash cascade[8] (Figure 12) – characterised by the Instantaneous Unit Hydrograph in analytical form controlled by two parameters only (the number of reservoirs/sub-reaches (n) and the residence time of reservoir/sub-reach, often referred

[8] It is interesting to note two points regarding the KMN cascade: 1. Kalinin and Milyukov (1957) and Nash (1957) arrived independently at the same result, i.e. formulation of the Instantaneous Unit Hydrograph (IUH) by using a completely different approach; the former by serially linking sub-reaches of a river reach with one another and assuming that the flow out from a sub-reach (the so-called characteristic reach) is linearly proportional with the amount of water stored in that particular reach; the latter by assuming that overland flow in a catchment could be represented by a series of linked elementary reservoirs, where the outflow from one elementary reservoir is the inflow into the next one and is linearly proportional to the storage in the elementary reservoir. 2. In essence, Kalinin–Milyukov's IUH represents a linear flow-routing model; models of this type are well known, have been used in hydrology for quite some time for flood routing (better known as 'hydrological flow-routing models') and could all be derived from the kinematic wave equation, representing the first-order approximation, i.e. 'the bare bone structure', of the full hydrodynamic Saint-Venant equations that govern one-dimensional, unsteady, gradually varying flow in open-channel.

to as storage coefficient). With some adjustments and modifications, the KMN cascade model is a good candidate to fulfil the requirements of an operational real-time hydrological forecasting. For this purpose, it needs to be a) converted from a time continuous model into an adequate discrete time model; b) transformed into a form that allows easy updating; and c) able to deal with the inherent forecast and other uncertainties, such as those in the processes involved as well as in their measurement. Model simplification (by using the kinematic wave equation as first-order approximation of the Saint-Venant equations) inevitably yields increased model uncertainty, so it is important to handle this type of simplification-imposed randomness as well. Note that, as a matter of fact, the discrete version of the KMN cascade represents a discrete linear cascade model (DLCM), the term that is usually used for this class of models.

The DLCM or discrete KMN cascade model types (deterministic component) coupled with a stochastic component within the unified deterministic-stochastic state-space model formulation are being successfully used for real-time streamflow and flood forecasting in the Danube basin, in particular Hungary where it is operational used in the whole country since 1984 [17], but also outside Hungary, i.e. in Germany, Thailand and Malaysia. This modelling approach, but using instead the Kalinin–Milyukov non-linear model formulation as deterministic component, has also been successfully tested in the Serbian part of the Danube river [2] [3] [4].

Downstream flow forecast module

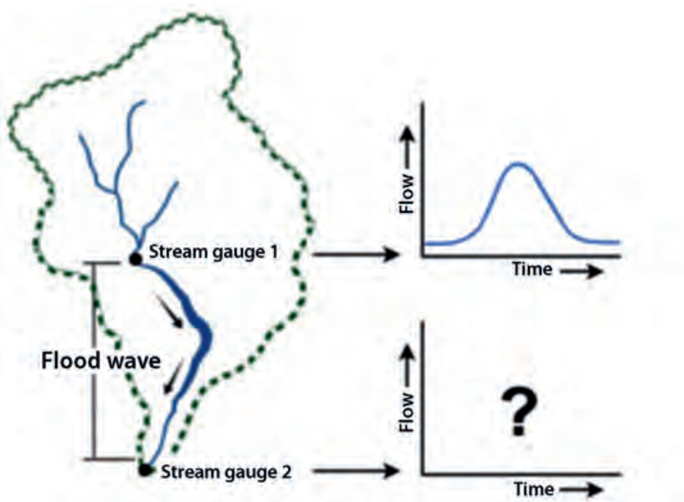

Figure 13. A catchment and river reach with upstream and downstream hydrological profiles [2]

Given the observed or forecasted flow hydrograph at the catchment's outlet profile upstream, this module deals with the dynamics of river flow with the main objective to come up with a model capable of real-time streamflow and flood forecasting at the downstream river reach or reservoir profile (Figure 13).

In general, the runoff-runoff phenomenon is addressed by various routing techniques that are broadly classified as either hydrological or hydraulic routing [24] as shown in Figure 14.

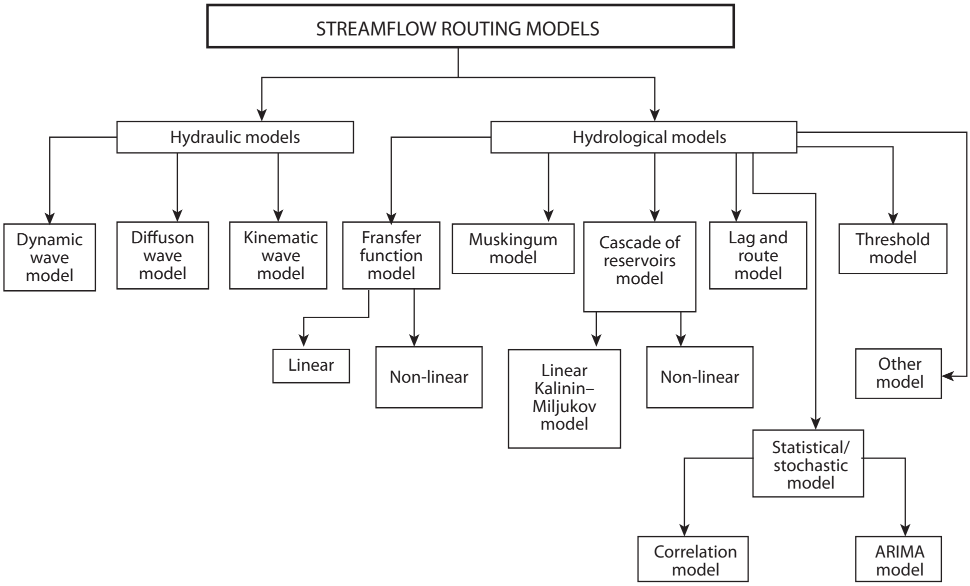

Figure 14. Classification of streamflow routing models [24]

As in case of catchment models, there exists a number of different forms of routing models spanning a wide range of complexity and computational requirements. In comparison to hydrological routing, which focuses on the relationship between the hydrographs at the upstream and downstream sections of a channel reach, hydraulic routing provides a more physically-based description of the dynamics of flow, such as for example the velocity and depth as functions of space (distance) and time. Hydraulic routing using dynamic or diffusion wave model requires details of the channel sections and much greater computational effort; for this reason, in comparison with hydrological routing it is less used as a tool for real time forecasting. As noted earlier, the kinematic wave model represents a 'bare bone structure', of the full hydrodynamic Saint-Venant equations and is often considered hydrological rather than hydraulic routing model; as such, it is not too demanding in terms of data requirements, and is widely used in various forms for real-time streamflow forecasting. In general, the picture becomes pretty blurred when one attempts to make a clear cut classification of various models used in hydrology; this is also the case with the classification presented in Figure 14.

Without going into exhaustive details, a short account of the following three main approaches in modelling the runoff-runoff process is presented in the text below:

– stochastic black box approach
– deterministic (dynamic) streamflow routing methods
– coupled structural (dynamic) – stochastic models

1. *Stochastic approach.* The problem of flow forecasting many modellers are trying to solve by using a "black box" input–output approach, whereby the physics behind the streamflow (runoff-runoff) or the catchment (rainfall-runoff) process is not defined explicitly. Stochastic hydrologists essentially look at the unit hydrograph theory and consider input–output relation only, but with one major difference. It is assumed that both input and output are two stationary random time series connected by system 'transfer function'.[9] The stochastic approach in modelling hydrological time series has its root in, and has decisively been influenced by a brilliant book written by the statisticians George Box and Gwilym Jenkins on time series analysis, forecasting and control [6]. Time series analysis became a popular tool in hydrological research and various identification techniques were used to estimate the system transfer function, or the stochastic IUH in hydrological parlance, from observed data.

Even though it is still being used, the approach performs poorly when used for real-time hydrological forecasting. However, it paved the way to a favourable marriage between the deterministic groom and the stochastic bride (or was it vice versa?). In the previous chapter, we have already touched on the benefits of this marriage for real-time hydrological forecasting, and a few more arguments in its favour will be discussed under the coupled structural– stochastic models later in this chapter.

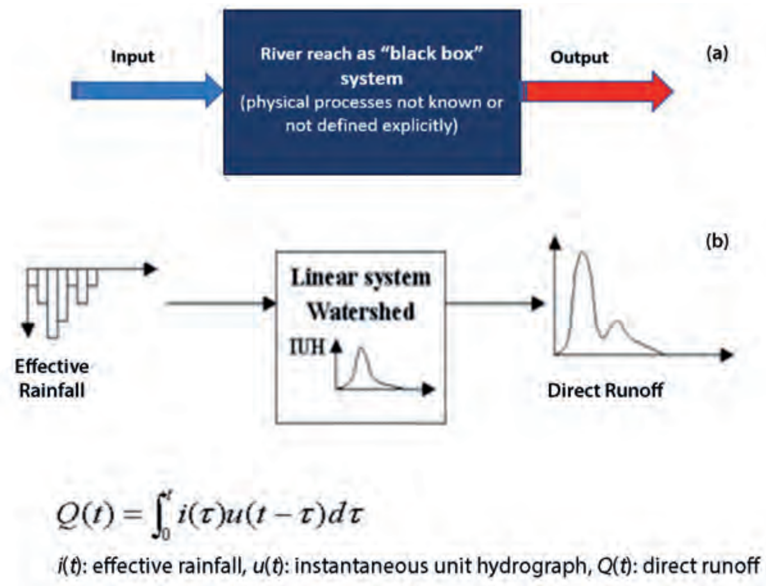

Figure 15. Illustration of a) the stochastic black box approach in modelling runoff-runoff process; and b) unit hydrograph approach in modelling effective rainfall – direct runoff process (Adapted from [2])

[9] This function some hydrologists termed a Stochastic Instantaneous Unit Hydrograph (SIUH)! There is no doubt this is an incorrect term as the stochastic transfer functions simply transform input into output series without any consideration of physical processes involved.

2. *Dynamic streamflow routing.* All the hydraulic routing models (i.e. dynamic, diffusion and kinematic wave) and a few of the hydrological routing procedures (Muskingum, discrete linear cascade of reservoirs, and the Kalinin–Milyukov model) listed in Figure 14 above are to a lesser or greater degree deterministic physically-based models.[10]

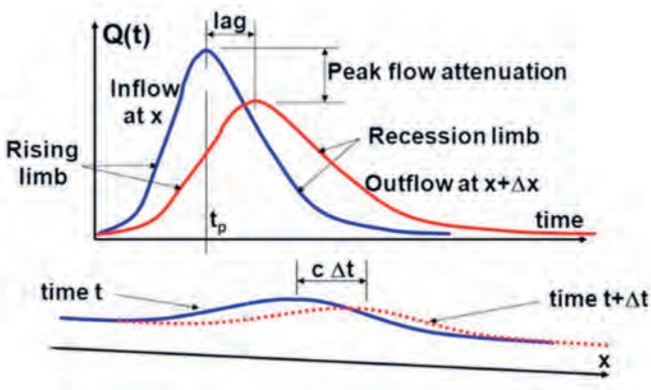

Figure 16. Dynamic flood routing: definition of key notions and variables (PPT – Flood Routing definitions PowerPoint Presentation, free download – ID:719053 slideserve.com)

All hydraulic routing models are nowadays being used in various forms for real-time flood forecasting as well as for delineation of flood risk areas.

3. *Structural (dynamic) – stochastic models.* As indicated in the foregoing text, the uncertainty is implicit in any human endeavour aimed at forecasting the future, and the hydrological forecasts are no exception to this dictum.

Broadly speaking, any hydrological model (either of the precipitation-runoff or the runoff-runoff type, or combined) used for real-time forecasting that has a modus operandi capable of assessing explicitly the probabilistic measure of uncertainty associated with each forecast as in Figure 17, belongs to this model category.

Uncertainty estimation is often carried out using a combination of numerical meteorological or hydrological models and statistical techniques. These techniques comprise of Monte Carlo analysis and statistical post-processing. Monte Carlo analysis is at the heart of ensemble techniques, where multiple plausible, equally probable initial conditions or model parameters are used as inputs to multiple model runs. Statistical post-processing aims to characterise the relation between forecasts and observations and, assuming that this relation is valid in the future as well, applies this relation to future forecasts.

Another and probably the best way to deal with uncertainty of forecasts is identical to many of the control systems applications in the sixties of the last century in rocket and

[10] Some pure hydraulicians might argue that none of the hydrological models for streamflow routing belongs to this category, but it has been demonstrated that a linear cascade model is discretely coincident with the hydraulic kinematic wave model. Similar reasoning applies for a variant of the original Muskingum, i.e. the Muskingum–Cunge model that has more physical/hydraulic basis.

communication engineering: forecasting the future trajectory of a dynamic system subject to random disturbances by minimising forecast error variance. Rudolph Kalman worked out his famous Kalman filter in 1960 for these types of problems. Back to hydrological forecasts, in essence the Kalman filter enables recursive estimate of conditional density functions (i.e. uncertainty) of our 'deterministic' dynamic hydrological forecasts. In fact, the Kalman filter gives the best estimation for linear dynamic systems and no better tool could be designed. (The trouble is that catchment processes cannot always be approximated by, and modelled as linear dynamic systems.)

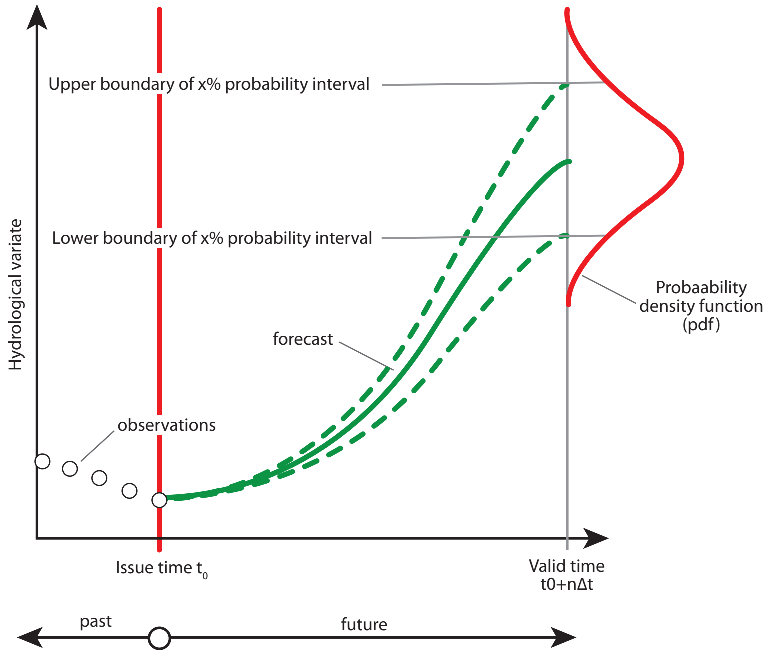

Figure 17. Illustration of a forecast with probabilistic measure of forecast uncertainty [16]

The above techniques aim to produce probabilistic forecasts that are reliable and as certain as possible. The very notion of reliability pertains to the probabilistic nature of the forecasts: predicted probabilities have to be matched by observed relative frequencies. Measure of uncertainty pertains to the width – or rather, the narrowness – of the predictive intervals (see Figure 17), i.e. the variance of the forecasting errors. Ideally, the variance should be as close to zero as possible, with the ultimate but unattainable goal of having zero value.

There are several procedures the hydrologists use nowadays in practice, but the discussions are still ongoing on what constitutes the most appropriate approach for estimating predictive uncertainty of hydrological forecasts.

Common procedures include, but are not restricted to:

Use of meteorological ensemble forecasts as input to a precipitation-runoff model (with subsequent flow routing downstream the river reach), thus producing a 'plume' of hydrological forecasts at each time step.

Multi-model approach whereby the ensemble of river flow forecasts of a number of rainfall-runoff or runoff-runoff models is used synchronously to produce a combined flow forecast at each time step, through aggregation of the results of available alternative model structures, thus avoiding reliance on a single model.

The multi-model approach can also be applied to the ensemble of forecasts obtained from alternative parameter sets of the same model, which produces near equal model efficiency. This approach is similar to that used in generating meteorological ensemble forecasts and may be considered a special category of multi-model forecasts. Using a single hydrological model, multiple numerical prediction are conducted using slightly different initial conditions that are all plausible given the past and current set of observations or measurements. Sometimes the ensemble of forecasts may use different forecast models for different members or formulations of a forecast model. The multiple simulations are conducted to account for the two sources of uncertainty in forecast models: first, the errors introduced by chaos or "sensitive dependence on the initial conditions"; second, the errors introduced because of imperfections in the model. Using the output from a number of forecasts or realisations, the relative frequency of events from the ensemble can be used directly to estimate the probability of a given flood event.[11]

Lastly, for tackling predictive uncertainty of real time hydrological forecasts, probably the best approach is to couple a suitable deterministic hydrological model with a stochastic component within a unified deterministic-stochastic state-space model formulation, thus enabling the use of the Kalman filtering techniques. This approach satisfies to a high degree the prerequisites for a reliable real-time hydrological forecasting model and has been extensively tested and operationally used in the Danube basin and elsewhere.

Before concluding the discussion on hydrological modelling, let us just reaffirm something what already seems logical and obvious: all the modules/models considered (see also Figure 9) – i.e. the precipitation forecast and the catchment rainfall-runoff – may each be used either standalone or combined in an integrated system for flood forecasting.

There are numerous examples of real-time forecasting and warning systems in use in the world. Some of them will be shortly summarised in the next section, focusing primarily on the systems in operational use in the Danube basin.

[11] Ensemble forecasts are more widely applied to NWP models than to hydrological models, with the probabilistic outcome to a number of NWP runs being used to provide the "most likely" scenario for input into a hydrological model. Applying ensemble approaches to both NWP and hydrological models would undoubtedly be prone to producing results with a wide range of uncertainty.

Examples of the FFWS in operation in the Danube basin

More than 40 rivers cross at least one border in Europe, with the most transnational river being the Danube, which is shared by 18 countries. In case of flooding, this means that different authorities involved in water resource management, civil protection and the organisation of aid must communicate, share data and information and, ideally, take concerted actions to reduce the impact of the flooding along the course of the river. There are a number of FFWSs in operation within the Danube basin, ranging from local, national and regional ones. There is also a unique pan-European system that also covers the Danube basin. In the text that follows, an outline is given about this European system, the newly developed regional Sava FFWS and the national Hungarian flood forecasting system.

European Flood Alert System[12]

Significant floods that occurred across Europe at the beginning of the 21st century led the European Commission (EC) to initiate the development of the European Flood Awareness System (EFAS). The objective of EFAS is to provide a pan-European medium-range streamflow forecast and early warning information in particular for large transnational river basins, in direct support to the national forecasting services. From 2003 to 2012, EFAS was developed and tested at the Joint Research Centre (JRC), the in-house science service of the EC, in close collaboration with various national hydrological and meteorological services across Europe, and other research institutes. In 2011 EFAS became part of the Copernicus Emergency Management Service (EMS)[13] in 2012 it was transferred from research to operational service, and over the past 10 years EFAS has become increasingly integrated into national and European flood risk management.

EFAS Structure. EFAS follows many operational hydrometeorological systems in generating forecast products based on the output of a hydrological model forced by numerical weather predictions (NWPs). For each forecast, the initial conditions of the hydrological model are derived using observed meteorological data. The forecast products are placed on a web platform available to the EFAS partners. These products are then analysed and, if necessary, the awareness of responsible authorities to the potential for upcoming flood events is raised.

The operational EFAS has been outsourced to four centres: 1. Hydrological data collection centre, which collects historic and real-time river discharge and water level

[12] For more information see www.efas.eu

[13] Copernicus Emergency Management Service (Copernicus EMS) provides information for emergency response in relation to different types of disasters, including meteorological hazards, geophysical hazards, deliberate and accidental man-made disasters and other humanitarian disasters as well as prevention, preparedness, response and recovery activities. The Copernicus EMS is composed of an on-demand mapping component providing rapid maps for emergency response and risk recovery maps for prevention and planning and of the early warning and monitoring component which includes systems for floods, droughts and forest fires (www.emergency.copernicus.eu).

data; 2. Meteorological data collection centre, which runs onsite at the JRC[14] and collects historic and real-time observed meteorological data; 3. Computational centre at the European Centre for Medium-Range Weather Forecasts (ECMWF), which collates NWPs, generates the forecast products and operates the EFAS Information System web platform; and 4. Dissemination centre, which analyses the results on a daily basis, assesses the situation, and disseminates information to the EFAS partners and to the EC.

Data acquisition. EFAS requires hydrological and meteorological data from in situ observations to calculate the initial hydrometeorological conditions and forecasting data to drive the flood forecasting system. Various meteorological and hydrological national services or river basin authorities, including a number or agencies from the Danube region, provide real-time and historic data to EFAS. For EFAS, the meteorological and hydrological data collection centres are in charge of managing the existing network of providers of observed data. The centres can also contact potential providers and negotiate standard data license agreements between the provider and the Copernicus services.

Data are collected on a 24/7 basis. Hydrological data collection provides real-time and historic in situ hydrological observed data. Real-time data are used in the generation of post-processed forecast products while the historic data are also used in model calibration. Currently, data are collected for over 800 sites. The meteorological data collection centre collates several variables from gauges including precipitation, temperature and wind speed, though not all variables are collected from all stations. Alongside the in situ observations HSAF[15] satellite-derived soil moisture and snow coverage products are also collated for visualisation purposes. Where the flood alerts issued by the national agencies are available, these are displayed in a common framework. For example, EFAS Information System (EFAS-IS) shows the warnings issued by the Swedish Hydrological Service to the public in the same way as it illustrates the warnings by the Bavarian water services. This provides a feedback loop from the officially issued warnings to the EFAS system.

Model components. Within EFAS, hydrological forecasts are generated by cascading an ensemble of meteorological forecasts through a deterministic hydrological model. This section briefly outlines both the models that provide meteorological forcing and the hydrological model LISFLOOD.

1. Meteorological Models: In order to capture some of the uncertainty in the weather predictions, EFAS has been designed to operate with several NWP systems capable of providing the required forcing for the LISFLOOD hydrological model. Currently, EFAS makes use of four NWP products, including two based on the ECMWF Integrated Forecasting System.

2. Hydrological Model LISFLOOD: LISFLOOD is a GIS-based spatially distributed hydrological rainfall-runoff model developed at the JRC for operational flood forecasting at pan-European scale. Driven by meteorological forcing data (precipitation, temperature, potential evapotranspiration, and evaporation rates for open water and bare soil surfaces),

[14] For more information see https://ec.europa.eu/jrc/en
[15] For more information see http://hsaf.meteoam.it

LISFLOOD calculates a complete water balance at a 6-hourly or daily time step and for every grid cell. Processes simulated for each grid cell include snowmelt, soil freezing, surface runoff, infiltration into the soil, preferential flow, redistribution of soil moisture within the soil profile, drainage of water to the groundwater system, groundwater storage and groundwater base flow (Figure 14.)

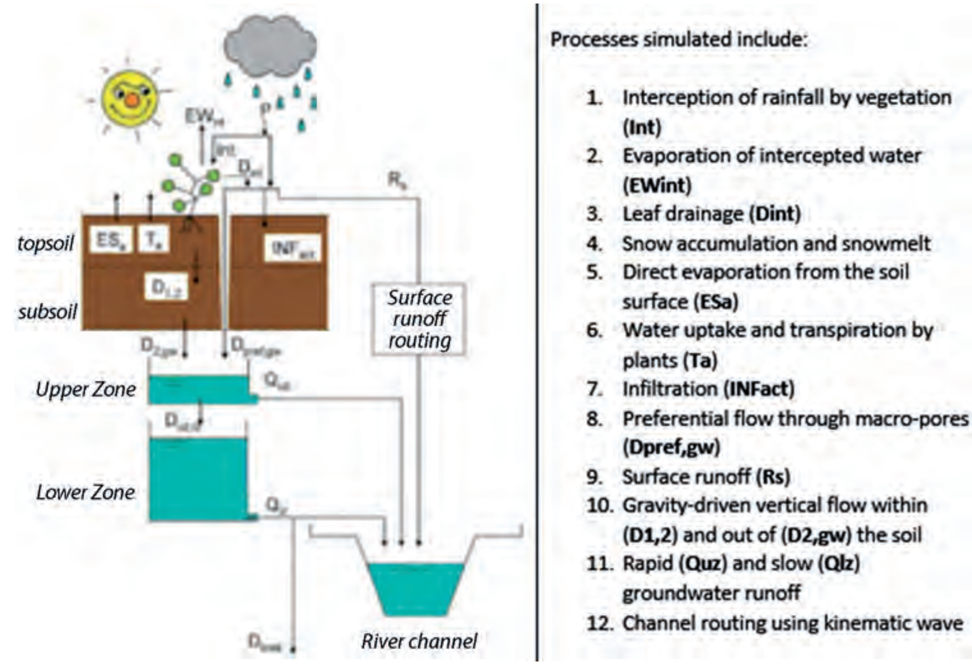

Figure 18. Schematic description of the hydrological LISFLOOD model used in EFAS (www.efas.eu/about-efas.html)

Runoff produced for every grid cell is routed through the river network using a kinematic wave approach. The pan-European set-up of LISFLOOD uses a 5 km grid on a Lambert Azimuthal Equal Area projection. Spatial data are obtained from various European databases with emphasis on having a homogeneous base for all over Europe.

Generation of forecasts. The generation of forecasts is the responsibility of the computational centre. The task can be subdivided into three main components: 1. collating all the necessary forcing and input data; 2. running LISFLOOD; and 3. preparing results for visualisation.

Full details about the EFAS – including model calibration, computer hardware and software, scheduling of execution, forecast products, flood alerts, post-processed forecasts, flash flood alerts, forecast dissemination, case studies and much more – interested students can find at the EFAS website and in numerous papers published about this system (e.g. [15]).

Sava Flood Forecasting and Warning System (Sava FFWS)

With the demise of Yugoslavia, the hydrometeorological services (HMSs) of the six new independent states have taken over the responsibility for execution of most activities of the former Yugoslav Federal Hydrometeorological Institute. However, some important tasks – such as exchange of hydrological and meteorological real-time and non-real-time data and related information, and coordination of hydrological forecasting activities in the new international river basins remained uncovered. This became in particular apparent in the Sava River basin (SRB), which became the international river shared by five independent states – Bosnia and Herzegovina, Croatia, Montenegro, Serbia and Slovenia. The gap was in part offset by the establishment of the International Sava River Basin Commission (ISRBC) in 2005. Yet, the weaknesses persisted in the real-time data exchange and operational flood forecasting domain and inspired a number of initiatives, led by the ISRBC and the national HMSs, to address and resolve the nagging issue.

Disastrous floods that had hit hard Serbia in May 2014 led eventually to the initiation of the World Bank-funded regional project, with the main objective to develop and establish an integrated real-time flood forecasting and warning system for the entire Sava River Basin (the Sava FFWS). The project was awarded to the international consortium led by the Deltares (along with the Royal HaskoningDHV, Eptisa, the Hydro-Engineering Institute of Sarajevo and Mihailo Andjelic), while its implementation started in mid-2016 and was completed by the end of October 2018. In fact, the developed Sava FFWS has just become fully operational in the 5 countries of the region and the ISRBC.

Main features of the Sava FFWS. The Sava FFWS is based on the Delft-FEWS operational forecasting open data and model platform [23]. The platform (Figure 19) is essentially a sophisticated collection of software modules designed for building a hydrological forecasting system that can easily be customised to suit the specific users' requirements.

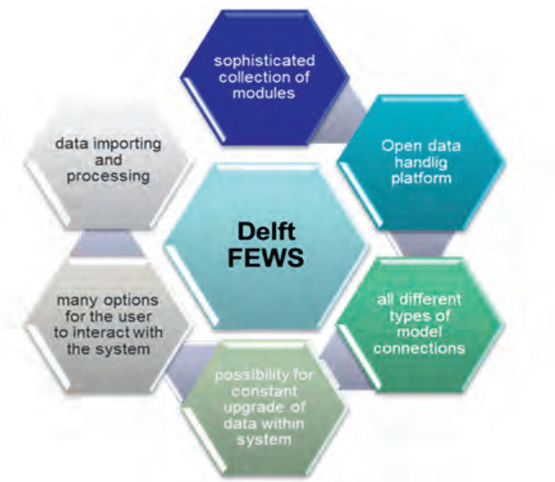

Figure 19. Characteristics of the Delft FEWS open data and model platform [23]

It is a freely available expert software that handles efficiently large amounts of data, integrates various hydrological and/or hydraulic models with real-time observations and the most recent meteorological forecasts, and enables consistent data quality control, standardised work processes, visualisation and reporting. In addition, the Delft-FEWS can orchestrate massive computations – on dedicated hardware and/or in cloud – and allows for remote collaboration between multiple experts and parties working and interacting with the same data. This means that the countries can use independently the models in operational mode, forced with the different meteorological input used by the riparian countries.

In developing the Sava FFWS [12] [21] [10], the Delft-FEWS is used as the backbone for integration of available telemetry data from the network of hydrological and meteorological stations in the region, radar and satellite data, meteorological NWP products, the hydrological and hydraulic models and user interface (Figure 20).

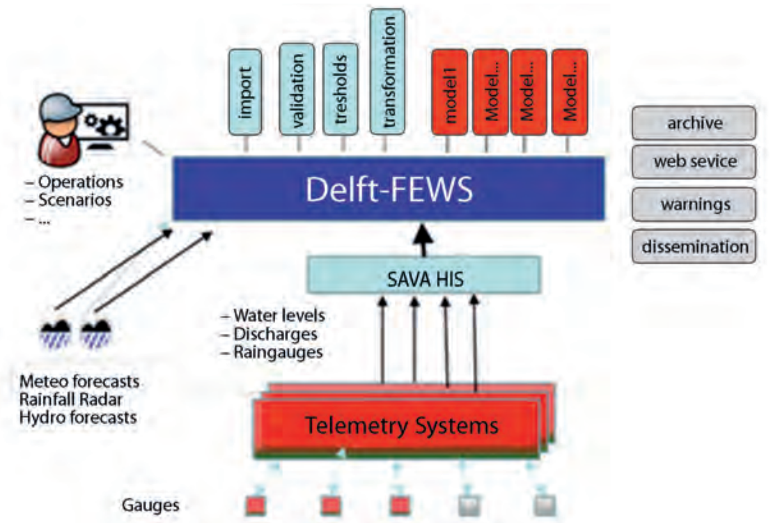

Figure 20. Use of the Delft-FEWS as the backbone for integration of data, models and user interface of the Sava FFWS [12]

The target users of the Sava FFWS are the hydrometeorological and/or water resources management institutions of the riparian states responsible for flood forecasting in their respective parts of the Sava river basin, and they jointly operate and maintain the Sava FFWS.

Real-time data and numerical weather predictions. The Sava FFWS connects to the real-time telemetry data, which are automatically collected in the already existing Sava HIS application hosted by the ISRBC and provides a web service based on the WaterML2 protocol. The telemetry data consist of water levels and discharges at 345 fluvial gauges as well as precipitation, air temperature and snow depths at 257 meteorological gauges in the basin. Automatic validations on doubtful or unreliable measurements are executed, based on set criteria (e.g. exceeding of validation limits, same readings or too high rates of changes). Thresholds based on operational warning levels have been implemented to visualise warnings. Meteorological forecast input is derived from various Numerical Weather Prediction (NWP) models that provide up to 5-day and 10-day forecasts of precipitation, temperatures, snow information, soil moisture, etc. The NWPs include Aladin, NMMB, WRF and ECMWF deterministic models, next to the ECMWF ensemble forecasts for the whole basin. The NWP products used in the Sava FFWS and their characteristics are listed in Table 2. The relevant meteorological information is transformed to catchment average data that is used as input to the hydrological models. Various aggregations have been realised to assist the duty forecasters in interpreting the current and the expected weather situation. To this effect, precipitation radar and satellite images have been implemented, but due to lack of radar and satellite images in the whole Sava basin, they are not currently used as model input, and can readily be included once they become available.

Table 2. NWP products used in the Sava FFWS [12]

Model/source	Spatial resolution (km)	Temporal resolution (h)	Forecasting period (days)	Updated every… (h)
ECMWF	8–10	1	10	12
ECMWF EPS	16–20	1	10	12
Aladin	4.5	1	3	6
Aladin HR	4	1	3	6
NMMB	3–4	3	3	12
WRF SRB	4–6	3	3	12
WRF BiH	2.5	1	4	24
WRF MNE	1	1	5	12
WRF MNE	3	3	5	24

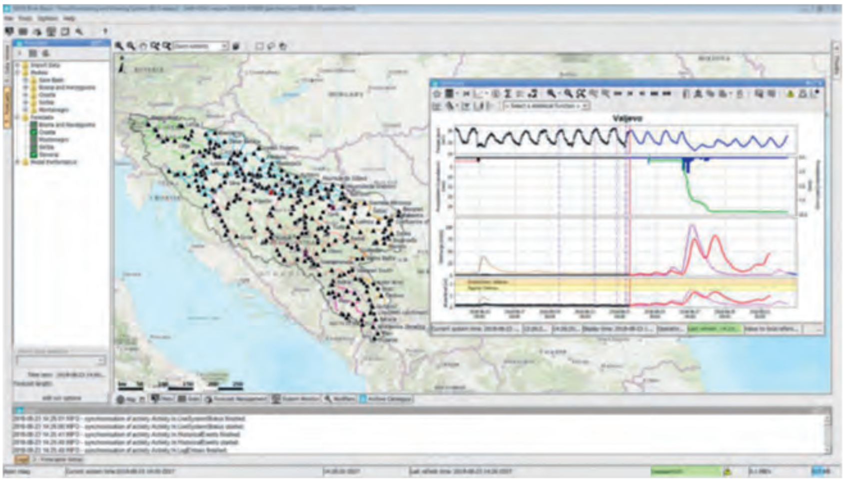

Figure 21. Illustration of the Sava FFWS user friendly interface – given for the hydrological profile Valjevo as an example [10],[12],[21]. It presents GIS map of the Sava River basin, detailed hydrographs, precipitation, temperatures as well as observed and forecasted discharges, water levels and warning status for each forecasting location

Hydrological and hydraulic simulation models. A large number of hydrological, hydraulic models have been implemented within the Sava FFWS. Most of them are based upon the existing models available in the beneficiary countries. Where applicable, these have been adapted and made compatible with the Delft-FEWS operational forecasting system. The detailed hydraulic models are coupled with the most suitable hydrological model, and the results of both are stored within the system. Within the Sava FFWS, distinction can be made between default runs, which are scheduled to run automatically at certain predefined time intervals, and the manual user runs, which can only be run manually.

Apart from the models given in Table 3, a framework of the distributed grid-based Wflow hydrological model has been implemented for the beneficiaries in Bosnia and Herzegovina, Serbia and Montenegro, and a separate Wflow model has been setup for Montenegro.

Table 3. Hydrological and hydraulic models in the Sava FFWS and the forecast workflows with NWP [12]

NWP			Default Run				Manual User Run			
Models	**Hydrological**	**Hydraulic**	**Aladin SI+ ECMWF**	**Aladin HR +ECMWF**	**ECMWF EPS**	**NMMB**	**WRF BiH**	**WRF MNE 1km**	**WRF MNE 3km**	**WRF SRB**
Basin BA/RS/ME	HEC-HMS Sava	HEC-RAS Sava	X		X	X			X	X
	WFlow (BA/RS/MNE)		X		X	X			X	X
Local	Mike-NAM (HR)	Mike 11 Croatia		X						
	Mike-NAM Una (BA)	Mike 11 Una		X						
	HBV-light Bosna (BA)		X		X	X	X		X	X
	HEC-HMS Sava	HEC-RAS Bosna (ISRBC)	X		X	X			X	X
	HEC-HMS Sava	HEC-RAS (BA) (9)	X		X	X			X	X
	Mike-NAM Vrbas	Mike 11 Vrbas	X		X	X	X		X	X
	WflowMNE		X		X	X		X	X	X
	HEC-HMS Kolubara (SRB)	HEC-RAS Kolubara (SRB) [17]	X		X	X			X	X
	"HBV (SRB) (5) Jadar, Kolubara, Tamnava,Ub, Ljig"		X		X	X			X	X

Data assimilation and predictive uncertainty. Within the Sava FFWS, the Asynchronous Ensemble Kalman Filter (AEnKF) algorithm is used to update the initial conditions for the WFlow hydrologic model to fit the observed discharges over the last couple of days thus improving the forecasts. For the HEC-HMS Sava model with the ECMWF Ensemble forecasts, a statistical post-processing approach [22] is used to arrive at a confidence band around the ARMA corrected streamflow ensemble traces.

Performance indicators. Sava FFWS calculates daily the performance of the NWP and hydraulic and hydrological models for preconfigured lead times by comparing stored forecasts with later obtained observed data. Performance is expressed with two indicators; the absolute bias and the root-mean-squared-error (RMSE). The RMSE gives a good indication of overall performance, while the bias also shows whether the error is a result of over- or underestimation. For each lead time, the performance is assessed over all forecasts available for a configured period of time. The results of the performance assessment can be used in a later stage to decide on the operational use and further improvement of the existing NWP, hydrological and hydraulic models.

Further details about the Sava FFWS interested readers can find at the ISRBC and national forecasting organisations of the 5 Sava riparian states.

Hungarian Hydrological Forecasting Service (HHFS)[16]

The Hungarian Hydrological Forecasting Service was founded on 1 March 1892. Starting from 1929, it operated within the Institute of Hydrology. In 1952 the Research Institute for Water Resources Development (VITUKI) was established, and the HHFS operated within this research institute until it was closed in 2012. From the 1st of August 2012, the HHFS operates in the General Directorate of Water Management (OVF). The main activities of the HHFS are:

- collection and delivery of observed hydrological data
- hydrological forecasting
- collection, processing and delivery of information on the shallow river sections (due to low water levels) of importance for navigation
- collection, processing and delivery of meteorological observations and forecasts
- collection, processing and delivery of data on snow depth and ice phenomena on rivers

Hydrological Forecasting System. Currently, the HHFS is using operationally a system known as OLSER (the Hydrological Simulation and Forecasting Model System) illustrated in Figure 22. Its development (under different names) started as far back as the beginning of the 1980s. As a result, a coupled, deterministic-stochastic model started its operative life at the HHFS in 1983 [17] and continued to be further improved and expanded in the subsequent years and decades (e.g. [9]).

Owing to decades of continuous development and upgrading, the Hungarian forecasting system has grown into a mature and complex tool containing snow accumulation and snowmelt, soil frost, effective rainfall, runoff, flood routing and backwater effect modules, extended with statistical error correction modules (Figure 18).

16 For more information see www.hydroinfo.hu/en/

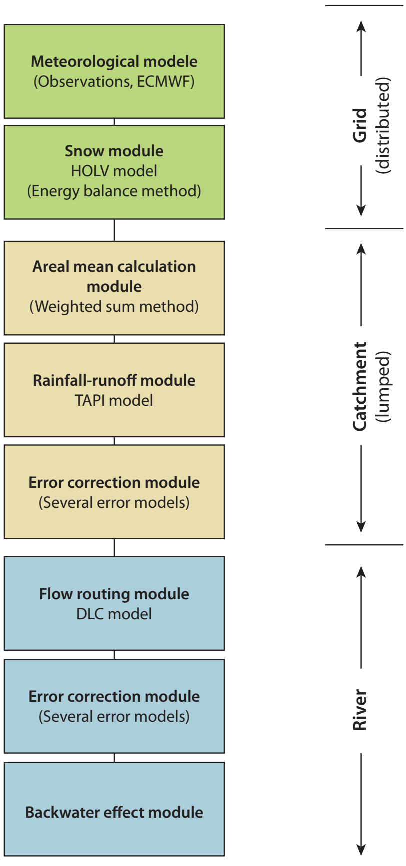

Figure 22. Modules of the OLSER Forecasting System [8]

Key functions of the OLSER modules are as follows:

- the meteorological module takes into account meteorological observations and forecasts on a 0.1 x 0.1 degree resolution grid
- the snow module is based on the HOLV model performing snow package calculations for 33 sub-catchments on a 0.1 x 0.1 degree grid over the entire forecast domain and handles all snow-related processes
- the areal mean calculation module producing spatial averages which serve as meteorological input for all sub-catchments
- the rainfall-runoff module is based on the TAPI rainfall-runoff model using Antecedent Precipitation Index (API) for its calculations
- the flow routing module is using the Discrete Linear Cascade Model (DLCM)
- the error correction and predictive uncertainty module contains special Kalman filter algorithms developed to consider for example the patterns in hydropower plants operation
- the backwater effect module handles the interaction on a tributary flow and the receiving river

The forecasting system is in daily operation and produces every morning 6-day lead time water level forecasts for the Danube, Tisza and Drava rivers, and their tributaries. Normally, it is run daily, but in emergency, i.e. during floods, it is run more frequently. The forecasts are published and provided to the public and all the users concerned with flood risk and water resources management.

Further details on the forecasting system used in Hungary is available from the HHFS website and the cited papers.

FFWS: New trends and possible future developments

Flood forecasting and warning systems have been shown to reduce impacts and save lives, and these systems are becoming an increasingly important tool for flood risk and water managers, and emergency response services.

The distinct advantage the hydrologists have nowadays over their predecessors is the advancement of science and technology. Better scientific understanding of physical phenomena and processes allows us to produce better and more realistic models, improve measurements of hydrological and meteorological variables, and improve prediction of model inputs. Dramatic advancements in affordable technology have made the application of modern hydro and meteo science and hydraulics to flood forecasting possible mainly due to the following three factors: 1. The availability of very fast computers, with significantly more memory and data storage capability than was available even just 10 years ago; 2. Widespread availability of high-resolution GIS-based data sets from remote-sensing sites, which are needed for model parameter estimation and calibration; and 3. Highly reliable telecommunications systems for data transmission from ground-based and satellite data collection platforms (DCPs). In addition, the expansion of the Internet and the proliferation of mobile phones have had a dramatic impact on the distribution of flood warnings.

Looking into the future, there is no doubt that further advancements in affordable technology would provide ample opportunities for improvement and expansion of the existing flood forecasting and warning systems in the world as well as in the Danube basin.

What is more, there is the fast-growing development of global and continental-scale flood monitoring, modelling and prediction systems. Such is, for example, the European Flood Awareness System (EFAS) shortly reviewed above. EFAS represents an operational continental-scale flood forecasting system, which became operational in 2012 with several European organisations having responsibility for producing and providing the flood information to EU and other European countries. EFAS was developed as a complementary system to the existing national and regional flood forecasting systems in European countries, and proved to be of great benefit to the national agencies and hydrological forecasting services.

Similarly, the Global Flood Awareness System (GloFAS),[17] which was developed jointly by the European Commission and the European Centre for Medium-Range Weather Forecasts (ECMWF), provides countries with information on upstream river conditions as well as continental and global overviews. GloFAS couples output from NWP model ensembles from the ECMWF Ensemble Prediction System with a hydrological model covering continental domains. In the meantime, the Global Flood Awareness System has been upgraded to version 2.0. It consists of a number of improvements to the hydrological forecasting chain. Version 2.0 follows the migration of the GloFAS prototype to the ECMWF operational environment as a 24/7 service, and includes the following major features:

- version numbering system for the GloFAS cycles
- calibrated LISFLOOD routing component
- updated reference discharge climatology
- improved initialisation of the real-time forecasts
- available GloFAS datasets

Capitalising on ever-increasing scientific understanding of physical phenomena and processes coupled with new advances in technology, it is not difficult to foresee that the global hydrological models will further be expanded and refined in the future, in parallel with development and improvement of the local, national and regional flood forecasting and warning systems.

A bright future of the FFWSs is more than welcome as counterweight to a rather gloomy picture of the world succumbing to numerous negative impacts of climate change, including the increase in intensity and frequency of disastrous floods.

Concluding notes

In the hope that these notes may prove beneficial to the prospective students, it is very important to recognise that a flood forecasting and warning system is a complex, live structure operating in real-time. As such, it is only as good as its weakest component, i.e., each component of the system is crucial for the system to fulfil its mission.

As an illustration, consider the real place of model development in building up an effective operational forecasting system. Model development and usage represent just a small fraction of the costs of establishing and running an operational hydrological forecasting system. Yet, the system is worthless without models; they play the same role as the heart in the human body. Small, no doubt, but one cannot exist without it.

At the very end, let us recall a few proverbial Murphy-inspired Laws of Hydrological Forecasting:

The flood always hits on Sunday at 2 a.m. when there is nobody in the forecasting centre.

[17] For more information see www.globalfloods.eu/

If the above Law does not apply, then the flood comes when the staff is windsurfing on the nearby lake.

If one is lucky, one meets only once in a lifetime the flood that is greater than the design flood.

If one is unlucky this happens regularly.

The 100-year-flood returns every ten years minimum twice.

When the Big Flood comes the online data collection system fails within minutes.

When the Big Flood comes all our precious hardware breaks down in maximum T hours, where T is one fifth of the concentration time of the catchment.

The probability of the joint occurrence of unfixable computer bugs in the code of our forecasting model and the Big Flood is equal to one.

A decent forecast of a flood peak specifies either the flood peak value or the time of occurrence, never both!

List of acronyms used

EUSDR: European Union Strategy for the Danube Region
EUSDR PA 5: European Union Strategy for the Danube River – Priority Area 5
DR: Danube Region
DFRMP: Danube Flood Risk Management Plan
DSPF: Danube Strategic Project Fund
FFWS: Flood Forecasting and Warning System
NWP: Numerical Weather Prediction
ECMWF: European Centre for Medium-Range Weather Forecast
QPF: Quantitative Precipitation Forecast
Sava FFWS: Sava Flood Forecasting and Warning System
ISRBC: International Sava River Basin Commission (ISRBC)
SRB: Sava River Basin
HMS: Hydrometeorological Service
DCP: Data Collection Platform

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Paul Ray Richard

Soil Mechanics of Flood Control

Introduction

Soil mechanics plays and important role in the design and performance of flood control systems. Flood protection levees are often embankments made entirely from locally available soil. Other structural elements such as control structures, diversions and monitoring stations must have a resilient foundation. The objective of this learning module is to summarise soil behaviour concepts, including soil stability, groundwater seepage and interaction with structures. The first section is a very brief review of basic soil mechanics applied to flood control. The sections that follow address seepage through and under embankments and slope stability of embankments.

From the perspective of a non-expert, the subject of soil mechanics may seem unimportant, or merely a pedestrian exercise. However, when viewed from a failure perspective (Figure 1), its importance should be quite clear. The two largest categories are “Quality Problems” and “Overtopping”. The Quality Problems category is broken down into the sub-categories shown; mostly dealing with soil. Overtopping can also be worsened by soil problems where the soil at the crest erodes and generates much more flow, and possibly failure of the entire embankment. The topics of this learning module should be obvious, given the main causes of failure: Piping (seepage) and Sliding (slope stability).

Case studies

Case studies of field performance (both successes and failures) are critical to understanding how embankments perform. Monitoring projects during construction, then during operation give insight into the behaviour of the levee or hydraulic control system, not just a single component or material. This is often overlooked by designers (and professors) who are more often focused on specific aspects of a design or performance of a particular material. Full-scale monitoring is very difficult. It is expensive, time-consuming and often boring. The monitoring program is at the mercy of nature[1] who rarely cooperates. Forensic studies of failures are also useful since the engineer knows that the system has definitely failed. By back-calculating stability or seepage analyses, one may gain

[1] I recall two such projects where funding lasted for three years. For both projects, the region experienced drought conditions for the entire duration of the research. It is indeed difficult to gather flood control data when there is no water. Three years later, one of the project locations experienced a 100-year event, destroying much of the instrumentation. We could only run forensics on the damage we found.

insight into the actual (versus predicted or designed) performance. Lessons from failures constitute a large percentage of geotechnical knowledge gained in the field.

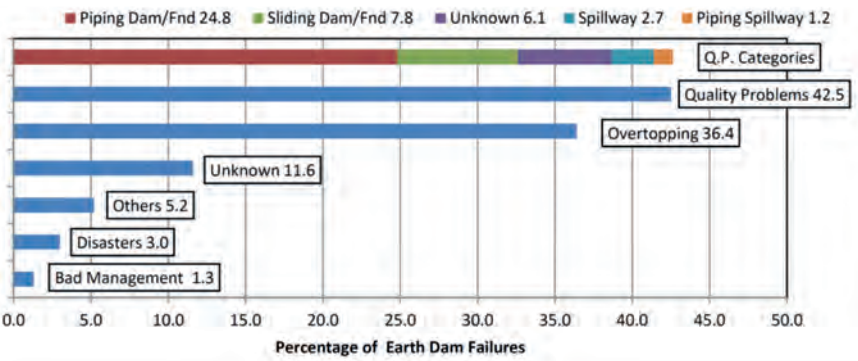


Figure 1. Causes of earth dam failures from a database of 591 studies (top bar is subdivided into specific categories) [26]

Case Study – Overtopping

Tous Dam, Spain was a 70 m high rockfill dam with a central clay core located near Valencia, Spain, failed due to overtopping [7,17]. It was designed and built as a flood defence structure and was also used for flow regulation and irrigation. Construction started in 1958 as a concrete dam 80 m tall but geotechnical conditions forced a stoppage in 1964. The design was modified and resumed in 1974 where the central embankment now consisted of a clay core with rockfill cover and finished in 1978. Final dimensions were 70-m tall and 400-m crest length. The emergency spillway used radial gates with a capacity of 7,000 m^3/s while the service spillway had a capacity of 250 m^3/s. During 19–20 October 1982, very heavy rain fell in the Jùcar basin upstream from the Tous Dam. The heaviest rain was recorded in the Cofrentes area, about 25 km northwest of Tous Dam with a total greater than 550 mm and 285 mm falling in only 3 hours. The estimated inflow was 5,000 m^3/s requiring the spillway gates to be opened. Unfortunately, the electrical grid was out of order due to the weather conditions and emergency generators could not be started. Efforts to raise the gates manually were fruitless. The overtopping started at 17:00 with water breaching 1.10 m over the main crest about at 19:15 p.m. So, about 16 hours after trying to open the flood gates, the dam was overtopped, and it washed out within 1 hour by erosion of the central rock-fill. After such an extraordinary flood, in the downstream basin 8 people lost their lives and about 100,000 people had to be evacuated. The damages were estimated to reach $400 million, even if part of these damages were likely to be caused by the floods before the arrival of the break wave (Figure 2). A new Tous Dam was built on the same site and part of the clayey core material, which had shown a relatively high resistance to water flow, was reused for constructing the new dam.

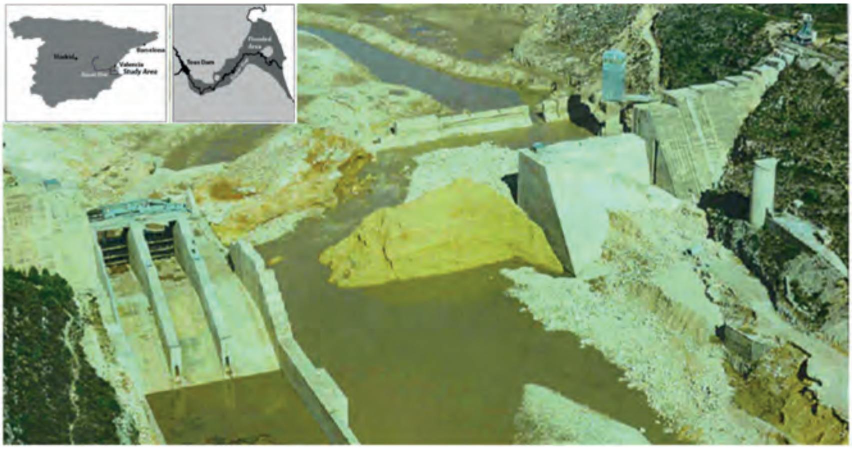


Figure 2. Tous Dam near Valencia, Spain after failure. Note the concrete abutments on both sides and washed-out clay core (yellow material) in the centre [5]

Case Study – Seepage piping failure

Teton Dam, Idaho, USA. Teton Dam, a 93-m-high earthfill dam across the Teton River in Madison County, southeast Idaho, failed completely and released the contents of its reservoir at 11:57 a.m. on 5 June 1976 [3]. Failure was initiated by a large leak near the right (northwest) abutment of the dam, about 40 metres below the crest. The dam, designed by the U.S. Bureau of Reclamation, failed just as it was being completed and filled for the first time. Eyewitnesses noticed the first major leak between 7:30 and 8:00 a.m. 5 June, although two days earlier engineers at the dam observed small springs in the right abutment downstream from the toe of the dam. The main leak was flowing about 0.5–0.8 m^3/s from the rock in the right abutment near the toe of the dam and above the abutment-embankment contact. The flow increased to 1–1.4 m^3/s by 9 a.m. At about the same time, 0.05 m^3/s seepage issued from the rock in the right abutment, approximately 40 metres below the crest of the dam at the abutment-embankment contact.

Between 9:30 and 10 a.m., a wet spot developed on the downstream face of the dam, 5–6 m out from the right abutment at about the same elevation as the seepage coming from the right abutment rock. This wet spot developed rapidly into seepage, and material soon began to slough, and erosion proceeded back into the dam embankment. The water quantity increased continually as the hole grew. Efforts to fill the increasing hole in the embankment were futile during the following 2 to 2 1/2-hour period until failure. The sheriff of Fremont County (St. Anthony, Idaho) said that his office was officially warned of the pending collapse of the dam at 10:43 a.m. on 5 June. The sheriff of Madison County, Rexburg, Idaho, was not notified until 10:50 a.m. on 5 June. He said that he did not immediately accept the warning as valid but concluded that while the matter was not too serious, he should begin telephoning people he knew who lived in the potential flood path.

The dam breached at 11:57 a.m. when the crest of the embankment fell into the enlarging hole and a wall of water surged through the opening. By 8 p.m. the flow of water through the breach had nearly stabilised. Downstream the channel was filled at least to a depth of 9 m for a long distance. About 40% of the dam embankment was lost, and the powerhouse and warehouse structure were submerged completely in debris.

Figure 3. Teton Dam showing progressive piping up to breach [24]

Case Study – Seepage along outlet works

Lawn Lake Dam, Colorado, USA. The dam was in Rocky Mountain National Park upstream of Estes Park, Colorado. It was an embankment dam constructed in 1903 and owned by an irrigation company. It fell within the National Park boundary when the Park was established in 1915. The reservoir was at almost 3,350 m elevation and the dam enlarged a natural glacially formed lake. The dam was raised in 1931 to 7.5 m high and stored a maximum of 1.5 million cubic metres of water [13]. A 1-m diameter, riveted steel outlet pipe was used for releases. A direct-buried gate valve was in this pipe directly under the crest of the dam. The dam was assigned a "moderate" downstream hazard potential. Due to its remote location with challenging access, inspections of the facility were relatively infrequent. Several issues were identified at the dam documented in inspection reports in 1951, 1975, 1977 and 1978.

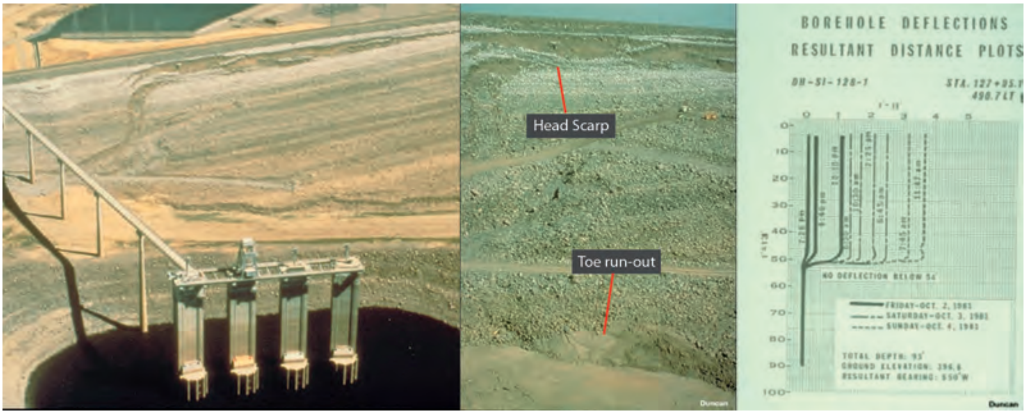
Between 5:00 and 6:00 a.m. on 15 July 1982, the dam failed suddenly, releasing 1.2 million cubic metres. There was no warning (Figure 4a). The peak flow was approximately 550 m^3/s. The flood wave changed as it went downstream due to the changing topography and the presence of a downstream dam. From the dam, the flood charged down the steep channel of Roaring River. It eroded areas up to 15 m deep. After dropping 760 vertical metres over 7 km, the flood poured out into Horseshoe Park – a relatively flat basin. There it dropped its load of boulders and debris and created an alluvial fan of over 16 hectares. The flood went out the east end of Horseshoe Park, filled and then overtopped a 5-m-high concrete dam called Cascade Dam. The maximum overtopping was 1.2 metres. After 17 minutes of overtopping, Cascade Dam gave way and a new flood surge of 450 m^3/s poured through the breach. In the town of Estes Park, debris-laden, muddy water up to 1.5 m deep (170 m^3/s) poured through the business district. It damaged 177 businesses (over 90% of the businesses). Damages totalled $31 million and a total of three lives were lost. The State Engineer performed an investigation and issued a report 8 months following the failure. The report concluded that "…the failure occurred due to leakage under high pressure from the leaded connection of the outlet pipe and valve, causing progressive piping of the dam embankment in the vicinity of the outlet pipe during periods of high reservoir levels and gate closure and sudden collapse of the embankment allowing rapid evacuation of the reservoir".

(a) *(b)* *(c)*

Figure 4. Lawn lake a) embankment after failure; b) gate valve improperly installed; c) recovered gate valve 70 m downstream [14]

Case Study – Slope stability

San Luis Dam, California, USA. The 76-m-high San Luis Dam, about 140 km southeast of San Francisco, California, stores water on the California Aqueduct System. These photos (Figure 5a) show a slide that occurred in the upstream slope of the dam in September 1981 [2], as water was being withdrawn from the reservoir. The slide extended for about 330 metres along the embankment. At the north end, near the inlet-outlet structure visible in this photo, the scarp at the top of the slide was about 9 metres high. At the bottom of the slope the toe of the slide moved horizontally about 9 metres out into the reservoir. The head scarp and toe bulge are more clearly visible in Figure 5b. Temporary roads have been cut into the slope to provide access for drill rigs which were used to retrieve samples for testing and to install "slope indicators" that measure movements underground.



(a) (b) (c)

Figure 5. San Luis Dam showing a) upstream slope failure due to rapid drawdown; b) closer view of head scarp; c) slope indicator output showing the sharp deflections where failure surface runs [2]

The plot of data from a slope indicator (Figure 5c) shows a distinct rupture surface at a depth of 52 feet (16 m). At this depth the soil is highly plastic clay called “slope wash”, on which the embankment was constructed. In its dry condition the slope wash was nearly as hard as a brick. However, the tests showed that the slope wash became very weak when wetted contributing to the principal cause of the slide.

Case Study – Liquefaction

Lower San Fernando Dam, California, USA. The upstream slope of the Lower San Fernando Dam, in California, failed due to liquefaction during the 1971 San Fernando earthquake. The dam was constructed by “hydraulic filling”, which involves mixing the fill soil with a large amount of water, transporting it to the dam site by pipeline, depositing the soil and water on the embankment in stages, and allowing the excess water to drain away. The fill that remains is loose and is subject to liquefaction as the result of earthquake shaking. About 1 m of freeboard remained after the upstream shell slid into the reservoir (Figure 6a). The paved crest of the dam can be seen descending into the water at the top of this photo. Fortunately, the intake structure was undergoing repairs, requiring a reduced level in the reservoir. The slide in the upstream shell is shown in Figure 6b with the reservoir emptied. The paved road surface identifies the former crest of the dam. With every case study comes new information and insight, enabling engineers to avoid mistakes and produce better and safer designs. The interested reader is invited to visit the website http://damsafety.org/ and http://damfailures.org/.

(a) (b)

Figure 6. Lower San Fernando Dam, upstream face showing liquefaction slide due to earthquake (M 7.1) shaking, a) immediately following the earthquake; b) after the reservoir was drawn down [17]

Considering failure modes in design

Part of the design process is to envision the types of failure scenarios that may occur. Figure 7 illustrates some scenarios that may occur and require consideration.

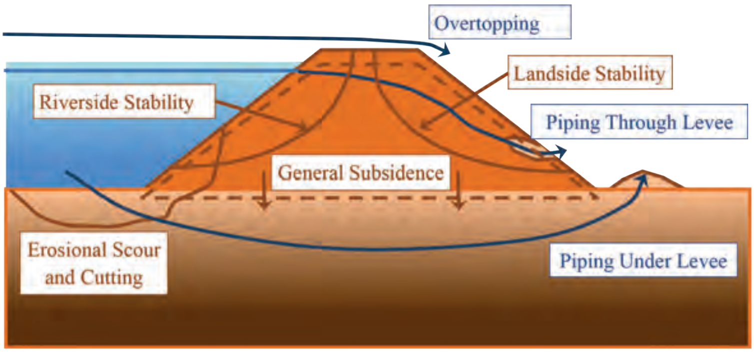


Figure 7. Some failure modes for levees and dams (compiled by the author)

While overtopping is perhaps more in the area of flow prediction and reservoir sizing, there are geotechnical aspects as well. Some embankments are meant to be overtopped as emergency spillway. The critical soil property is then resistance to erosion so that the overtopping water will not cut a deep channel through the embankment. Other conditions where erosion resistance is important occur when flood waters try to scour away the toe of a riverside slope. This may be due to wave or current action, or both. Scouring may occur under (concrete) foundations of hydraulic control structures as well.

One of the most critical tasks in levee design is concerned with water seepage through and under the levee during flood events. Seepage under or through the levee may go undetected until it is too late. It is also complicated by the fact that soil hydraulic conductivity can vary by 12 orders of magnitude. Such a wide variation means that thin, undetected soil layers may control where and how much seepage occurs.

Stability of the levee is controlled by the strength of the embankment materials and the soils beneath it [16]. Unfortunately, embankment materials that perform well at blocking seepage are not very strong. Conversely, soils that provide good slope stability are poor seepage barriers. This is the primary reason for zoned embankment dam design. Each zone in the dam performs a different function, and by working together, they achieve the necessary stability and seepage blocking requirements.

General subsidence (settlement) may occur when the embankment soils, or soft foundation soils, compress or consolidate over time. This leads to reduced crest height in the dam or levee. Differential settlement may cause cracking in the embankment or functional loss of control gates or other mechanical features that require precise dimensional tolerances.

Connecting failure modes, methods of analysis, required data

Based on the possible failure scenarios above, one must consider the methods to evaluate their likelihood as well as the data necessary to perform meaningful analyses. Table 1 shows the pertinent soil properties, laboratory and field tests, and common analyses required in order to assess the level of safety for a levee.

This table is by no means exhaustive but is meant to demonstrate the relationships between different possible failure scenarios and the methods to evaluate them. Not shown in the table are the field and laboratory testing that is performed to better define the extents of different soil layers throughout the site as well as other index and classification tests used to confirm that soil in one location is indeed the same (or not) as soil in another location.

Table 1. Failure modes, soil properties, tests and analyses to evaluate the possibility of failure (compiled by the author)

Failure Mode	Required Soil Properties	Field Tests	Laboratory Tests	Analysis
Overtopping Toe Erosion and Scour	Erodability Dispersivity Soil strength	Jet Erosion	Erosion Function Apparatus Clay Dispersion	SRICOS HEC-18 EUROSEM Infinite slope analysis
Seepage and Piping	Hydraulic Conductivity SWCC (unsat.) Dispersivity	Well tests CPT injection Tensiometers	Constant Head Constant Flux Pressure Plate Clay Dispersion	Groundwater Flow, Unsaturated Flow
Slope Stability	Soil Strength (cohesion, phi)	CPT Dilatometer Sample Boring	Triaxial Strength Direct/Simple Shear Proctor Compaction	Slope Stability FEM Displacements
Embankment Subsidence	Soil Compressibility	CPT Dilatometer Sample Borings	Consolidation Creep Proctor Compaction	Hand Computation of Settlement FEM Displacements

Note: Classification, Grain Size, Atterberg Limits and other index tests would be part of all of these assessments

Principal design cross sections

Based on the concept that soils can rarely perform both seepage and stability functions well, typical cross sections for dams and levees have evolved to zoned earth dams. A purely homogeneous embankment will allow seepage water to exit the landside face and create piping and stability problems there. So, even rudimentary dams have a drainage system to direct seepage out of the embankment in a controlled manner (Figure 8a, b). Dams with a core material (typically clay) greatly reduce seepage volumes but require transition zones to help keep the core in place and provide more dependable stability.

Additional challenges with flood levees

Some obvious problems occur when engineers try to build a levee system alongside a meandering river such as the Mississippi River in the U.S. or parts of the Danube and Tisza in Hungary [1] [10] [22]. While the terrain is flat, the underlying Holocene geology is very heterogeneous and complex. A typical section may look like the illustration in Figure 9 where old river channels and flood features underlie the present river system.

This also means it underlies the levee system, as well and will connect seepage sources to different places behind the levee, defying any two-dimensional approach. There may be deposits that are moderately deep beneath the levee that later reach the surface over 300 m from the levee embankment.

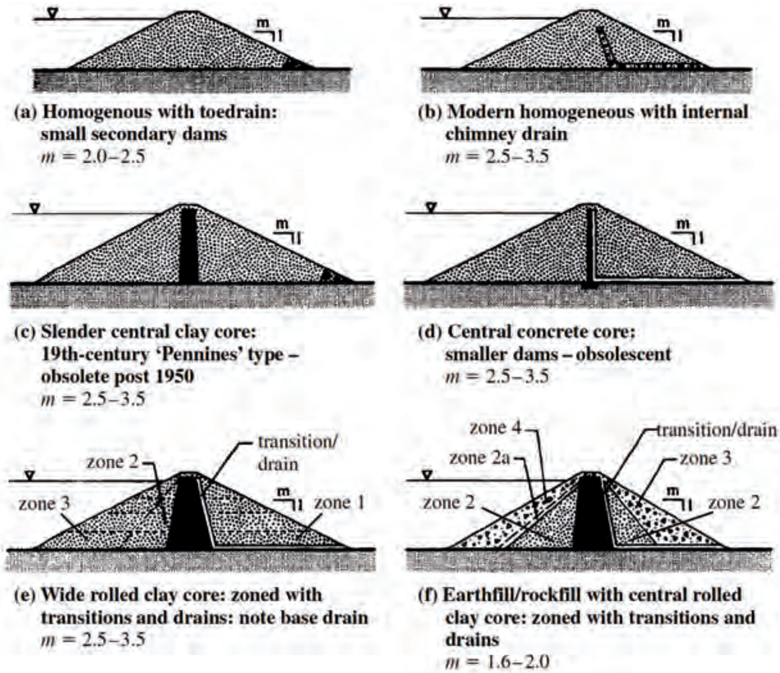


Figure 8. Typical dam or levee cross-sections, simplest (a) to most complex (f) [22]

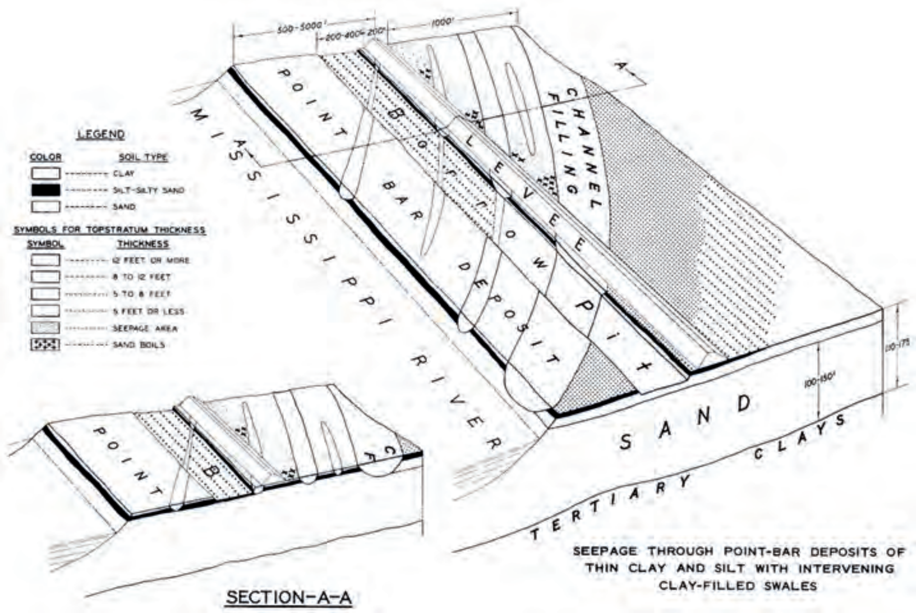


Figure 9. Holocene deposits along meandering river channel [20]

Basic soil mechanics

Description and classification of soils

Soil is defined for engineering purposes as a natural aggregate of mineral grains separable by gentle mechanical means, e.g., agitation in water. Rock in contrast, is a natural aggregate of minerals connected by strong and permanent cohesive bonds. The boundary between soil and rock is to some degree arbitrary, as exemplified by soft or weathered rocks, e.g. weathered limestones and shales, or weakly cemented sandstones. All engineering soils of non-organic origin (i.e. excluding peats, etc.) are formed by rock weathering and degradation processes. These may occur in situ forming residual soils. Alternatively, if the rock particles are removed and deposited elsewhere by natural agents, e.g., glaciation or fluvial action, they will form transported soils. Soft or weathered rocks form part of the range of residual soils. Transportation results in progressive changes in the size and shape of mineral particles and a degree of sorting, with the finest particles being carried furthest.

All engineering soils are particulate in nature, and this is reflected in their behaviour. An important distinction must be drawn between two generic inorganic soil groups which result from different weathering processes [15]. The larger, more regularly shaped mineral particles which make up silts, sands and gravels are formed from the breakdown of relatively stable rocks by purely physical processes, e.g. erosion by water or glacier, or disintegration by freeze–thaw action. Certain rock minerals are chemically less stable, e.g. feldspar, and undergo changes in their mineral form during weathering, ultimately producing colloidal-sized 'two-dimensional' clay mineral platelets.

These form clay particles, the high specific surface and hence surface energy of which are manifested in a strong affinity for water and are responsible for the properties which particularly characterise clay soils, i.e. cohesion, plasticity and susceptibility to volume change with variation in water content. Differences in platelet mineralogy mean that clay particles of similar size may behave differently when in contact with water, and hence differ significantly in their engineering characteristics. Soil particles vary in size from over 100 mm (cobbles) down through gravels, sands and silts to clays of less than 0.002 mm size. Naturally occurring soils commonly contain mixtures of particle sizes but are named according to the particle type that controls its general behaviour. Thus, a clay soil is so named because it exhibits the plasticity and cohesion associated with clay-mineral-based particles, but the mineral matrix invariably contains a range of particle sizes, and only a minor proportion of the fine material in the matrix may be clay sized, i.e. < 0.002 mm (2 μm) as shown in Figure 9. One system (Unified Soil Classification System) used for defining and classifying the particle size ranges for soils is provided in Figure 11.

The divisions between the named soil types correspond broadly to significant and identifiable changes in engineering characteristics. Particle size analysis is therefore employed for primary classification, to distinguish between gravels, sands and fine-grained silts and clays [6]. However, particle size analysis is insufficient for the complete classification of fine-grained soils or coarser soils where the matrix includes a proportion of plastic fines, i.e. clays (e.g. Figure 10c–f). Classification by plasticity is then necessary, using limits expressed in terms of percentage water content by mass.

The liquid limit, wL, is the water content defining the change in soil consistency from plastic to liquid; the plastic limit, wP, defines the change-point below which a soil is too dry to exhibit plasticity. The range between wL and wP is plasticity index, IP, with IP = wL – wP. Secondary classification is determined through IP and wL using classification charts.

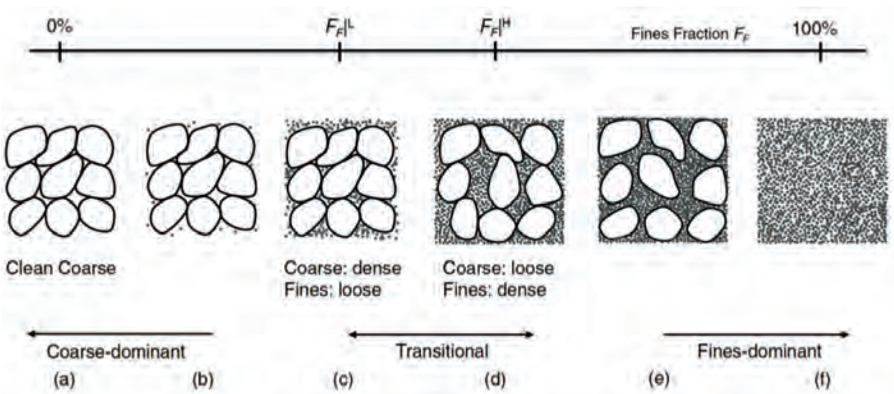


Figure 10. Transition from coarse to fine-grained soil [14]

Note that fine material has dominant effect, even at low weight percentage

The Unified Soil Classification System (Figure 11) divides soils into groups, each of which is denoted by a two-letter symbol. The first letter is the dominant soil constituent, i.e. G, S, M and C for gravels, sands, silts and clays respectively. The second provides descriptive detail based on particle size distribution for coarse soils, e.g. SW = well-graded sand, or on the plasticity of fines.

Criteria for assigning group symbols				**Group Symbol**
Coarse-grained soils More than 50% retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3^c$	**GW**
			$C_u < 4$ and/or $1 > Cc > 3^c$	**GP**
		Gravels with Fines More than 12% fines [a,d]	*PI* < 4 or plots below *"A"* line	**GM**
			PI > 7 and plots on or above *"A"* line	**GC**
	Sands 50% or more of coarse fraction retained on No. 4 sieve	Clean Sands Less than 5% fines[b]	$Cu \geq 6$ and $1 \leq C_c \leq 3^c$	**SW**
			$Cu < 6$ and/or $1 > C_c > 3^c$	**SP**
		Sands with Fines More than 12% fines [b,d]	*PI* < 4 or plots below *"A"* line	**SM**
			PI > 7 and plots on or above *"A"* line	**SC**
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and Clays** Liquid Limit less than 50	Inorganic	*PI* > 7 and plots on or above *"A"* line	**CL**
			Liquid Limit less than 50	**ML**
		Organic	$\frac{\text{Liquid Limit oven dried}}{\text{Liquid Limit not dried}} < 0.75$	**OL**
	Silts and Clays Liquid Limit 50 or more	Inorganic	*PI* > 7 and plots on or above *"A"* line	**CH**
			PI < 4 or plots below *"A"* line	**MH**
		Organic	$\frac{\text{Liquid Limit oven dried}}{\text{Liquid Limit not dried}} < 0.75$	**OH**
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			**Pt**

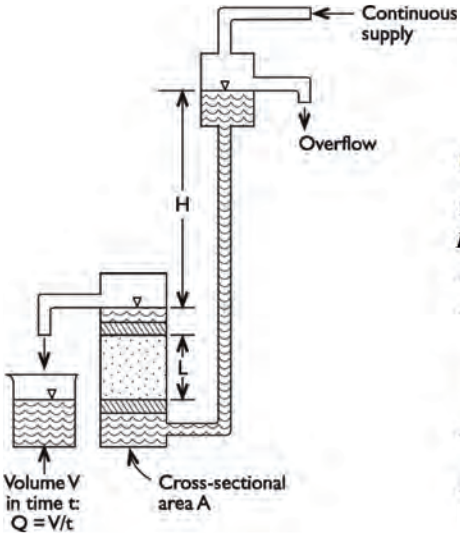
(a) Gravels with 5 to 12% fines require dual symbols GW-GM, GW-GC, GP-GM, GP-GC
(b) Sands with 5 to 12% fines require dual symbols SW-SM, SW-SC, SP-SM, SP-SC
(c) $C_u = \frac{D_{60}}{D_{10}}$ $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$
(d) If $4 \leq PI \leq 7$ and plots in hatched area use dual symbol GC-GM or SC-SM
(e) If $4 \leq PI \leq 7$ and plots in hatched area use dual symbol CL-ML

Figure 11. Classification criteria for soils [6]

Normally, sieve analysis for coarse soil; sieve, hydrometer and liquid/plastic limits for fine grained soils are required. Such tests are routinely performed on a daily basis in a laboratory.

Hydraulic conductivity of soils

One of the key soil performance properties for flood control is hydraulic conductivity. It is a measure of how easily water travels through soil. By simply observing different soils, one may appreciate the vast differences between gravels and clays. The degree of conductivity is related to the size and connectivity of pore spaces within soils. Gravel, with larger pores that are well connected exhibits high conductivity while clay particles often have pores only microns in size have very low conductivity. The numerical property is generally based on the assumption that water flow through soil is independent of the degree of pressure (gradient) being used to push it through. Measuring this value is often done in the field through pumping tests and borehole tests and in the laboratory by controlled head or controlled flow tests. It is easier to discuss measurement using a simple laboratory test called the constant head test, shown in Figure 12.



Darcy's Law States

$$Q = kiA = k\frac{H}{L}A \quad \text{or} \quad k = Q\frac{L}{HA}$$

Q = measured as V / t (m^3/sec)

H = measured as difference in height shown (m)

L = length of specimen (m)

k = hydraulic conductivity (m/sec)

Set H = 30 cm, Sample L = 10 cm, A = 12 cm^2

Measure V = 200 cm^3, t = 120 sec

$$k = \frac{200}{120}\frac{10}{30 \times 12}\left(\frac{cm^3}{sec}\frac{cm}{cm\ cm^2}\right) = 0.046\left(\frac{cm}{sec}\right)$$

$$k = 4.6 \times 10^{-4}\left(m / sec\right)$$

Figure 12. Darcy's Law applied to laboratory constant head test [14]

The example calculation is a reasonable set of numbers for a laboratory test. There are many correlations between soil type and hydraulic conductivity. Table 2 lists one such set. Note the very wide range of values possible and the influence of the clay fraction in sands and gravels (GC, SC). Maintaining some level of quality control in the selection of embankment materials and treatment of foundation soils is very important because even a small variation in soil mixture can change conductivity by 1000x. One might also appreciate the dangers of having a thin layer of dissimilar material that would block (low k) or pipe (high k) seepage through the levee. Since a laboratory specimen is indeed small compared to the site, field tests are often performed to verify conductivity values determined in the laboratory. Another useful function of laboratory tests is to evaluate how soil improvements (compaction, injections, additives) will affect conductivity. Hydraulic conductivity will play a key role in determining the influence of seepage through and under the embankment. This will be addressed in a later section on seepage.

Table 2. Approximate hydraulic conductivity values for different soil types [14]

Description	USCS	min. (m/s)	max. (m/s)
Well graded gravel	GW	5.00E-04	5.00E-02
Poorly graded gravel	GP	5.00E-04	5.00E-02
Silty-sandy gravels	GM	5.00E-08	5.00E-06
Clayey gravels	GC	5.00E-09	5.00E-06
Well graded sands	SW	1.00E-08	1.00E-06
Poorly graded sands	SP	2.55E-05	5.35E-04
Silty sands	SM	1.00E-08	5.00E-06
Clayey sands	SC	5.50E-09	5.50E-06
Inorganic silts	ML	5.00E-09	1.00E-06
Inorganic clays	CL	5.00E-10	5.00E-08
Organic silts	OL	5.00E-09	1.00E-07
Silts of high plasticity	MH	1.00E-10	5.00E-08
Clays of high plasticity	CH	1.00E-10	1.00E-07
Compacted silt	(ML-MH)	7.00E-10	7.00E-08
Compacted clay	(CL-CH)	–	1.00E-09
Organic highly plastic clays	OH	5.00E-10	1.00E-07
Peat/highly organic soils	Pt	–	–

Soil strength, compressibility and stability

In order to remain stable, a dam or levee must have material that resists sliding and does not consolidate or compress too much under its own weight. Sliding stability is related to shear strength of soils while compressibility is a function of the bulk stiffness of the soil. The most common tests to determine shear strength are triaxial tests. A cylindrical specimen is subjected to confining pressure (simulating burial at a particular depth) then vertical load is applied until failure. This test is often repeated on several similar samples using progressively higher confining pressure. Once this is completed, an estimate of strength properties, based on cohesion (c) and friction angle (ϕ), can be deduced (Figure 13). Typical values for strength for different soils are shown in Table 3.

Table 3. Typical strength values for soils [6]

Clays–Description	Strength	Sands–Description		Cohesion–Friction
Hard soil	Su > 150 kPa	Compact Sands		35°–45°
Stiff soil	Su = 75–150 kPa	Loose Sands		30°–35°
Firm soil	Su = 40–75 kPa	Overconsolidated Clay		
Soft soil	Su = 20–40 kPa	Critical State	c' = 0	ϕ' = 18°–25°
Very soft soil	Su < 20 kPa	Peak State		c' = 10–25 kPa ϕ' = 20°–28°
		Residual		c' = 0–5 kPa ϕ' = 8°–15°

Cohesion is more likely to be associated with clayey soils, while friction angle comes from sandy soils. Of course, a soil can have both, and often does. If the soil on-site is saturated (almost always with dams and levees), then soil testing should be performed under effective stress conditions. Effective stress includes the effects of pore water pressure within the soil, usually denoted as:

$$\begin{aligned}
\boldsymbol{\sigma'} &= \boldsymbol{\sigma} - \boldsymbol{u} \\
\boldsymbol{\sigma'} &= \text{effective stress} \\
\boldsymbol{\sigma} &= \text{total stress (soil pressure and water pressure)} \\
\boldsymbol{u} &= \text{pore pressure (pressure of water within soil)} \quad (1.1)
\end{aligned}$$

Pore pressures can increase or decrease during the shearing process and therefore the effective stress may change as well. Once effective stresses are accounted for (most software will do this automatically), the strength properties are developed in exactly the same way. Pore pressures (if there are any developed) are subtracted from the confining stress, effectively moving each failure circle (σ_{3a}-σ_{1a}, σ_{3b}-σ_{1b}, σ_{3c}-σ_{1c}) to the left on the graph a distance equal to the pore pressure generated during the test.

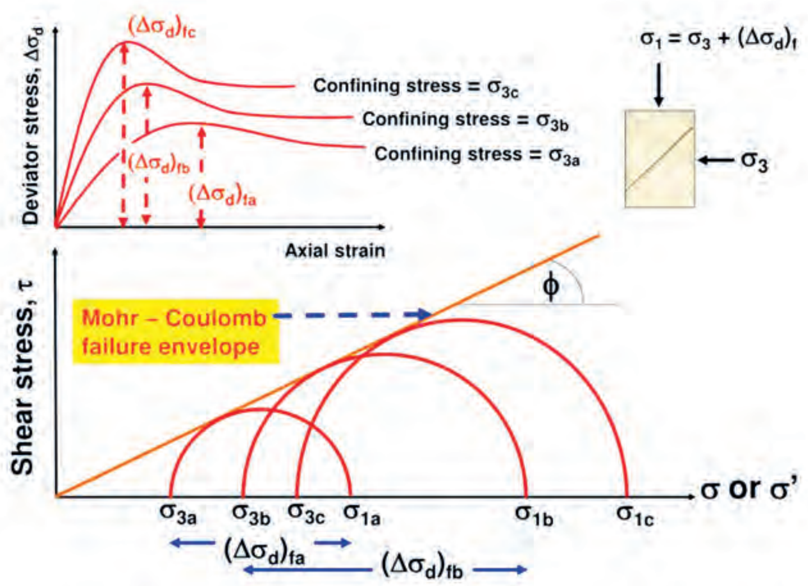
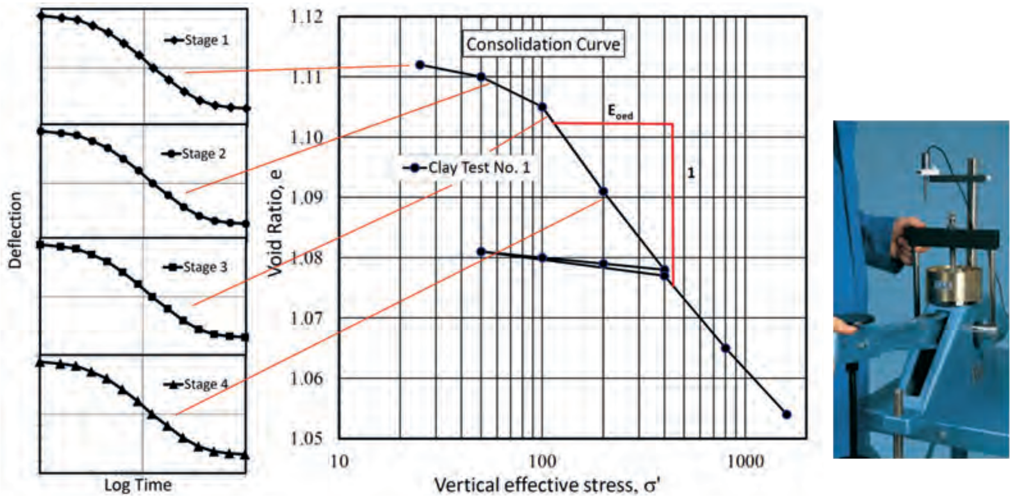


Figure 13. Series of three triaxial tests at increasing confining stress [6]
Note differences in stress-strain curves. For this soil, c = 0 and $\phi = 25°$

Consolidation is a process where firm or soft saturated clays are loaded from above and attempt to squeeze out pore water. Since clays have a low hydraulic conductivity, this process is slow, perhaps taking years to complete. As the pore water migrates out of the clay, its pores become smaller, causing settlement. Engineers need to know how much consolidation settlement it is likely, and how long will take to complete. Both answers come from a consolidation test in the laboratory. A typical test consists of several stages of loading with each stage requiring about 24 hours to complete (Figure 14a). As one stage is completed, load is doubled and the next stage started. The specimen may be unloaded as well to determine rebound behaviour. When the stages are completed, a summary plot is generated and the oedometer modulus E_{oed} can be determined (Figure 14b). Time to complete 50% or 90% of consolidation is also determined. The oedometer modulus can be applied to the field soil profile to determine how much settlement may occur. The time to complete consolidation is scaled from laboratory conditions to field conditions; depending mainly on the distance required for the pore water to reach a drainage layer.



(a) *(b)* *(c)*

Figure 14. One-dimensional consolidation testing to determine (a) rate of settlement; (b) magnitude of settlement; (c) device [6]

Methods of seepage analysis

When evaluating a levee's ability to resist seepage breakthrough, the conceptual model mentioned earlier shows the various combinations of problems that may occur (Figure 7). Note that these effects often interact with each other to weaken stability or provide a more ready pathway for ground water to flow. Seepage is not necessarily bad; however, uncontrolled seepage can lead to severe problems. In this section, the main ideas about determining the quantity and direction of seepage through a dam or levee are presented.

Flow nets and flow paths

The simplest way to estimate where seepage will travel is to construct a flow net. It is a graphical method for deducing the path water will follow as well as water pressure (head, pore pressure) and gradient ($i = dh/dl$ = change in head/distance travelled). All of these quantities are important for the proper functioning of a dam or levee.

A flow net consists of two families of lines: equipotential lines and flow lines. Equipotential lines represent locations where water head (pressure + elevation head) are equal. Figure 15 shows them with red lines. Flow lines represent the path seepage water takes from the source (river, reservoir, lake) to the exit point (hopefully a drain or outflow). Figure 15 shows them with green lines.

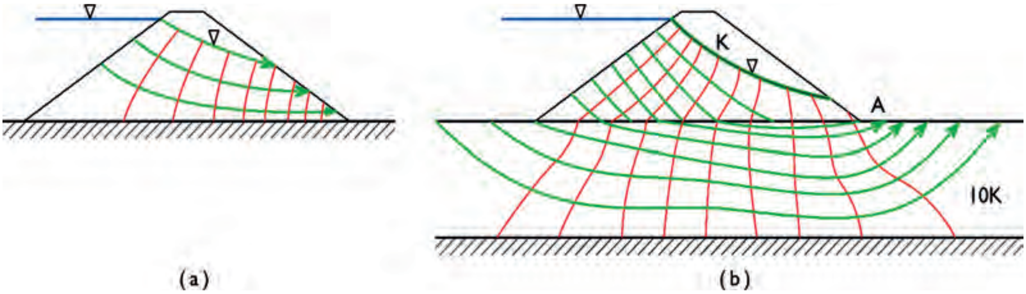


Figure 15. Flow nets drawn through an earth dam with (a) impervious foundation; (b) pervious foundation (compiled by the author)

The embankment in Figure 15a is resting on an impervious foundation; therefore, no water is seeping below it. The configuration in Figure 15b includes foundation soils with approximately 10 times higher hydraulic conductivity. Important design factors to note include the exit point on the downstream slope of Figure 15a and somewhat reduced exit on Figure 15b. Both would be unacceptable as designs. Exit gradient can be estimated if the head loss for each equipotential line (Δh) is known as well as the physical distance between the lines (Δl). Gradient is then:

$$\boldsymbol{i} = \frac{\boldsymbol{\Delta h}}{\boldsymbol{\Delta l}} \quad (1.2)$$

As the flow line is directed upward near point A in Figure 15b, it will tend to lift the soil up and out of place. This occurs when the gradient reaches approximately i = 0.85. Exit gradients of this magnitude may create exit seeps along the downstream edge of the levee or dam [14]. Drawing flow nets for conditions that are even moderately complex requires a great deal of practice and technique beyond this course. However, most seepage software programs produce equipotential lines quite readily, and will produce flow lines as part of its post-processing activity.

Seepage analysis software

There are a great number of software programs that will analyse seepage below and through dams and levees. The most common approach is a two-dimensional finite element method. Finite elements are a way to approximate non-linear field behaviour (in this case, seepage through soil) by dividing the problem domain into a finite number of elements where each has a behaviour that can be described by a simple (e.g. linear or second-order equation). Each element may have its own material properties, such as hydraulic conductivity and are connected by sharing common nodes with neighbouring elements. So, instead of defining a problem with an intractable mathematical formulation, finite elements break the problem into smaller, simpler pieces. For seepage problems, the result is a physical problem where the soil transmissivity matrix (hydraulic conductivity and element geometry combined) is first assembled. Boundary conditions are then applied, typically as constant head boundaries (such as a reservoir or flood level) then the unknown values of the total head are computed for every nodal point in the domain. The price of simplification is that many nodes and elements are required to generate an accurate model of the physical problem. Models with over 5,000 nodes (and unknowns) are common. As complexity increases, the software may have to compute more than just a steady-state solution. Flood waves are time-dependent, as are scenarios for reservoir filling and emptying. This requires a multi time step approach using unsaturated hydraulic conductivity values. The problem must then be solved for every time step. Additional boundary conditions may be needed such a rainfall, pumping or other transient events. If the problem requires a three-dimensional model, the numerical solutions become very time consuming with perhaps 50–100,000 nodes and equations.

Shown below are just two examples of a 2D problem with a dam and reservoir. The boundary conditions are 8-m total head elevation at the reservoir and 0-m total head elevation at the downstream toe. Figure 16 shows the flow regime for a homogeneous dam. There is only one type of soil throughout. The contour lines of equal total head are labelled for every 0.5 m. Note the very high exit velocity arrows near the downstream toe. This would indicate that the water would carry away the downstream slope until it failed. Flow lines move evenly through the dam from the reservoir to the downstream side. The flow net produced is very similar to Figures 15a, b. This particular software also models unsaturated flow, so there is no free (phreatic) surface through the dam. It could be drawn by connecting points on total head contours at their corresponding elevation. A more engineered approach is shown in Figure 17 where there is a centre core of lower conductivity soil (clay) and a toe drain of higher conductivity soil (gravel). Note that many of the total head contours are inside the clay core. That is because the core dissipates seepage energy, leaving much less energy toward the downstream toe. The toe drain draws flow to it, which is good, because the gravel in the toe drain is much less likely to be lifted out and away from the downstream slope. The total flow of water through this dam is about one-tenth of that shown in Figure 16.

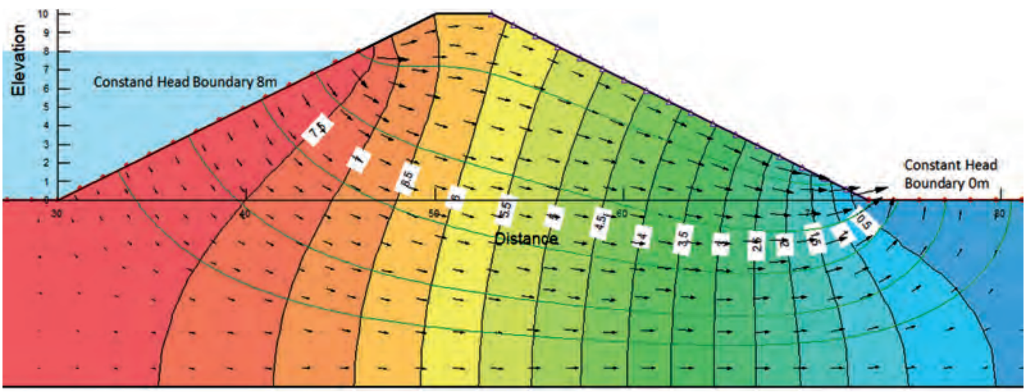


Figure 16. Flow through homogeneous dam (compiled by the author based on [8])
Note the very strong exit velocity at the downstream toe

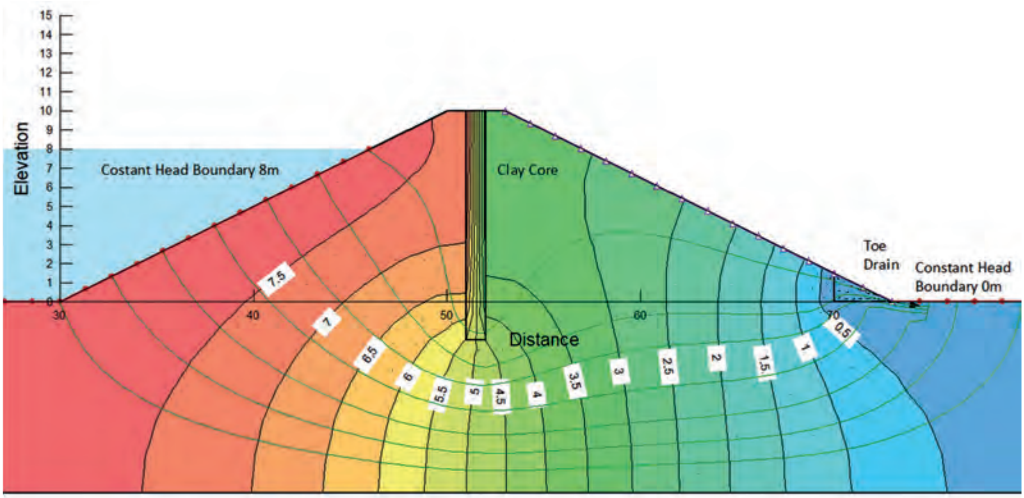


Figure 17. Seepage through a dam with a clay core and gravel toe drain (compiled by the author based on [8])

Special seepage problems with flood levees

Assessing seepage problems in levees is more difficult than in dams. Levees can be many kilometres long and are required to hold back water until a flood event occurs. It is difficult to determine how well a levee will perform until the flood wave arrives; not the best time to find out if the levee is performing properly. Since levee systems are so extensive, many embankments are nearly homogenous. Others may have an upstream blanket and some sort of toe drain system, but this is not always the case. One of the most persistent problems in levee performance is the generation of exit sand boils (Figure 17). They occur regularly during floods and are very difficult to predict and remediate [1] [18].

Since the analysis and evaluation of levees are limited in complexity, several methods have been developed to evaluate the levee embankment and foundation soils as an entire unit. Some methods apply only to specific boundary conditions or when the hydraulic conductivity of the levee is very small (1/1000) compared to the foundation soils. Other assumptions may include homogeneity of the foundation soils, or the levee embankment, or simple layered systems.

Assumptions concerning anisotropy are important as well. All methods can be used to obtain seepage quantities but may not give seepage gradients, pressures, or forces. Table 3 provides guidance about the information that can be obtained from each procedure.

Figure 18. Sand boil created by high exit gradient (sandbags have been piled around the opening to increase the downstream head and reduce the gradient) [10]

Table 4. Methods of analysis and what they compute (Comp.) or estimate (Est.) (compiled by the author)

Method	Gradients	Pressures	Seepage Quantity
Flow Net	Comp.	Comp.	Comp.
Embankment Phreatic Line	Comp.	Est.	Comp.
Unconfined Aquifer (Dupuit's assumption)	Est.	Est.	Comp.
Confined Aquifer of finite length and uniform thickness	Est.	Est.	Comp.
Blanket-Aquifer (Continuous and Discontinuous)	Est.	Comp.	Comp.
Finite Element Models	Comp.	Comp.	Comp.

Analysis by Hungarian designers is very similar to that shown above. In section 7 of the Hungarian floodwater defence handbook, the categories of models are identical to the phreatic line discussed by Casagrande and by Kozeny; and applied to several configurations. Additional configurations were analysed using the Blanket-Aquifer approach developed by [1] and extensively modified by others as it was incorporated into the design procedures for the U.S. Army Corps of Engineers [20] [22] [23] for use in the Mississippi River system. Other methods of analysis used in Hungary were based on modifications to work by Dachler, Davidenkoff and Pavlovsky [25].

Through berm analyses

The through berm (embankment) analyses generally assume flow in the foundation soils. This is a reasonable assumption if the foundation soils have conductivities 1/100–1/1000 times that of the levee berm. Some analyses consider the berm and foundation soils to have nearly the same conductivity. Most of these analyses focus on the point of exit of seepage on the landside of the berm. If the exit point is too high and the seepage velocity too great, internal piping will take place followed by a land-side collapse.

Under-berm analyses

Flow of seepage under the levee receives more attention because it is more difficult to detect and prevent. Often, some form of blanket layer is assumed to exist that restricts horizontal flow near the surface of the foundation soils. However, it will not restrict vertical flow into lower, more permeable layers, nor is it able to stop the upward movement of water on the landside. This gives rise to sand boils or blanket heave where seepage gradients are high enough to erode soil away from the exit point. The under-berm analyses generally assume no flow in the berm soils, vertical flow in the blanket soils and horizontal flow in the foundation soils. Many simplifying assumptions are made in these analyses to compute overall flow and exit gradient. Typical general blanket flow geometry is shown in Figure 19.

The figure is taken from [11], perhaps the most comprehensive reference on blanket theory today. Shown in the figure are three foundation zones: 1. Inflow on riverside; 2. Horizontal flow below levee; 3. Outflow on landside. Also shown are the pressure-head line within the foundation layer (there is no flow through the levee embankment) and either side of the levee, as well as critical head values used in calculating flows and gradients. On the extreme right and left are shown assumptions of flow or no-flow conditions which will affect the derivation of formulae and final results. An impermeable layer at depth simplifies assumptions of continuity. The authors use a coordinate system with the origin at the centre of the levee which allows for a more compact solution.

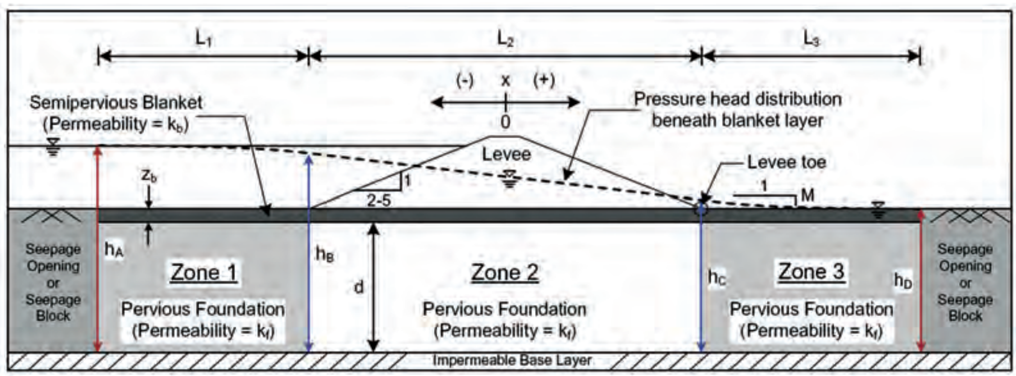


Figure 19. Typical blanket configuration [11]

Using the configuration shown above reduces the seepage assessment to families of equations that can be solved on a spreadsheet. This greatly simplifies analysis and allows the engineer to evaluate performance of a wide variety of geometric and conductivity values.

Levee field performance in the U.S.

Most levees in the U.S. are under the supervision of the U.S. Army Corps of Engineers (USACE) with a large percentage associated with the Mississippi/Missouri River system. Seepage performance of flood levees have been evaluated and analysed for over 70 years on the river system. Some of the observations are listed in the following paragraphs.

Lower Mississippi River. [19,20,21] reported the analysis of piezometer data obtained at 15 piezometer sites during the 1950 high water and selected sites at other times. Conclusions pertinent to this study included the following:

a) Sand boil occurrence. The locations of sand boils were highly correlated with local geologic conditions. In point bar areas, most sand boils occurred in ridges adjacent to swales. Sand boils also tended to occur between levees and parallel clay-filled plugs and in landside ditches.

b) Sand boil gradients. Where sand boils occurred, measured gradients were in the range 0.5 to 0.8, often about 0.65, and generally lower than the 0.85 value used in the analysis procedure. Two influencing factors were suggested: old boils may be reactivated at relatively low pressures, and the pressure relief resulting from the boil may lower piezometer readings in the area.

c) Entrance and exit distance. Both the entrance (L1) and exit (L3) distances varied with river stage. In certain cases, a reduction in the entrance distance with river stage was attributed to scour in riverside borrow pits. It was observed that calculated entrance and exit distances were quite variable, and that a 0.015-m reading error in each of two piezometers could result in substantial error in calculating these distances.

d) Permeability ratios. Ratios of the substratum horizontal permeability to the landside top stratum vertical permeability, back figured from the entrance and exit distances, were typically in the range 100 to 2,000.

e) Permeability. Apparent top blanket permeability decreased as top blanket thickness increased as a result of sealing defects, such as root holes and cracks. Also, the permeability of the landside blanket was 2 to 10 times that of the riverside blanket because of downward flow sealing defects and upward flow opening defects.

Mid-Mississippi River. [21] and [25] reviewed the performance of the Alton-to-Gale (Illinois) levee system along the middle Mississippi River during the record flood of 1973. The review was based on approximately 20,000 piezometer readings obtained from approximately 1,000 piezometers along 384 km of levee.

a) Characterisation by two soil layers. Critical reaches with respect to under seepage had a thick (6- to 15-m) layer of sandy silt or silty sand beneath the top blanket and above more pervious sands. In the present analysis and design procedure, this "intermediate" stratum must be mathematically transformed and combined with either the top blanket or substratum. When wells were designed and installed, the intermediate stratum was blanked off as the materials were too fine for the standard filter and screen. During floods, such wells may flow profusely yet piezometers at the base of the top blanket indicate excessive residual heads. This phenomenon occurs because the horizontal permeability of the intermediate stratum is greater than the vertical permeability of the substratum, causing seepage in the intermediate stratum to be more readily conducted landward than toward the well screen.

b) Corners. Where a levee bends or turns a corner (frequently encountered where a riverfront levee meets a flank levee), the landside toe is subject to seepage from two directions and the measured residual heads may be significantly higher than those predicted from the 2D analysis.

c) Back levees and flank levees. Where levees are built to provide protection from small creeks and streams traversing the main river valley that are not efficiently connected to the pervious substratum, piezometric levels may reflect slowly rising regional groundwater levels rather than being a function of the variables involved in under seepage analysis.

d) Entrance and exit distances. Entrance and exit distances calculated at piezometer ranges were frequently found to be shorter than assumed for the original design. Where values of 180 to 300 m were assumed in design, measured values were often 120 m or less.

e) Permeability ratio. The ratios kr/kb were smaller than assumed for design (400 to 2,000) [4] but were reasonably consistent with later design guidance (100 to 800 in Rock Island) [5].

Occurrence of sand boils. Sand boils occur at less-than-predicted gradients. This was noted as early as 1952 and is well documented in Figure 20 taken from [21]. It was also noted by [5] in his analysis of Rock Island performance data. Nevertheless, boil occurrence is rare in terms of the many kilometres of levee subjected to similar gradients. It is apparent that local geologic conditions must have a more significant influence on where boils occur than does the gradient. There is considerable evidence that boil occurrence is often related to concentration of seepage at discontinuities and defects in the top blanket. Such non-uniform blanket geometry is not accounted for in the uniform, 2D model used for design. Despite the discussion concerning geologic conditions in [20] and the 3D cross sections illustrating floodplain deposits (Figure 9) and their relationship to under seepage, the same analysis and design criteria are applied in the same manner for all types of deposits.

Relationship of boils to blanket thickness. The correlation presented by [5] between boil occurrence and top blanket thickness implies that boils are the only concern and overlooks the possibility of rather sudden rupture of thick clay blankets retaining high piezometric pressures (heaving). This was apparently the case of the 1943 floodwall failure at Claryville, MO, described by [13].

Critical Gradient Criteria. [5] notes that the calculation of the critical gradient was based on a homogenous top blanket with no cohesion and flexural strength and noted that these assumptions would often be invalid. This was also challenged in a discussion of [19]. This discussion recommended the use of a factor of safety against uplift defined as the ratio of the saturated weight of the blanket to the piezometric pressure at the base of the blanket.

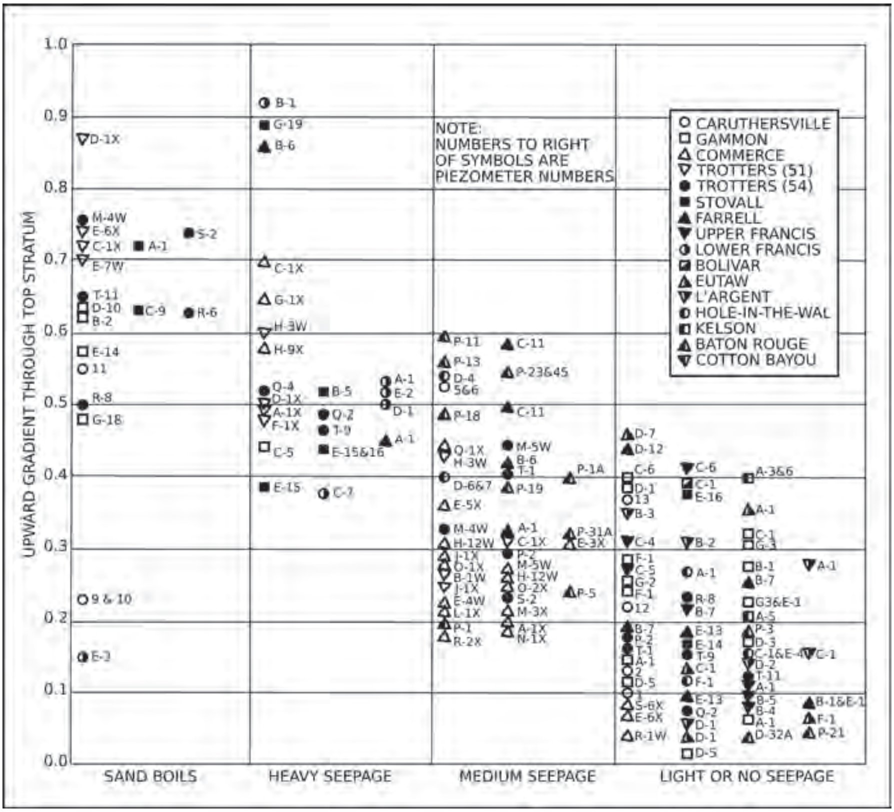


Figure 20. Degree of seepage vs. upward gradient for many levee locations [12]

Calculation of gradients. As pointed out by [5], the calculation of gradients is an uncertain process because of the difficulty in properly estimating the blanket thickness. It becomes very judgmental where a non-homogeneous blanket must be transformed to an equivalent homogeneous blanket, or where the blanket changes thickness along or beyond the levee toe. In ridge and swale topography, the top blanket may be highly stratified, and development of an idealised design profile by the engineer may seem to be a meaningless process.

Calculation of entrance and exit distances and residual head. The review of [5] suggested that accurate values of the entrance distance, X1 and the exit distance, X3 are almost impossible to obtain. The problems are not as severe in practice as it would appear, even though they are functions of four uncertain parameters. This arises because the prediction of interest is the residual head, at the levee toe. Working backwards through the analysis equations, h0 is determined by simple proportion involving the entrance and exit distances:

It is apparent that h0 can be accurately calculated if the proportion between X1 and X3 is reasonably correct, even if their actual values are grossly in error. For a levee reasonably distant from the river, where riverside values of the parameters are used to calculate X1, and landside values are used to calculate X3. As the landside and riverside values are often significantly correlated, the equations yield values for the entrance and exit distances that are generally in correct proportion. Furthermore, the extraction of the square root tends to minimise the effects of error in the parameters, and errors in z and d are just as likely to be compensating as biased. [4] implies the same idea; that is, that one can reasonably predict the residual head even with the wrong permeability ratios.

Permeability values and ratios. Although hydraulic conductivity (or permeability) is difficult to quantify, the Corps' recommendations are not arbitrary as suggested by [5] but are based on considerable experience and piezometric measurements. Residual heads and gradients are dependent only on the ratios of the permeabilities, not their absolute values. As the values used are back calculated from observed piezometric grade lines and then reused in the same equations to estimate the piezometric grade line for other conditions, it is not surprising that they provide generally good results. The permeability ratios and the blanket formulas form a closed loop; thus, they tend to work whether they are correct or not.

Nevertheless, data obtained from the 1973 flood in St. Louis indicated lower ratios than those typically recommended for use in the Lower Mississippi Valley, and the Rock Island analysis indicated still lower values. While the reasons for this trend require more study, it is noted that these sites represent significant differences in the geologic environment. The Lower Mississippi is a classic meandering stream in a wide valley. Levees are at relatively great distances from the river, and discontinuities such as clay plugs, and oxbows are common. The river carries a high sediment load. At the other extreme, the characteristics of the valley in the Rock Island District are primarily related to glacial melting. The valley is rather narrow and there are relatively few meander deposits. Levees are relatively close to the river. Much of the sediment load enters the river downstream of the Rock Island District. The St. Louis District and the Middle Mississippi Valley represent transitional conditions. Concentrations of seepage adjacent to clay plugs or other blanket discontinuities increase residual heads and may result in apparently higher permeability ratios than would be measured under relatively uniform blanket conditions.

Determination of parameters from piezometer data. Estimates of entrance distances, exist distances and permeability ratios have generally been made only at piezometer ranges because a linear hydraulic grade line can be fitted through a number of points. Too many assumptions appear necessary to estimate these factors from a single piezometer at the levee toe. However, all reports of such analyses have mentioned the difficulty in obtaining reasonable values because of the sensitivity of the calculations to minor errors in the differences between piezometer readings.

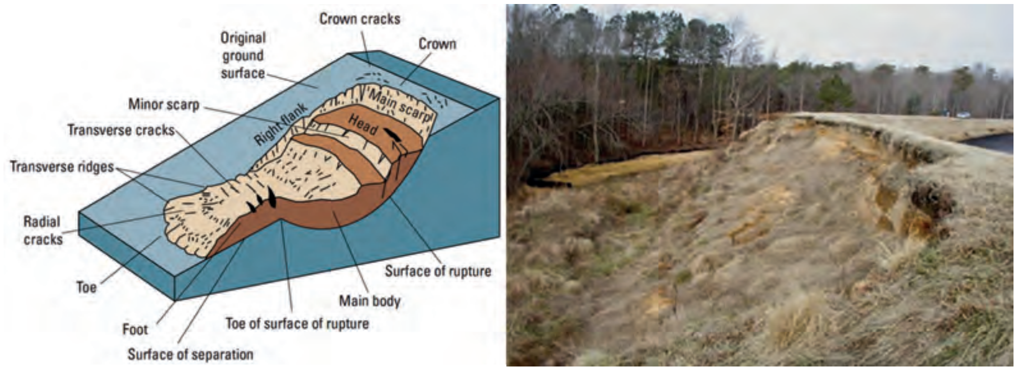
Using simplified equations and the measured residual head from a single piezometer at the levee toe, and making a few reasonable assumptions, considerable insight can be gained regarding the probable values of X1, X3 and the permeability ratio.

Deficiencies in procedures, summary. Based on the various reviews of performance data, a summary of the assumptions made in under-seepage analysis and the special cases in which they may be deficient has been summarised. The performance data also indicate that there can be a wide variation in the observed values of parameters assumed or calculated in the design. Possible improvements to the analysis procedures lie in four areas:

- *a)* Computerised analysis using existing procedures to allow more expedient solutions.
- *b)* Probabilistic adaptation of existing procedures to allow for uncertainty in the parameters.
- *c)* Extension of the existing procedures to more general cases to allow more realistic modelling of actual conditions.
- *d)* Improvements in the exploration process to allow better identifications of the subsurface conditions to be modelled.
- *e)* The equations for seepage analysis as well as for design of seepage berms and relief wells have been adapted to computer programs by several parties.

Slope stability

Dams and levees are particularly sensitive to slope stability problems. This is because of seepage as well as other site factors which tend to magnify small problems. Shown in Figure 21a are terms related to landslides in general and Figure 21b shows a typical slope failure on the downstream side of a dam. One must first realise that the slope is seeking a better equilibrium point by sliding, so if the material at the toe is removed, more sliding will occur. Another important point is that the slope forces are very large and structural elements such as stabilising piles or walls generally will not help. On a hillside or natural slope there is often firm ground or rock behind the surficial soils that will provide an anchoring point for tiebacks and supports. Of course, in an embankment, such support is not available. Additionally, seepage from the waterside, rainfall erosion, animal activity or differential settlement of the embankment may all encourage instability.



(a) (b)

Figure 21. (a) Landslide terminology and (b) typical slope failure on the downstream side of the dam [9]

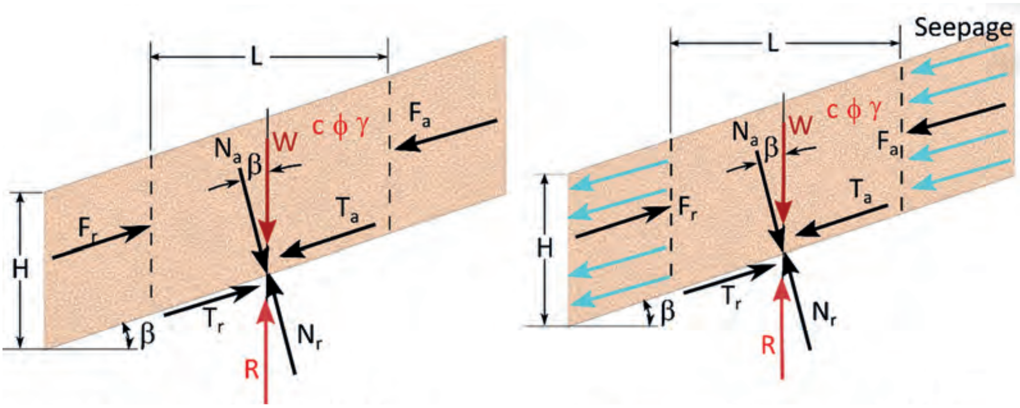
Evaluating slope stability

The numerical evaluation of slope stability requires calculating two competing elements: 1. The forces (or rotational moments) driving the slope downward; and 2. The forces (or rotational moments) resisting the downward movement. The driving forces originate from the weight of soil and water in the slope while the resisting forces come from the soil's strength, usually expressed as cohesion (c) and friction (ϕ).

Infinite and finite slope analysis

The simplest and most useful analysis assumes the slope to be infinite in length and the slide to be rather shallow in depth. The calculation of stability is straightforward and gives a first approximation of the stability of the embankment. Figure 22a, b shows two scenarios where the first is a dry slope (no water action) and the second accounts for seepage forces moving down the slope. This may occur if there is seepage near the surface of the downstream face, and also if the reservoir or upstream water drops suddenly, leaving pore water trapped in the soil because the soil itself cannot drain fast enough. "Suddenly" is a relative term, depending on the hydraulic conductivity of the soil in the slope. Gravels and sands might drain in minutes or a day. Silts would take several days to several weeks, and clays may take six months. So sudden drawdown for a clay embankment would be anything faster than a few months.

The factor of safety for stability is the ratio of resisting forces (soil strength expressed as (c, ϕ)) to the action forces (downward weight of the soil (γ), and seepage or water forces if present). One could formulate equilibrium of the system as shown in the equations below each section. Both equations have the resisting forces on top and action forces on the bottom. If the safety factor is less than 1.0, the slope will fail by sliding downhill. For the dry conditions, the contributions of cohesion and friction angle are separated. For sandy slopes (c=0), the slope remains stable if it is flatter than ϕ' For the same sandy slope with seepage occurring, the effect of the water reduces resistance and increases actions.



$$SF = \frac{c'}{\gamma H \cos^2 \beta \tan \beta} + \frac{\tan \phi'}{\tan \beta}$$

$$SF = \frac{c'}{\gamma_{sat} H \cos^2 \beta \tan \beta} + \frac{\gamma' \tan \phi'}{\gamma_{sat} \tan \beta}$$

(a) *(b)*

Figure 22. Infinite slope forces for (a) dry conditions and (b) seepage conditions (compiled by the author)

Typically, $\gamma_{sat} \approx 2 \times \acute{\gamma}$ and the tangent of the slope (*tanβ*) must be less than 1/2 (*tanϕ'*). Infinite slopes fail along a shallow plane below the surface. Sandy materials will often fail this way. However, materials that are mixed sand and clay ($c,\phi \neq 0$) experience more deep-seated failure. The weakest surface where failure takes place may be irregular, but more circular than planar. In order to better model such failures, the method of slices is often used. The approach first assumes a failure plane through the embankment which may be circular but does not have to be. The mass of soil above the failure plane is then divided into slices (Figure 23) and forces acting between each slice are considered. Rotational equilibrium is considered here so resisting moments due to soil strength and acting moments due to soil weight are considered. Note that some of the soil weight will produce a negative rotation (slices 1–4). The soil at the base of each slice is resisting rotational movement (S_i in upper diagram) while the larger slices are acting to de-stabilise the slope (slices 5–10). The computation for this example is moderately complex; I used a spreadsheet to perform the calculation and I spent less than an hour setting it up, computing and displaying the results in Excel. The computational procedure is iterative: you must guess the eventual factor of safety before you compute it. The final solution converges after 2 or 3 intelligent guesses. This analysis produced a factor of safety = 1.5. As with most geotechnical analyses, two or three digits of precision is as accurate as one can compute since there are so many uncertainties about soil strength, layer geometry and pore pressure conditions. Since this is only one possible failure surface, a full analysis would try other circles with different centre points and different radii, compute factors of safety and select the candidate with the lowest value. Obviously, a software model is normally used to perform stability analyses. The same danger with sudden drawdown or seepage forces applies here as it did before with the infinite slope analysis. The soil is heavier and weaker during those conditions; therefore, it must be carefully modelled and checked as part of the embankment stability assessment.

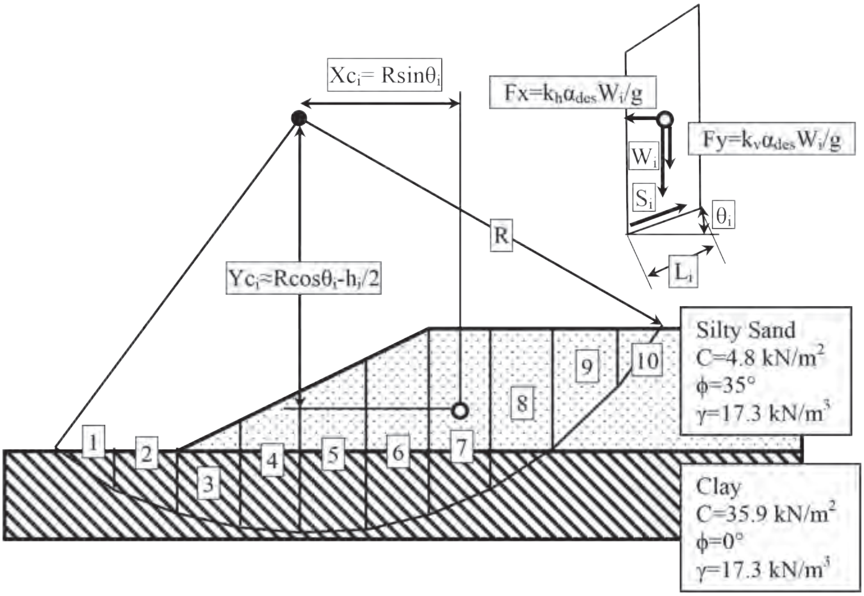


Figure 23. Slope stability by method of slices (compiled by the author)

Note that the upper diagram shows forces acting on one slice. Trial failure surface cuts through two different layers of soil

Software program for slope stability

Slope stability analysis by limit equilibrium methods, such as the one presented in Figure 23, are the standard approach to determine a factor of safety against sliding. Soil properties, layer geometry, boundary conditions and limits on the number of possible failure surfaces to attempt are input to the model analysis. The software will test perhaps 100 trial surfaces and return the worst (lowest FS) 20 or so. A typical output is shown in Figure 24. The yellow fence is the slices used by the software to compute the stability problem in a manner similar to Figure 23. The other red lines are other failure surfaces where FS = 1.9 to 2.0. There were over 100 trial surfaces in the analysis; however, these few give a good enough impression about how it searches for solutions.

The finite element method can be used to examine slope stability as well; however, a different approach is required. Since FEM is a continuum approach, there will not be a sharp failure surface where sliding takes place. Instead, a strength reduction method is used where the strength of materials are reduced until a prescribed level of strain occurs. The level of strain is enough to indicate impending failure, but not enough to cause the numerical computation to become unstable. Figure 25 shows a 3D problem with finite element mesh. There are 43,000 nodes with about 120,000 equations and unknowns and a total of about 13,000 elements. The execution time was 41 minutes on a high-performance workstation. The displacement surface is shown and has a shape similar to a circular failure arc. Based on the strength reduction method, the factor of safety is 0.97.

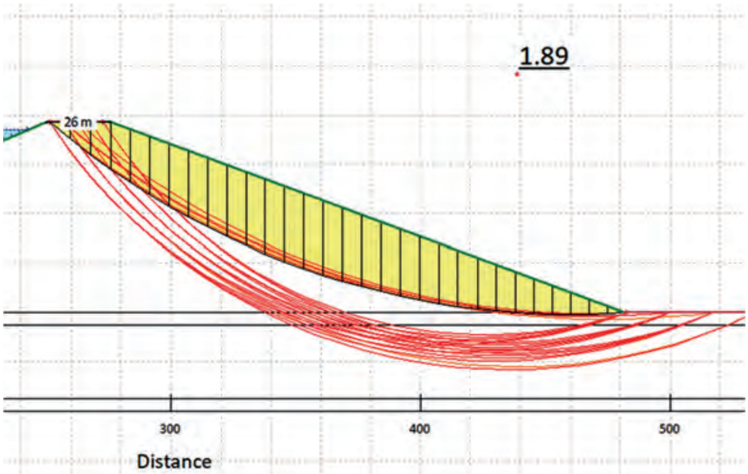


Figure 24. Slope stability analysis showing critical failure surface and slices used to compute FS = 1.89 (analysis by author using [8])

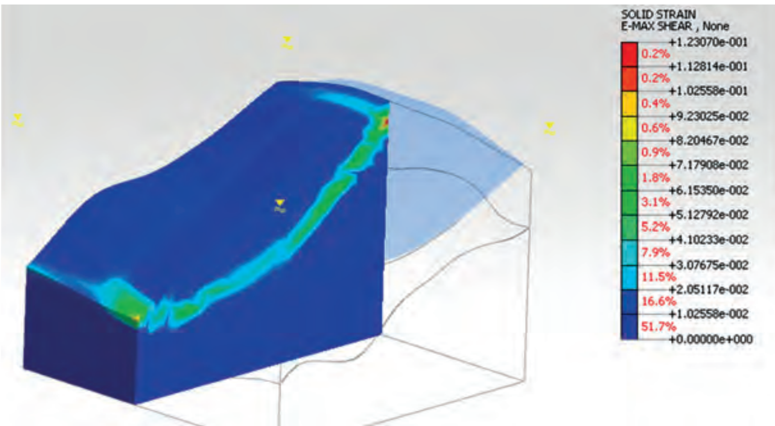


Figure 25. 3D view of maximum shear strains during strength reduction FS = 0.97 (analysis by author using [12])

Conclusions

Soil mechanics is indeed a complex system of natural materials and forces. Predicting behaviour due to flooding or other extreme events is even more difficult. At the present time, the level of sophistication in analysis outstrips the level of accuracy of data and known boundary conditions. Engineers and water resource managers are cautioned that wonderful 3D renderings do not make an analysis any more accurate. Careful observation and measurement in the field during flood events are still the best resource for better management and engineering.

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Gábor Baranyai

The Legal and Policy Framework of Transboundary Flood Management

The challenge of transboundary flood management in international relations

The joint management of river floods has always been one of the most prominent challenges of transboundary water cooperation in international river basins. Naturally, this condition is also reflected in the relevant international regulatory and policy frameworks. Thus, contemporary public international law stipulates the collaboration of riparian states to prevent and mitigate floods as one of the core obligations of transboundary water cooperation. The European Union actually dedicates a stand-alone regulatory instrument – the Floods Directive – to the collective management of floods in the EU. River basin organisations, basin-wide and bilateral treaties devote significant effort and attention to cross-border flood control.

This is all the more necessary as the various megatrends of our era – most prominently climate change, urbanisation and ensuing land use change – all contribute to the global rise in flood risks. The regular assessment reports of the Intergovernmental Panel on Climate Change project that one of the most critical freshwater-related impacts will be the increased exposure to 20th century 100-year-river-floods. This will go hand in hand with the likely increase in the frequency of meteorological droughts (i.e. less rainfall) and agricultural droughts (i.e. less soil moisture). The rise of hydrological extremes will have a knock-on effect on the safety of humans, material assets, other natural resources and ecosystems. Not only the magnitudes of flood events are expected to increase, but their patterns also show significant changes. The loss of snow and the decrease in total meltwater yields in glacier-fed rivers implies a shift of peak discharge from summer to spring [3 p. 232–234]. In the European context recent research suggests that flood peaks with return periods above 1 in 100 years are expected to double in frequency in the next thirty years [1].

Against this background the importance of proper policy and regulatory frameworks to govern transboundary cooperation in the field of flood protection will only grow. Luckily, flood management belongs to the more “benign” challenges of co-riparian relations. Floods are typically short-term events with a(n almost) mechanical knock-on effect on downstream areas. The downstream motion of water can be predicted fairly precisely by widely available satellite-based technologies. On mid- and downstream areas, where population density tends to be the highest, this allows authorities and citizens to choose the adequate level of protection. Therefore, flood management is usually perceived by riparian states as a politically less contentious issue whose collective resolution is mutually beneficial. (This is in sharp contrast with such “malign” issues of high political conflict potential as water allocation between riparian states or transboundary water pollution) [5]. This condition significantly enhances the development of international governance frameworks and the success of their implementation.

The structure of the international governance framework

General international water law – notably customary international law, the 1997 UN Watercourses Convention and the 1992 UNECE Water Convention – addresses the issue of transboundary flood cooperation at a high level of abstraction. It prescribes horizontal obligations that are derived from the general duty of riparian states to cooperate and not to cause transboundary harm. The main rules of transboundary floods can be clustered into procedural responsibilities and substantive obligations. Procedural responsibilities of riparian states cover emergency cooperation and contingency planning. Substantive obligations, on the other hand, relate to the duty to maintain national flood protection infrastructure and to implement flood control in such a way that it does not increase flood risks in fellow riparian states.

At basin level the relevant multilateral or bilateral treaties usually address the issue of flood management explicitly, even though detailed rules on the subject are adopted only exceptionally (e.g. in the case of the Sava River). Yet, coordination of flood protection has become a major undertaking by all relevant river basin organisations. This development has, especially in the European continent, yielded considerable results in the past two decades. Naturally, flood management also features in the lowest level of transboundary water cooperation: bilateral (frontier) water cooperation treaties.

The European Union (EU) – a regional supranational body of European states – maintains an autonomous water governance regime that covers the issue of transboundary flood management substantially. In fact, the EU's Floods Directive constitutes the World's most elaborate and robust dedicated international cooperation scheme with regards to flood management.

Flood management in international water law

General international water law

International law is the body of law that governs the legal relations among states and international organisations. Its main function is to provide the institutional framework and rules for treaty-making, interpretation and dispute resolution for countries to work together peacefully. International water law is a sublet of public international law concerned with the use and protection of freshwater.

Over the past two centuries international water law has been largely shaped by claims and counter-claims concerning the possession and use of shared water resources. Much of this state practice has been subsequently codified through regional and global treaties, confirmed by international judicial practice or summarised by the works of non-governmental scholarly bodies, most prominently by the 1966 Helsinki Rules[1] and the 2004 Berlin Rules[2] of the International Law Association.

Today, the use and protection of shared watercourses is governed by a number of fundamental principles rooted in general (customary) international law, two global legal instruments that lay down general cooperation frameworks for transboundary river basins – the 1997 UN Watercourses Convention[3] and the 1992 UNECE Water Convention[4] – as well as the considerable jurisprudence of the International Court of Justice and other international courts and tribunals. Evidently, most of daily cross-border water management, however, takes place through the vast body of regional, basin and bilateral treaties that regulate co-riparian relations at various levels of detail.

Flood management under general international water law

Concerns about the natural variability of transboundary river flow are not a new phenomenon in international relations. Yet, until relatively lately water treaties did not pay sufficient attention to the issue. As a result, general international conventions law scarcely address flood management in any explicit and extensive fashion. They nonetheless provide an important framework to address the issue in detail in basin or bilateral context.

Thus, the core requirements of the UN Watercourses Convention – i.e. equitable and reasonable utilisation of shared river basins, the obligation not to cause significant harm and the obligation to cooperate over planned measures – regulate the issue only indirectly. These general principles, however, imply the duty of watercourse states to manage floods with due attention to the interests of other riparians. The Convention also calls on watercourse states to prevent and mitigate, individually and/or jointly, "harmful conditions", including floods that may have a negative impact on other riparian states.[5] When such conditions amount to an emergency situation, i.e. a sudden event actually or potentially causing serious harm to other watercourse states, the state of origin must immediately notify the (potentially affected) other riparians and take all practicable measures to prevent, mitigate or eliminate the harmful effects of the emergency.[6]

These rather general treaty obligations are further interpreted by the so-called Berlin Rules on Water Resources, a scholarly compilation of customary international water law developed by the International Law Association. The Berlin Rules cluster the relevant duties of riparian states as follows:

1 International Law Association: The Helsinki Rules on the Uses of the Waters of International Rivers, 14–20 August 1966.
2 International Law Association: The Berlin Rules on Water Resources, 21 August 2004.
3 Convention on the Law of Non-Navigational Uses of International Watercourses. New York, 21 May 1997.
4 Convention on the Protection and Use of Transboundary Watercourses and Lakes. Helsinki, 17 March 1992.
5 Art. 27, UN Watercourses Convention.
6 Art. 28. Ibid.

- general obligation to cooperate in the development and implementation of flood control measures with due regards to the interests of states likely to be affected by flooding
- immediate communication of situations likely to create floods or dangerous rises of water levels in their territory to other riparian states and the competent international organisation (river basin commission, bilateral joint commission, etc.)
- joint monitoring of flood conditions and planning of flood protection measures – these include contingency plans, collection and exchange of relevant data, preparation of surveys, the planning and designing of relevant measures (e.g. flood plain management and flood control works), flood forecasting and warnings, development of a regular information service, etc.
- maintenance of flood control works and the prompt implementation of flood control measures to assure the minimisation of damage from flooding[7]

The UNECE Water Convention prescribes similar obligations for riparian states in the Pan-European regional context. The starting point under the Convention is the general obligation to prevent, control and reduce transboundary impact.[8] While transboundary impact is defined as "significant adverse effect […] caused by a human activity", the interpretation practice of the Convention, however, confirms that the impacts of naturally occurring hydrological extremes such as floods also fall under this obligation [4, p. 369]. Hand in hand with the prevention/mitigation obligation goes the general duty of riparian states to cooperate on a multitude of water management issues, such as the joint monitoring and regular assessment of transboundary impacts (including the floods, ice drifts, etc.)[9] or the early exchange of information.[10] Also, in their basin treaties and/or bilateral arrangements riparian states have to establish warning and alarm procedures as well as contingency plans that cover floods.[11] In case of critical situations, parties are under a duty to assist each other following the procedures laid down by the Convention.[12]

In addition to the above general framework, the various Convention bodies have adopted a range of soft-law instruments that provide further assistance to basin states as to the short- and long-term management of floods. First such instrument was the Guidelines on Sustainable Flood Prevention adopted in 2000.[13] The Guidelines cover:
- basic principles, policies and strategies for transboundary flood management
- tasks of joint bodies (river basin organisations)
- the provision of information
- mutual assistance and public awareness
- education and training

7 Art. 34, Berlin Rules.
8 Art. 1.2, 2.1, UNECE Water Convention.
9 Art. 4, 9.2, 11.1, 13.3, UNECE Water Convention.
10 Art. 6, 13.1. Ibid.
11 Art. 3.1, 9.2, 14. Ibid.
12 Art. 15. Ibid.
13 Guidelines on Sustainable Flood Prevention, ECE/MP.WAT/2000/7.

They recommend that joint bodies take a central role in flood control. To that end it suggests that they:

- develop long-term flood prevention and protection strategies as well as action plans for the shared basin
- draw up an inventory of structural and non-structural measures
- help countries cooperate in establishing the water balance for the entire catchment area

The Guidelines also include good practices, inter alia on retention of water in the soil, proper land use, zoning and risk assessment, early warning and forecast systems, awareness raising and planning. Finally, the Guidelines address the health impacts of floods.

The Guidelines were followed by the UNECE Model Provisions on Transboundary Flood Management endorsed in 2006.[14] The Model Provisions comprise a concrete legislative text that can be used by riparian states in their specific basin-wide or bilateral arrangements to tackle the challenges of transboundary flood control. The Model Provisions contain a similar range of obligations as outlined in the Guidelines and the Berlin Rules.

Basin and bilateral water treaties

As the scale of geographical scope decreases, specific variability management schemes become more frequent. Thus most basin and bilateral treaties in the world dedicate significant attention to flood issues. E.g. the Mekong Cooperation Agreement contains general and specific rules for water quantity management for the monsoonal wet and dry seasons.[15] In "cases of historically severe droughts and/or floods", however, the application of regular allocation rules is suspended.[16] Such exceptionally severe hydrological events are subject to early notification and the mandatory involvement of the Joint Committee of the Mekong River Commission with a view to adopting appropriate remedial action.[17] The Charter of Waters of the Senegal River also foresees such consultation procedures in the event pre-fixed water allocations must be revisited due to floods or other natural disasters or water shortages of natural character.[18]

Apparently, water treaties primarily concerned about water allocation are more likely to contain some kind of mechanisms to handle extreme flow variations. For instance, the 1996 Ganges Treaty between India and Bangladesh calls for immediate consultations should the flow at Farakka Dam fall below a commonly agreed threshold so as "to make

[14] Model Provisions on Transboundary Flood Management, ECE/MP.WAT/2006/4.

[15] Art. 5 and 6, Agreement on the Cooperation for the Sustainable Development of the Mekong River Basin. Chieng Rai, 5 April 1995.

[16] Art. 6. Ibid.

[17] Art. 10. Ibid.

[18] Art. 6 and 7, Charter of Waters of the Senegal River, 28 May 2002.

adjustments on an emergency basis, in accordance with the principles of equity, fair play and no harm to either party".[19]

Flood management in the European basin and bilateral water treaties

Despite its primary ecological focus, the Danube Protection Convention[20] contains a number of substantive and procedural provisions that help riparian states address hydrological variability in a systematic and structured fashion. The preamble to the Convention pays specific attention to "the occurrence and threats of adverse effects, in the short and the long term, of changes in conditions of watercourses within the Danube River Basin".[21] It follows that Danubian states are required to cooperate in the prevention, control and reduction of transboundary "adverse impacts and changes occurring or likely to be caused".[22] Joint action thus must also encompass the monitoring and evaluation of the natural water household and all of its components (precipitation, evaporation, surface and groundwater run-off) in the entire basin.[23] From this general objective flow a number of precisely defined obligations. First, riparian states must monitor, record and assess, jointly and individually, the conditions of the Danube's natural water resources through a number of quantitative parameters, including water balances, flood forecasts or any change in the riverine regime.[24] Second, under the general obligation to prevent, control and reduce transboundary impacts riparian states are obliged to exchange all relevant data, including the operation of existing hydrotechnical constructions (e.g. reservoirs, water power plants) and measures aimed at preventing the deterioration of hydrological conditions, erosion, inundations and sediment flow, etc.[25] Regular exchange of information must be supplemented by coordinated or joint communication, warning and alarm systems as well as emergency plans to address critical water conditions, including floods and ice-hazards.[26] Should such a critical situation of riverine conditions arise, riparian states must provide mutual assistance upon the request of the affected basin state.[27]

The sister treaty of the Danube Convention, the Sava Framework Agreement[28] goes even further when it comes to managing hydrological variability. Thus the Sava Framework Agreement specifically refers to droughts and water shortages as critical

[19] Art. II, Treaty between the Government of the Republic of India and the Government of the People's Republic of Bangladesh on sharing of the Ganga/Ganges waters at Farakka. New Delhi, 21 December 1996.
[20] Convention on Cooperation for the Protection and Sustainable Use of the Danube. Sofia, 29 June 1994.
[21] Second Recital, Preamble, Danube Protection Convention.
[22] Art. 5.2, Danube Protection Convention.
[23] Art. 1.c.g), Danube Protection Convention.
[24] Art. 5.2.a) and 9.1. Ibid.
[25] Art. 3.2 and 12. Ibid.
[26] Art. 16. Ibid.
[27] Art. 17. Ibid.
[28] Framework Agreement on the Sava River Basin. Kranjska Gora, 3 December 2002.

hazards jeopardising the integrity of the water regime of the river.[29] In that spirit it calls upon riparian states to establish a coordinated or joint system of "measures, activities and alarms in the Sava River Basin for extraordinary impacts on the water regime, such as […] discharge of artificial accumulations and retentions caused by collapsing or inappropriate handling, flood, ice, drought, water shortage […]".[30] To that effect, parties have even committed themselves to conclude a special protocol "on the protection against flood, excessive groundwater, erosion, ice hazards, drought and water shortages".[31] The 2010 Protocol on Flood Protection to the Framework Agreement on the Sava River Basin[32] – that undertakes the coordinated implementation of the EU's Floods Directive in the basin (even though half of the riparian states are not EU members) – calls for the:

- undertaking of preliminary flood risk assessment
- preparation of flood maps
- development of flood risk management plan in the Sava River Basin[33]

Moreover, the Protocol creates an operative system of flood defence, comprising forecasting, warning and alarm, information exchange as well as the handling of emergency situations and mutual assistance.[34]

The Rhine Protection Convention[35] addresses flood protection in a far less elaborate fashion. The key objectives of the Convention – i.e. the maintenance and restoration of the natural functions of the Rhine basin waters, the environmentally sound management of water resources and general flood protection and prevention – imply the broad cooperation of riparian states over flood protection and other hydrological hazards.[36] Thus, riparian states must inform the International Commission for the Protection of the Rhine (ICPR) and other riparian states likely to be affected of imminent flooding.[37] They must also draw up warning and alert plans for the Rhine under the coordination of the ICPR.[38]

The Meuse Agreement[39] defines the mitigation of the effects of floods and droughts as one of the key objectives of transboundary cooperation.[40] In both cases joint riparian action should extend to the development of preventive measures. To that effect the International Meuse Commission is tasked to develop recommendations on flood prevention and protection, flood management coordination as well as on the mitigation of the effects

29 Art. 2.1 and 13, Sava Framework Agreement.
30 Art. 13.1. Ibid.
31 Art. 30.1.a). Ibid.
32 Protocol on Flood Protection to the Framework Agreement on the Sava River Basin. Gradiška, 1 June 2010.
33 Art. 4. Ibid.
34 Art. 9–11. Ibid.
35 Convention on the Protection of the Rhine. Bern, 12 April 1999.
36 Art. 3. Ibid.
37 Art. 5.6. Ibid.
38 Art. 8.1.c). Ibid.
39 International Agreement on the River Meuse (Accord international sur la Meuse). Gent, 3 December 2002.
40 Seventh and eight recitals, Preamble, Accord International sur la Meuse.

of droughts.[41] The Meuse riparians are also obliged to inform each other of any major hydrological events, including imminent floods.[42]

The 1990 Elbe Convention[43] and the 1996 Oder Convention[44] make no reference whatsoever to hydrological variability, not even flood protection cooperation. The two relevant basin commissions are however tasked to monitor the general hydrological situation in their respective catchment areas.[45]

Bilateral water treaties

The most comprehensive of all bilateral water treaties, the Albufeira Convention between Spain and Portugal[46] sets out concrete measures parties must implement in case of floods. The applicable flood control regime goes actually further than the usual forecasting–warning–emergency–preparedness provisions most regional or bilateral similar arrangements contain. It also gives upper and lower riparian states a right to demand the other party to implement pre-defined (or any other) interventions that are necessary to prevent, control or mitigate the effects of floods.[47] The conditions of exceptional situations – both floods and droughts – are to be defined for every two years and subsequently reviewed. The Convention also calls for the joint study of water floods with a view to long-term prevention and mitigation.[48]

Several other European bilateral water treaties make some reference to cooperation over flood prevention and protection. Most of these treaty provisions, however, tend to be rather basic, reinstating the general will or duty of the parties to cooperate and/or referring the subject to the activities of the joint commissions.[49] In a limited number of cases, bilateral water treaties contain substantive obligations parties must observe in flood protection or other emergency situations. E.g. the Hungarian–Ukrainian frontier water treaty requires parties to refrain from permitting any interventions that may raise flood volumes above previously agreed-upon levels. In the spirit of solidarity, riparian states are also obliged to provide technical assistance in times of exceptional floods

41 Art. 2.c, 4.4.a), b). Ibid.
42 Art. 3.2.d). Ibid.
43 Convention on the International Commission for the Protection of the Elbe. Magdeburg, 8 October 1990.
44 Convention on the International Commission for the Protection of the Oder. Wroclaw, 11 April 1996.
45 Art. 2, Elbe Convention; Art. 2, Oder Convention.
46 Convention on the Co-operation for the Protection and the Sustainable Use of the Waters of the Luso-Spanish River Basins. Albufeira, 30 November 1998.
47 Art. 18.5. Ibid.
48 Art. 18.7 and 19.5. Ibid.
49 E.g. Art. 2.1.b), Agreement between Finland and Sweden Concerning Transboundary Rivers. Stockholm, 11 November 2009; Art. 2.2.b) and 6, Agreement between the Federal Republic of Germany and the European Economic Community, on the one hand, and the Republic of Austria, on the other, on cooperation on management of water resources in the Danube Basin. Regensburg, 1 December 1987.

upon demand (the costs of such technical assistance is to be borne by the beneficiary).[50] The so-called Discharge Rule between upstream Finland and downstream Russia for the Vuoksi river basin[51] calls on riparian states to maintain the flow quantity of the river in a "normal zone", defined by the Rule with reference to historically prevailing natural flow volumes. Should extreme floods or extreme low water levels appear, discharge rates must be changed by Finland with a view to minimising adverse effects.

Flood management in European Union water law

The European Union and the question of freshwater

The European Union (EU) is a supranational form of political and economic integration of 28[52] European states. Over the past 60 years the EU has developed an autonomous legal system that – to a large extent – functions independently from international law and enjoys supremacy vis-à-vis the national legal order of its member states. The EU implements a large number of thematic public policies according to a division of competences laid down in the EU's founding treaties.

One of the most extensive sectoral policies of the EU relates to the protection of the environment. Under the broad heading of environmental policy the EU has, since the 1970s, adopted a large number of legislative acts and strategic documents relating to freshwaters. These legal acts and strategic documents amount to a comprehensive water policy regime.

EU water policy and law represent a very high level of ambition and success in global comparison. No other transnational water governance scheme aims at such a comprehensive protection of human health and the aquatic environment as the EU does. In fact, EU water law amounts to a much more uniform and stringent common water protection regime that most of the world's 28 federations.

The distinctive characteristics of EU water policy and law

The fact that water is regulated in the EU as a sublet of environmental policy has substantial repercussions on the nature and scope of the EU's own water regime. First of all, water quality management and water ecology dominate water policy and legislation. Other aspects of water management fall under the competence of the EU only to the extent their regulation is justified by its relevance to environmental protection. Consequently, the quantitative dimensions of surface water management are hardly addressed by EU

50 Art. 9.1 and 9.4, Convention between the Government of the Republic of Hungary and the Government of Ukraine on water management questions relating to frontier waters. Budapest, 11 November 1997.
51 Vuoksi Agreement on Discharge Rule in Lake Saimaa and the Vuoksi River, 1989.
52 Before the withdrawal of the United Kingdom from the European Union.

law. Second, EU water law is very closely linked to the broader environmental policy and legislation of the bloc, comprising a set of procedural obligations (impact assessment and authorisation of projects and plans) as well as substantive requirements affecting the ways water can be used (nature conservation constraints, industrial uses, pollution, etc.). Finally, environmental policy itself forms an integral part of the EU's the general policy framework. This implies that certain aspects of water management can be affected by other policy fields that fall outside the scope of environmental policy, such as agriculture and fisheries (water pollution, irrigation, aquaculture), transport (navigation), industrial policy (water use efficiency, water pollution) or general economic policy (provision of water services).

Linkages to international and national water law

As mentioned above, EU water law is a comprehensive supranational water governance scheme that – to a very large extent – functions independently from international water law. Yet, the two regimes do not exist in complete isolation. Their relationship can be best described as complementary. International water law is predominantly concerned with the use and protection of transboundary surface waters by riparian states, in other words: transboundary water governance. The usual topics of transboundary water governance include the quantitative management of surface water, economic uses of water (including navigation, hydropower generation, etc.), environmental protection, the management of hydrological variability in shared basins as well as the institutional frameworks of cooperation. Although the raison d'être behind regulating water at EU level is the presence (or likelihood) of transboundary impacts, EU water law addresses cross-border management questions surprisingly lightly. In fact, in this very context it mainly creates non-sanctioned procedural obligations for international river basin planning and management. Consequently, the more extensive scope and provisions of international water law usefully complement the somewhat unidimensional ecological approach of EU water law. Importantly, the EU is also party to a number of multilateral water treaties. Thus, these treaties must also be implemented by EU institutions and member states. In theory, they enjoy precedence over the EU's internal water legislation (even though collisions among the two regimes are hardly identifiable).

The structure of the EU legal order is such that national water governance regimes are subject to the supremacy of EU water law. It follows that the national legislation of member states must comply with the relevant policy objectives as well as the procedural and substantive obligations set by the EU. This, of course, does not imply that member states do not enjoy a considerable margin of discretion with regards to those aspects of water policy that are not regulated by EU water law. In fact, such critical questions of water management as surface water quantity management, economic utilisation of water, protection against hydrological extremes, property rights over water, regulating water services, water infrastructure management, etc. largely remain under national control. Here EU law is only relevant in so far as it defines distant constraints: no measure can be taken at national level that would jeopardise the attainment of the environmental objectives of EU water policy (e.g. good water status) or would otherwise run counter to the basic requirements of other policy fields (e.g. the provision of services).

The general legal framework of water management in the European Union

The Water Framework Directive

The centrepiece of today's EU water law and policy is Directive 2000/60/EC establishing a framework for Community action in the field of water policy, i.e. the Water Framework Directive (WFD). The WFD represents a broad overhaul of previous water policy and regulatory philosophy: it has either replaced or called for the gradual repeal of 25 years of previous EU water legislation, leaving only a handful of pre-WFD legislation in force. The broad framework of the WFD is complemented by two policy documents: the EU's 7th Environment Action Programme and the Blueprint to Safeguard Europe's Water Resources.[53]

The WFD lays down a comprehensive framework for the protection and the improvement of the aquatic environment in the Union that is supplemented by a set of water and environmental directives.

The WFD has a universal scope covering all inland freshwater (surface and groundwater) bodies within the territory of the EU as well as coastal waters. It also covers wetlands and other terrestrial ecosystems directly dependent on water.[54] Its regulatory approach is based on the integrated consideration of all impacts on the aquatic environment, extending the focus from purely chemical to biological, ecosystem, economic and morphological aspects.

The WFD establishes environmental objectives for surface waters, groundwater and so-called protected areas (areas designated under other EU legislation for their particular sensitivity for water). The environmental objectives for water are summarised as "good water status", described in the Annexes to the Directive by precise ecological, chemical and quantitative parameters. Importantly, the WFD considers quantitative issues as "ancillary" to water quality, conspicuously leaving surface water quantity to a regulatory grey zone. Member states are obliged to carry out extensive monitoring of the quality of the aquatic environment along EU-wide coordinated methodologies.

The planning and implementation framework of the WFD is the river basin. Member states are obliged to identify river basins in their territory and assign them to river basin districts (formal administrative management units comprising one or more basins).

[53] Communication from the Commission to the European Parliament, the Council, the European Economic and Social Committee and the Committee of the Regions: A Blueprint to Safeguard Europe's Water Resources, COM (2012) 0673 final.

[54] Art. 1, WFD.

If a river basin is shared by more than one member state it has to be assigned to an international river basin district.

The environmental objectives of the WFD have to be achieved through a complex planning and regulatory process that, in case of international river basin districts, requires the active cooperation of member states. The main administrative tools of member state action are the river basin management plans and the programmes of measures to be drawn up for each river basin district (or the national segment of an international river basin district).

The WFD lays down strict deadlines for the preparation of the management plans and for the compliance with the environmental objectives. As a general rule, all water bodies in the EU had to reach good status by the end of 2015. If, objectively, that was not possible and was clearly justified under any of the statutory exemptions specified under the Directive, good water status will have to be ensured by the end of the following planning cycle of 2021, or ultimately, by the final compliance deadline specified by the WFD, that is 2027.

Other water-related EU directives

The WFD, as its name suggests, provides only a framework for water policy. There exists a range of additional EU legislative acts addressing various specific water-related issues.

The first group of such measures is concerned with various sources of pollution or the chemical status of water. The most important such measure is the Urban Waste Water Directive,[55] the single most costly piece of environmental legislation ever to be implemented in EU history. It obliges EU member states to collect and subject to appropriate (i.e. at least biological) treatment all urban waste water above 2,000 population equivalent and the waste water of certain industrial sectors. Another important source of nutrient input, i.e. nitrates pollution from agricultural sources is regulated by the so-called Nitrates Directive.[56] It aims to protect surface and groundwater by preventing nitrates from agricultural sources polluting ground and surface waters and by promoting the use of good farming practices. Discharges into surface waters of the most prominent hazardous substances is governed by Priority Substances Directive[57] that sets limit values for 33 priority hazardous substances and 8 other pollutants with a view to their progressive elimination. The Groundwater Directive[58] establishes a regime which defines groundwater quality standards and introduces measures to prevent or limit inputs of pollutants into groundwater.

[55] Council Directive 91/271/EEC of 21 May 1991 concerning urban waste-water treatment.

[56] Council Directive 91/676/EEC of 12 December 1991 concerning the protection of waters against pollution caused by nitrates from agricultural sources.

[57] Directive 2008/105/EC of the European Parliament and of the Council of 16 December 2008 on environmental quality standards in the field of water policy.

[58] Directive 2006/118/EC of the European Parliament and of the Council of 12 December 2006 on the protection of groundwater against pollution and deterioration.

The EU's general industrial pollution legislation, the so-called Industrial Emissions Directive[59] (formerly: IPPC directive) lays down an integrated permitting system for the most important industrial installations, with strict conditions relating to surface water, groundwater and soil protection. It subjects all existing and future permits to a periodic review in light of the developments in the best available technique, a set of evolving industry-specific technological and management benchmarks.

Mention also must be made of the Drinking Water Directive[60] and the Bathing Water Directive[61] that regulate two important health aspects of water management: the prevention of water-borne diseases through the contamination of water intended for human consumption and the microbiological pollution of natural bathing waters.

EU environmental directives

Other EU environmental measures have important effects on water management. These include horizontal legislation such as the directives relating to environmental impact assessment and strategic environmental impact assessment,[62] access to environmental information[63] or environmental liability,[64] EU nature conservation measures, especially those concerning the Natura 2000 network[65] or the legislative framework on the EU's marine strategy.[66]

Flood protection under the Water Framework Directive

Flood protection as a derogation from the general environmental objectives

EU water law approaches water management from an environmental perspective. The WFD defines the general environmental objectives of EU water law and policy as follows:

59 Directive 2010/75/EU of the European Parliament and of the Council of 24 November 2010 on industrial emissions (integrated pollution prevention and control).

60 Council Directive 98/83/EC of 3 November 1998 on the quality of water intended for human consumption.

61 Directive 2006/7/EC of the European Parliament and of the Council of 15 February 2006 concerning the management of bathing water quality.

62 Directive 2011/92/EU of the European Parliament and of the Council of 13 December 2011 on the assessment of the effects of certain public and private projects on the environment; Directive 2001/42/EC of the European Parliament and of the Council of 27 June 2001 on the assessment of the effects of certain plans and programmes on the environment.

63 Directive 2003/4/EC of the European Parliament and of the Council of 28 January 2003 on public access to environmental information.

64 Directive 2003/35/EC of the European Parliament and of the Council of 21 April 2004 on environmental liability with regard to the prevention and remedying of environmental damage.

65 Council Directive 92/43/EEC of 21 May 1992 on the conservation of natural habitats and of wild flora and fauna, Council Directive 79/409/EEC of 2 April 1979 on the conservation of wild birds.

66 Directive 2008/56/EC of the European Parliament and of the Council of 17 June 2008 establishing a framework for community action in the field of marine environmental policy.

- the prevention of the further deterioration, protection and the enhancement of the status aquatic ecosystems as well as of terrestrial ecosystems and wetlands directly depending on the aquatic ecosystems
- the promotion of sustainable water use based on the long-term protection of available water resources
- the protection and improvement of surface water status, among others, through the progressive reduction of discharges, emission and losses of pollutants
- the progressive reduction of pollution of groundwater and prevention of its further pollution
- the mitigation of the effects of floods and droughts[67]

In view of these general objectives, the WFD also defines specific objectives for surface waters, groundwater and so-called protected areas so as to achieve the gold-standard of water management: good water status.

The presence of flood risks can influence the achievement of these objectives in multiple ways. Therefore, the WFD creates a number of temporary or permanent derogations from the environmental objectives of the WFD with reference to the imperative of flood control:

- Designation of heavily modified or artificial water bodies for flood protection: member states may define less stringent environmental objectives for so-called heavily modified or artificial water bodies. Notably, they do not have to achieve the so-called "good ecological status" for surface water bodies, only the more moderate conditions summarised as "good ecological potential". The WFD allows member states to designate such water bodies for the purposes of flood protection, if the achievement of good ecological status would have significant adverse effects on flood protection and that there are no significantly better environmental options that are technically and/or financially feasible.[68]
- Temporary derogation from the environmental objectives in case of extreme floods: the WFD makes it clear that member states may temporarily deviate from applicable environmental objectives for a particular water body, if non-compliance is the result of exceptional or unforeseeable floods. Even in such cases member states must, however, take all practicable steps to prevent further deterioration of all affected water bodies.[69]

Flood protection in the context of long-term adaptation to hydrological variability

Long-term adaptation to hydrological variability is a key element of the planning and implementation cycle of the WFD. Thus, it imposed an obligation on member states to

[67] Art. 1, WFD.
[68] Art. 4.1 and 4.3. Ibid.
[69] Art. 4.6, WFD.

undertake a detailed analysis of the main characteristics of each river basin by 2004 that had to contain an analysis of all relevant water uses, human and natural impacts on river flow and groundwater status, including abstractions. Ever since member states have had to continuously monitor any developments in these factors, including the volume and rate or level of flow.[70] The impacts of natural and man-made fluctuations in stream flow had to be reviewed by 2014 and appropriate adaptation measures had to be included in the revised river basin management plans and programme of measures.[71]

The administrative and implementation framework of flood protection

While the Floods Directive – as shown below – defines a work programme and set of instruments tailor-made to flood protection, such work programme and instruments are integrated into the broader scheme of the WFD as both water quality improvement and flood protection form a part of broader river basin management. Consequently, the Floods Directive builds upon the administrative setup of "river basin districts" established under the WFD.[72] The WFD's implementation toolkit, notably the river basin management plans (RBMPs) and the programmes of measures (POMs), must also take floods into consideration.

River basin management plans are an innovative tool for the basin-wide management of surface and groundwater resources, aquatic and related terrestrial ecosystems. RBMPs must contain the following minimum:

- description and characterisation of the river basin, including the environmental assessment of human activities, economic assessment of water uses, description of pollution sources and risk analysis of failing to achieve the objectives
- the list of environmental objectives and exemptions established for surface and groundwater
- the list of protected areas
- the map of the monitoring stations
- the measures to achieve cost recovery for water services
- the summary of programme of measures and specific additional measures to achieve the environmental objectives[73]

In case of international river basins member states are required to ensure co-ordination and co-operation with the aim of producing one single international River Basin Management Plan. If such an international RBMP cannot be produced for some reasons, member states are still responsible for producing River Basin Management Plans for the parts of the international river basin district within their territory.[74]

[70] Art. 5 and 8. Ibid.
[71] Art. 5, Annex VII, WFD. Also see [2].
[72] Art. 3, Directive 2007/60/EC.
[73] Annex VII, WFD.
[74] Art 3. Ibid. See also Section III.3.1.a) above.

The comprehensive nature of RBMPs requires that the issue of floods is addressed in the plans as a critical condition influencing the achievement of the environmental objectives of the WFD. Where floods or the requirements of flood defence are expected to lead to long-term or temporary derogations from the environmental objectives, this must be specifically identified in the River Basin Management Plans.[75] The same applies to the programmes of measures, which are compilations of regulatory and administrative tools for the implementation of the environmental objectives of the WFD in the context of a particular river basin district. While not a formal requirement under either directives, member states are nonetheless encouraged to integrate flood risk management plans into river basin management plans. These plans are also meant to be implemented ideally with a synchronised timing as well as a coordinated consultation and reporting process.

Floods Directive

The regulatory approach: Risk assessment and management planning

The EU's key legal act in the field of flood management is the so-called Floods Directive.[76] The directive was adopted in 2007 in response to the growing number of devastating inundations in various parts of Europe. It represents an outlier in EU water law as its main objective is safety (rather than environmental quality), it is closely linked to water quantity management (a constitutional misfit in EU law) and its focus is transboundary water cooperation (instead of parallel domestic actions).

Importantly, the Floods Directive does not address flood management in the general sense of the term, but tackles the issue from a risk management perspective. Consequently, its regulatory approach is mainly of procedural character, focusing on the assessment of flood risks and the planning of flood risk management. Thus, the directive does not address such fundamental questions of flood protection as infrastructure development and maintenance, spatial planning measures, etc. in any substantial fashion.

It, nonetheless, lays down certain basic requirements the national and international management of floods must meet:

- the objectives of flood management must focus on the reduction of potential adverse consequences of flooding for human health, the environment, cultural heritage and economic activity
- flood risk management plans must take into account the following key factors:
  - costs and benefits
  - flood extent and flood conveyance routes
  - areas which have the potential to retain flood water, such as natural floodplains
  - the environmental objectives of the WFD

[75] Art. 4.3. and 4.6. Ibid.

[76] *Directive 2007/60/EC* of the European Parliament and of the Council of 23 October 2007 on the *assessment and management of flood risks.*

- soil and water management
- spatial planning, land use, nature conservation
- navigation and port infrastructure

– flood risk management must cover prevention, protection, preparedness, including flood forecasts and early warning systems, taking into account the characteristics of the particular river basin or sub-basin

Preliminary flood risk assessment, flood hazard and flood risk maps

Under the Floods Directive, EU governments are (were) required to carry out a preliminary flood risk assessment and, subsequently, to establish flood hazard and flood risk maps.

As a first step, member states were required to complete, by the end of 2011, a preliminary assessment of flood risks. This initial assessment served as a filter to identify areas subject to flooding and eliminate those where floods are unlikely to occur or their negative impacts remain insignificant. The preliminary assessment contained, among others, a description of the floods which have occurred in the past and which had significant adverse impacts on human health, the environment, cultural heritage and economic activity. It also had to include an assessment of the potential adverse consequences of future floods on all these conditions.[77]

For areas identified as being at potentially significant risk of flooding, flood hazard maps and flood risk maps had to be drawn up by the of 2013.[78] For other areas, no further steps are necessary.

Flood hazard maps show the probability of different flood events. They had to be drawn up for three different scenarios that could appear within a geographical area:

– floods with a low probability, or extreme event scenarios
– floods with a medium probability (likely return period ≥ 100 years)
– floods with a high probability

Each of the three scenarios must be described through the following elements:

– flood extend
– water depths or water level
– where appropriate, the flow velocity or the relevant water flow[79]

Flood risk maps, on the other hand, outline the potential adverse consequences associated with the above flood scenarios (Figure 1). They must display such adverse consequences broken down along the major subjects of flood protection as follows:

– the indicative number of inhabitants potentially affected

[77] Art. 4.2. Ibid.
[78] Art. 7. Ibid.
[79] Art 6.3–4, Directive 2007/60/EC.

- the type of economic activity of the area potentially affected
- installations covered by the Industrial Emissions Directive
- other information which a member state considers useful[80]

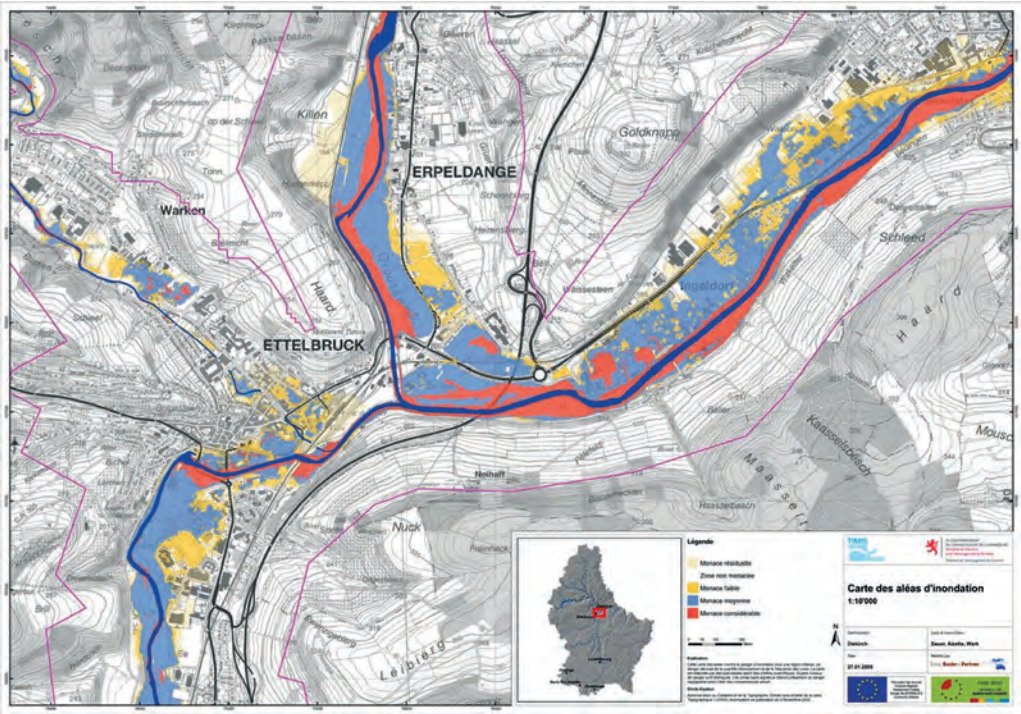

Figure 1. Flood risk map of Ettelbruck, Luxemburg (www.climatetechwiki.org/content/flood-hazard-mapping)

Flood risk management plans

Based on these maps, member states must adopt flood risk management plans that are coordinated at basin or at least sub-basin level (the original deadline was end of 2015). The starting point of flood risk management plans are specific objectives that member states must establish for each of those areas that have been identified with potential significant flood risks or likelihood. These objectives must focus on the reduction of potential adverse consequences of flooding for human health, the environment, cultural heritage and economic activity. Member states must also consider the consequences of non-structural initiatives (e.g. green infrastructure) and the reduction of the likelihood of flooding.[81] Attention must be paid to the environmental objectives of the WFD with

80 Art. 6.5. Ibid.
81 Art. 7.2, Directive 2007/60/EC.

regards to surface water, groundwater and protected areas as well as the requirements of other dependent sectors such as spatial planning, soil management, land use, nature conservation, navigation and port infrastructure.[82]

Flood risk management plans must be based on a comprehensive approach towards flood prevention and control. In terms of geographical scope, they must cover the entire flood extent and flood conveyance routes, including areas which have the potential to retain flood water, such as natural floodplains. The scope of measures must extend to all aspects of flood risk management including prevention, protection and preparedness, taking into account the characteristics of the particular river basin or sub-basin. Plans may also include the promotion of sustainable land use practices, improvement of water retention as well as the controlled flooding of certain areas.[83]

An important, but rather general substantive requirement vis-à-vis flood risk management plans is that they cannot include measures which, by their extent and impact, significantly increase flood risks upstream or downstream of other countries in the same river basin or sub-basin (unless these measures have been coordinated and an agreed solution has been found among the member states concerned).[84]

The main content of the flood risk management plans is defined by the directive as follows:

- conclusions of the preliminary flood risk assessment (only for the first plan) in the form of a summary map of the river basin district, delineating those areas where potential significant flood risks exists or might be considered likely to occur
- flood hazard maps and flood risk maps and the conclusions that can be drawn from these maps
- description of the appropriate objectives of flood risk management
- summary of the measures and their prioritisation aiming to achieve the appropriate objectives of flood risk management – this should also include flood related measures taken by other EU environmental directives, e.g. those relating to environmental impact assessment and strategic environmental assessment, industrial installations with major accident hazards, etc.
- description of monitoring
- summary of the public information and consultation measures/actions taken
- list of the national competent authorities and, in case of international river basin districts, description of the relevant cross-border coordination process[85]

The regular updates of flood risk management plans every six years must also include:

- any changes since the publication of the previous version of the plan, including the summary of reviews

[82] Art. 7.3. Ibid.
[83] Ibid.
[84] Art. 7.4. Ibid.
[85] Annex, Part A, Directive 2007/60/EC.

- an assessment of the progress made towards the achievement of the objectives of flood risk management
- description of, and an explanation for, any measures foreseen in the earlier version of the flood risk management plan which were planned to be undertaken and have not been taken forward
- description of any additional measures since the publication of the previous version of the plan[86]

Coordination with the Water Framework Directive

Implementation of the Floods Directive must take place in close coordination with that of the Water Framework Directive. Such coordination has multiple dimensions.

As mentioned earlier, the assessment and management units of floods are the river basin districts defined by the Water Framework Directive, although exceptionally member states may assign individual river basins to a unit of management different from those under the WFD (i.e. they can divide the river basins differently for flood control).

As in the case of river basin management plans, if a river basin district (or other nationally determined administrative unit of flood protection) is shared by more member states, they have to coordinate with a view to producing one single flood risk management plan (or a set of harmonised plans) for the entire international river basin district. This, however, is not an obligation of result. Should such coordination efforts fail, member states just have to go ahead with their individual (uncoordinated) plans. Where an international river basin district falls partly outside the EU, member states are merely required to "endeavour" to produce a single flood risk management plan. If no such plan is eventually produced, individual member states must adopt their own national plan(s).[87]

Given that the original timeframe of implementation of the WFD and the Floods Directive differ significantly, the latter does not call for the merger of the two systems of plans. Yet, the Floods Directive urges member states to coordinate the application of the two directives "focusing on opportunities for improving efficiency, information exchange and for achieving common synergies and benefits". To that end they must ensure that the development of the first flood hazard maps and flood risk maps and their subsequent reviews are carried out in such a way that the information they contain is consistent with relevant information gathered and used under the Water Framework Directive. Evidently, this requires the coordination of river basin management plans and flood risk management plans and allows member states to integrate the former into the larger framework of river basin management plans.[88]

86 Ibid.
87 Art. 8.2. and 8.3, Directive 2007/60/EC.
88 Art. 9. Ibid.

Public participation

The full protection of all relevant values and interests against floods cannot be ensured at reasonable costs to society. Therefore, flood risk management plans must take into the costs and benefits of flood defence when prioritising interventions. Given the conflict potential of flood risk optimisation, the Floods Directive aims to ensure public participation in and transparency of the planning of flood risk management. This includes the following obligations on the part of the competent authorities:

- publication of all relevant documentation: member states must make available to the public the documents of the preliminary flood risk assessment, the flood hazard maps, the flood risk maps and the flood risk management plans
- active involvement of all interested parties: member states must "encourage" active involvement of interested parties in the production, review and updating of the flood risk management plans

This process must be coordinated with the engagement of the public under the Water Framework Directive with regards to adoption and review of river basin management plans.[89] Where flood risk management plans and river basin management plans are produced together, the public participation requirements of the WFD apply automatically. It means that all draft plans must be published one year before adoption, allowing at least six months for comments.[90] For the most probable case, however – i.e. when flood risk management plans are produced independently from the WFD planning cycle – the Floods Directive does not specify concrete procedural steps. Mutatis mutandis, however, a consultation process that is identical or at least similar to the WFD's should be undertaken.

Implementation of the EU's flood management regime

According to a scoreboard published by the European Commission, the Floods Directive has been transposed into national legal systems in time in all EU member states. Moreover, not a single member state has failed to meet the deadlines for the preliminary flood risk assessment, the flood hazard and flood risk maps, respectively.[91] This is a remarkable achievement in view of the complexity of the task, but also vis-à-vis the mush more inconsistent implementation record of the WFD. Similarly impressive is the compliance rate with the requirement to produce flood risk management plans. Here, only three member states out of 28 have failed to deliver the plans in time (Figure 2). Such impressive compliance figures seem to suggest that member states consider flood protection a high priority and find the toolbox of the Floods Directive adequate.

89 Art. 9 and 10, Directive 2007/60/EC.
90 Art. 14.1, WFD.
91 Floods Directive Scoreboard, http://ec.europa.eu/environment/water/flood_risk/implem.htm

National experiences

Following the deadline for the completion of the flood risk management plans (i.e. end of 2015), the European Commission carried out a survey with a view to assessing member states' initial experience with regards to the implementation of the Floods Directive.[92] Since the implementation cycle of the flood risk management plans had just begun, the survey could not evaluate the real impacts of the directive on effective flood control. Yet, it could already identify, even at this early stage, the main impacts of the Floods Directive on national water governance and transboundary cooperation.

Figure 2. Adoption of flood risk management plans by EU member states (by 2018) (http://ec.europa.eu/environment/water/flood_risk/implem.htm)

Green: Reporting for all Units of Management with significant flood risk

Yellow: Reporting for some, but not all, Units of Management with significant flood risk

Red: No reporting

One of the most significant organisational impacts of the Floods Directive identified by the survey is the fact that it helped enhance the collaboration and coordination among different sectors (e.g. water, disaster management, emergency planning) on the one hand, and among various decision-makers and other stakeholders, on the other hand.[93] As a result, the directive actually influenced policy areas outside water in a considerable way. The Floods Directive also contributed to the consolidation of national and international

[92] For more information see https://circabc.europa.eu/sd/a/8768cbc2-85f3-428f-b859-f9aee7a27e56/FD%201st%20cycle%20questionnaire%20report_formatted_07%20March%202017.pdf

[93] Ibid. 25.

methods and measures for flood risk management. Previous diversity (or often cacophony) of plans and methods was replaced by a systematic, coordinated and holistic approach to flood control. This was manifested in changes in legislation and policies, prioritisation of measures, reorganisation of administrative competences, etc.[94]

Against all these positive developments, the implementation of the Floods Directive continues to pose significant challenges. On the technical side problems have been identified with regards to lack and quality of data, the absence of methodology and models for certain types of floods, the handling of uncertainties, etc. On the organisational/administrative side, difficulties include inadequate financial and human resources, coordination among stakeholders with different competences and/or at various geographical scales, the proper engagement of the public, etc. The latter aspect is particularly important as experience in many member states reveals a very limited interest or engagement by the public in the planning procedure, even though various groups of stakeholders – including the general public – have different flood protection priorities that should be reconciled.[95]

International river basins

All major transboundary river basins in the European Union are subject to a governance treaty that (usually) establishes an implementation body in the form of the joint commission of riparian states. Since flood protection has been one of the key areas of transboundary water cooperation from the outset, these basin commissions have taken a central role in the coordination of the implementation of the Water Framework Directive as well as the Floods Directive. In fact, some basin commissions – such as the International Commission for the Protection of the River Danube (ICPDR) – even played a key role in the conceptualisation of the Floods Directive.

Therefore, the ICPDR also serves as the coordination platform for the implementation of the Floods Directive and for the preparation and update of the Danube Flood Risk Management Plan. Its relevant activities, however, predate the adoption of the directive. In relation to the catastrophic floods in the Danube basin between 2000 and 2002, the ICPDR undertook a comprehensive flood mapping and planning exercise that resulted in the adoption in 2004 of the basin's key strategic document entitled *Action Programme on Sustainable Flood Protection in the Danube River Basin*.[96] The goal of the Action Programme was to achieve a long-term and sustainable approach for managing the risks of floods to protect human life and property, while encouraging conservation and improvement of water related ecosystems. The Action Programme only created an overall framework whose objectives had to be operationalised at lower geographical scale. Therefore, it called for the preparation of flood action plans for all sub-basin in

94 Ibid. 46.
95 Ibid. 123, 125.
96 For more information see www.icpdr.org/flowpaper/app/services/view.php?doc=ICPDR_Flood%20_Action_Programme.pdf&format=pdf&page={page}&subfolder=default/files/

the Danube catchment area. This was achieved by the end of 2009 when 17 flood action plans for the sub-basins of the Danube were adopted by the ICPDR. Importantly, the flood action plans for sub-basins paved the way for the implementation of the EU Directive on Floods in the Danube River Basin.

Even though not all Danube riparian states are members of the EU, in 2010 all basin countries committed themselves to implement the EU Floods Directive throughout the whole Danube river basin and to develop one single international Flood Risk Management Plan or a set of flood risk management plans, based upon the ICPDR's Action Programme for Sustainable Flood Protection and the sub-basin plans. As a first step of this process the preliminary flood risk assessment has been completed for the entire basin by December 2011. This was followed by the preparation of flood hazard and flood risk maps for individual member states as well as for the entire basin.[97]

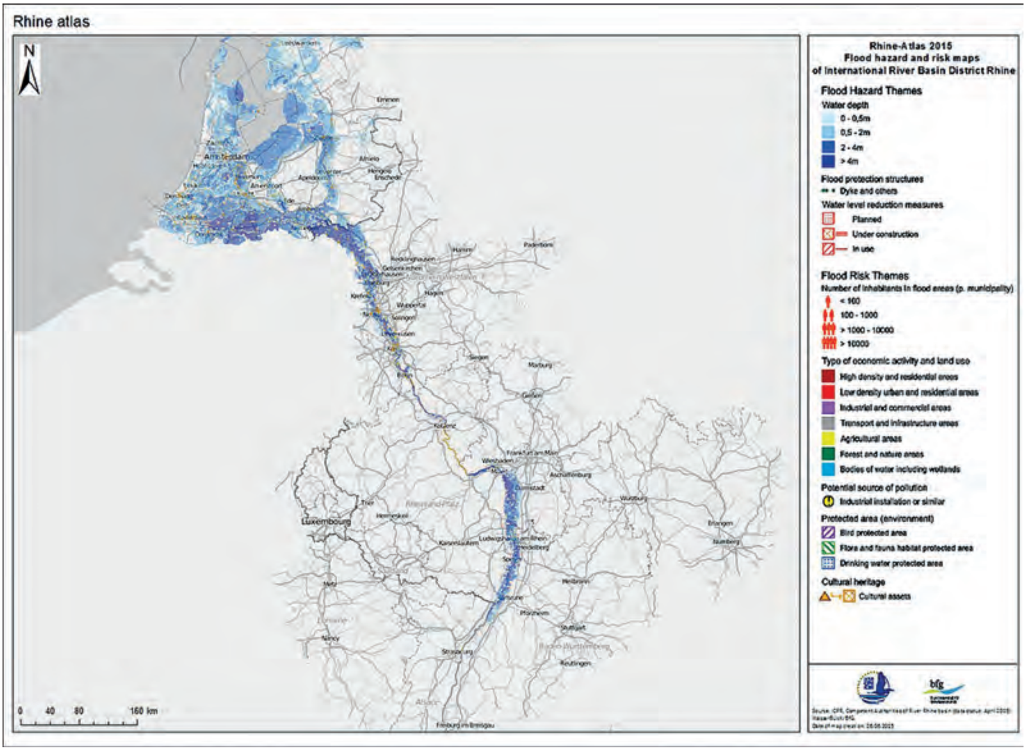

Figure 3. The Rhine Atlas (www.iksr.org/en/documentsarchive/rhine-atlas/)

While the Rhine Protection Convention is less explicit about the coordinated prevention and control of floods than the Danube regime, this has not prevented Rhine riparian states to task the river basin organisation – the International Commission for the Protection

[97] For more information see www.icpdr.org/main/activities-projects/danube-floodrisk-project and www.icpdr.org/flowpaper/app/#page=1

of the Rhine (ICPR) – with the coordination of the implementation of the Floods Directive. The preliminary flood risk assessment for the entire basin and the development of the ensuing flood hazard and risks maps has been completed by 2015, summarised in a publication of the ICPR entitled *Rhine Atlas* (Figure 3).

The EU Solidarity Fund: Post-flooding financial assistance

As shown above, the Floods Directive creates a framework for cooperation to prevent and manage floods with a view to avoiding significant damage to persons, property, infrastructure and the natural environment. Importantly, however, the EU's scope of action is not limited to damage prevention and control as member states (and EU candidate countries) can apply for financial assistance to finance ex post certain emergency measures necessitated by major flood events. Such financial assistance is provided by the so-called EU Solidarity Fund (EUSF) that was established in response to the devastating floods in Central Europe in 2002.

Disbursements from the EUSF are limited to public expenditure related to relief operations. Thus, it does not cover private claims, long-term restoration, infrastructure construction for future floods, etc. The underlying legislative act – Regulation 2012/2002[98] – defines eligible actions as public expenditure for the following essential emergency operations:

- restoring the working order of infrastructure and plant in the fields of energy, water and waste water, telecommunications, transport, health and education
- providing temporary accommodation and funding rescue services to meet the needs of the population concerned
- securing preventive infrastructure and measures of protection of cultural heritage
- cleaning up disaster-stricken areas, including natural zones, as well as immediate restoration of affected natural zones to avoid immediate effects from soil erosion[99]

Funds from the EUSF can be mobilised only in the case of "major disasters" at national or regional scale. National floods qualify as "major disasters" where the ensuing direct damage exceeds EUR 3 billion (in 2011 prices) or 0.6% of the GNI. "Regional natural disasters" are those that result in damage in excess of 1.5% of the affected region's GDP.[100]

While the EUSF is also available to cover natural disasters other than floods (e.g. earthquakes, droughts), since its establishment in 2002 eligible countries have mainly applied for funding mainly for flood-related emergency operations.[101]

[98] Council Regulation (EC) No 2012/2002 of 11 November 2002 establishing the European Union Solidarity Fund.
[99] Art 3. Ibid.
[100] Art 2. Ibid.
[101] For more information see https://ec.europa.eu/regional_policy/sources/thefunds/doc/interventions_since_2002.pdf

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Convention on the Protection and Use of Transboundary Watercourses and Lakes. Helsinki, 17 March 1992.

Convention on Cooperation for the Protection and Sustainable Use of the Danube. Sofia, 29 June 1994.

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Antje Bornschein

Flood and Excess Water Control Techniques and Technologies

The general subject of floodwater and excess water control

The EU Floods Directive (Directive 2007/60/EC on the assessment and management of flood risks) came into force on 26 November 2007. Member States were required to assess if watercourses and coastlines are at risk from flooding. In a first step, the flood extent, assets and people at risk in these areas should be mapped. In a second step, the flood risk should be reduced by adequate and coordinated measures. The directive states that the public has the right to access this information and to have a say in the planning process.

All measures to reduce the flood risk should be in accordance with the EU Water Framework Directive (Directive 2000/60/EC of the European Parliament and of the Council establishing a framework for the Community action in the field of water policy). Therefore, it is not allowed to make the water environment worse while implementing flood damage reduction measures.

In Art. 2 of the EU Floods Directive the term “floods” is defined as follows: “flood” means the temporary covering by water of land not normally covered by water. This shall include floods from rivers, mountain torrents, Mediterranean ephemeral water courses, and floods from the sea in coastal areas, and may exclude floods from sewerage systems.

The sources of floods and excess water are described in section *Sources of excess water.* This includes pluvial flooding, fluvial flooding, coastal flooding and groundwater flooding. Section *The management of excess water within the catchment* includes non-structural and structural measures to reduce run-off as well as to decrease the flood peak discharge. In section *The management of excess water within floodplains and along rivers* measures are more focused on reducing flood-induced damages by preventing flooding entirely or by adapting buildings and infrastructure to flooding. Section *The management of flood risk* contains further information about the management of flood risk. Definition of flood risk, the flood risk cycle and information about flood early warning systems are presented. The EU Floods Directive focuses on flood damage assessment and mapping the flood extent. This can be done by analysing historical flood events. Another technique is hydrodynamic modelling which uses numerical methods to calculate the water level in a river reach for a certain design flood event. Using GIS tools, the flooded area can also be determined. A short introduction to the numerical methods to calculate flood extent is given in section *Modelling floods.*

Sources of excess water

Pluvial flooding

Local rainfall and snowmelt can create pluvial floods. The amount of water from rain or melted snow that accumulates on the surface depends on the surface characteristics, the rainfall event and the topography of the area affected.

The surface in the area affected can be characterised by the surface material, soil type, land cover and land use. In order to describe the rainfall event, we use the rainfall intensity and duration. If the rainfall intensity is higher than the infiltration capacity water remains on the surface and accumulates in local flow paths and low lying areas. In an urban area, flow paths can be on the surface or under the surface (in the sewerage or sewer system). In low lying areas, overland flow can be increased when water is flowing out from the sewer or stormwater system.

Urban areas, where for the most part the surface is paved, areas with soils with low infiltration rates or areas with an even surface profile are more prone to pluvial floods. Rainfall on a frozen surface is also prevented from infiltration. Water from melting snow can add to the runoff in that case.

Fluvial flooding

Fluvial floods occur if the runoff and/or groundwater enter the waterways and the river water levels rise above a certain threshold. Fluvial floods depend on the rainfall event and valley and river characteristics.

Figure 1. Fluvial flooding at the river Elbe (left photo taken by the author, note the temporal flood protection walls) and coastal flooding in Wismar, Germany (right photo taken by Matthias Menzel)

Rainfall events with high intensity in small catchments with a steep land surface slope like mountainous catchments can create so-called flash floods. They are characterised by a short time to peak as well as a relatively short flood event time, critical or supercritical flow, significant sediment transport and mobilisation of additional floating debris like tree trunks and rootstocks. It is likely that sediment and floating debris block narrow river cross sections, bridge cross sections or weir cross sections during such an event increasing the water level upstream of these structures additionally.

The term "fluvial flood" is often assigned to long-lasting floods in wide river valleys after long-lasting rainfall and snowmelt events in wider catchment areas (see Figure 1). The flood event depends firstly on the intensity and duration of the rainfall as well as the additional contribution to the flow by melting snow. The river catchment, particularly its size, shape, soils and land use are important here. The level in the waterways themselves is affected by river characteristics such as size and shape. In addition, land elevation, soil material, land use and land cover are influencing the flow propagation. The material of the river bed and the vegetation on its banks determine the main channel flow. Structures in and adjacent to the river like weirs or dikes are important to consider, too. The land elevation of the floodplain determines mainly the local recurrence period of flooding [1]. In case of floods, flood water levels depend significantly on the land cover and the land use in the floodplains. Higher and rigid vegetation decelerates the flow and is connected to a higher water level.

Groundwater flooding

During a rainfall event, water infiltrates into the soil and flows in the groundwater layer towards areas with lower surface level. Over the years, these low lying areas often developed into urban areas. Groundwater flooding is more likely to occur in areas with permeable soils (aquifers). In general, the groundwater level rises after long periods with high rainfall. Due to the low flow velocity of groundwater, the flooding is likely to occur over a long period because the water has no other paths to flow away.

The flooding depends on the rainfall characteristics and the type of soil and rocks which determine the underground flow paths. But also man-made flow paths (e.g. sewer pipes or stormwater pipes) can contribute to these events.

Buildings can be damaged with or without visible exfiltration of water. In urban areas, the high groundwater level can flood basements or underground car parks, tunnels for streets or public transport. In agricultural areas, the gropes can also be affected by high groundwater levels.[1]

Coastal flooding

When coastal areas which are normally dry are flooded by seawater, this is called coastal flooding (see Figure 1). Higher sea levels can be originated from unusually high tide level, storm surges or tsunamis. In addition, the sea level itself is rising [20].

[1] Further information can be found e.g. on www.groundwateruk.org/faq_groundwater_flooding.aspx

Exceptional high tide levels at the time of new moon or full moon together with storm surges due to strong onshore wind can create very high sea water levels. The flooding is than limited to the time of the high tide but flooding can occur during several high tide intervals in a row. The city of Venice is a well-known example for an urban area prone to flooding due to high tide and onshore wind. Last exceptional flooding occurred on the 29th of October 2018 [25].

The onshore wind itself may cause the water to pile up onto the shoreline. Together with the wave run-up of the wind-induced waves sea water can reach very high levels. The duration of these storm surges depends on the duration of the storm event. The wind wave run-up can erode the shore and sea levees or damage other coastal protection structures like sea walls. In case of a levee or sea wall failure wide areas which may be lower than the normal sea water level e.g. areas in the Netherlands can be flooded [26].

Another cause for coastal flooding is a tsunami wave. Tsunami, which means harbour wave in Japanese, is created by a seaquake. If the sea bottom moves significantly during a seaquake huge amounts of water are moved and a wave on the sea surface is created. These waves propagate in all directions and are characterised by relatively shallow wave height and very long wavelengths [27]. A tsunami wave is a so-called shallow water wave so its propagation velocity depends only on the local depth of the sea. When the tsunami reaches the coastal area, the propagation velocity decreases and the wave height increases. That is why it is called "harbour wave" because it is more distinguishable near the coast and near a harbour than on the open sea. Tsunami waves create a significant flow of seawater into the coastal areas and are responsible for huge damages there. The high water level together with the high flow velocity mobilise a lot of sediment and debris, like ships, cars, trees, etc. and transport them into the hinterland. Examples are the tsunami which occurred on the 26th of December 2004 due to the Sumatra–Andaman earthquake in the Indian Ocean and the tsunami on the 11th of March 2011 due to the Tohoku earthquake in Japan. But many tsunami waves in the Mediterranean Sea are documented, too [19].

The management of excess water within the catchment

Structural and non-structural measures

The management of excess water aims at reducing flood-induced damages e.g. by reducing flood peak discharge and maximum water level or by preventing urban areas or agricultural areas from flooding. Possible measures are divided into two groups: non-structural and structural measures. Non-structural and structural measures can be applied in the catchments as well as along the main watercourse.

Non-structural measures aim to improve flood awareness and flood management as well as try to change the characteristics of the catchment area and with that the creation of floods itself. Financial preparedness of people who live in the floodplains can be increased by taking out flood damage insurance. Proper spatial planning and early flood warning systems can reduce flood risk by reducing possible flood damages. Further information is found in sub-section *Early flood warning systems.*

Changes in land use like reforestation and other measures at smaller tributaries can reduce the amount of runoff and with that the flood peak and volume of the flood wave. More information is given in sub-section *Structural and non-structural measures.*

Structural measures aim at reducing the flood discharge, the flood water level or at the protection of the buildings and infrastructure along rivers. The measures discussed in the following sections are:

- the improvement of rainwater retention and infiltration in the urban watershed (sub-section *Small scale retention measures*)
- the creation of flood retention basin as permanent storage or flood detention basin as temporary storage (sub-section *Retention measures in urban areas*)
- the installation of levees, flood-walls or mobile flood protection elements (sub-section *Flood protection structures*)
- the flood control channel or flood relief channel (sub-section *Flood control channels*)
- dike relocation and flood polder (sub-sections *Dike relocation* and *Flood polder*)
- a flood-proof home design increasing the resilience of buildings and allowing for flooding without or with only little damage (sub-section *Flood-proof home design*)

Small scale retention measures

The creation of run-off during a rainfall event depends on land use, soil characteristics and rainfall intensity and duration. In order to increase the infiltration and retention of water on the surface, land use change and smaller green infrastructure or nature-based solutions can be applied.

In general, these measures have a small effect on floods with small return periods and tend to have no effect on extreme flood events. Nevertheless, if applied in a whole catchment, their effect can sum up. Normally, these measures cost much less than traditional (so-called grey infrastructure). These retention measures may provide additional services like new habitats for birds and insects, recreational areas, preventing erosion or mitigating droughts [9].

In general, these measures are uncontrollable, automatic and potential water retention capacity is difficult to predict. They should be carefully assessed regarding their cost and water retention effect including all the additional ecosystem services, too.

As an example, reforestation can increase water infiltration significantly. Nevertheless, the selection of areas for reforestation may be restricted by other land uses like agriculture.

Other small retention measures aiming at delaying the run-off in a catchment include the creation of plants protective belts, woodlots shrubs or the creation of terraces [11]. Measures on agricultural land include the selection of proper agro-technics, the increase of organic matter content in the soil, anti-erosion measures or the run-off regulation from drainage systems. The widening of the area of wetlands, peatlands and swamps and rewetting of peatland are also possible. Their main objective is to increase water storage in these areas.

Retention measures in urban areas

Urban areas with their paved and sealed surface are prone to fast run-off during a rainfall event. Impervious surfaces include roofs, streets or parking lots. Other highly sealed areas are ports, airports or construction sites. The soil sealing in percent of the urban zone ranges from over 20% to almost 80% for European capitals (see Figure 2).

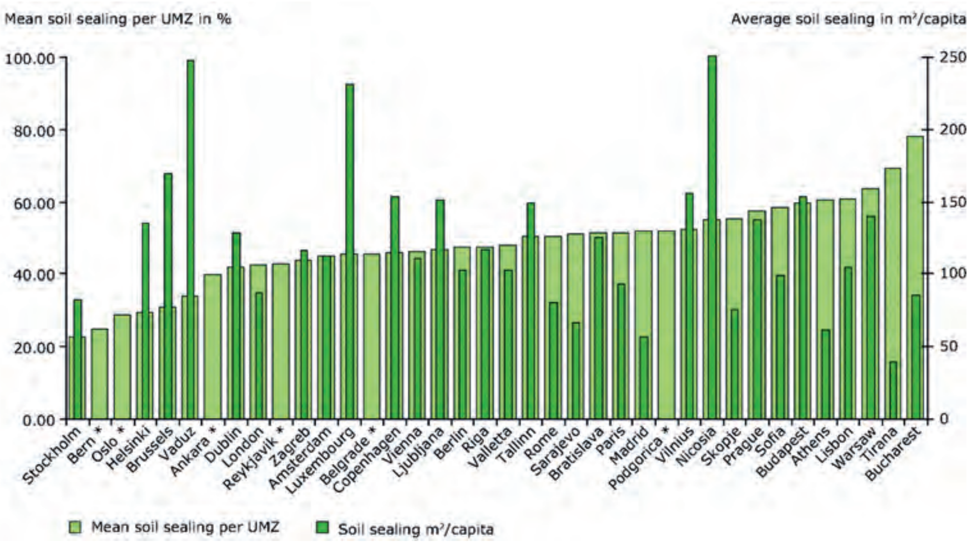


Figure 2. Soil sealing in European capitals [28]

Regarding urban areas, retention measures aim at reducing the run-off by increasing infiltration, storing it in areas where no damage occurs or delaying it by temporal storage.

Possible measures are:

- dams and reservoirs (see sub-section *Dams and reservoirs*)
- ponds and infiltration wells for storage and increasing infiltration
- temporal flooding of playgrounds or sports grounds
- green roofs for storage and delaying run-off
- the adaptation of streets as temporal water flow ways

All these measures are summarised in the concept of a “Sponge city” [7]. Implementing this concept needs a combined effort of hydraulic engineering, land-use planning, urban politics and the citizens themselves because many measures are implemented on private land.

Dams and reservoirs

In order to hold back water in the upper parts of a river catchment, dams can be used. When building a dam, a reservoir is created in the area where the water is stored. The design of dams depends mostly on their height. In general, a large dam is a dam with a height of ≥ 15 m above the lowest foundation or a dam between 5 metres and 15 metres high impounding more than 3 million cubic metres (so-called ICOLD[2] criterion [29]).

Reservoirs (see Figure 3) can have a permanent water volume stored behind the dam (permanently filled reservoirs, flood retention basins) or can be empty or partially filled during normal weather and only completely filled when an increased inflow due to rainfall or melting snow occurs (non-permanently filled reservoirs, flood detention basins). The advantages of non-permanently filled reservoirs are free migration routes for water-bound animals like fish and amphibians. There is no permanent disruption of the wildlife corridor because during normal weather the river flows through the empty reservoir like in the natural parts of a valley.

The reservoir can be situated in the main valley or beside the river like a polder (see sub-section *Flood polder*) or at some distance from the river like the upper reservoirs of pumped storage power plants. In order to fill the latter, additional channels or pipes have to be built.

The dam is a structure resisting the hydrostatic pressure force of the stored water column. Different types of dams are established:

- gravity dams made from masonry or concrete
- earth filled dams made from compacted soil layers with (zoned dam) or without (homogenous dam) additional sealing structures
- rock filled dams built using rock fragments or large boulders with additional sealing structures
- roller compacted concrete dams
- arch dams made from concrete
- buttress dams made from concrete

According to the World Register of Dams with 59,071 registered dams, the majority of large dams in the world are earth dams (38,426 dams, 65.1%). The next biggest categories are rock filled dams (7,670 dams, 13.0%) and gravity dams (7,450 dams, 12.6%). The following categories have much fewer dams registered: buttress dams (419 dams, 0.7%), barrages (280 dams, 0.5%), arch dams (2,332 dams, 3.9%) and multiple arch dams (133 dams, 0.2%). Then there are 2,361 dams (4.0%) which are categorised as "others".

Much less common and often small are metal dams and timber dams.

[2] ICOLD: International Commission on Large Dams.

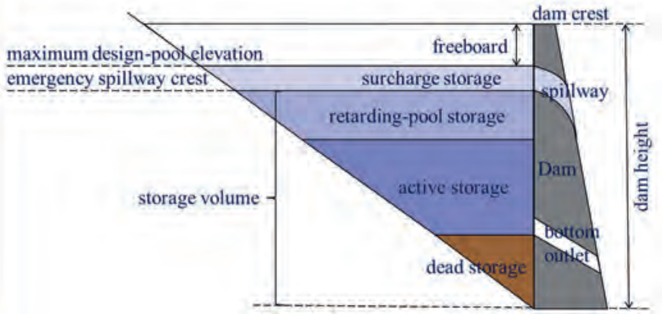


Figure 3. A dam and its reservoir: oblique view with 1 – dam, 2 – water surface in case of a permanently filled reservoir or in case of flood inflow for an empty or partially filled reservoir, 3 – water surface of a partially filled reservoir when there is no flood inflow, 4 – emergency spillway (left), definitions regarding structures, water levels and storage volume for a permanently filled reservoir (right) (compiled by the author)

The volume of water below the minimum pool level is called the dead storage (see Figure 3). This volume can be filled with sediment without endangering the correct operation of the dam and reservoir.

During normal weather and in case of a permanently filled reservoir, water is kept in the dead storage and the active storage. In case of a design flood event, the flood water will be stored in the retarding-pool storage and the reservoir water level will rise until it reaches the emergency spillway crest. This means a full reservoir level. For many dams, this design flood event is a 1-in-50-years or 1-in-100-years flood event. The recurrence period depends on the dam height or storage volume. No outflow over the emergency spillway happens. No damages downstream should be caused by the maximum total outflow discharge during this event.

If a larger flood event occurs, the additional water will be stored in the surcharge storage. Water will flow over the emergency spillway. The type and the dimensions of the emergency spillway have to be chosen in a way that the reservoir water level does not exceed the maximum reservoir level in case of this flood event. In general, flood events for designing the emergency spillway have a recurrence period of 100 to 200 years for smaller dams and up to 1,000 years for large dams.

The general dam safety is assessed using flood events with recurrence periods between 500 and 1,000 years for smaller dams and up to 10,000 years for large dams. No damage to the structure itself has to occur. Minor damages can affect infrastructure or measurement devices at the dam which are not vital for dam operation.

The difference between the dam crest level and the maximum reservoir level is called freeboard. The freeboard is necessary to accommodate additional water level increasing effects e.g. wind wave run-up, water level raised by wind or waves induced due to landslides.

Emergency spillways are very important structures regarding the operation of dams in case of floods. Their type and design have to be appropriate to the type of dam.

Spillways can be located within the body of the dam, at one or both sides of the dam or even completely separated from the dam as a by-pass spillway.

Common types of emergency spillways are (see [22]):

- free overfall or straight drop spillway (for thin arch dams e.g. the Gebidem dam, Switzerland)
- ogee or overflow spillway (the water flows over a weir crest parallel to the dam crest, often at gravity dams, e.g. Eibenstock dam, Germany)
- side channel spillway (the water is collected by a side channel with a weir crest perpendicular to the dam crest in a horizontal plane, after that the water could either flow through an open channel or a tunnel to the dam toe, e.g. Malter dam, Germany)
- chute or open channel or through spillway (the water is conveyed through a very steep channel to the dam toe, often used at earth dams, e.g. Mosul dam, Iraq)
- conduit or tunnel spillway (e.g. reservoir Sylvensteinspeicher, Germany)
- drop inlet or shaft or morning glory spillway (water drops through a vertical shaft to a horizontal conduit that conveys the water past the dam, often used for earth dams, e.g. Monticello dam, California, USA)
- siphon spillway (used mainly for concrete dams, e.g. Murrum Silli dam, India)

Because the water flow accelerates while passing over the spillway or dropping down freely as a water jet, the energy of the flow has to be dissipated before the water enters the downstream river course. Slowing down the water means to create a hydraulic jump which is the transition from supercritical to subcritical flow. The structure where the hydraulic jump should occur is called a stilling basin.

In order to calculate the needed storage volume of a reservoir (retarding-pool storage), it is stated that the change of the water storage volume dS within a reservoir or polder equals the difference between the reservoir inflow Q_{in} and reservoir outflow Q_{out} during the short time period dt:

$$\frac{dS}{dt} = Q_{in} - Q_{out} \tag{1}$$

The inflow into a reservoir consists of river inflow, overland runoff from the valley sides and precipitation within the reservoir itself.

The outflow is defined by outlet structures like mid-level and/or bottom outlets, a spillway or a turbine. The calculation of the outflow discharge takes into account different formula for these different outlet structures.

The calculation is based on the relation between the reservoir level and the storage volume, which can be derived from topographic data. When calculating reservoir retention, the storage as well as the outflow discharge depends on the reservoir water level. That is why an iterative solution is needed.

It is important to note that the peak discharge occurs when the water level within the reservoir reaches its maximum. At the same time, the reservoir storage volume S has its maximum (see Figure 4). This is the case when the outflow structures are fully opened.

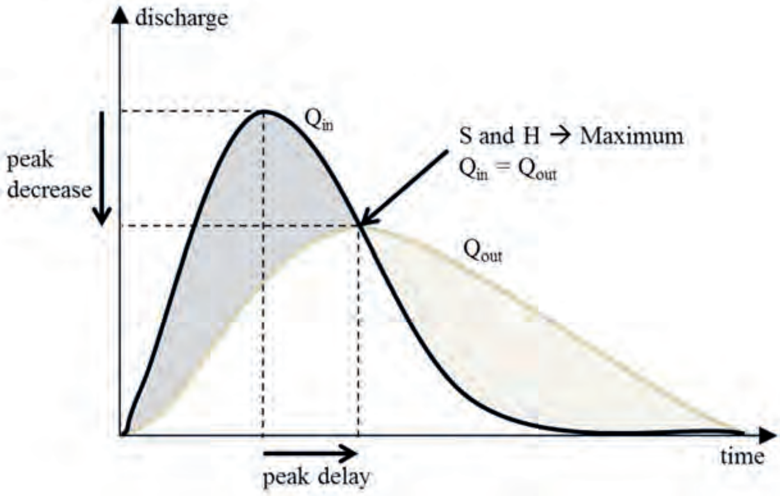


Figure 4. Inflow and outflow hydrograph for a reservoir with fully opened outflow structures, the coloured areas represent the required storage volume S (reservoir flood retention) (compiled by the author)

For a design flood event, the required reservoir storage and dam height respectively depend on the maximum allowable reservoir outflow Q_{out}. The value has to be defined such, that the flow downstream a reservoir or polder is retained within the river banks and no overflow is created. This also depends on the land use in this area because an urban area is more worth protecting than agricultural land. In general, it can be said, the higher the allowable outflow the lower the required storage volume and vice versa (Figure 5).

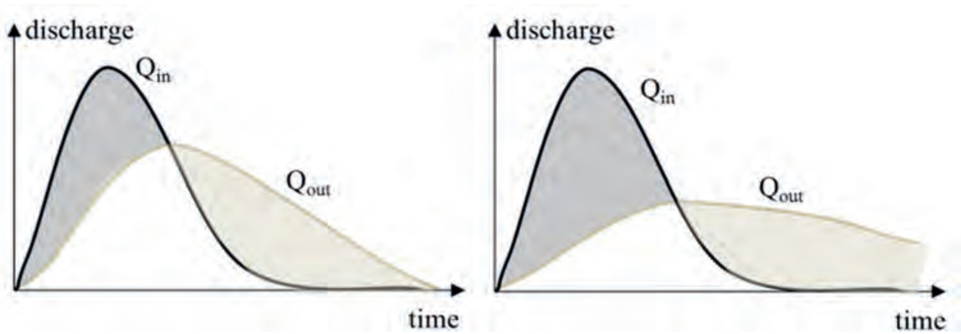


Figure 5. Reservoir flood retention: A higher allowable reservoir outflow in the left picture is connected to a lower required reservoir storage volume (coloured areas) (compiled by the author)

The water volume which is stored in a reservoir or polder is controlled by regulating reservoir outflow. This is an advantage of these flood control structures. In case of the design event or a smaller event, the decreasing effect on the flood peak is very high. Even if an extreme flood event happens, the flood peak can be significantly delayed.

Data from the 2002 flood event in Saxony, Germany [17] or from other flood events [2] [16] showed that very clearly.

If hydrological data e.g. design flood hydrograph are based on short historical flow series, it is possible that they underestimate possible extreme flood events. This is sometimes the case with older dams because when they were built hydrological knowledge was not as accurate as today. But what happens when the flood control or retaining capacity (reservoir volume) is too small compared to the flood wave volume? If the flood storage volume is already filled and the inflow exceeds the spillway capacity, the dam crest could be overtopped. Overtopping is a major cause for dam failure which can lead to catastrophic flooding downstream [18].

The management of excess water within floodplains and along rivers

Flood routing in rivers and floodplains

In case of a flood, water overflows the river banks and streams into the floodplain. If the tributaries do not add further water from their sub-basins, the flood peak declines during flood wave propagation and the flood duration gets longer. This process is called river flood retention. To illustrate this, three river reaches with their according discharge hydrographs for a hypothetical flood event are shown in Figure 6. The discharge hydrograph for the most downstream river reach has a lower peak discharge and a longer duration of high flow than the most upstream river reach.

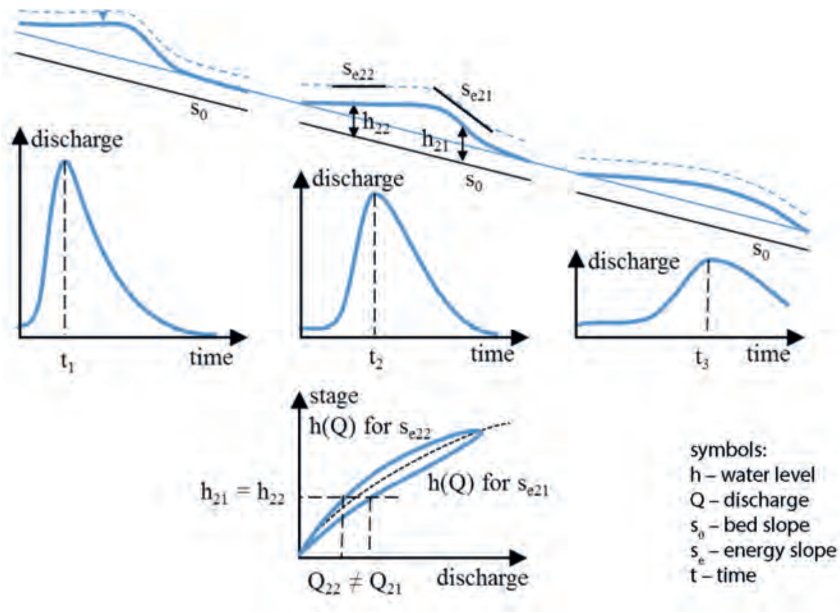


Figure 6. Characteristics of the flow and the hydrograph at three selected cross sections of a river (compiled by the author)

The diagram below shows the hysteresis effect due to the change in energy gradient during flood propagation.

River flood retention is governed by two major processes which are significantly influenced by the land use and land cover within the floodplain.

In general, the water moves slower over the floodplain than within the main channel. In wide floodplains, it is also possible that the water travels with such a low velocity that the floodplain creates an effect equal to a reservoir. That part of the water volume that travels in the floodplain reaches the downstream end of a certain river reach significantly later than the water traveling in the main channel. This lowers the flood peak and makes the flood duration longer.

The other process is driven by the difference in the energy slope during an increasing and a decreasing flood flow. During the rising limb of the flood hydrograph, the water level increases over time. Thus, the energy slope is higher than the longitudinal bottom slope in a river reach (Figure 6, cross section 21). When the flood peak occurs, the energy slope and longitudinal slope are equal for a moment. That is why it is feasible to use quasi-steady conditions when calculating long flood events. And at the falling limb of the flood hydrograph, when the water recedes, the energy slope is lower than the longitudinal bed slope (Figure 6, cross section 22).

Looking at Manning's Formula [8]

$$Q = A \cdot \frac{1}{n} \cdot {r_{hy}}^{2/3} \cdot \sqrt{s_e} \tag{2}$$

with A – cross-sectional area, n – Manning's coefficient, r_{hy} – hydraulic radius, s_e – energy slope, one can easily derive, the gentler the energy slope the smaller the discharge in a channel. It can be concluded, that the flow near the flood peak is more decelerated than the flow during the rising limb of the flood hydrograph. And the flow during the falling limb of the flood hydrograph is more decelerated than the flow around the flood peak.

The relation between the discharge in a certain river reach and the water level is very complex. As the energy slope is higher during the rising limb of the flood wave (see Figure 6 with $s_{e21} > s_{e22}$), so is the velocity. That is why a higher discharge is connected to the same water level ($h_{21} = h_{22}$) when the water level is rising than when the water level is falling. Thus, for the stage–discharge–curve in Figure 6 follows $Q_{21} > Q_{22}$. For this reason, the time when the maximum water level occurs is not equal with the time when the discharge reaches its peak. The maximum water storage in a river reach happens later than the peak discharge. This is called the hysteresis effect.

If a floodplain is covered with higher vegetation e.g. wood (see Figure 7) the flood peak discharge downstream the inundation area would be lower and the flood wave duration longer as in the case of an inundation area used as meadow or field (Figure 8). Accordingly, water would remain longer within the inundation area and the flood water level there would be higher [10].

Figure 7. Riparian forest at the river Elbe near Meißen (left) and agricultural land in a flood plain (right; photos taken by the author)

However, this effect has a lower impact on extreme and seldom floods than on smaller and more frequent floods.

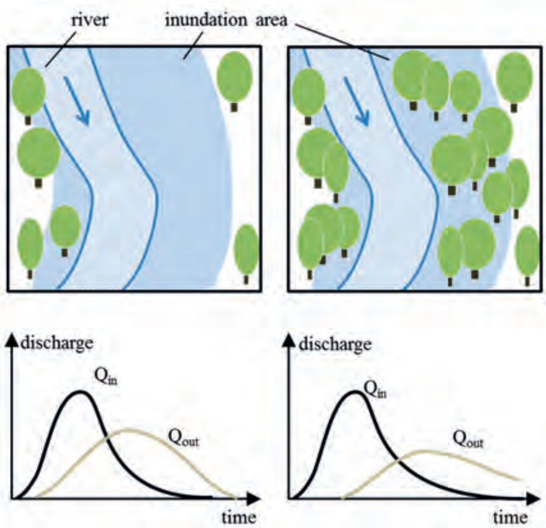


Figure 8. Two river reaches with floodplains characterised by different land use (above = oblique view) representing a lower roughness (left) and higher roughness (right) (compiled by the author)

The diagrams below show the flow hydrographs at the beginning (Q_{in}) and at the end (Q_{out}) of the river reach.

The reestablishment of floodplain woodland and the promotion of river meandering are two measures of river and floodplain restoration projects which try to employ river retention. It will take a long time between the planting of floodplain woodland and any significant effect on flood flows. But in order to initiate these projects, it would be good to quantify the effectiveness of this type of woodland as a mechanism for lowering the flood flow. The effect of high vegetation on flood wave propagation can be modelled using hydrodynamic models (see section *Modelling floods*).

The flood control function of floodplain woodland along a 2.2 km long reach of the River Cary in the United Kingdom was modelled by [15]. The river reach was assumed to be completely forested. This would mean the establishment of 133 ha wet woodland. The study showed an increase in flood storage by 71% as well as a delay of the flood peak arrival downstream by 140 min in case of a 100-year-flood.

Additionally, river retention depends on the characteristics of the river. 200 years ago, rivers were more shallow and wider (Figure 9). In order to use rivers for transport, engineers took measures like dragging and cutting of meanders which created deeper and narrower river beds. Focusing on flood retention it can be beneficial to reverse this development. "The Wise Use of Floodplains" project [1] investigated the potential of reinstating a river channel to its pre-engineered state. This would induce more frequent flooding of its floodplains and with that an increasing effect of flood retention. The study included calculations for the Cherwell catchment in the United Kingdom. It was shown that the flood flow downstream the study area could be reduced by 10 to 15% if the river channel and floodplain are restored to its pre-engineered dimensions.

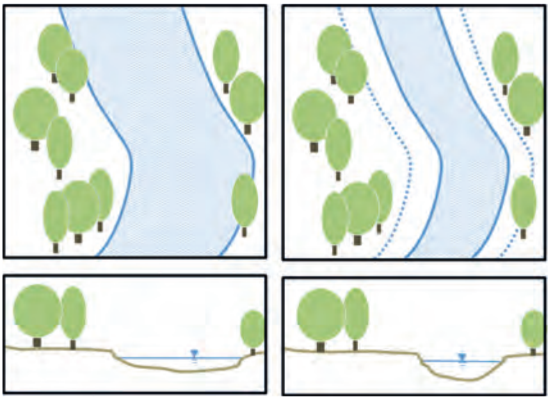


Figure 9. One river reach in its pre-engineered state (left) with a more shallow and wider cross section and in its engineered state (right) with a deeper and narrower cross section (above: oblique view, below: cross section) (compiled by the author)

The implementation of these measures could be restricted if the river is used for transport (navigable river).

Flood protection structures

Flood protection structures along rivers or along the coastline are levees or dikes, flood walls or demountable flood barriers or demountable flood protection structures. They prevent areas from flooding by increasing the local surface level. The design height of a flood protection structure depends on the local ground level, the design water level and the height of an additional freeboard.

The local design water level can be set to a certain height, can be the water level of a historical flood event or can be calculated by means of hydrodynamic models (see section *Modelling floods*) for a design flood event. The freeboard height is necessary to accommodate additional water level increasing effects e.g. wind wave run-up, water level raised by wind or the backwater effect of ice jams.

Levees or dikes are permanent embankments. They are situated in parallel to the coastline or the river channel axis and made out of the local soil. In general, their surface is covered with grass. This provides protection against erosion and additional sealing. Embankments need a lot of free space because their two faces are sloped between 1V:3H to 1V:6H. A 5 metres high dam with a dam crest width of 2 metres and a riverward side sloped 1V:3H and a landward side sloped 1V:2H has a width at its foundation of 27 metres.

Flood walls are more suitable for urban areas with less open space available. They are also permanent structures. They can be made out of concrete, masonry, steel or even glass. They have to withstand the hydrostatic pressure force and have to be well grounded to prevent tilting. Their construction design has also been checked if it can withstand the impact of floating debris.

Demountable or moveable flood barriers can be of various shapes, design and out of very different materials. They are non-permanent structures. They are often used to close gaps in permanent flood protection structure like wall openings for access to the riverside or street crossings. A certain time is needed to establish them. A good flood forecast is necessary to apply these structures. The construction should be easy enough to be conducted fast and get a reliable structure at the end. The structures have to be stored elsewhere in a time of normal river water level. The design has to withstand against the hydrostatic pressure force as well as against the impact of floating debris. The waterproof connection to permanent structures like buildings or walls is a critical point regarding demountable flood barriers.

The simplest and most common form of a moveable flood protection barrier is a sand-bag wall. Other structures are made out of aluminum beams (e.g. in Prague, at the right bank, downstream the famous Charles Bridge), also big water filled bags, sand filled big bags or inclined systems are used.

Flood control channels

In order to accelerate the flood flow or to redirect flood discharge away from an urban area flood control channels or flood relief channels are established. Two different set-ups are possible.

Firstly, the channel can start downstream an urban area. When a certain water level is reached, the flow discharge starts to enter the flood relief channel. Due to the bigger flow cross section, the water level decreases increasing the local energy slope for the flow in the upstream urban area and accelerating the upstream flow. This effect is only temporal but may help to contain the flood water within the set flood protection structures.

An example is the system of the two flood relief channels at the river Elbe in the city of Dresden, Germany. The flooding of the two channels starts at different flood water levels.

Figure 10. Inflow structure "Pretziener Wehr", controlling the inflow discharge to the flood relief channel near Magdeburg, Germany (photo taken by the author)

Secondly, the flood relief channel starts upstream of the urban area. The flood flow is bypassed reducing the flow discharge in the river itself. An example is the "Elbeumflut", an older river bed of the river Elbe near Magdeburg, Germany. This old river bed is only filled in case of a flood event. The filling is controlled by a weir (Pretziener Wehr) built in 1875 with 9 sluice gates (see Figure 10).

Dike relocation

Sometimes it is necessary to build a new levee more afar from a river. This can be in order to give "room to the rivers", to build a higher dike, to build the dike on a more suitable ground or in order to get a better layout of the dike line. An existing levee would then be removed and an additional area could be inundated in case of a flood (Figure 11, left). The setback of levees is one of many flood retention measures which improve ecological parameters as well [6].

If the existing levee remains beside the new levee both can create a so-called polder (see sub-section *Flood Polder*). Two additional structures – an inflow and an outflow structure – have to be built in order to control the moment when polder filling starts and to regulate the inflow and the outflow discharge (Figure 11, right).

The main difference between these two scenarios is the controlled flooding in case of a polder versus the uncontrolled flooding in case of a levee setback [3]. If there is a flood event with a reliable flood forecast, the polder inflow is controlled aiming at lowering the flood peak (see Figure 11). The additional flood area created by the levee setback is flooded always when the water level in the main channel reaches the bank level and is filled as the water level rises. It could be possible that the flood area is already filled when the flood peak reaches the location of the levee setback and no significant peak reduction is achieved.

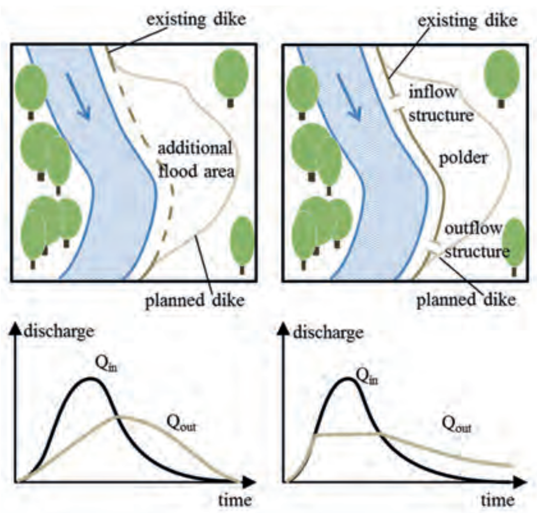


Figure 11. A river reach (above, oblique view) with an additional flood area due to a levee setback (left) or a polder (right). Below are the corresponding flood hydrographs upstream and downstream of the considered river reach (compiled by the author)

Assuming a quite short river reach with subcritical flow conditions which in general applies to our rivers and considering the Bernoulli-Theorem

$$z + h + \alpha \cdot \frac{v^2}{2 \cdot g} = const. \qquad (3)$$

with z – height above a datum, h – water depth, α – kinetic energy correction factor, v – flow velocity and g – gravity acceleration constant, together with the principle of continuity

$$Q = v \cdot A = const., \qquad (4)$$

it can be stated that a wider flood cross section due to a levee setback causes a lower flow velocity and hence an increasing water depth.

It is important to know which length will be needed for a levee setback reach to get a water level reduction instead of a higher water level. This so-called effective minimal length of levee relocation L_{Aeff} can be estimated using the following formula [10]:

$$L_{Aeff} = \frac{h_0}{s_0} \cdot \left[\frac{r_h \cdot h_3}{h_o} - \frac{h_3}{h_0} + \left(1 - {Fr_0}^2\right) \cdot f_R\left(\frac{h_3}{h_0}\right) - f_R\left(\frac{r_h \cdot h_3}{h_0}\right) \right] \qquad (5)$$

with h_0 – normal depth, s_0 – bed slope, h_3 – flow depth downstream of a dike relocation and Fr_0 – Froude number at normal flow conditions (see Figure 12). The expression $f_R(x)$ depends on the shape of the river cross section. As a first estimation, the formula below for a rectangular shaped cross section can be used [10]:

$$f_R(x) = \frac{1}{6} \cdot ln\left(\frac{x^2+x+1}{(x-1)^2}\right) + \frac{1}{\sqrt{3}} \cdot arctan\left(\frac{1+2\cdot x}{\sqrt{3}}\right) \tag{6}$$

with x as a general independent variable. The water depth ratio $r_h = h_1/h_3$ can be derived from:

$$r_h{}^3 - \left(\frac{Fr_3{}^2}{2} \cdot (1+\varsigma_c) + 1\right) \cdot r_h{}^2 + \frac{Fr_3{}^2}{2} \cdot \frac{(1+\varsigma_c)}{r_b{}^2} = 0 \tag{7}$$

With ς_c – a coefficient describing the local energy loss at the downstream end of a dike relocation, $r_b = b_1/b_3$ and Fr_3 – Froude number downstream the levee relocation.

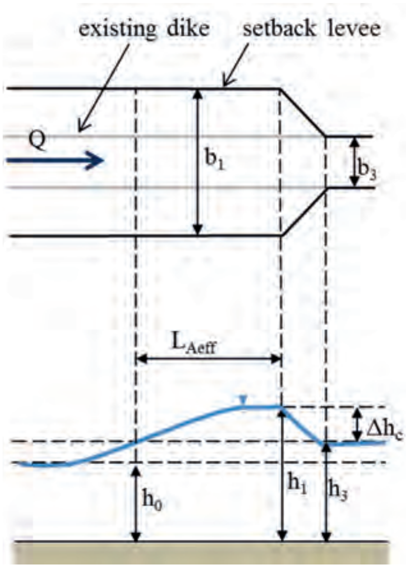


Figure 12. Definition of some parameters (see equations 5–7) within a levee relocation reach (plan view – above) (compiled by the author)

There is a lower water depth within the levee relocation reach and upstream of it if the length of a dike relocation is longer than L_{Aeff}.

[10] stated further, that levee relocations reduce the peak discharge only in case of smaller floods. For floods with higher recurrence periods very wide and long additional flooded areas are needed to achieve a significant effect. The longer and the wider a levee setback area, the higher the decreasing effect on the flow depth. Additionally, it was found that peak reduction is higher in case of shorter and steeper flood hydrographs than in case of long-lasting floods.

Dike relocations provide additional ecosystem services which have to be included in cost–benefit analyses. In the last decades, there was a huge improvement in the knowledge and perception of the river-floodplain system as an ecosystem. Areas which are flooded regularly may provide habitats for plants and animals from the Red Red List (see www.iucnredlist.org/).

Flood polder

There are two main definitions of the term “polder”. One meaning, originating from Dutch, describes a low-lying area which is usually flooded by the sea or a river and which was separated from the water body by a levee. This area would then be drained and used for agriculture and settlements. Here, the protected area is situated inside the levee which is often formed as a ring.

The extreme flood events in the last centuries in Europe let to another point of view regarding these protected areas. If no settlements were created and the land is only for agriculture, it can also be used to store flood water temporarily. In that case, the area which is flooded is inside the levees. The latter meaning is the one which will be used further.

A polder is an area alongside a river. Its filling in case of a flood event can be controlled by an inflow and an outflow structure. These are normally weirs with gates like sluice gates, flap gates, mitre gates or radial gates. As an example, the polder Löbnitz at the river Mulde (Germany) has a weir with flap gates as an inflow structure and a weir with mitre gates as an outflow structure. The old dikes prevent filling during flood events with a recurrence interval of $T \leq 25$ years. If a larger flood occurs, the polder area is flooded providing up to 15 million m^3 additional flood storage.

The controlled flooding is an advantage of a polder because it gives a significant potential for reducing downstream flows [1]. If there is a flood event with a reliable flood forecast, the polder inflow could be controlled aiming at lowering the flood peak (see Figure 11). In the last two decades, many polders and polder systems which can significantly influence the flood flow were newly established at big European rivers. Additional structures are still in the planning stage. Some examples are:

- river Havel (Germany), this polder system is able to reduce the flood peak in the river Elbe, too
- river Rhine, e.g. polder Ingelheim and polder Söllingen/Greffern in Germany, further measures are planned (see program “Rhein 2020”)
- river Danube, e.g. polder Riedensheim in Germany (under construction), further measures are planned but they are highly controversially discussed by stakeholders

In case of a more technical approach and with focus on flood retention, land use in polders should be restricted in order to maintain a maximum storage volume and to propagate a fast inflow and filling process [3]. In addition, land use should not stimulate additional sedimentation of fine river sediments. That is why polders are often used as grassland.

In addition, this temporarily flooded land can offer additional ecosystem and other services like providing habitats or the opportunity for recreation. Then optimising the additional uses is mandatory and the polder area should consist of different types of land use like riparian woodland, wetland and agricultural land.

Near the inflow and outflow structure, flow velocity is higher and the bottom should be covered with a material which is less prone to erosion. Inflow and outflow structures are constructions including a paved or concrete bed, concrete pillars and a control room.

The management of flood risk

Flood risk

The EU Floods Directive states in its Art. 2 that 'flood risk' "means the combination of the probability of a flood event and of the potential adverse consequences for human health, the environment, cultural heritage and economic activity associated with a flood event".

In general, flood risk r can be calculated using the following formula:

$$r = c \cdot f \tag{8}$$

where c stands for the consequences of a flood event and f is the recurrence frequency of this flood event.

The consequences include all the negative impacts of a flood event equal to the definition of the EU Floods Directive. Consequences for the human health include injuries or death but also long-lasting effects of mental stress. Floods can destroy habitats with their flora and fauna and can have long-lasting effects like changes in the ecological system. Cultural heritage sites like old city centres with their old churches, libraries and town halls or monasteries are often very prone to flooding because the material of the buildings is old and standards in design and construction were lower in former centuries. In addition, cultural heritage sites may hold many old and valuable artefacts which can also suffer from damages due to the flood water. The economic activities in an inundated area suffer immediately during the flooding because of lost production but also afterward when a lot of money is needed for cleaning and rebuilding.

If it is possible to quantify all the consequences, the risk can be expressed in cost per year.

The flood damage depends mainly on the type of the structure and the local water level as well as the flow velocity. In order to calculate the possible damages, so-called flood damage functions are used. They are empirically established based on data of historical flood events about real damages and flood characteristics [24].

Flood risk management and flood resilience

Flood risk management includes all measures relating to floods, like measures affecting run-off creation or flood flow, damage mitigation measures, flood rescue missions, land management and also financial schemes providing money in case of flood damages [12].

The so-called flood risk management cycle includes all the measures and shows their relation to each other (see Figure 13).

When a flood event is about to happen, response measures follow first. The flood forecast centre gives information about possible flood water levels, flood warnings are distributed, fire brigades or inhabitants prepare for the flood. They establish mobile flood protection structures and monitor other flood protection structures like levees.

If necessary, roads and bridges are closed, the evacuations of potentially flooded buildings start and affected inhabitants are provided with temporal shelter, food and other aid supplies.

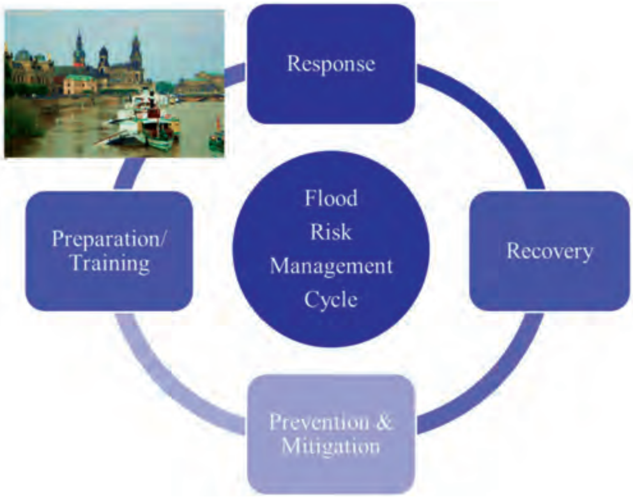


Figure 13. The flood risk management cycle (compiled by the author)

After the flood, recovery measures are important. Basements or underground parking lots have to be pumped empty, flood damages at buildings, infrastructure and the waterways have to be repaired. Financial schemes to help the affected people or communities are very important here.

After recovery or even during this time, it is important to assess the causes of flood damages and to take possible measures to mitigate or prevent them should the same flood event occur again.

The next step in the flood risk management cycle is preparation and training. This focuses on implementing further flood mitigation measures and maintaining a high level of flood awareness and preparedness. Flood awareness and preparedness usually decrease over time [4]. High flood water marks at prominent buildings, exhibitions or special TV programs about historical flood events and cultural events help to remember past floods.

Flood resilience describes the ability of communities to deal with floods. Small flood-induced damage together with a fast recovery after a flood is connected to high flood resilience.

Communication about floods and its consequences has been significantly enhanced over the last two decades. Information to the public is easily available on many governmental websites (e.g. www.chiefscientist.qld.gov.au/publications/understanding-floods/; www.gov.uk/browse/environment-countryside/flooding-extreme-weather).

Hazard maps, risk maps or inundation maps are important for exact action planning in case of a flood. Flood risk management plans together with decision support systems and appropriate maps provide information to take quick and adequate measures. Operational forces need information about the area affected, required measures as well as lists with affected people and companies, hospitals, senior residents, schools and others. Management plans and flood maps should contain information about frequent as well as rare and extreme flood events to cover all possibilities.

Early flood warning systems

In urban areas, structural flood protection measures are often restricted by limited space, environment protection or the attempt to conserve the cityscape. That is why demountable flood protection elements (see sub-section *Flood protection structures*) or non-structural measures like an early flood warning system might be more welcome by the inhabitants to mitigate potential flood damage.

A flood warning system provides information about the flood arrival and peak water level necessary for flood damage mitigation and rescue measures by fire brigades, private persons and companies. The success of an early flood warning system depends on the availability of forecast models and their input data as well as short information paths (see Figure 14) and clearly and understandably formulated warning messages [5]. A flood warning system is one of the non-structural measures with best cost–benefit ratio. In Europe, there are many flood warning systems in operation but only a few are applied to small and urban catchment areas.

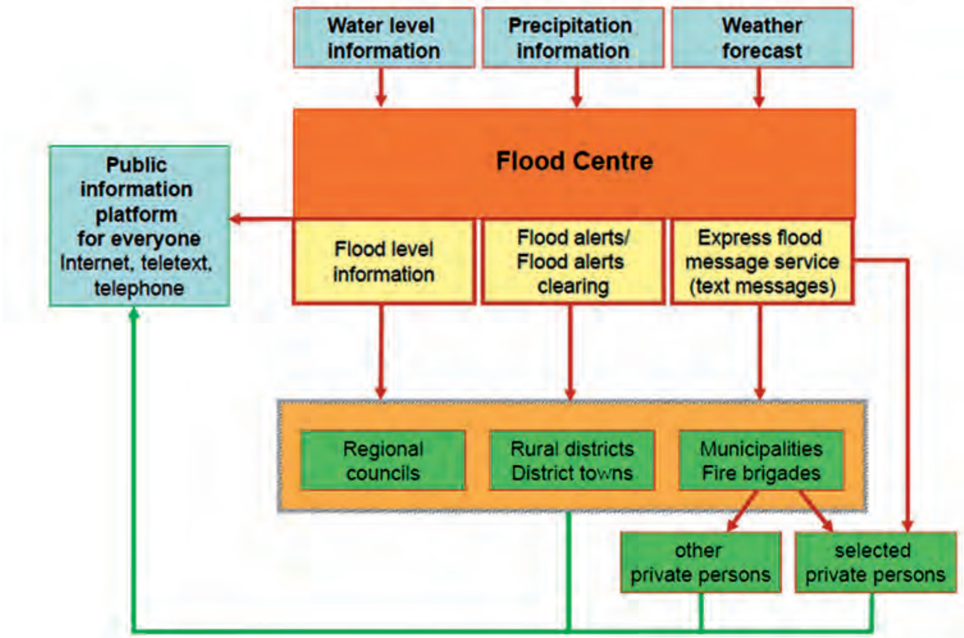


Figure 14. Example for short distribution paths within a centrally organised flood warning system [5]
Note: Flood Centre stands for an organisation which is responsible for the distribution of early flood warning messages

Flood centres are responsible for centralised data collection, running of forecast models and warning messages distribution to all affected entities at the same time. They should be established on a national scale in smaller countries or on the scale of federal states, regions or wider catchment areas in larger countries. It has to be assured that all data transmission to and from the flood centre is uninterrupted.

It is as important to get accurate information about upcoming flood events in small catchment areas as a very fast warning message distribution. In Figure 14 an example of an advanced flood warning system is shown with short information paths and simultaneous warning distribution to local administration, responsible fire brigades or other rescue workers and inhabitants.

In general, flood warning systems are characterised by a discontinuous information path or warning chain. The staff of a flood warning centre have to gather the raw data first and run some checks to eliminate errors from the measurement devices like river discharge gauges or errors due to data transmission or to validate the results of rainfall-runoff models. After that, warning messages are released [5]. In the case of flash floods, this approach shortens the available forecast lead time significantly.

Automatic flood warning systems are today research topics or in the planning stage. Here, the link between the forecasted rainfall and the flood-induced damage is very complex. Automatic warning systems concerning accidents and disasters involving hazardous material are very common. Automatic warning systems concerning rockslides or landslides are in operation lately but operational experiences are still not available. In these cases, damage-inducing processes are directly related to mostly local impacts. We can learn from these systems that automatic warning systems in small (catchment) areas should be highly equipped with different types of sensors to obtain valid data and to provide the opportunity to check released warning messages. In addition, local observers and uninterrupted observation and/or measurement systems can help to compensate for the general lack of knowledge.

The prediction of rainfall in small catchment areas is not exact today. Small catchment areas are often prone to convective rainfall events which are smaller than the spatial resolution of today operational national weather models. Measurement data of rainfall are only local (weather stations) or only qualitative (radar data). Transformation in quantitative areal data needs time and errors can occur. Rainfall data in small catchment areas with no or few rainfall gauges should be improved by information of local observers and installation of additional rainfall gauges.

Flood forecast models including rainfall-runoff calculation are very complex models which need a lot of input data. Inaccuracies in modelling are mainly based on the real-time input data like precipitation or soil moisture. When including river gauge measurements, the amount of input data decreases while the accuracy increases. The flood wave propagation modelling uses hydrodynamic models. These models are more common for rivers with middle and large catchment areas. A short introduction to hydrodynamic numerical modelling is given in section *Modelling floods.*

Short and complete warning message distribution is essential to give the inhabitants of a city or town the vital hours they need to prepare and protect their properties. Action forces have to establish demountable flood protection structures. Rescue workers need this information to prepare the evacuation of flood-prone areas. Short warning dissemination chains are important for the quick warning. Multiple information distribution paths (acoustic devices, TV, radio, Internet) assure that affected as well as concerned inhabitants get all needed information if available.

Weather and flood forecast is characterised by increasing uncertainty considering higher forecast lead time. In a very early state, flood warning messages are often vague and refer only to lower flood water levels which cause no extra precautions. If an event proceeds, higher flood levels are expected but then there is only a very short time to prepare for flood damage mitigation actions. This is why experts often confirm the existence of flood warning messages but inhabitants report that there was no early warning. This is described in the very detailed and extensive report about flood management during the extreme flood event in Saxony in 2010 [14]. That is why it is important to give an honest and understandable description of what kind of information actual forecast models can provide.

Sirens, diaphones or loudspeakers are cost-efficient methods to disseminate warning messages to a local limited set of affected people even if they have no access to public media like TV, radio or internet. In case of flood, people must know what the signal means and what measures are required. To provide more information sirens which can also disseminate voice messages or megaphones are helpful. Additional information (information about proper behaviour, evacuation, development of situation) should be provided by community helpline or TV and radio. Light signals or traffic signs are necessary to block roads or close other public places.

The review of warning systems and documentation of operation experiences was often only triggered by extreme flood events. As intended by the EU Floods Directive and due to the great number of flood events all over in Europe, this improvement process has become more continuous in the last two decades and should go on.

Flood-proof home design

Flood-proof home design was developed in riverine and coastal communities and is very old.

The most obvious form of a flood-proof home is an elevated house (see Figure 15). The flood level does not reach the important components of the building and no damage occurs. The house can be built on stilts, on a raised platform, or on a bank of earth or concrete. This design is more suitable for flood events with low recurrence periods when no other measure is feasible. A disadvantage of this design is the restricted accessibility in case of a flood.

During the “Room for the Rivers” program in the Netherlands farmers living in the Overdiepse Polder were relocated on higher dwelling mounds (so-called terps) which have been newly built against a levee (www.ruimtevoorderivier.nl/river-widening-over-diepse-polder/). Now the farmers and their families are safe in case of a flood event but still have easy access to their farmland in the polder.

New designs and approaches aim at floating houses. These are houses which are normally on land and only float in case of a flood.

Figure 15. Left: Elevated restaurant at St. Peter-Ording at the North Sea coast. Right: elevated houses near the river Elbe (photos taken by the author)

Sometimes it is not possible to protect an urban area by levees, flood walls or demountable flood protection structures. Causes may be that the area is situated in a low lying area and the protection structures would be too high. Or the protection structure is too expensive in relation to the prevented flood-induced damage. In these cases, it is better to improve the design of buildings already situated in flood-prone areas (so-called floodproofing).

Figure 16. Elevated door sill and door flood barrier at Sóller in Mallorca (photo taken by the author)

In case of flash floods with short flooding periods, it is sufficient to elevate the door sill or thresholds or to install a barrier in front of the door (Figure 16). In case of long-lasting floods, the whole building has to be watertight. This includes the walls too. Windows, doors, air-bricks and garage doors have to be flood proof. Non-return valves for all above and below the ground inlet and outlet pipes together with sealing technology for all gaps around pipes and cables entering the property are required.

If it is not possible to keep the water out of the building, measures could be taken to minimise the damage caused by flooding. Solid floors instead of wooden floors minimise damage. It is even better to have tiled floors and walls to make cleaning after the floods easier. There should be no power outlets or heating devices below the expected flood water level. And if any furniture is necessary, it should be fixed and made out of a material that can safely take a soaking.

Modelling floods

In general, the design of flood protection structures like levees and floodwalls are based on the local flood water level for a design flood event. During the planning of flood damage mitigation measures, the assessment of excess water and flood control techniques and technologies is important requiring a cost–benefit analysis. The effect of each measure like a polder has to be quantified in terms of its costs as well as its flood water level reduction effect downstream.

The local flood water level or the flood reduction effect can be calculated by means of hydrodynamic models. In case of unsteady open channel flow, these models are based on the Saint-Venant equations (de Saint Venant 1871). It is a set of partial differential equations which describe the mass and energy conservation.

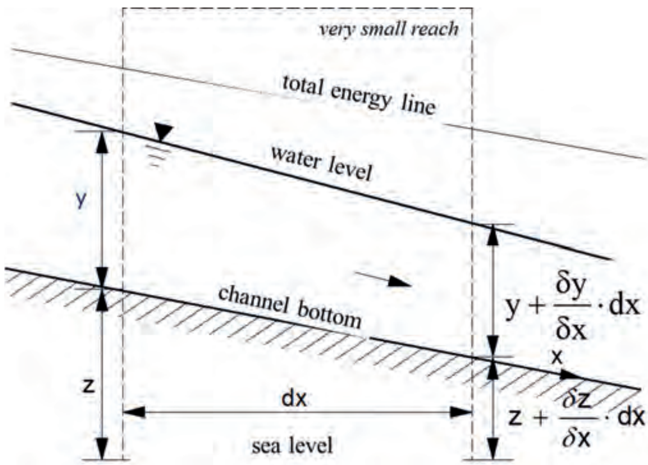


Figure 17. A very small river reach (y – water depth, z – elevation head, x – length coordinate) (compiled by the author)

For a one-dimensional consideration, it is assumed that the difference between the inflow and outflow volume into and out of a very small river reach equals the change in storage dV (see Figure 17):

$$q - \left(q + \frac{\delta q}{\delta x} \cdot dx\right) = \frac{dV}{dt} \qquad (9)$$

with q – discharge per m channel width, x – length coordinate, V – water volume and t – time. With the description of dV/dt as

$$\frac{dV}{dt} = \frac{\frac{\delta y}{\delta t} \cdot dt \cdot dx}{dt} \tag{10}$$

the continuity law is derived:

$$\frac{\delta y}{\delta t} + \frac{\delta q}{\delta x} = 0 \tag{11}$$

The equilibrium of forces can be written as:

$$\frac{\Sigma F}{b} = d(\rho \cdot q \cdot v) = \rho \cdot d(q \cdot v) \tag{12}$$

with ρ – the density of water. The following forces should be considered for open channel flow:

The pressure force F_D

$$\frac{F_D}{b} = -\rho \cdot g \cdot y \cdot \frac{\delta y}{\delta x} \cdot dx \tag{13}$$

the gravity force F_g

$$\frac{F_g}{b} = \rho \cdot g \cdot y \cdot \frac{\delta z}{\delta x} \cdot dx \tag{14}$$

and the friction force F_f

$$\frac{F_f}{b} = -\tau_0 \cdot dx = -\rho \cdot g \cdot y \cdot S_f \cdot dx \tag{15}$$

Defining the discharge q per metre channel width as

$$q = y \cdot v \tag{16}$$

and the flow velocity v as

$$v = \frac{dx}{dt} \tag{17}$$

as well as the longitudinal slope of the channel bottom S_0 with

$$S_0 = \frac{\delta z}{\delta x} \tag{18}$$

and if one uses equation 12, 13 and 14 in equation 11 one gets:

$$-\rho \cdot g \cdot y \cdot \frac{\delta y}{\delta x} \cdot dx + \rho \cdot g \cdot y \cdot S_0 \cdot dx - \rho \cdot g \cdot y \cdot S_f \cdot dx = \rho \cdot y \cdot dv \cdot \frac{dx}{dt} \tag{19}$$

Including what the total derivative of v with respect to t gives

$$\frac{dv}{dt} = \frac{\delta v}{\delta x} \cdot v + \frac{\delta v}{\delta t} \tag{20}$$

the energy conservation equation can be derived

$$S_0 - S_f = \frac{v}{g} \cdot \frac{\delta v}{\delta x} + \frac{1}{g} \cdot \frac{\delta v}{\delta t} + \frac{\delta y}{\delta x} \tag{21}$$

As the numerical solution of the equation system (equation 11 and 21) is time-consuming and there is a need for fast models, different simplifications were introduced with regard to different types of flow.

In the case of flood wave propagation which is an unsteady and non-uniform flow all terms should be considered (so-called dynamic wave approach):

$$S_f = S_0 - -\frac{\delta y}{\delta x} - \frac{v}{g} \cdot \frac{\delta v}{\delta x} - \frac{1}{g} \cdot \frac{\delta v}{\delta t} \tag{22}$$

Considering a steady and non-uniform flow, the change of the velocity over the time can be neglected. This can be applied when only focusing on the flow around the flood peak of long flood waves because they can be described as a quasi-steady flow (see sub-section *Flood routing in rivers and floodplains*).

$$S_f = S_0 - -\frac{\delta y}{\delta x} - \frac{v}{g} \cdot \frac{\delta v}{\delta x} \tag{23}$$

A more simple approach is the so-called diffusive wave approach which neglects the change of the velocity along the longitudinal river axis x too:

$$S_f = S_0 - -\frac{\delta y}{\delta x} \tag{24}$$

In case of a steady and uniform flow, there is also no change in the water level along the longitudinal river axis and the energy slope is equal to the bed slope (so-called kinematic wave approach):

$$S_f = S_0 \tag{25}$$

The diffusive wave approach and the kinematic wave approach are often used in rainfall-runoff models. The focus lies here on the description of the runoff generation and the river flow is only described by these simpler wave approaches.

The steady and uniform flow was considered when the Manning's formula (equation 2) was derived. This last statement describes a problem of calculating unsteady and non-uniform flow by means of hydrodynamic models. Most of them use the Manning's coefficient to describe the energy loss caused by water flowing over the river bed or the floodplain. But the Manning's values tabled in the literature (e.g. [8]) came from empirical studies considering uniform and steady channel flow.

Hydrodynamic models can consider flow in one direction along the main channel axis (so-called 1D models). The river and its banks and floodplains are described by different valley cross sections perpendicular to the river axis. The model can calculate the mean flow velocity in the flow cross section and the water level in it.

2D models are more suitable for the flow over a wide surface like a floodplain where the water can flow in different directions. The surface is represented by a mesh containing triangular and/or rectangular cells. The flow is described by a water level for each cell and constant velocity over the water depth. 1D models and 2D models are widely used to calculate flood water level and flood extent.

3D models describe the water flow in a 3D water volume. These models are used to calculate the flow in a reservoir or a tank or around submerged hydraulic structures like sluice gates or turbine propellers. The modelled volume is described by grid cells which can be shaped like tetrahedrons. Structured and non-structured grids are used. The application of these models on flood wave propagation is still a more scientific approach.

To establish a hydrodynamic model the following data are necessary:

- cross sections or topographic information, today often available as high-resolution digital terrain models with a spacing of 1 m, 2 m or 5 m for the flood plain, the river cross section is often derived from additional terrestrial survey data
- information about buildings standing near a river or in the floodplain (e.g. building outlines), buildings create flow obstacles and have to be incorporated into the numerical models
- information about land use and land cover; this can be derived from Orthoimages or is available as polygons (e.g. as shp files) from governmental geographic services
- information about bridges (e.g. height and bridge cross section), weirs (e.g. weir height and width, weir operation in case of a flood)
- hydrologic information like design flood discharge, data from river gauges

Measurement data from a flood event including the water level and the discharge at many locations is necessary in the investigation area as well as information about other flood event characteristics e.g. log jams, bridge clogging, accumulation of floating debris (see Figure 18), heightening of levees using sandbags, levee failure and so on. These data are used to calibrate a hydrodynamic model.

Land use and land cover are represented in hydraulic formulas (equation 2) as well as hydro-numerical models as roughness coefficient [2]. Higher vegetation like riparian woodland is associated with a higher roughness coefficient in a hydro-numerical model. Floating debris which was retained during a flood event e.g. in trees could create additional local roughness elements (Figure 18).

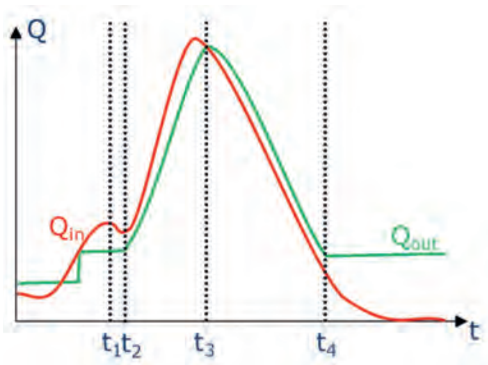
Figure 18. Flood water has deposited all kinds of debris in a tree within a wet woodland (photo taken by the author)

It is also noticeable that vegetation and its influence on flood propagation differ between winter and summer season because vegetation in winter is less dense and/or lower. Hydro-numerical models which were calibrated using a winter flood event may be unfitting in forecasting flood water levels for a summer flood event [13].

Self-study questions

What are the differences between fluvial flooding and pluvial flooding?

You can see below a measured inflow and outflow discharge hydrographs at a dam in case of a flood event. At which time did the maximum reservoir water level occur?



Explain why the calculation of river routing is more complex than calculating reservoir routing and explain appropriate methods for river routing calculation.

What is the hysteresis effect considering flooding?

Describe the flood risk management cycle.

What data are needed to calibrate a hydrodynamic numerical model?

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Zsuzsanna Engi

Logistics of Flood and Excess Water Control[1]

Logistics of flood protection and excess water control management on the state level in Hungary in the mirror of the National Water Strategy

Hungary is located in the catchment area of the Danube River, in a mostly flat area of the Carpathian Basin. Because of this special hydro-geographical condition Hungary has great vulnerability on flood and excess water. Nearly half of Hungary's arable land is prone to inland water inundation which is caused partly by the non-appropriate land use and cultivation technology. The size of flood risk areas is 21,207 km^2. The length of state managed flood protection dyke sections is 4,157.1 km and 176 km section of dykes is managed by local governments. The number of endangered population is 1.9 million people.

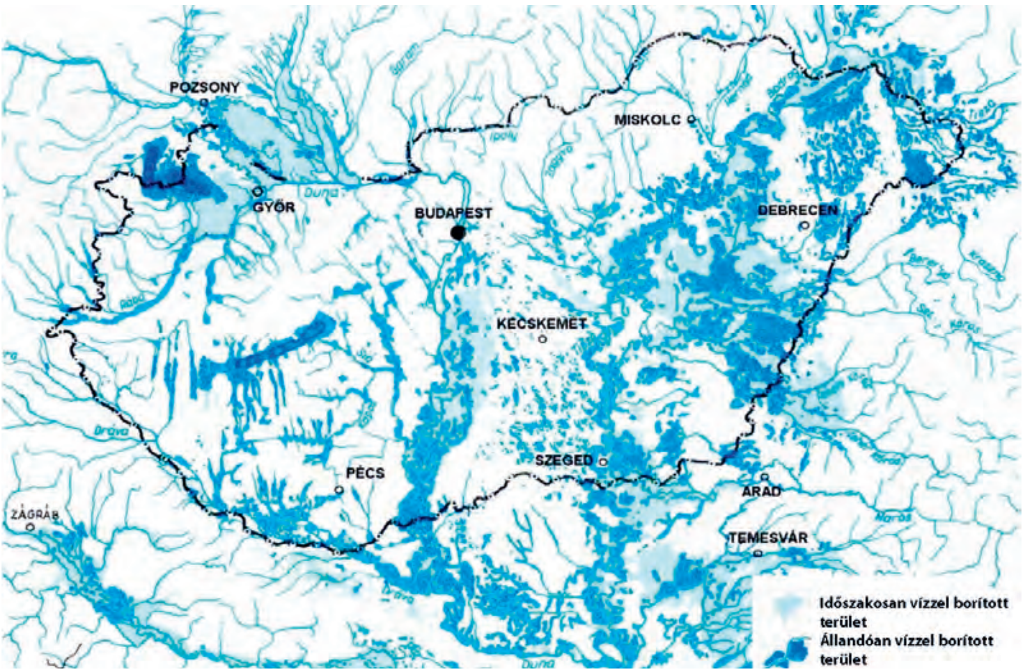

Figure 1. Hydrographic and water management data of Hungary (Adapted from [6a] p. 29)

[1] The text was adapted from the Hungarian courses of the University of Public Service, Baja.

Due to the changes in natural processes and the effects of human interventions, flood risk has increased and as it seems, this tendency will not change in the future. The main reasons of this process are:

- climate change
- the narrow inundation or floodplain areas
- the reduced conveyance capacity caused by the impenetrable barrier of vegetation in the floodplain and by silting up of the floodplains
- deforestation on the upper water catchment area
- the reduced natural flood retention capacity caused by land use
- vague financial background of the maintenance of flood protection structures
- the growth of the value of properties at risk on the floodplain

Institutional background

The Ministry of the Interior, through its Deputy State Secretariat for Public Employment and Water Management is responsible for water management issues and for the direction of the organisations of water management and for water prevention and protection from damages. The General Directorate of Water Management (OVF) carries out the tasks of central operative management.

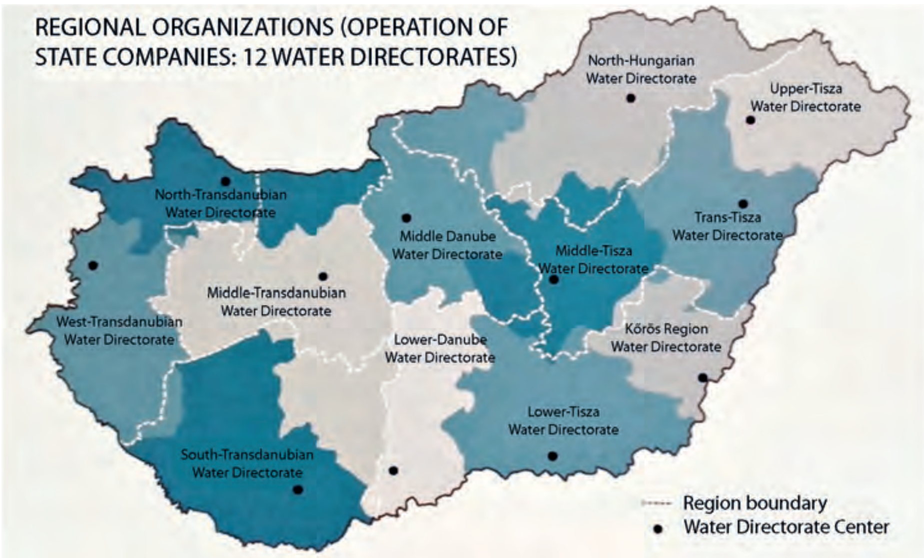

Figure 2. Regional organisations (operation of state companies: 12 Water Directorates) (adapted from [6b] p. 58.)

Areal water management is carried out by 12 water directorates (VIZIG) which are organised according to water catchment areas. The tasks of the directorates include the handling of state owned works including the control of flood and excess water.

12 disaster management directorates have been established, as required by the law, in the counties and in the capital to act as the local authorities of water protection and catastrophe prevention of the country. Some related tasks remained in the auspices of the notary of the local governments. The operational area of the first level authorities of water management and protection mostly coincide with the areas of the water catchment area based water directorates. The second level authority is the National General Directorate for Disaster Management, acting also in the frame of the Ministry of the Interior.

Legal background

The main tasks relating to water management are defined in the following laws:

- Act LVII of 1995 on water management
- Act LIII of 1995 on the general rules of environmental protection
- Act CXLIV of 2009 on water users' associations
- Act CCIX of 2011 on water public utility service

Regulations:

- Decree 10/1997 (VII.17.) of the Ministry of Transport, Communications and Water on flood protection and excess water control
- The EU directives are observed in Hungarian laws such as:
- 2000/60/EC on community action framework on water policy
- 2007/60/EC on the assessment and management of flood risk

The organisation of governmental division of labour in the practice of flood fighting and excess water control

The national management of flood fighting until the extraordinary alert falls within the Minister of Water Management's cognizance, during the extraordinary alert falls within the Government Commissioner's cognizance and in case of especially high hazard (state of emergency) falls within the Governmental Committee's cognizance.

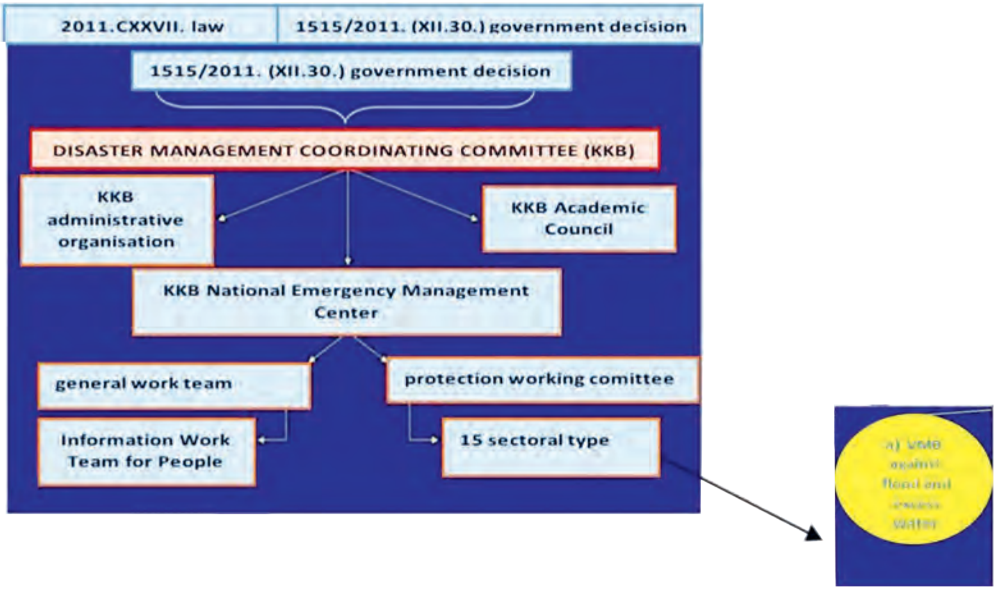

Figure 3. The organisational structure of the Disaster Management Coordinating Committee (KKB) (compiled by the author based on [2])

The Disaster Management Coordinating Committee (KKB) is responsible for the preparation of decisions and the sectoral coordination of tasks related to flood protection. The Government Commissioner's task is the national management of water damage control technical duties in every level of flood warning under the control of the General Technical Managing Committee (OMIT) which consists of staff management, technical duty, technical and supplier professional group, information service.

The OMIT oversees the operation of the defence organisation of the 12 Water Directorates and the capital of Budapest. The local control of flood protection is solved by the Water Directorate's own organisation and by the supply of technical and supplier professional group of the OMIT.

The scope of the OMIT

The following measures fall within the scope of the OMIT's cognizance:

- national management and decision support
- the control of flood protection (Water Directorates, protection staff of the capital of Budapest, local government)
- judgment of emergency
- control in the protection against ice floods

Figure 4. OMIT meeting during the flood in 2013 (adapted from Vízügy: DUNAI ÁRVÍZ 2013 JÚNIUS videóklip.mp4)

The duties of the OMIT

The task of the OMIT is to collect and process all information for the managing of flood protection, to compile decision preparatory materials and reports, to publish measures appropriate to decisions, and the monitoring of their implementation.

The OMIT can order the following interventions:

- to prepare flood storage reservoirs (the disposition of its opening and filling), its incidental overfill, sluice (the drainage of water stored in it) taking into account the operating rules of the given reservoir or reservoir system
- opening and closing of levees
- building of localisation lines not included in the localisation plan
- the continuation of protection against threatening phenomena like serious malfunction, levee breach or inundation in critical situation
- the limitation of major drainage structures' function (pumping, water inlet to drainage structure, etc.)
- emergency storage of excess surface waters with the opening of drainage canal's levee (spoil bank) or another way
- overfilling of excess water retention reservoir
- make a proposal for traffic limitation order (limitation of shipping, road, railway closure)
- data recording (aerial survey, fixing elevation of water surface, etc.)
- measures that are different from the updated flood protection plans
- measures about forecast and the publication of it in case of great hydrometeorological uncertainty, runoff, propagation situation and/or greatly contradictory forecast based on the proposal of the Hungarian Hydrological Society

- any other, technical matter connected to protection which is performed with full responsibility
- preparation of government decisions connected to protection in form of written drafts
- flow control related to water quality damage prevention

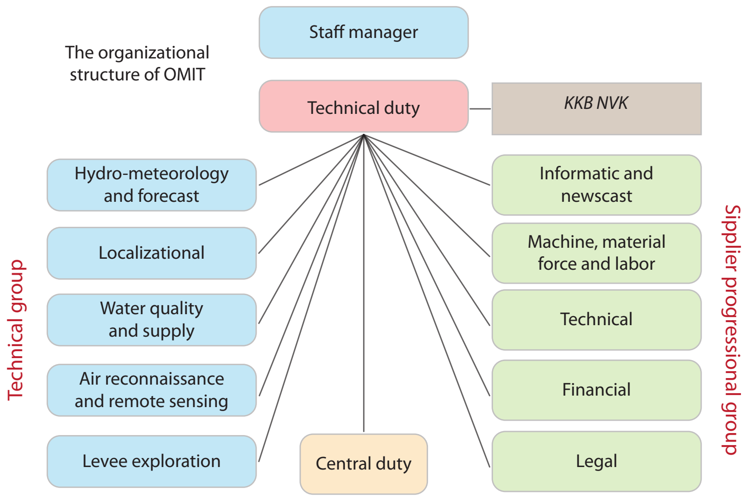

Figure 5. The organisational structure of the OMIT (compiled by the author based on [2])

The OMIT carries out resource coordination on the following aspects or cases:

- using the national set
- using the services of the armed forces and the law enforcement agencies
- alerting the protection squad of the Regional Water Directorates
- protection preparedness can be ordered by the Regional Water Directorates, furthermore by the central and regional municipalities controlled by the Minister
- asking for the services of other Regional Water Directorates depending on the level of emergency and taking into consideration the updated cooperation order
- secondment of technical leaders for protection in local government
- secondment of external experts (universities, colleges, planners, etc.)
- measures related to any other resource which is performed with full responsibility
- permits and oversees of the protection of ice blasting activity
- permits and oversees for alerting and operation of the ice breaker fleet
- the Regional Water Directorates can be alerted and deployed in any part of the country as well as in foreign countries

County organisation of flood fighting and excess water control

In case of extraordinary alert (emergency), the county organisation of flood fighting (Defence Committee) assists the President of the Defence Committee in carrying out the administrative tasks of defence. Performing the tasks of proposal-making and decision-preparation, it participates in the coordination of the protection on regional level and controls the operation of the local flood defence committees.

The Organisation of the Regional Water Directorates in flood fighting and excess water control

The flood fighting is controlled by the Regional Water Directorates at local level in close cooperation with the county organisation of flood fighting (Defence Committee). The 12 Regional Water Directorates have the same organisational structure. As an example, we introduce the organisational structure of the West Transdanubian Water Directorate in the period of flood fighting and excess water control, which is illustrated in Figure 6. The director of the Regional Water Directorate is the head of flood protection who relies on the flood protection staff, the professional groups and the flood protection squad in his/her work.

The duties of the head of flood protection and its assistants

The central head of flood protection is the all-time director of the Regional Water Directorate who controls and leads with personal responsibility the protection works on the operational territory of the Directorate. These protection works are controlled by regulations which are determined in the supplement of the Water Damage Prevention Regulation and it is about the rules of protection against water damages. During alert, the water damage prevention organisation is set up according to the instructions of the director (during flood protection works his position is: central head of flood protection). The leaders of the flood protection organisation and its employees are assigned in the annually published Organisational Assignment of Water Damage Prevention. The central head of flood protection controls the flood protection works through the Flood Protection Staff.

In the absence of the head of flood protection, its assistants proceed with the flood protection works. The assistants must inform the head of flood protection as soon as possible about measures which influence directly the flood protection activity.

Flood Protection Staff

The duties of the Flood Protection Staff is the technical and technological serving of the flood protection activity, the systematisation and analysis of the necessary information, the preparation of the central head of flood protection's decisions, their substantive plan, organisation, the implementation of technical duties and professional service related to flood protection activity. The Protection Organizational Assignment contains the list of the Flood Protection Staff.

Central technical tasks

The task of the leader of the special departments is ensuring the special equipment, means, registrations, data sheets. In case of alert, service is done by the members who belong to the Organizational Assignment of Water Damage Prevention. In case of water damage prevention alert, the members of the central technical duty (head of duty, people on duty, administrators) are subordinated directly to the head of flood protection's assistant who is on duty, and the tasks must be done according to his/her orders. The main tasks of the central duty are determined by the specifications of laws, regulations, directorial orders which are imparted in the water damage prevention regulation. Every important measure with respect to the implementation of protection proceeds from the central technical duty on the orders of the central head of flood protection or its assistant and arrive there also.

The Regional Water Directorate ensures the regional presence through the head of the district flood protection and the dike keepers. Levee guard assistants work under the guidance of dike keepers for the observation of flood phenomena.

General duties of the head of district flood protection

The head of the district flood protection is bound to do every necessary measure for effective flood protection. The head of the district flood protection ensures the performance of the following measures:

- the implementation of every technical measure necessary for drainage of the water without damage and prevention of danger
- constant observation of the condition of protective facilities
- registration of the harmful phenomena
- application and control of the work and equipment necessary for protection
- continuous employment, supply, registration of the attendants in protection
- continuous registration and account of data necessary for the report of protection target, especially of works of the employees in flood protection, and of using equipment required for protection

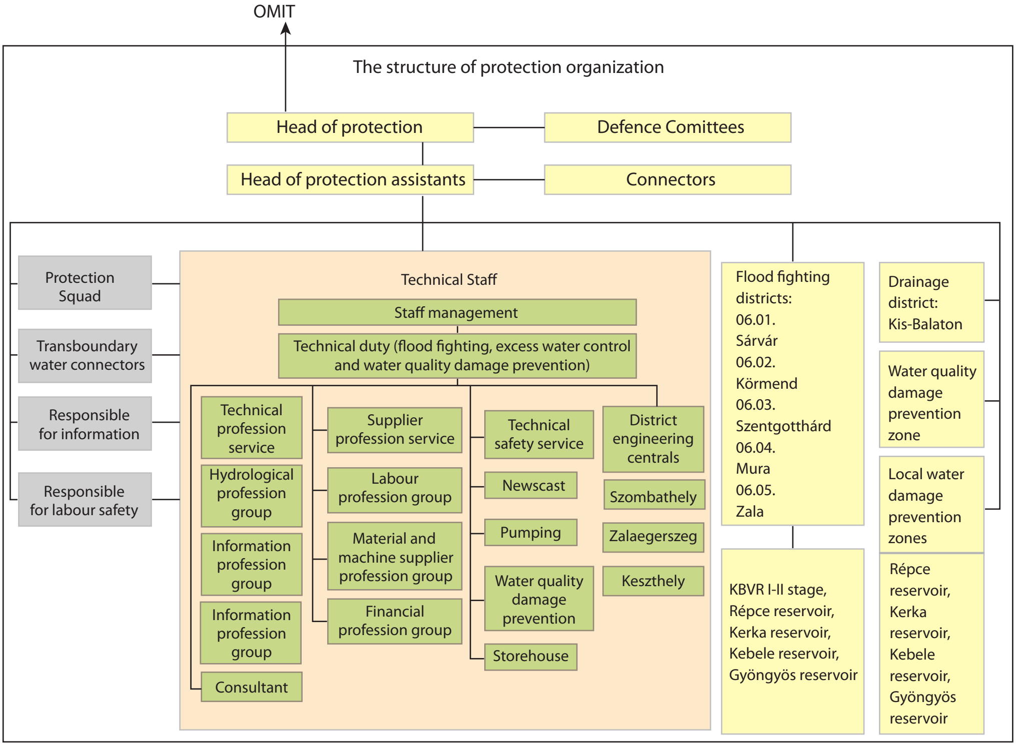

Figure 6. Protection organisational structure of the West Transdanubian Water Directorate (compiled by the author based on [2])

We introduce in a few steps in more detail how this looks in practice. The head of the district flood protection and the technical employees must perform the following tasks:

The construction of the protective facility's district must be marked, in case of heightening the levee its height must also be marked (Figure 7).

Figure 7. In case of heightening the levee, its height must also be marked (photo archive of NYUDUVIZIG)

The dumping grounds must be marked, sandbag filling stations must be established (Figure 8).

Figure 8. The sandbag filling stations (photo archive of NYUDUVIZIG)

The helicopter landing space, mooring of boat, barge possibilities must be marked (Figures 9–10).

Figures 9–10. The helicopter landing space (photo archive of NYUDUVIZIG)

The transport traffic must be organised and controlled; in case the dyke crest is not usable for this purpose, alternative routes must be marked including air and water transport routes.

The continuous material resupply must be organised to the dumping grounds and to the place of construction of the protective facility.

The work of the flood protection teams must be organised and controlled; when needed, headcount reorganisation must be implemented. In case of inadequate headcount, more people shall be requested.

The social conditions (mobile toilet, occasional warmer rooms) have to be ensured for the outside workers.

In case of night work, the illumination of the protection line has to be cared for (Figure 11).

Figure 11. Illumination of the protection line (photo archive of NYUDUVIZIG)

The head of the district flood protection can petition the command of the Directorate Protection Squad in a reasonable case, and he can ask for the help of the Technical Security Servi0ce (machine supply, pump installation, in cases of telecommunication and other things). If this measure is insufficient then he can recourse to the help of further organisations according to succession contained in the Resource Requisition Plan.

The protection district can be divided into dike keeper sections, the dike keepers who do their jobs here control the smaller protection works and the building of protective facilities; in case they observe harmful phenomena, they announce the head of the district flood protection who decides about the necessary measures. Levee guard assistants help the job of the dike keepers who work in pairs (one of them on the dyke crest, the other at the protected side of the toe of the dyke) and give reports to the dike keepers in case they observe harmful phenomena. In especially dangerous districts and locations that require continuous observation, the employment of an emergency guard can be reasonable. This emergency guard monitors the dangerous place (e.g. sand boil, leakage, softening, etc.) and gives reports to the dike keepers in case they experience any change.

The flood protection organisation is bound to supply the technical duties of protection preferably with own power (labour, material, equipment, machine and device). The Regional Water Directorate ensures the resources, headcounts for protection by concluding pre-contracts. If the available own power for the supply of the technical duties of protection is not enough then the head of the district flood protection of the Regional Water Directorate takes action about the completion of documents of resources, such as flood protection measures and duties in case of different alert levels.

Duties in case of 1^{st} degree alert

- actualising and synchronising the flood protection plans
- checking the status of the line of protection and the structure
- closure of sluices
- checking/ensuring personal conditions (technical workers, dike keepers, levee guard assistants)
- providing accident prevention education for protectors
- survey, ensuring (requiring) the protection equipment, materials
- checking the operation of news casting, computational systems
- survey of transport and carrying devices, ensuring them if necessary
- checking, ensuring the conditions of the flood protection administration (flood protection diary, attendance sheets, sample contracts, block of delivery notes, etc.)
- preparing the social conditions of protection (accommodation, meal)
- noticing, giving information to the head of the local government connected to protection
- reading the gauges twice a day (at 6 and 18 o'clock)
- making a daily report about the previous day (closing the day at 6 o'clock in the morning), and sending it to the centre of the Water Directorate, then to the OMIT
- supplying the flood protection staff and vehicles with "Flood Protection" signs

Figure 12. "Flood Protection" sign (photo archive of NYUDUVIZIG)

Due to unfavourable weather, pot-holing and ballast of the damaged dyke crest and surrounding roads is necessary, as well as snow clearance of the dyke crests, roads, stowage and de-icing the interlock of the structures.

During the period of 1^{st} degree alert, a 12 hour daytime watch has to be kept. In addition to the dike keepers, an appropriate number of levee guard assistants must be placed (according to the status of the protection line and protection conditions).

Duties in case of 2nd degree alert

- ordering a necessary number of levee guard assistants next to every dike keeper, ensuring emergency guards to structures
- ensuring news casting devices on the newer duty stations (structures)
- ensuring lighting devices (backlight) for night duty
- reading the gauges four times a day (at 6, 12, 18, 24 o'clock)

Figure 13. Discharge measuring (photo archive of NYUDUVIZIG)

- checking the flood protection district, control the work of the dike keepers, levee guard assistants, emergency guards

Signs of flood phenomena:
- white flag – sign of failure
- yellow flag – permanent, intensified observation
- red flag – immediate intervention

During the period of 2nd degree alert, 24 hour non-stop watch has to be kept. In addition to the dike keepers, an appropriate number of levee guard assistants must be placed (according to the status of the protection line and protection conditions).

In the course of carrying out the protection administration, the followings are needed to be done:
- recording the received and given orders and phenomena on the protection line in the protection diary
- carrying out the protection works, registering work time and work performances
- preparing the request of labour, machine and material needs, announcing the flood protection staff of the Water Directorate
- giving information about the expected water stages and the chances of flood protection with regard to protectors (company, local government) who work on summer dikes

- providing information for the head of local governments connected to flood protection
- preparing daily reports and sending them to the flood protection centre of the Water Directorate

Duties in case of 3rd degree alert

Emergency guards have to be ordered to that location of the protection line where a serious threatening phenomenon to stability had formed or this can be expected. The changing of the emergency guards is happening on the spot, the phenomenon cannot remain unattended.

The water stages have to be read, noticed and reported in every 2 hours (in paired hours).

The fixing and ranging of the flood cresting elevation of flood water stages have to be done (usually points fixed per 500–1,000 metres, and with uptaking of typical sections).

All other further work has to be carried out that is typical of lower degrees (organisation, administration, information, control, report etc.).

Figure 14. Controlling the height of the levee (photo archive of NYUDUVIZIG)

During the period of 3rd degree alert, 24 hour non-stop watch has to be kept. In addition to the the dike keepers, an appropriate number of levee guard assistants must be placed (according to the status of the protection line and protection conditions).

Duties in case of extraordinary alert

Maximum protection work is needed for the prevention of dike failure!

Duties: as in case of 3rd degree alert.

Gauge reading if necessary (in every 2 hours, when needed, in every hour).

Usually, the needed flood squad and specially equipped profession groups are required from the flood protection staff (like air observation, levee expert profession groups, armed scouts).

Figure 15. Controlling the built protection structure (photo archive of NYUDUVIZIG)

During the period of extraordinary alert, 24 hour non-stop duty is kept not only on the connected Water Directorates. In addition to the dike keepers, levee guard assistants have to be placed (according to the status of the protection line and protection conditions).

Figure 16. Bridge protection during the flood (photo archive of NYUDUVIZIG)

Duties after the termination of alerts

The degrees of alerts have to be terminated in cases when the falling limb had decreased and the discharge of the river returned to normal and a new flood is not expected, therefore the reason of ordering the alert had terminated. The dike failure or implemented cutoff generated on the flood protection levee and on the spot of the damage threatening the stability of the protective facility, a 3rd degree alert cannot be terminated until renovation. Duties which have to be carry out after the termination of alerts are as follows:

- removal of scum, floating debris from the foreshore slopes
- renovation of the damaged protective facilities (foregrounds, slopes, dyke crest)

- collection, repair and storage of the protective materials, devices and equipment
- measures for the replacement of the missing protective materials
- reckoning of protection outputs
- reckoning or return of services, devices, equipment required from other organisations, people
- preparing the summary report (by the head of the district flood protection, or the directorate)

Duties of the urban local governments concerning flood protection and local water damage prevention

In case of flood protection and local water damage events, the duty of the urban local government is the implementation of the rules described in regulations and laws and the supply of public help. The duties of local water damage prevention are the followings:
- prepare for protection
- take part in operational protection
- measures after the termination of protection

Figure 17. Protection practice in Kerkaszentkirály in 2010 with public participation (photo archive of NYUDUVIZIG)

The duties of urban local government outside the period of flood is the implementation of rules described in regulations and laws, the supply of public help, which means the following duties in connection with flood protection:

– preparing a local flood protection plan for mobilisation to ensure resources, etc.
– preparing for protection, among others keeping protection practices

Prepare for protection

The building of drainage facilities: In an ideal case, the drainage system of the settlement is built for the warning level and has water permit of operation.

If the building is not full, it has to strive for the planned, conscious implementation of the surface drainage. It is important that the local government has at least a study plan about facilities which are necessary for the drainage of the warning water mass.

Maintain a defensible state: Regular maintenance of the drainage facilities, mowing at least twice a year (desired twice, ideally three times), and silt removal 3–5 times a year.

Maintenance of pumping stations, machine equipment.

Draining, maintenance of storm storage basin in time.

Repair of facilities, coverings.

The regular and professional revision of facilities has to be done at least once a year (expedient at autumn). Determining failures, measures for their termination.

Operative protection

The management of the flood protection of the settlement is ordered by the mayor and the head of the flood protection appointed by the mayor is responsible for the ongoing works. In view of the protection works, the degree of alert can also be ordered for the settlement according to the flood protection plan of the settlement. This is recommended in favour of the support of the later force majeure certificate.

Measures after the termination of flood protection

The competent Water Directorate must be notified of the termination of flood protection measures.

The protective facilities damaged during protection have to be repaired.

The protective materials, devices and equipment have to be collected and returned to their owners.

A summary report has to be prepared about the protection activities from which a version must be sent for the competent Water Directorate.

Flood protection register plans – Flood control programmes

The flood protection register plans refer to a flood protection district and contain those information that are necessary from a flood protection concept point of view for the judgement of the dike's state. The flood protection register plan contains a technical description and thematically the followings:

- overview and a detailed map of the flood protection district
- a written longitudinal profile section of the flood protection district with drawings
- typical cross-sections marked with the historical dike progress
- flood phenomena observed during earlier floods
- characterisation of the dike state
- soil layer and drilling sections
- listing, location and typical data of the facilities which cross over the dike
- the plans of the crossed facilities

The flood protection plan must be updated once in every year, the technical and personal changes must also be shown. In addition to the flood protection plan, a preparation plan must be made for every flood protection district which has to be updated every year at the beginning of the year. The preparation plan relying on the flood protection plan contains those information which may occur in case of a flood event and which have to be responded to. The preparation plans can be made for typically two scenarios: HWL (Highest Water Level) – peak stage of design flood + 1.0 m. In case of these two water stages the following measures have to be taken:

- the treatment of altitude shortages
- ensuring the levee stability (levee body, slope and subsoil)
- protection against wave action
- ensuring access to roads, routes, logistics
- measures concerning districts requiring special attention

Those parameters contained in the flood protection plans (crest height, dike section size, geomechanical data, etc.), furthermore those districts and works can be determined knowing the above two water stages which have a plan if the given situation occurs. For the creation of a design, the followings have to be planned:

- determining the building technology, specific technical design (e.g. emergency dike, coffer dam, formation of protection against wave action, etc.)
- determining the necessary staff needed for the supply of the protection line's monitoring service and for the building of the protective facility according to the organisational assignment of water damage prevention
- design of human resource needs necessary for the building of the protective facility
- determining the needed machinery for the building of the protective facility
- registration, design of material delivery tools, protective machines
- ensuring the transportation, technical background of loading (supplying routes, etc.)

- determining the material needs necessary for the building of protective facilities, ensuring the material resupply, registration, replacement of protective stock
- registration, design of protective material stocks (materials on both central and protection districts)
- formation of warehouse, storage places

The practical logistical duties of flood protection

The definition of logistics and determining its aim

Logistics consists of material flow and the connected information, value, energy and labour flow. The aim of logistics is that the appropriate quality material gets in the appropriate quantity, from the appropriate place to the appropriate place, with the appropriate way and device, at the appropriate time and with the appropriate cost.

The logistics of flood protection

In flood protection, the speed of countermeasures has a key role. That is why the organisational structure, the decision levels and the logistics of flood protection have to be formed to serve the effective protection. Providing fast countermeasures require not only local knowledge but also excellent organisational skills. The flood protection specialist who knows the places well, has a special flood protecting knowledge, uses the conveyor materials, travel routes, conveyor machines routinely and has an appropriate human knowledge, in other words the well-trained flood protection specialist cannot be ignored in the future when mentioning flood protection. In a lot of cases, the success of flood protection depends on good solutions of the logistic duties. The applied delivery ways and protective materials depend on the opportunities and requirements.

The chain of the delivery of protective materials

In the 19th century, everything was used for flood protection that was available in the proximity of the flood event and was cheap to obtain: straw, floating debris, corn stalk, manure, etc. At the beginning of the 20th century, a significant amount of protective material was placed in the flood protection storehouses because of the weak delivery opportunities. In the last 30–40 years, flood protection ways have changed. The fast delivery of great amounts of protective materials made flood protection possible in a quick and efficient way. Flood protection thus can begin immediately because the most often needed protective materials like sandbag, geotextile, foil, etc. are stored or delivered in greater amounts. Nevertheless, the transport of this large amount of protective material requires developed logistics.

The most important materials and devices of flood protection

The materials of flood protection must be simple, easy to handle and useful in large amounts. Those protective materials and devices enjoy advantage which can be used easily and do not require significant professional skills.

The most important material of flood protection ever is the sandbag. The sandbag can be filled with sand, gravel or clay. Its use depends on the flood protection team. The material of the sandbag is a good quality jute or white plastic bag without UV protection that can be used several times. The emptied bags are washed by the dike keepers, they dry them on the slope and then store them. The used sandbags are not classified as hazardous waste and they are handled accordingly.

Figure 18. The most important material of flood protection is the sandbag (photo archive of NYUDUVIZIG)

The ideal extender of the classic sandbag is the ground-wet muddy sand or fine sand which in its ground-wet state can be formed well and can be easily built upon one another. It is important that the acquisition place of the sandbag's extender have to be searched in "peacetime" and they have to be marked in the flood protection plans.

For the transport of large amounts of filled sandbag and bulk sand or gravel, the application of 1 m^3 volume containers is widespread.

The most important device of flood protection is the vane, small machines, generators, lighting equipment, sheet pile machines, etc.

Concerning the materials and devices, there is a three-levelled storage system: national supplies (kept in the available storehouses of the water directorates); the supplies of the Water Directorates (usually kept on the squad yard); and local supplies in the storehouses of the dike keeper's lodge. In case of a flood event, the most often needed protection devices and materials are stored in limited quantity in the flood control storehouses of the Water Directorates. The local supplies are only sufficient to start the flood protection procedure.

Figure 19. Application of 1 m^3 volume container bag (photo archive of NYUDUVIZIG)

Transportation, trans-shipment, spoil banking

The protective material can be transported on land (on terrain and/or built road), in the air and water. The building place of the protective material can be approached the best by air; usually, land transport needs the biggest labour demand.

While transporting protective materials, it is important to organise that the drivers carry the cargo to the desired place.

In the chain of the transport of protective materials, trans-shipment places must be inserted to ensure seamless delivery. On the trans-shipment places, transport has to be switched from land to another type of transport; in this case the special problems are the following:

- for water transportation at trans-shipment the available area is usually small
- for air transportation the trans-shipment place can be uploaded practically at night

The chain of the transportation of protective materials has to be set up to match the capacity of the associated delivery points. Only the required spoil banking and reservation should be needed.

Sandbag logistics

It is a long way from the sandpit to the place of building. The first logistic step is the choice of the bagging place (bag charge). Usually, the leader of the local flood protection team can choose from the followings:

- Sandbag filling happens on the place of building; in this case sandbags, sand and vane shall be ensured except for the flood protectors. This can be recommended if the location of protection is easily accessible. In case the surrounding roads are inaccessible, flood protection can start using the devices of the neighbouring levees in a way that it does not damage noticeably the flood protection safety. In many places, sand and spare spoil bank is prepared on the protected side of the dike but in case of emergency the ramps and the protected side of the crest edge can be exposed to sliding. The vane and the sandbags are transported by the protectors to the location.

- Sandbag filling can happen in the nearest settlement (or if the settlement is out of the way, in a suitable place such as a road junction).
- Sandbag filling can happen in a central filling station. Although the transport route may be long, it still ensures continuous service.

In case of sandbagged protection, in the chain of the transport of protective materials, the filling of the sandbag is important. Sandbags shall be filled to 40–50% of their volume. In order to avoid having to determine how much is in each bag, according to the volume of the bag and the size of the vane it must be determined at the beginning of the work how many vanes should be taken to one bag. Crowded sandbags are useless, with their circled cross-section they are not impervious when placed next to each other because of the gaps between them.

The end point of the transport of protective materials is the building of the sandbags. Usually, sandbags arrive in a single file from the last transport device to the place of building. This is the work that increases the labour demand of flood protection.

Figure 20. Sandbag filling in a central filling station of Győr during the flood in 2013 (photo archive of NYUDUVIZIG)

Figure 21. The single line arrival of sandbags during the flood in 2013 on river Marcal (photo archive of NYUDUVIZIG)

Sandbags arriving at the place of building must be placed. The empty part of the bag shall be folded over and placed to its place joined in a row and column. Sandbags have to be stamped firmly into place to close gaps and create a tight seal. The protective facility made from sandbags will be stable only in this way.

Figure 22. The correctly prepared protection line made of filled sandbags (photo archive of NYUDUVIZIG)

Transportation of protective material

Transport by road

Recently, the protective material transport by vans to the dyke crest has spread increasingly. Vans damage less the unpaved crest and do not break the paved crest; due to the low platform loading and unloading is also faster. Vans can travel in both directions which big trucks cannot perform.

Figure 23. Protective material transport on the dyke (photo archive of NYUDUVIZIG)

At local water damage or migration of eruptive waters, it often occurs that the water washes away on longer or shorter sections the built road, the transport of protective materials and the supply of the population becomes impossible.

Transport on terrain

Since 1999, the protective material transport with military trucks (PTSZ) has proved its excellent applicability. Vehicles travelling on the same route for several days make such mud that these vehicles can get stuck also. The Ural trucks made for heavy terrains performed well at flood protection.

It is a very important step in transport of materials on terrain the delivery of the protection material that arrived at the site to the final place of installation. This place is usually difficult to reach by machine, so the sandbags are placed passing from hand to hand in a chain. In this process a lot of labor force is required and the protection material should be often moved up or down on the embankment slope. With appropriate mechanization, like the conveyor belt, physical work can be replaced. It is simplifying and speeding up the work if the sandbag slides down on the embankment slope on foil or in a plastic half-pipe.

Figure 24. The sandbag slides on foil down the slope (photo archive of NYUDUVIZIG)

Water transportation

Water transportation can reach places that are difficult to access on land. Let us not forget about the crossway transport on the river. From the other bank of the river, protection can be served better and easier with water transportation! Near the intervention – for example at a ramp – a port must be formed at the paved road in favour of the movement of protective materials.

Water transportation has an extraordinary significance, but its effectiveness is really revealed when, due to raining, the crest and the protected side of the toe of the dyke become impassable. Water transportation during flood protection means the following tasks:

- operative activities
- protection from water, savings, construction of insurance
- implementation of transport tasks, like person and device transport, machine and material transport
- observational and control tasks
- removing of floating debris and scum
- other additional and server activities
- closure of dike breaches (sheet piling) works
- rescue tasks

Preparation and logistic tasks during water transportation

Tasks of the preparation period:

- marking of staging and stationing places
- marking of bank connected places – stowage places
- reconnaissance, control of water ways between the main protection line and shipping way
- survey, keeping clean the shipping line
- control of the dike foreground navigability, ensuring the obstacle clearance of the appropriate sizes
- ensuring the watercraft's stationing and logistic supply
- marking of launch and boatyard possibilities
- during a flood protection order – knowing the forecasts
- providing standby service
- implementation of staging to the marked stations
- preparation of material handling and transportation plans

The accident prevention regulations must also be kept during flood on the ships and on watercraft.

Air transportation

The helicopter material transport is a special case of flood protection when land and water transport ways which were applicable for approaching the protection place became impossible.

In precipitated weather circumstances and on leaking aqueous areas, the material transport can be solved by PTSZ, but traces caused by the trucks damage the state of the thin top sheet. For this reason access to the protection line must be limited.

The spread of air transportation made the spread of container bags possible in flood protection, it offered further opportunities for the organisation of large mass stowage and transport. The most important element when organising air transportation is the provision of the appropriate material, length and attachment of the transport rope to the helicopter. The helicopter can transport 2 container bags one-by-one with 0.8–1.0 ton mass.

Figure 25. The helicopter can transport two container bags (photo archive of NYUDUVIZIG)

When placing the container bags, the biggest problem is that the pilot cannot see the area below, so it moves directly with land control on the target area. The rotating bags must be placed under such conditions so that the 2 tonnes do not cause excessive displacement. At the same time, a third person unloads the bags independently of the pilot.

The building of the container bags remains a difficult problem to solve in spite of the fact that the pilots do their jobs as accurately as possible. Several factors make the accurate deposition difficult, for example the two container bags attached to the helicopter rotate along its centre of gravity. It is also a disadvantage that no activity can be performed in the immediate vicinity because of the wind and noise. Furthermore, the building of an accurate alignment cannot be controlled by human operators because the wire rope hanging from the helicopter can hit the person.

Figure 26–27. Placing sandbags by helicopter during the flood in 2013 in Győr (photo archive of NYUDU-VIZIG)

During transport by helicopter, the following organisational tasks must be performed in the period of preparation:

- the marking of the basis of container fillings, organising material supply
- the marking of landing and loading places which can be approached by helicopter
- organising land transportation between filling and loading places (transport device, their quantity and transportation routes)
- restoring the carrying ropes

In case of concrete flood protection work it is necessary:

- the choosing of the optimal number of transport helicopters taking into account the material transport requirements and circumstances
- determining the technologic processes of the concrete protection measures (e.g. load, underpinning of slope)
- organising the communication between the place of building and the helicopter

The protective material transport by helicopters has advantages, disadvantages and consequences. It is an undoubted advantage that it does not require a staging area, it can be easily redirected, the practiced pilots place their loads with a 30–40 cm precision, little central bag filling basis is needed, it works independently from every other protective material transport route and little special technical controlling activity is required. Its disadvantage is that no activity can be performed in the immediate vicinity because of the wind and noise. The consequence of protective material transport by helicopter is that the berms are wider and a little bit more irregular.

The building of flood protection roads

The protective facilities made from stone can be classified into two groups according to their functions:

- serving as access road for the inaccessibly compromised districts
- the toed underpinning serving the dike slope failure

The parameters of road building planned from stone are the followings:

- it must be fixed to the toe of the dyke wherever flood protection activity has not yet started (previous protection)
- on the place of flood protection activities it must be tightly built (where slope stability berm is already there)
- the road must be 60–80 cm deep and 5–7 m wide

The stones have to be placed on a geotextile geomesh with back tilting.

 

Figure 28–29. Flood protection road during the flood in 2013 in Győr (photo archive of NYUDUVIZIG)

The built road must ensure:

- the continuous transport of protective materials
- the draining of water from the dike
- the filtered draining of water from undersoil
- if necessary the road can be a solid ground for the increasing of slope stability berm appropriately
- the geotextile has a separator role, the stone is not imprinted into the soft protected side of the surface soil
- the function of the geomesh is weight distribution

The practical logistic tasks of the flood protection squad's staging and application

The task of the flood protection squads

The operation of the protection squads is provided for in the "Special Protection Tasks" section of Decree 10/1997 (VII.17.) of the Ministry of Transport, Communications and Water on flood protection and excess water control:

8. § (1) To carry out protection tasks requiring special preparation and equipment, local protection teams operate at the Water Directorates and regional protection teams at the designated Water Directorates.

(2) The task of the protection squads is in particular to execute:

a) the temporary closure of dike failure

b) the temporary closure of the embankment breach

c) the lighting of workplace

d) the water transport of protective materials

e) the installation of temporary pumping platform

f) water quality damage prevention

g) diving works

(3) The special protection tasks, the number of personnel and the equipment required for each protection squad are determined by the Minister on the proposal from the Director-General of the Water Directorate.

The involvement of the protection squads in flood protection

The commander of the protection squad is selected by the central head of protection. It is subordinated directly to the central head of protection, in its absence to the central deputy head of protection. According to the rules of the organisation, the members of the squad are under the command of the commander of the protection squad. In the event of an emergency, the squad commander organises the preparation of the team and the purchase of equipment. If necessary he/she organises the accommodation of the squad, the installation and professional tasks in accordance with the order. The squad is subordinated to the head of the district flood protection according to the rules determined in the command instruction. The commander of the squad organises the provision of staff, materials, services and devices necessary for the flood protection tasks. In case of acquisition of special protective materials or devices, he/she helps the work of the supply profession service. If necessary – with special permission – he/she acts on his/her own authority for procurement. The squad commander controls the flood protection activity of the squad and is single-handedly responsible for the team. He/she cares about the compilation of the daily report, the activity of the squad and that the report is forwarded with appropriate timing, format and content towards the central management through the valid data traffic network every day by 7 a.m. He/she implements the instructions of the head of the district flood protection referring to the squad. Before the beginning of the flood protection work, members of the squad must be provided with health and safety training, which must be registered according to rules.

Figure 30. The involvement of flood protection squads (photo archives of TIVIZIG, KÖVIZIG and KÖTIVIZIG)

Excess water control measures in case of different alert levels

The formation of excess water

45% of the area of Hungary is endangered by inland water. The length of drainage canals is 48,513 km, of which irrigation and dual function canals are 4,326 km long. The number of pump stations is 624 of 952 m^3/s capacity. During the excess water control, portable pumps can also be installed.

Figure 31. Excess water near the Little Balaton (photo archive of NYUDUVIZIG)

The causes of the formation and development of excess water are due to environmental or anthropogenic factors as listed below.

Environmental factors:

- the unequal territorial distribution of extreme precipitation
- the small surface slope
- soils with poor water-conductivity
- the basin type
- limited gravity guided possibility for drainage caused by high water level of rivers

Anthropogenic factors:

- the change of the ownership of land was not followed by the transformation of the drainage system
- inappropriate land use
- incorrect soil cultivation
- failure to maintain the drainage systems
- the unreasonable filling of the drainage system
- less forests than needed or incorrect tree planting

The content of the excess water control register plans

The defence processes of bodies responsible for excess water control are set out in Act LVII of 1995 on water management and internal instructions of defending organisations. Decree 10/1997 (VII.17.) of the Ministry of Transport, Communications and Water obliges all organisations participating in the excess water control to prepare and update the defence plans, e.g. excess water control register plans.

Types of plans prepared by the Water Directorates:

- general excess water control plans are prepared for the areas of operation of the Directorates
- detailed inland water protection plans are made at the level of protection stages

The Protection Organizational Assignment contains the personal and availability data of participants in the excess water control.

Legislation requires a periodic revision of protective structures and their accessories, protective materials and tools and equipment (autumn review is based on the instructions about the review of the preparation for flood and excess water control).

Tasks in case of different alert levels concerning excess water control

1st degree of excess water control should be ordered if the gravity conditions of the free conveyance of the drainage canals toward the rivers cease to exist or measures should be taken to assure the conveyance of excess waterways.

2nd and 3rd degree of excess water control should be ordered if pump station operation is needed taking into account the water retention in drainage canals or the excess water conveyance regulation.

Extraordinary alert should be ordered if the excess water flooding takes on dimensions that endanger populated areas, industrial sites, main roads, railways and further flooding is expected.

The practical logistical duties of excess water control

The process can be divided into three distinct phases: measures taken in the previous period of excess water control, operative measures during the excess water control and measures after the termination of excess water control.

Measures taken in the previous period of excess water control

Hydrographic observations and data provision (water level, precipitation, temperature)

- observation of water level (drainage canals, recipients)
- detecting and processing rainfall data
- detection and processing ground water data
- detecting and processing temperature data
- controlling the recipients' conditions for retention of excess water (retention basins, oxbow lakes irrigational canals)

Warning to other organisations participating in excess water control

In the knowledge of the data, calling the attention of the concerned Water Management Associations, Municipalities and Defence Committees to the expected excess water status. Sending information to local governments to properly document the expected force majeure events.

Operative measures during excess water control

Ordering the degree of excess water control alert and the interventions needed are regulated in Decree 10/1997 (VII.17.) of the Ministry of Transport, Communications and Water.

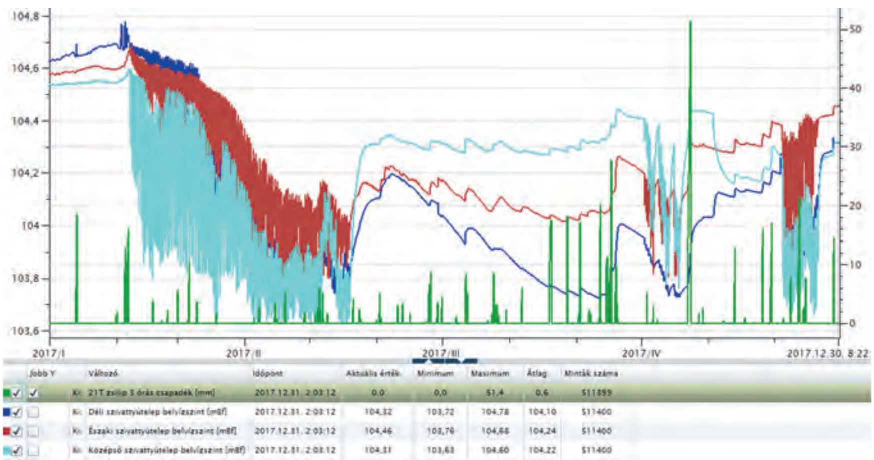

Figure 32. Water level of the pump station on Little Balaton during operation (archive of NYUDUVIZIG)

Mobilising a defence organisation

On the basis of the Protection Organizational Assignment, the number of workers corresponding to the degree of protection is set to work.

Figure 33. Clearing the drainage canal (photo archive of NYUDUVIZIG)

Operation of protective structures

Activities in accordance with the ordered degree are carried out at inland water protection facilities, which are:

Drainage canals and their structures

The drainage canal transfers the collected waters of the excess water system to the main recipient, which is usually a river between dykes. Water is supplied to the recipient by gravitation, pumping or the combination of the two.

Pump stations

Pump stations provide channel water to the recipient if gravity cannot be solved.

Portable pumps

If the capacity of stable pump units is insufficient for drainage, and local pumping interventions are required by installing mobile pump units, this can help flood protection activities.

Measures after the termination of excess water control

After the completion of the excess water control, the protective materials and equipment should be collected and the assessment of their status should be done: damage assessment, maintenance, making accounts, creating the summary report, etc.

The logistic tasks of the procurement, installation and operation of the portable pumps

The logistic tasks of the installation of the mobile pumps

In peacetime, portable pumps are located in flood control storehouses. When ordering the excess water control, the logistic process starts, the steps of which are explained below:

The logistic process can be divided into nine main steps:

1. The process of declaring the level of alert and necessary interventions:
   - ordering the installation
   - assessment of installation sites (determining device, equipment and material requirement)
   - determining the installation method
   - warning – personal announcement
   - providing transportation and loading

Figure 34. Incorrect soil cultivation (photo archive of NYUDUVIZIG)

2. Transport:
   - moving and loading
   - providing a shipping route
   - organising the structure of supply background

- provision of personal care: catering, work safety, social background
- providing fuel supplies
- determining delivery schedule
- quantity and date scheduling
- structure of a communications chain
- transport of materials and tools (supplier, loading, personal and installation vehicles, earthwork machines)

3. Starting the installation:
   - organising the traffic order of the site – traffic management
   - providing the equipment for the site, landscaping
   - loading, pre-assembling
   - placement and adjustment of pumps
   - installation of piping
   - building a fuel supply system
   - creating illumination
   - performing test runs
4. Operation:
   - providing an operational staff with 12 hour shift
   - on-site maintenance of machine-condition repair (crane truck, workshop car, spare pump, etc.)
   - provision of fuel supply according to schedule according to operation data
   - providing lubricants for operation and intermittent oil change
   - management of engine logs, registration of fuel and lubricant use
5. End of operation:
   - termination of the reasons for the operation
6. Disassembly ordering:
   - providing a shipping route
   - personal insurance of traffic control permits and crossing points
   - ensure the personal and mechanical conditions for assembly
   - management required for assembly
   - availability of transport and loading machines
   - preparing a reception site
7. Beginning of demolition works:
   - disconnection of the fuel supply system
   - disassembly and loading of pipelines and pumps
   - other materials and equipment loading
   - delivery to the site
8. Maintenance performing:
   - repair and maintenance of pumps
   - maintenance of pipe kits
   - maintenance of fasteners, gaskets, auxiliary materials
   - placement and storage of equipment
   - restoration of pumping site, landscaping

9. Evaluation:
    - evaluation of the operation data
    - evaluation of the whole process
    - modification and refinement of defence plans as needed

Expectations for installation sites

Portable pumps can be installed in a built-in location, on a place that was built in advance but the excess water control should also proceed unhindered when the installation is carried out on an unplanned site. If the site is prepared in advance, less pipe volume will be required and less assembly time and the location could be reached easier. If excess water control is to be started in an unplanned place, it is important to pay attention to the fact that a larger amount of pipe will be needed, so expect a more difficult access because of the roads, weather, etc.

Figure 35. Excess water control without transport possibility on the dike during the flood in 2013 (photo archive of NYUDUVIZIG)

Ensure the possibility of using an off-road vehicle and prepare for the possibility that assembly will be complicated – because of terrain. It will be necessary to ensure access through the pressure pipes, e.g. when crossing on the flood control dikes and fuel supply difficulties may also occur.

Figure 36. Excess water control in an unplanned place in 2013 (photo archive of NYUDUVIZIG)

Case study – Mezőtúr – Temporary flood gate

In the estuary of the Hortobágy-Berettyó, there are three very important facilities for flood protection and excess water control: so-called triple sluice, the flood gate and the temporary portable pump of 12 m^3/s capacity. The triple sluice was built in 1899 in the mouth of the Hortobágy-Berettyó. The three-hole structure's board was initially made of wood; after the reconstruction works, it was changed to steel board. Its task is to guide the collected excess water of the Hortobágy-Berettyó to Hármas-Körös and to exclude the flood waves of the river. In 1940–1942, the flood gate was built in the new riverbed at the same time as the Békésszentandrás Dam. On the one hand, the flood gate ensures the conditions for shipping on the distance between Hortobágy-Berettyó and Mezőtúr, on the other hand it excludes the flood waves of Hármas-Körös. The function of the temporary portable pump of 12 m^3/s capacity is to pump the water of the Hortobágy-Berettyó into the Körös in the case of the permanently high flood wave of Hármas-Körös. The pumping structure was built in 1993–1994 on the island between the old and the new river section. 24 pump units of 0.5 m^3/s capacity can be installed in the pre-formed positions which was also the case during the excess water control in the spring of 2013.

Figure 37. The temporary portable pump during the excess water control in the spring of 2013 (photo archives of TIVIZIG, KÖVIZIG and KÖTIVIZIG)

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Ivana Kunc

Management of Navigation during Floods

Introduction

The main goal of the course is to provide knowledge for better utilising the inland waterways potential in the Middle (Central) Danube Region, improving the safety of inland navigation in critical hydro-meteorological periods (especially: high navigable water level), using the Danube, Sava and Tisza Rivers as environmental friendly transport routes which takes into account further risks due to climate change.

In networking with the Sava and Tisza Rivers, the Central Danube (from River-km 1,791.33 to River-km 931.00) is more and more developing into a main Region traffic axis, thereby directly connecting seven countries via waterway. The provision of minimum fairway parameters, crucial for cost-effective inland navigation must be achieved by improved waterway maintenance, better communication and monitoring.

Figure 1. IWT Area – Danube, Sava and Tisza River [31]

The objectives of the course are the following:

- understand the River Information Services (RIS) as a tool of navigation safety and good potential to simplify cooperation between inland waterway transport users and different public national and international authorities in critical hydrological conditions
- meet some practical aspects of RIS technologies as a basis for a variety of services including fairway information services and recognise when there is a probability of very high water levels, RIS can be used as a warning system for the users of the fairway

- take into account that emergency rescue teams in the event of an accident can monitor vessels, detect the transport of dangerous goods and actions related to ship-borne waste
- Outcomes have to be:
- understanding that RIS assist freight and passenger shipping and is full regulated by the relevant EU directives
- accepting rivers as a highly complex, multidimensional, dynamic ecosystem that represent much more than just longitudinal channel networks; ship-borne waste along the rivers can affect the river ecosystem

The second chapter reports the core targets and strategies of inland waterways which are essential for inland navigation on the middle section of the Danube and its navigable tributaries, the rivers Sava and Tisza. The current state of the waterways of international importance and strategic objectives are addressed in the third chapter, which describes elements of waterway infrastructure and presents the classification of inland waterways. This chapter also highlights the safety of navigation during transport of dangerous goods. The fourth chapter of this course presents general information of RIS possibility as a tool of navigation safety, including practical aspect of RIS service and activities during high navigable water level. The fifth chapter pays attention to the problems of ship waste management and presents the classification of ship-borne waste. It also shortly describes recommendations on collection of waste from vessels of the Danube.

In the fourth chapter of the course, students will be provided four hours of RIS Training in the Belgrade Harbour Office and/or Directorate of Inland Waterways, Belgrade.

Figure 2. Passenger ship, Sava River, Belgrade, 2014, high navigable level (PPD Company, Serbia)

Background – Targets and strategies of IWT

This chapter describes the core targets and strategies which are relevant to inland navigation on the middle section of the Danube and its navigable tributaries, the Sava and Tisza rivers. The focus in Europe and the Danube Region is clearly oriented toward sustainable and efficient transport and Inland Waterway Transport (IWT) is recognised as an environmentally friendly and safe mode of transport.

New EU Transport Infrastructure Policy

The new infrastructure policy of the European Union (EU) was established by Regulation No. 1315/2013 of the European Parliament and of the Council on guidelines for the development of the trans-European transport network, issued in December 2013. This Regulation fundamentally reformed the infrastructure policy of the 1980s. The European Commission published new maps depicting nine major corridors that will act as a backbone for transportation within Europe's single market and considerably change the connections between the East and the West. In accordance with the new policy, the allocated EU funds will be focused on establishing a powerful common European transport network.

The Rhine–Danube Corridor is one of the nine European corridors of the new unified trans-European transport network (TEN-T network). It covers the waterway of the Rhine, via the Main and Danube connecting the central regions around Strasbourg and Frankfurt via Southern Germany to some capitals of riparian states of the Danube (Vienna, Bratislava, Budapest and Belgrade) and downstream stretches (Romanian, Bulgarian, Moldavian, Ukrainian) and finally the Black Sea.

In the past, the Danube was a priority corridor on its own, but limited as a waterway. Now the Rhine–Danube Corridor is a unique system of waterways, connecting important railways and roads of Central and Southeast Europe to the industrial centres of Germany and France. With this approach it will be possible to connect and integrate transport infrastructure, including ports, and to remove technical and administrative barriers in the multimodal transport and ensure free flow of information in navigation. In this sense, with the transport network consisting of the Danube waterway in the length of 860 km, the Middle Danube section with all its tributaries holds a special importance within the overall European transport policy.

EU Strategy for the Danube Region – Priority Area 1a – "To improve mobility and multimodality: Inland waterways"

The goals of the Priority Area 1a of the EU Strategy for the Danube Region – "To improve mobility and multimodality: Inland waterways" (Danube Strategy), are to increase the cargo transport on rivers by 20% by 2020 compared to 2010, remove obstacles to navigability taking into account the specific characteristics of each section of the Danube and its navigable tributaries, and to establish efficient inland waterway infrastructure management.

Having in mind the European Strategy 2020 for smart, sustainable and inclusive growth, the Danube Strategy, and the 2011 White Paper *Roadmap to a Single European Transport Area,* and taking into account the Convention regarding the Regime of Navigation on the Danube (Belgrade Convention) and the European Agreement on the Main Inland Waterways of International Importance (AGN), within the Priority Area 1a of the Danube Strategy, the transport ministers of 8 riparian states of the Danube which are EU members, signed in 2012 the *Declaration on Effective Waterway Infrastructure Maintenance on the Danube and Its Navigable Tributaries* (Luxembourg Declaration).

The Luxembourg Declaration is a document produced as a result of interdependency of strategic areas such as transport, environment and sustainable development of the Danube Region, and due to increasing significance of the IWT for the development of the European economy, in particular the Danube and its navigable tributaries as a part of the TEN-T network. The document recognizes the challenges of the IWT and takes into account further risks due to climate change.

This joint document placed an emphasis particularly on the importance of national and cross-border coordination procedures in order to efficiently respond to extraordinary circumstances, or low water periods, ice and floods, for the purpose of establishing the best conditions for smooth navigation. The Luxembourg Declaration further emphasised the significance of maintaining continuous and up-to-date communication on the fairway situation, especially fairway depth and width data in the critical sections, through national administrations, and in particular via relevant River Information Services (RIS) operators.

The Steering Group of the Priority Area 1a of the Danube Strategy is responsible for monitoring the implementation of the Luxembourg Declaration and the partner signatory governments are required to take measures at the national level. The Steering Group has initiated a Fairway Maintenance Master Plan for the Danube Region, taking into account the mentioned objectives of the Priority Area 1a, and the fact that those shippers, terminal operators, logistic service providers and other users of IWT have constantly asked for the improvement of navigation conditions and elimination of the existing, mostly financial, technical and administrative barriers. The Plan proposes short-term measures and emphasises national projects to ensure smooth navigation in accordance with the existing international legal framework and objectives of the Priority Area 1a of the Danube Strategy.

Joint Statement on Inland Navigation and Environmental Sustainability in the Danube River Basin

Inland navigation can contribute to making transport more environmentally sustainable, particularly where it substitutes road transport. It can, however, also have significant influence on river ecosystems, jeopardising the goals of the Directive 2000/60/EC of the European Parliament and of the Council (EU Water Framework Directive), which aims for the "good ecological status" of all waters by 2015.

Recognising this potential conflict, the International Commission for the Protection of the Danube River (ICPDR) has linked up with the Danube Commission (DC), and the International Sava River Basin Commission (ICRBS) to conduct in 2007 an intense, cross-sectoral discussion process. As a result of a hard interdisciplinary discussion, a final document *Joint Statement on Inland Navigation and Environmental Sustainability in the Danube River Basin* (Joint Statement) was adopted in January 2008 by ICPDR, DC and ICRBS on a high-level.

The Joint Statement summarised principles and criteria for environmentally sustainable inland navigation on the Danube and its tributaries, including the maintenance of existing waterways and the development of future waterway infrastructure.

The Joint Statement is a guiding document for the development of the *Programme of Measures* requested by the EU Water Framework Directive (WFD), for the maintenance of the current inland navigation in the Danube, as well as for the planning and the investments in future infrastructure and environmental protection projects.

Figure 3. Length of national sectors and common stretches of the Middle and Lower Danube [26]

Current state and development of inland waterways: Danube, Sava and Tisza Rivers

The focus of the course are waterways of international importance which represent primary resource of the network of waterways within the territory of the Republic of Serbia consisting of the rivers Sava and Tisza along the Danube. Apart from the transport of goods and passengers, the activities carried out on those waterways also fall within the remit of river basin management (flood protection, land improvement, water supply of cities and industry), energy generation (hydropower plants), environmental protection, tourism, recreation, etc. Each of the mentioned functions of river flows leaves its unique mark in the process of decision-making on upgrading and development of inland waterways. Also, they correspondingly influence the maintenance costs, but this is why they also enjoy the benefits arising from these processes. This partaking, although apparent, often cannot be clearly isolated and quantified, but it nevertheless clearly suggests a need for an integral approach to the development of waterways in order to achieve optimal results in all the above mentioned areas with acceptable costs.

Regarding the infrastructure of the waterway, the main problem is related to an apparent lack of continuous technical maintenance, being a consequence of decades of neglect of this economic sector in terms of insufficient allocation of funds. The result of such an approach has also been partial utilisation of the waterway in comparison to available capacities, threatening to become a permanent state and therefore jeopardise the Republic of Serbia's strategic position on these rivers.

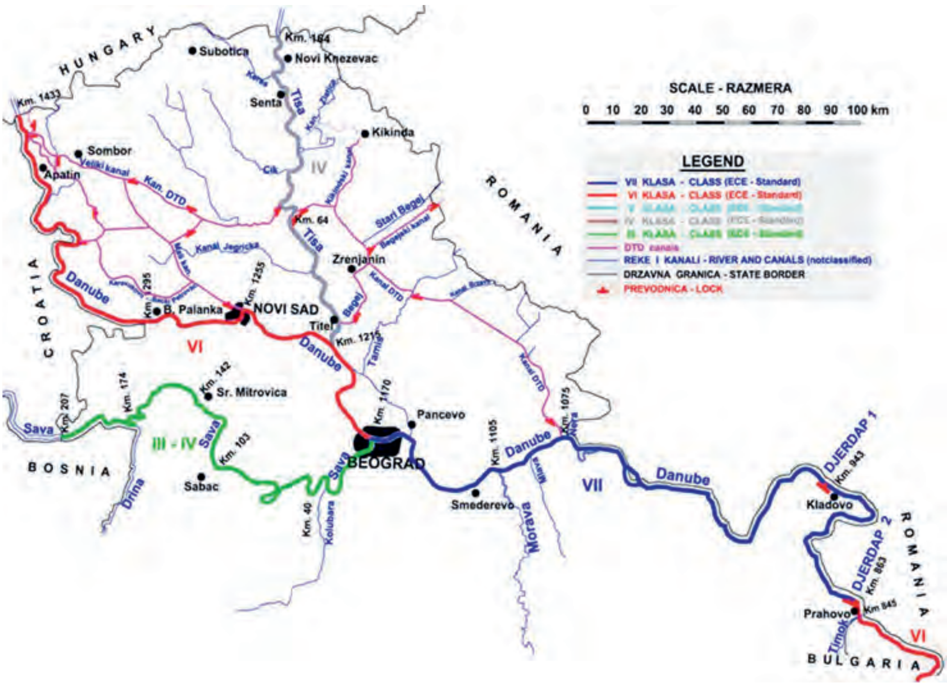

Figure 4. The Danube, Sava River and Tisza River – Republic of Serbia [29]

Elements of waterway infrastructure and classification of inland waterways

The size of inland vessels or convoys suitable for specific inland waterways depends mainly on the current infrastructure parameters of the waterway concerned. Determinants of waterway infrastructure for safe navigation are:

- fairway (depth, width, curve radius)
- lock chambers (available length, width, depth)
- bridges (clearing height and available passage under bridges)
- the width and the route of the fairway are marked by
- internationally standardised fairway signs such as buoys
- or marks on river banks

The parameters of international waterway classes are based on the type of inland vessels and convoys with maximum dimensions (length and width) and minimum drought loaded of vessels, which can navigate the waterway of the respective class. Waterway classes are identified by Roman numbers from I to VII. Waterways of class IV or higher are identified as important for beneficial inland waterway transport (IWT).

Table 1. Waterway classes and determinants of waterway infrastructure (AGN, UNECE)

Pushed convoys						
Type of convoys: general characteristics						
Waterway class	**Formation**	**Length L(m)**	**Width B(m)**	**Draught d(m)**	**Deadweight T(t)**	**Min height under bridges H (m)**
IV	1-lane/1 unit	85	9.5	2.5–2.8	1,250-1,450	5.25/7
Va	1-lane/1 unit	95–110	11.4	2.5–4.5	1,600–3,000	5.25/7.00/9.10
Vb	1-lane/2 unit	172–185	11.4	2.5–4.5	3,200–6,000	5.25/7.00/9.10
VIa	2-lane/2 unit	95–110	22.8	2.5–4.5	3,200–6,000	7.00/9.10
VIb	2-lane/4 unit	185–195	22.8	2.5–4.5	6,400–12,000	7.00/9.10
VIc	2-lane/6 unit or 3-lane/6 unit	270–280 195–200	22.8 33.0–34.2	2.5–4.5 2.5–4.5	9,600–18,000 9,600–18,000	9.10 9.10
VII	3-lane/9 unit	275–285	33.0–34.2	2.5–4.5	14,500–27,000	9.10

According to the AGN classification and Inventory of Main Standards and Parameters of the E Waterway Network, the so-called “Blue Book” as a supplement to the AGN (UNECE 2012):

- the Danube between Budapest and Belgrade is classified as a waterway class VIc (2-lane and 3-lane/6 unit convoys)
- the Danube downstream from Belgrade to the Danube Delta (Tulcea, Romania) is classified as a waterway class VII (highest class according to UNECE classification)
- the Sava River (navigable length through Croatia, Bosnia and Herzegovina and Serbia: 586 km) is classified from class III to IV (in Serbia)
- the Tisza River (navigable length through Hungary and Serbia: 685 km) is classified from class I to IV (in Serbia)

The most important reference water levels for IWT are:

Low Navigable Water Level (LNWL)

Highest Navigable Water Level (HNWL)

There are no guaranteed minimum fairway depths at LNWL, skippers and ship operators have to plan their journeys according to the fairway depths which are currently available at the shallowest stretches of the waterway or according to the acceptable maximum draught loaded (draught of immobile vessel) as foreseen by the waterway regulations.

If the HNWL is reached or exceeded by over a certain degree, the authority responsible for the concerned waterway section may impose a temporary suspension of navigation for reasons of traffic safety.

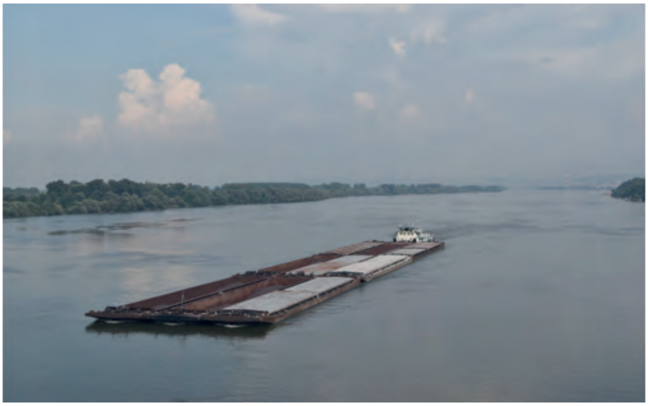

Figure 5. Ship "Karlovac" with a 9-barge convoy, high navigable level (JRB Company, Serbia) [32]

One of the key factors in the procedure of providing competitive transport services in waterway transport is the state of development and infrastructure of the waterway. Sufficient and stabile dimensions of a waterway (width, depth and clearance height and available passage width under bridges) create the possibility for continuous flow of mass transport at competitive prices. An additional upgrade of the category of the waterway enables the utilisation of larger waterway structures and thus further increases the efficiency of this form of transport. In this way, branching out of waterways network, its interconnectedness, physical characteristics of waterway as well as the very traffic management all represent key factors that directly affect the traffic price, share in modal distribution, energetic efficacy and environmental impact.

Reliability, as another relevant consequence of the physical condition of waterways, represents a decisive factor in modal choice. It is especially jeopardised in periods of low water levels that occur several times annually when navigation is disabled temporarily, partly or completely at critical navigation sectors. From the viewpoint of navigation, low water levels also mean small permitted vessel draft, which is additionally increasing the transport price and encouraging its fluctuation. Also, due to unfavourable navigation conditions during these periods, the danger of occurrence of navigational accidents increases, which may suspend transport in a critical sector for a longer period of time. Lack of adequate maintenance of waterways additionally worsens this situation along with a negative effect on pricing policy in water transport.

Extreme water level oscillations combined with the impact of climate change will have a significant part in the future scope of activities and thereby in costs of further maintenance of waterway infrastructure.

The current state of inland waterways in the Republic of Serbia

By its accession to the AGN, the Republic of Serbia with a strategic position that covers the confluence of the Sava to the Danube and the Tisza to the Danube has committed itself to build and develop its inland waterways of international importance in accordance with uniform technical and operational characteristics contained in the “Blue Book”. Aiming to make it easier for the states to focus their infrastructural projects on further development of an integrated network of inland waterways, the Serbian competent authority established a list of the most important bottlenecks and missing links in the network of Serbian waterways.

The Danube River

Throughout its entire inland waterway length of 587.6 km in the Republic of Serbia, the Danube River is an international waterway with free navigation for all flags. With a part of its river flow, the Danube River is creating a natural border with the Republic of Croatia (still not defined) and with Romania.

Criteria for the waterway categories at certain river sectors are not met and those sectors are considered critical. It is needed to meet the requirements for the category VIc in accordance with the AGN on the part of the Danube from its border with Hungary (km 1,433.1) to Belgrade (km 1,170.0). A total of 24 critical sectors has been identified in this section, not fitting into this waterway class and therefore limiting the navigation due to insufficient width or depth of waterway at low water level, as well as due to morphological instabilities. There are 17 critical sectors at the common section of the waterway between the Republic of Serbia and the Republic of Croatia, while the remaining 7 are to be found on the part of the waterway from Backa Palanka to Belgrade. The preparation of the project documentation for 6 critical sectors was finalised, and the completion of the realisation of hydro-technical work was planned for the end of 2017. On this part of the Danube River, there is a temporary Road–Railway bridge in Novi Sad identified as one of the critical sectors that is not meeting the prescribed criteria of waterway dimensions. At the same location, a part of the river in a hairpin turn with a radius less than 1,000 m remains a critical sector that has not been rationally eliminated.

The parameters of waterway class VII need to be fulfilled on the part of the Danube River from Belgrade to the border with Bulgaria (km 845.5). The waterway is in the zone of water surface of accumulations of Hydropower Plant (HP) “Djerdap 1” (km 943) and HP “Djerdap 2” (km 863); therefore, navigation conditions are favourable for the most part of the year and depend on the exploitation regime of the waterpower system. In some sectors within the zone of the Djerdap, problems of insufficient waterway width occur, though this cannot be changed given the morphology of the terrain. In the period of low waters, occasional obstacles to navigation downstream of HP “Djerdap 2” occur also due to the remains of German warships that sunk by the end of WWII.

Hydro-technical objects along the waterway that were built in the period between the years 1960 and 1995 are already at the end of their foreseen service life and, in order to extend their positive effect, an investment in their rehabilitation or reconstruction is necessary as soon as possible.

The waterway of the Danube River is completely marked in accordance with the applicable international regulations. There are officially proclaimed safety objects of navigation on the waterway: winter ports, shelters and anchorage. Existing dry docks near Apatin, Novi Sad, Ivanovo and Kovin are at the moment used as winter ports in times when there is ice over the Danube, but they only partly meet the prescribed requirements.

The Sava River

With one part of its river flow through the Republic of Serbia, at the length of 210.8 km, the Sava River represents an international waterway with free navigation for all flags. It also constitutes a natural border with Bosnia and Herzegovina.

At approximately 14% of its length in the Republic of Serbia, the waterway of the Sava River does not meet the minimum requirements for international waterway defined by waterway class IV. In accordance with the AGN, it is recommended to meet the parameters of at least waterway class Va during the modernisation of the waterways.

The project of the Preparation of the Project Documentation for Hydro-technical Works on the Sava River from Belgrade (km 0) to Brcko (km 231) and the Project of the Preparation of the Study of Environmental Impact Assessment for Hydro-technical Works on the River Sava from Belgrade (km 0) to Brcko (km 231) were initiated in November 2013 (IPA 2010, Bosnia and Herzegovina). Project documentation for works are not completed.

The waterway of the Sava River is not completely marked, while the activities on a marked part are carried out in accordance with valid international regulations. Winter shelters are proclaimed on the waterway, but there are no officially proclaimed anchorages.

The international regime of navigation also applies on the right tributaries of the rivers Sava, Drina (at length of 15 km) and Kolubara (at length of 5 km). At the moment, these waterways meet the requirements for waterway class I and there is no commercial traffic on these rivers apart from occasional recreational navigation.

The international waterway of the Sava River is becoming quite a priority in waterways development of the Republic of Serbia, especially after the catastrophic floods throughout the whole region. Namely, the natural disaster in May 2014 speeded up the highest-level agreements of member countries of the International Sava River Basin Commission about the promotion of cooperation within the Sava River basin by using all so far developed instruments of the Secretariat of the Sava Commission for the project implementation.

The agreement on better coordination of work on projects of common interest for the Republic of Serbia, Bosnia and Herzegovina, the Republic of Croatia and the Republic of Slovenia that are planned and implemented on the basis of the Framework Agreement on the Sava River Basin is of an exceptional significance for the entire region in the period from 2015 to 2025.

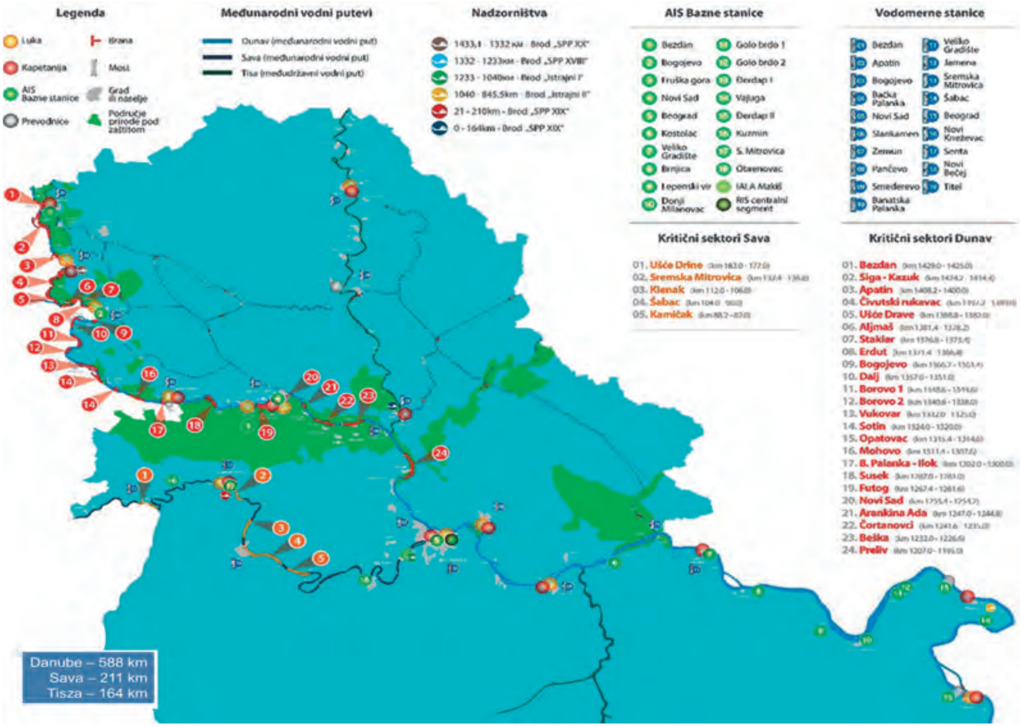

Figure 6. Current state of navigability – Republic of Serbia [30]

The Tisza River

By signing the AGN Agreement, River Tisza has become a river of international importance in all its length of 164 km within the Republic of Serbia.

Critical sectors on the River Tisza are related to hairpin turns and dimensions of ship lock at the Novi Becej dam that would need to be extended in order to be in accordance with the requirements of waterway class IV.

The waterway of the Tisza River is completely marked in accordance with the applicable international regulations. There are officially proclaimed safety objects of navigation: winter ports, shelters and anchorages.

By the Agreement between the Government of the Republic of Serbia and the Government of Hungary on Navigation on the Tisza River which was adopted in 2017, both sides agreed to develop the inland waterway transport of the Tisza River and closely cooperate on navigation management and environment protection.

HS DTD Canals

In terms of the regulation of water regime, the canals Hydro System Danube–Tisza–Danube (HS DTD) are of vast significance for sustainable development of this part of the Republic of Serbia. At the same time, throughout all its construction phases, this canal network is designed as a single waterway integrated in the waterways of the rivers Danube and Tisza in the region of Vojvodina. The total navigable length of the canal network is about 600 km, out of which 13.1 km belongs to waterway class Va, and 289.8 km to waterway class III as determined by the state waterways classification in 2013. Out of the total number of 17 built ship locks, 12 of them have the following dimensions: 85 × 12 × 3 (including the ship lock of the Dam on Tisza) and all are in function. The authority for technical maintenance of waterways on the canals HS DTD is the Public Water Management Company "Vode Vojvodine". At the same time, they are in charge of this water resource management structure.

Development plans for improvement of water transport on the HS DTD Canal Network include the elaboration of designs of new types of vessels principally aimed for canal network and watercourse navigation in waterway class III. In the HS DTD canals, there is a need for revitalisation of affected sections contaminated by mud (Vrbas, Zrenjanin), as well as for the rehabilitation of the Bezdan ship lock as an entry point from the river Danube into the DTD canal system (Great Backa Canal) near Bezdan, which was out of function for more than 30 years. It is also necessary to prepare the Spatial Plan of Special Purpose of the Region of the HS DTD Canal Network aiming to define the purpose of the bank and the bank area of the canal network.

Strategic objectives of the safety of navigation – Danube, Sava, Tisza

Serbia has initiated a set of activities in order to modernise inland waterways transport infrastructure and to secure the preconditions for safe navigation on the Danube, Sava and Tisza rivers, being part of the Rhine–Danube Corridor. The goal to improve the condition of waterways for the next period, that is, to create conditions where IWT will become a safer, more reliable and more efficient form of transport with respect to current environmental standards during the process of planning and designing is a reasonable and realistic goal. Maintenance and development of infrastructure on the waterways for navigation purposes is based on clearly defined international standards. Navigation conditions on the rivers Danube, Sava and Tisza have to improve in accordance with European development plans for the waterway transport and undertaken international commitments.

In general, strategic objectives of the management of navigation are:

– improving the quality level of technical maintenance of waterways in accordance with the new EU infrastructure policy and the Luxemburg Declaration, fulfilling the requirements of the AGN referring to the dimensions of ships defined for each category of the waterway, with full protection of the environment

- recognised benefits of international inland waterways development and importance of navigation safety in accordance with the modern communications technology
- preservation of favourable condition of ecologically important river areas and improving the deteriorated condition of parts of the ecological network consisting of ecologically significant areas, ecological corridors of international importance and protection zones
- fleet modernisation, in particular for the transport of dangerous goods
- integration of a navigation monitoring system on the Danube

It is necessary to begin, as soon as possible, with the activities of the bottleneck removal on 17 remaining critical sectors of the River Danube in the part where it is representing a border between the Republic of Serbia and the Republic of Croatia. In these activities, positive experiences and already established methods from previous projects should be utilised, especially in the part of application of innovative and generally acceptable solutions that will have a minimum environmental impact (an integrated approach to planning of river infrastructure from the very beginning of the project). The unsolved issue of the border between the Republic of Serbia and the Republic of Croatia can potentially influence the dynamics of the elimination of all bottlenecks. For greater efficiency, it is necessary to jointly determine priorities among the critical sectors that are not conditioned by the resolution of the state border issue.

The international waterway of the Sava River is becoming a priority for the region development. By doing so, a reliable and efficient traffic connection of Serbia with the states of the Sava River Basin (Slovenia, Croatia and BiH) will be achieved, as well as the full integration of this waterway with the Rhine–Danube Corridor, that is, the connection of the Sava River with the core network of European waterways. In that sense and in terms of technical maintenance of the Sava River waterway, it is necessary to provide access equal to the one applied on the rivers Danube and Tisza, that is, an equal level of quality of services and infrastructure should be offered on all rivers with the international regime of navigation.

In case of a flood event, ice or in other times of emergency within the network of international waterways, besides the existing dry dock ports, winter ports and shelters are used for the reception of vessels. Large material investments are needed for the equipment of winter ports, and given the economic situation, requirements for winter ports in case of emergency should be prescribed very soon based on the previous practice and concerning that there were no problems related to the protection of ships in the existing winter ports and shelters.

An additional solution for helping the vessels in case of flood, ice and other emergency situations is the establishment of adequate shelters on the parts of the river course with a favourable configuration of the coast, the utilisation of port pools of the DTD Canal, as well as the port pools at other locations. Countries such as Austria, Germany and Slovakia chose these solutions and they use exclusively port pools to protect national and foreign vessels in emergency situations.

The watercourse of the Danube River, as a key part of the Pan-European Rhine–Danube Corridor, is a huge potential for the cooperation between EU and non-EU Danube riparian countries. The possibility of further EU financing of the projects of river infrastructure in accordance with its new policy, especially given the cross-border projects, is realistic, especially if they prove to be the best projects of the highest priority for the wider region along the Pan-European Corridor.

Navigation safety – Dangerous goods

More intensive investments in preparation and equipment of ports and temporary shelters are needed, especially regarding ships that are carrying dangerous goods. Existing anchorages on international waterways are mostly parts of harbours and ports, areas of border crossings, as well as at the locations that have entered into the use due to nautical purposes, and until now they have proved to be favourable for anchoring vessels. In the following period, it is necessary to systematise the locations of the existing anchorages, to add the new ones, in places where they are needed, as well as to adapt to new regulations of the European Agreement concerning the International Carriage of Dangerous Goods by Inland Waterways (ADN), and to execute ADN procedures and obligation.

In accordance with the ADN, the competent state bodies have to control international and inland waterway transport of dangerous goods for the purpose of increasing the safety of people, property and protecting the environment, especially in case of flood event, ice or in other times of emergency. The strategic goal in fulfilling the safety requirements for inland waterways is establishing a network of ADN Safety advisors in all shipping companies, as well as thorough and permanent training of vessel crew members, i.e. the participants in transport of dangerous goods.

The procedure for issuing vessel approvals and licenses, i.e. the certificate form for the vessel which transports dangerous goods has to be in accordance with the ADN provisions. The technical rules for the construction and modernisation of ships which carry dangerous goods, separately for the transport of dry cargo, and separately for the tankers of type G, C and N, apply in all European countries, including Danube riparian countries, which are signatories to the ADN. The control of the application of the technical rules for ships within the strict timeframes, which are defined by the ADN rules, may be a limiting factor especially for tanker fleet. It is necessary to invest into the modernisation of the tanker fleet in accordance with the ADN rules and bearing in mind the required harmonisation with the technical standards for tanker fleet and the strict timeframes for the employment of double-hull vessels for specific types of liquid cargo, especially gasoline and crude oil (the deadline was on 31 December 2015) and diesel fuel (the deadline was on 31 December 2018).

Figure 7. "Zemun" tanker, tanks and tank barges (JRB Company, Serbia) [32]

River Information Services (RIS)

RIS is information and management service on the inland waterways. The implementation of RIS increases the safety and efficiency of inland navigation. RIS has been developed in Europe to assist the freight and passenger shipping.

RIS Technologies are specified in the EU Directive 2005/44/EC of the European Parliament and of the Council on Harmonised River Information Services (RIS) on Inland Waterways in the Community, which has been effective since 20 October 2005. EU "RIS Directive" regulates:

- technical standards for RIS implementation
- standards of vessel equipment
- standards of RIS data exchange

The other relevant EU regulations which are the basis for different services, including fairway information services and traffic information and management:

Regulation of the Commission No 414/2007 on Technical Instructions for the Planning, Implementation and Operational Use of RIS Referred to in Article 5 of Directive 2005/44/EC

Regulation of the Commission No 415/2007 Concerning the Technical Specifications for Vessel Tracking and Tracing Systems Referred to in Article 5 of Directive 2005/44/EC (VTT)

Regulation of the Commission No 416/2007 on Technical Requirements for Notices to Skippers as Referred to in Article 5 of Directive 2005/44/EC (NtS)

Regulation of the Commission No 164/2010 on Technical Requirements for Electronic Reporting from the Vessels on Inland Waterways Referred to in Article 5 of Directive 2005/44/EC (ERI)

Regulation of the Commission No 909/2013 on Technical Specifications for the Electronic Chart Display and Information System for Inland Navigation (Inland ECDIS)

The following mandatory RIS technical standards are available for use:

Vessel Tracking and Tracing (VTT), based on inland automatic identification system (Inland AIS) technology

Notices to Skippers (NtS)
System for generation and distribution of electronic navigational charts (ENCs)
Electronic Reporting International for voyage and cargo data (ERI)

Implementation in Serbia (non-EU country)

River Information Services (RIS) were implemented in Serbia under the EU IPA 2007 project *Implementation of River Information Services in Serbia* between 2009 and 2013. The RIS infrastructure includes a central segment of the system situated in Belgrade, an IALA dGPS system and a network of 18 base stations of the automatic identification system (AIS), which ensure that the entire stream of the Danube and the Sava through Serbia are covered by the automatic identification system (AIS network) signal.

The Directorate of Inland Waterways continually (24/7) monitors the integrity and maintains the functionality of the river information service infrastructure and provides support to all RIS users in the Republic of Serbia. Harbour offices are a part of the Ministry for Construction, Transport and Infrastructure in Serbia regularly using the RIS and monitoring the waterway traffic.

A program aimed at equipping public vessels and commercially-operated vessels with appropriate RIS equipment has been implemented in order to support the implementation of river information services. As a result, more than 150 vessels have been equipped with the necessary equipment for accessing RIS services.

Continuous development of the RIS involves harmonising the remaining regulations of the way of organisation and establishment of the RIS in the Republic of Serbia with the European standards. In the Republic of Serbia, RIS shall, by using modern and reliable technologies, improve services in accordance with the needs of development of water traffic from 2015 to 2025, i.e. it shall be fully harmonised with the RIS systems on the European network of waterways, of which it is an integral part.

Figure 8. RIS Centre, Belgrade [30]

The way RIS infrastructure works is shown in Figure 9.

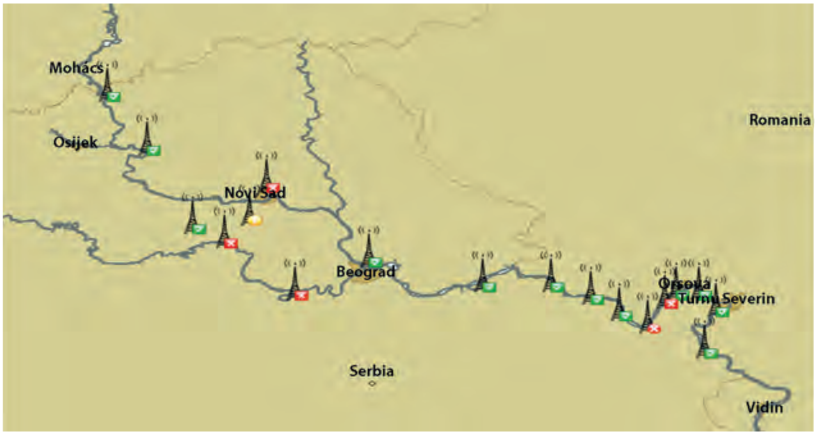

Figure 9. Base stations in Serbia [30]

Figure 10. AIS base station, Serbia [30]

Figure 11. Vessel equipment, Serbia – Harbour office Bezdan, Danube [30]

Table 2. Summary of the installation, Serbia [30]

Segment	Description	Quantity installed and accepted
Ship Segment	Government vessels	49
	Commercial vessels	160
Shore Segment	AIS Base Stations	15
	IALA DGPS	0
Operator Segment	SW applications	9
	RISCentre Common Applications	1
	Telecom Network	1
Authority Segment	LUWS	37
	VHF far Captaincies	9
Logistic Segment	Web interface	2

RIS services in Serbia are available:

VTT Service – in line with EC Regulation 415/2007 and obligatory in Serbia from 1 January 2014

NtS Service – in line with EC Regulation 416/2007 and obligatory in Serbia from 1 January 2014

Inland ECDIS on Board – in line with EC Regulation 909/2013 and obligatory in Serbia from 1 January 2015

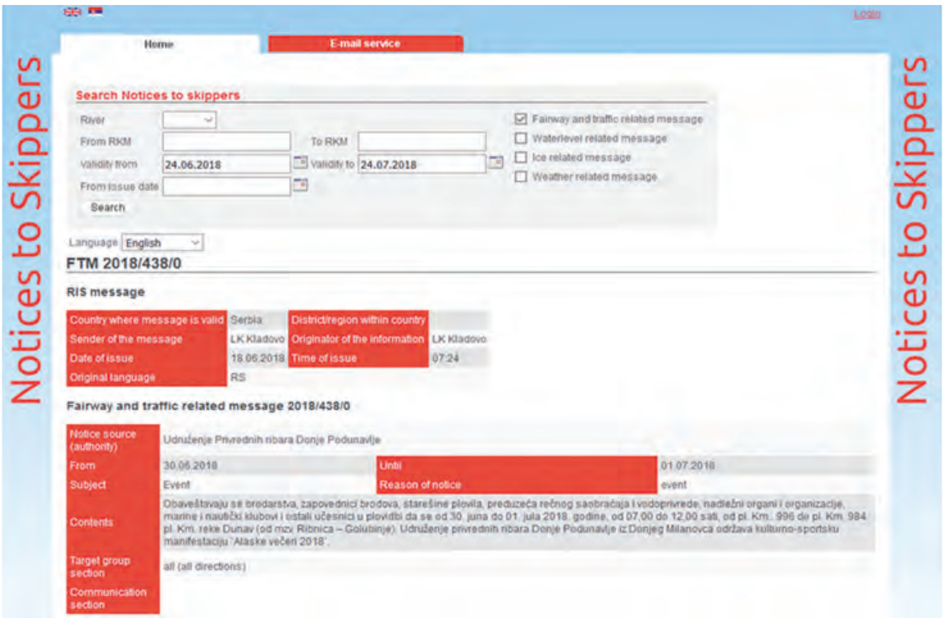

Figure 12. NtS, fairway and traffic related massage, km 996 – km 984, Danube, Serbia [30]

Figure 13. ENCs (electronic navigational charts), Sava and Danube River, Belgrade, Serbia [30]

Pilot operations, training of the users and administrators as well as the initial system operation support were also part of the services.

User training for all users of RIS services according to the Technical Specifications was held during the period 2010–2013.

Figure 14. Harbour office equipment, S. Mitrovica, Sava River (Harbour Office, Serbia)

Fairway Information Services – FIS gives two types of available information, static and dynamic (daily and hourly updated):

Daily water levels and hydrological forecast
Wind data
Active notices to skippers
Waterway marking plan
Available fairway depths and widths at critical sectors for navigation
Bridges – Available vertical clearances
Availability of locks
Availability of RIS (River Information Services)
Useful contacts are shown in Figure 15.

Figure 15. Useful contacts (compiled by the author)

Assessment of flood monitoring and forecasting in the Danube River Basin

In general, the floods which occur in the Danube River Basin can be divided into several main types as follows:

- Winter and spring floods caused by snow melting which can be combined with rain. This type of flood is most frequent in under-mountain areas but these floods can also affect lower reaches of the rivers.
- Winter floods caused by ice phenomena, which can occur also during the periods when the flows are relatively low. These floods occur in those river reaches which are exposed to formation of ice jams, etc.
- Summer floods caused by long-lasting regional precipitation. These floods usually occur on all watercourses in the area exposed to the precipitation with highest impacts along middle or large-size rivers. Summer floods caused by short high-intensity storms (frequently over 100 mm during several hours) affect relatively small areas. These floods can occur anywhere on small rivers with catastrophic consequences mainly in those basins that are highly declined and fan-shaped.

Activities associated with protection against floods are governed by the respective legislation of each Danube state. All measures are governed by the flood protection authorities as well as crisis authorities, especially in case of large disastrous floods. They are bodies of the state and/or municipal administration that should be fully responsible to co-ordinate and control all activities. The extent of the flood risk determines the order of the flood prevention activities:

I. State of Alert
II. State of Danger
III. State of Emergency
IV. Severe Situation

The major tasks of the meteorological services of the Danube states in the area of flood forecasting include monitoring and forecasting of the weather situation, and advisory and warnings on dangerous weather events such as heavy precipitation, storms, hail, etc. Quantitative precipitation forecast is the most important activities of the meteorological services, and it is provided through the use of numerical weather modelling. This information is supplemented by data from the meteorological satellites and maps of rain intensities provided by national meteorological radars.

The hydrological services should monitor the current situation on the rivers in the Danube River Basin by gauging stations which provide regular hydrological information that is supplemented with the data from the River Basin Authorities. The hydrological forecasting system is connected to RIS.

The flood forecasting service regularly provides hydrological forecasts to the Danube River Basin authorities and other stakeholders and publishes them on a website. During floods, it is accompanied by information of flood evolution and its further prediction.

Skippers and members of crew update navigation charts that exactly describe current hydrodynamic conditions. Good Electronic Navigation Charts (ENC), as one of the RIS services, are very useful tools for vessel skippers. Boat masters and skippers can use this information and the automatically updated navigation charts to safely and more efficiently steer their vessels. Electronic navigational charts that display both fairway directions and water depth enhance the comfort and security of sailing vessels.

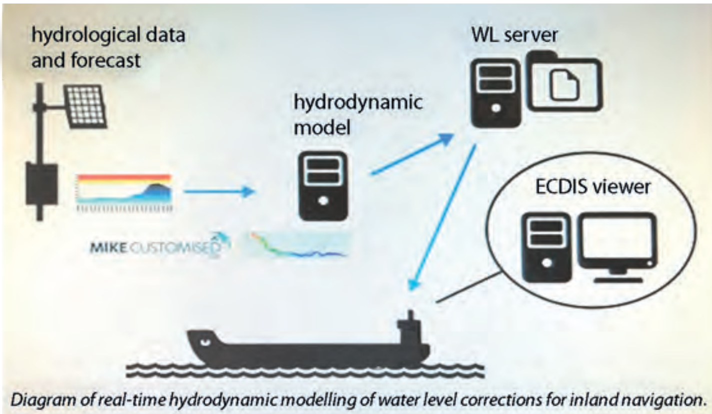

Figure 16. Diagram of real time hydrodynamic modelling of water level (MIKE HIDRO River modelling package)

Practical aspects and navigation during flood period

High navigable water level of free flow rivers at certain water gauge corresponds to the water level defined with the discharge duration of the 1% (Q1%). It is defined from statistical analysis of discharge duration taking into account 30 years of observation. Traditionally, it is used to define vertical clearance under the bridges or power line/cables. Usually, it is close to or higher than the regular level of flood defence.

During the flooding period, streams can have a very high velocity. It becomes hard for ships to stay on track and the risk of crashes increases. The fuel consumption increases, and navigation becomes less economically cost-effective. The waves of the ships can damage the riverbanks.

Figure 17. Navigation under the Belgrade bridges during floods, 2014 [30]

Sometimes, during floods the marking system for navigation gets useless.

Figure 18. Marking system during floods, 2014 [30]

Practical problems of navigation during floods, risk of pollution and RIS possibilities are the subjects of ongoing research by the competent authority on the rivers Danube, Sava and Tisza.

The first practical recommendation is to improve the internal communication and coordination of all activities in critical hydro-meteorological periods. That means to make a better Integrated Flood Management System (Flood Protection, IWT–RIS and Environmental Protection authorities) on international, national and local level.

Conclusions

- RIS makes it possible to determine transport times more precisely
- logistic service providers can link freight data to the traffic data provided by the RIS, enabling all partners in the logistic chain to track the transport cargo in real time
- port, berth and lock operators can achieve optimal usage of capacity
- when there is an expectation of very high water levels, RIS can be used as a warning system for the users of the fairway
- emergency rescue teams and authorities can monitor transport of dangerous goods, as well as the coordination of emergency rescue teams in the event of an accident
- RIS has potential for involved authorities to simplify cooperation between inland waterway transport users and all public authorities

Future researching is necessary in the field of:

- Emergency Management (Flood Risk Management) including activities of public water management companies, shipping companies, private recreation vessels and floating objects on inland waterways
- early warning system for the identification, management and prevention of spills due to accidental discharges, emergencies or accidents in order to avoid soil and water contamination

Ship waste management

It is extremely important for the region downstream of the Hungarian–Serbian border to urgently define the terminals which would serve as collecting stations for ship-borne waste in accordance with the Recommendations of the Danube Commission (DC) for waste management for inland navigation on the Danube, and the results of the current EU projects in the region.

Defining the terminals with the accompanying infrastructure for collecting, disposing and treatment of dangerous ship waste produced during the exploitation of vessels on the waterway network of the middle Danube sector, Sava and Tisza Rivers is extremely important for protecting the quality of water.

Based on the experiences of the countries in the Danube region, the length of waterways and the arrangement of terminals in other countries of the region, it can be concluded that it is necessary to plan at least four terminals in the Republic of Serbia, two on the Danube, one on the Sava and one on the River Tisza. Organised ship waste management would solve the problem of disposal of waste from many foreign and domestic users of the international waterways passing through the region, which could improve the ecological status of the waterways.

Figure 19. "Baja Green Terminal" – Mobile Service, Budapest, Hungary (Ivana Kunc, private photo collection, 2012)

Figure 20. JRB ship "Karlovac", engine service (Ivana Kunc, private photo collection, 2018)

Classification of ship-borne waste:

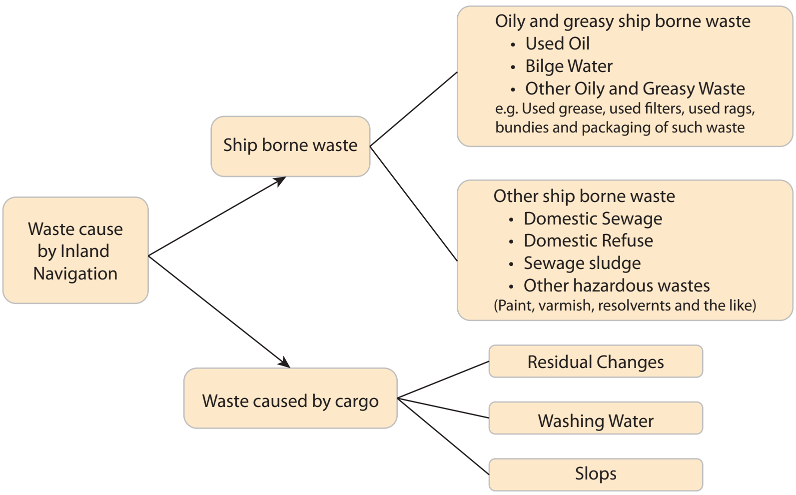

Figure 21. Classification of ship-borne waste on the Danube [4] [27]

Recommendation of the DC

Recommendations on collection of waste from vessels on the Danube were approved by the Danube Commission in 2007 (update 2009/2011). As of 1 January 2008, its application is suggested by the international organisation to all member countries. However, no country has so far applied it to its fullest effect.

The recommendations provide a detailed description for the categories of ship-borne waste, pollutant limit values and technological solutions for waste collection, as well as monitoring systems and financing methods. Their further development will be touched upon amongst the international harmonisation goals of the framework concept.

The geographical scope of application is the navigable sections of the Danube River and the areas of the ports. They are applicable to all authorities, skippers or other persons who are engaged in IWT on the Danube. They are applicable to all vessels navigating on the Danube; however, vessels falling under MARPOL 73/78 are seen to comply with the provisions. Newly built ships or ships that are currently modernised shall be equipped with the foreseen equipment on board for waste collection, for other vessels a transitional period of 8 years is allowed. Different types of waste are defined.

Waste shall be collected on board and have to be delivered to the foreseen waste reception facilities. Burning of household garbage and waste arising from the operation of the vessels can be allowed. However, the regulations of the competent authorities, which may also prohibit the use of incinerators, have to be followed. A discharge of bilge water is prohibited. Shut-off devices of pipes, which are foreseen for a direct discharge of bilge water, have to be formed in a way, that they can be sealed with leads in closed state. Vessels with an engine room/department have to carry an oil log in which the delivery of oily and greasy ship waste has to be recorded as well as the sealing with leads. Oil control log contains the information about the accepted oily and greasy ship operating waste: used oils, bilge water, used rags, used grease, old filters and packaging.

Tankers that transport dangerous goods obtain a “loading book” for recording activities related to hazardous substances. Authorities may check the oil log and the loading book.

Household garbage shall be collected and delivered separately (paper, glass, other useful materials and residual waste), if possible. In case of an accident, the skipper has to notify the responsible authority, reporting certain information. The delivery of oily and greasy ship waste as well as accidental pollution has to be registered in the appropriate ship documents. Local provisions for the collection of ship-borne waste have to be followed. Passenger vessels – including (daily) cruising boats as well as cabin vessels with a capacity of more than 12 passengers shall be equipped with on-board purification plants or collection tanks for domestic sewage. Limit values for on-board purification plants are foreseen.

The Danube States take measures individually or jointly in order to establish waste reception facilities. The responsible authorities announce the available network of reception facilities, the timetable of collection vessels as well as changes.

A waste reception facility has to be equipped with:

- a waste collection vessel which serves certain stretches of the Danube and/or

- a stationary reception facility (floating, onshore)
- onshore connection parts of the pipes for the delivery and for the reception of bilge water and domestic sewage have to correspond to the European Standard EN1305; adapters on the vessel have to correspond to the ISO 7608 standard

The authorities in charge have to check:
- the keeping of the books
- the sealing of the pipes from which harmful substances could be discharged into the water

If vessels are blamed for illegal discharge, they stop them, investigate the facts and write down a protocol. Violations can be noticed by governmental, cooperative and cooperating bodies as well as organisations for the protection of water quality and private individuals, who forward the information to the responsible authorities. The skippers are responsible for complying with these recommendations.

RIS can be a reporting tool for ship waste management in the future.

List of acronyms used

ADN: Agreement concerning the International Carriage of Dangerous Goods by Inland Waterways
AGN: European Agreement on the Main Inland Waterways of International Importance
AIS: Inland Automatic Identification System
CCNR: Central Commission for the Navigation of the Rhine
CDNI: Convention relative à la collecte, au dépôt et à la réception des Déchets survenant en Navigation rhénane et Intérieure (Convention on the Collection, Deposit and Reception of Waste Produced during Navigation on the Rhine and Inland Waterways)
CEVNI: Code Européen des Voies de la Navigation Intérieure
CO-WANDA: Convention for Waste Management for Inland Navigation on the Danube
DC: Danube Commission
DHI: Institut for Vand og Miljø (Danish Hydraulic Institute)
ECDIS: Electronic Chart Display and Information System
ENC: Electronic Navigational Charts
ENI: European Number of Identification (European Vessel Identification Number)
ERI: Electronic Reporting International
EU: European Union
EUSDR: EU Strategy for the Danube Region
GPS: Global Position System
ICPDR: International Commission for the Protection of the Danube River
IMO: International Maritime Organization
ISRBC: International Sava River Basin Commission
km: kilometre
MIKE: Hydro River Modelling Package
NtS: Notices to Skippers

RIS: River Information Services
t: ton
TEN-T: Trans-European Transport Network
UNECE: United Nations Economic Commission for Europe
VTT: Vessel Tracking and Tracing
WANDA: Waste Management for Inland Navigation on the Danube
WFD: Water Framework Directive

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Tibor Rácz

Flood Protection in Urban Environment

Flood protection – some basics of flood protection

Flood management – flood protection or flood control – has been changed in the past decades. Global challenges, such as climate change and the increasing human population, revealed the demand of change in our approach to floods in the frame of global water management. The World Meteorological Organization, as a non-profit professional organisation and the Global Water Partnership, as an open international network of organisations involved in water resources management, developed the Associated Programme on Flood Management (APFM). The APFM promotes the concept of Integrated Flood Management (IFM) as a new approach to flood management [1].

The traditional response to severe floods was typically an ad hoc reaction. It covered a quick implementation of a project that considered both the problem and its solution to be self-evident, and that gave no thought to the consequences for upstream and downstream flood risks. The flood management practices have largely focused on reducing flooding and reducing the susceptibility to flood damage. Traditional flood management has employed structural and non-structural interventions, as well as physical and institutional interventions.

The main tools of flood control are divided into two groups in the last decades. These two groups are the structural and non-structural tools, or measures. According to the definition of the United Nations Office for Disaster Risk Reduction (UNDRR), the structural measures are any physical construction to reduce or avoid possible impacts of hazards, or the application of engineering techniques or technology to achieve hazard resistance and resilience in structures or systems. The non-structural measures are measures not involving physical construction which use knowledge, practice or agreement to reduce disaster risks and impacts, in particular through policies and laws, public awareness raising, training and education [2].

These interventions can be applied before, during and after flooding, and can often overlap. The traditional flood management interventions are the following [1]:

- source control to reduce runoff (permeable pavements, afforestation, artificial recharge)
- storage of runoff (wetlands, detention basins, reservoirs)
- capacity enhancement of rivers (bypass channels, channel deepening or widening)
- separation of rivers and populations (land use control, dikes, flood proofing, zoning, house raising)
- emergency management during floods (flood warnings, emergency works to raise or strengthen dikes, flood proofing, evacuation)
- flood recovery (counselling, compensation or insurance)

The IFM covers land and water resources development in a river basin, within the context of the Integrated Water Resource Management (IWRM), and aims at maximising the net benefits from the use of floodplains and minimising loss of life from flooding. The IFM treats the river basin as a dynamic system in which there are many interactions and flux between land and water bodies. In the view of the IFM, the starting point is the river basin and the sustainability of its livelihood, looking for identifying opportunities to enhance the performance of the system as a whole.

An Integrated Flood Management plan has to address the following six key elements that follow logically for managing floods in the context of an IWRM approach [1]

- manage the water cycle as a whole
- integrate land and water management
- manage risk and uncertainty
- adopt a best mix of strategies
- ensure a participatory approach
- adopt integrated hazard management approaches

It is important to determine the role of water engineer in the context of IFM, since this concept points beyond the usual engineering way of thinking. There are several points of IFM which requires the collaboration of representatives of several professions, as economists, environmentalist, etc. Further on, the engineering aspects of flood management – with special regards to the urban flood management – will be presented.

Flood protection, or better to say protection against inundations, evolved parallelly with the development of technology. The first protecting technology was the careful selection of housing places, where the inundations could not occur, or the frequency of inundation was tolerable.

The second step of flood protection was the use of some intervention to achieve this aim, so filling up territories, mainly with limited extension, focusing basically on the buildings.

Later, the territorial flood protection has been achieved using engineering solutions, dykes, as the economic development made it possible. This solution was applied only if the protected territory could be used for some intensive production with high income. An unfavourable change of the flood regime was observed, caused by human interventions on the water courses and climate change issue. The exposition of social values to floods increased gradually, the engineering solutions of flood protection seemed to become more expensive and their unfavourable environmental effect was recognised. This process has caused the change of approach, in addition to the engineering solutions, other considerations emerged, as local protection, building protection, flood tolerable building technologies, or education for increasing the resilience of the society.

Urban flood protection shows similarly a development, but there are other considerations as the consequence of the urban environment. Principal characteristics of flood endangered urban environment is the a quite intensive land use, as traffic area, habitation or recreational area, zone of industry with ports etc. Because of this often shared, multiple purpose usage. To fulfil the increasing increased demand for complex solutions the engineering tools must not be used exclusively. The complexity of these facilities demands most of the cases unique solutions and strong collaboration of engineers, architects and landscape designers.

The public utilities which are inevitable parts of urban environment give an extra problem to face with during the realisation of urban flood protection. These utilities are generally underground structures, pipes, cables, and their manholes, and other special facilities. This environment is rather unusual and sometimes dangerous for the flood protection facilities, since these have to be waterproof and resistant for the effects of seepage.

It is important to mention that flood protection in general – and especially in urban areas – is composed of two basic elements. The first is the protection of the surface from direct inundation, making temporary dikes, raising the crown of levees. The second element is the collecting and pumping of water – waste water, or rainwater, or seepage water – from the defended territories. These two key activities are the basis of flood protection.

Characteristics of urban environment

The urban area can be characterised with high population density and infrastructure of built environment. According to the World Bank data, 54.8% of the human population of the Earth lived in urban areas in 2017. This percentage was 33.6% in 1960 [3].

The higher density of people inhabitants – over the personal properties – results in the a higher density of urban assets, public and company wealth. These facilities together ensure the continuity of the extremely complex process of value producing in towns. However, these assets can be rather sensitive. Any disturbance or blocking of their operation can cause extremely high damages. Because of this characteristic, the urban areas considerably exposed to any disasters, so as the floods, which causes hazard for human life. These events can cause physical damages of facilities or further indirect damages, by the consequence of blocking the normal way of urban value production. This vulnerability needs to be diminished to the lowest possible level.

The main types of flood damages are:

- loss of human life, or injuries (physical and/or psychological)
- harms of structures of several facilities in the inundated area
- harms of devices, fixtures of facilities (electronic devices, furniture, etc.)
- contamination of inundated area (flushed hazardous chemical substances, bacterial contamination from sewers or landfills, etc.)

The potential damages can be diminished by the following strategies:

- prevention, building of flood protection structures, raising the low-lying areas, avoiding construction in frequently inundated territories
- prevention by active flood protection, use of local protecting tools, demountable structures
- wise use of buildings, deploying water sensible devices on upper floors

The specific environment of towns means limitations for the above-mentioned interventions. These limitations are:

- built-in areas
- subsurface public utilities
- disturbed "urban soil" with high seepage capacity
- underpasses, subsurface premises
- continuous surface traffic, logistic difficulties of supply of flood protection works

Built-in areas. In order to understand the difficulties of urban flood protection, the previously mentioned limitations are needed to be explained in more detail. These circumstances can be better understood with some view to the development of towns.

Regarding the development of towns, two different ways of development can be identified. The first is the slow, organic way of development, the second is the fast growing of cities, properly from nothing.

Organically developing towns had sufficient time to suffer from flooding during their life. Some cities had thousands of years of development; this time span was enough to gain fundamental experience of weather and flood extremities. The chronicles, water level records has showed extension and level of flood events for later generations, and so the strategies were developed to prevent the harms. The prevention means the careful selection of built-in areas, shunning the frequently inundated regions and preferring the higher areas. Another way of prevention can be the well-chosen building technology, as using piles, leaving the bottom floor to be inundated in the case of floods, and so on. During the development of these cities there was a continuous attention to the flood issue; so on the frequently inundated locations:

- the buildings are less sensitive to flooding
- the areas would be raised to a secure level
- the areas are defended by dykes

The example of this process can be seen in historical floodplain cities. E.g. in case of Budapest, the Danube floods destroyed the assets on the floodplains in 5–10 years mainly by ice jam caused floods. This was strongly ingrained in the inhabitants, so the old towns were built on the isles of the floodplain, a few meters higher than the average terrain of floodplain, over the frequent flood levels. Later the city began to grow and the inhabitants had to confront with the question of floods. After the devastating flood of 1838, the community of the city decided to build flood protection facilities, raised the territories to a more secure level by the river, built dykes making a continuous defence line between the floodplain isles, raised areas and the high banks (the natural banks of rivers are higher than the frequent or admitted flood level). These solutions worked well enough until the change of the flood regime, which was caused by the meteorological variability and/or the change of runoff regime. As a consequence of these changes, previously safe parts of the town became endangered again. There are historical parts of the city that are more frequently inundated during floods in the recent decades.

The towns which develop rapidly within decades do not have historical experiences of floods. The flood exposition can be underestimated easily.

Subsurface public utilities. The densely built-in areas are supplied with water, gas, electrical and communication utilities which can drive the water toward the lower areas of towns. The density of conduits can be extreme, as it can be seen in Figure 2. The public utilities make the urban area penetrable for water, sometimes independently of the surface level. It means that if there are some water conducting utilities (not only water utilities but also cable ducts of electrical or telecommunication conduits) they can take the water behind the higher level territories, causing the inundations of cellars or deeper parts of the surface. Therefore, the ducts of cables need to be interrupted by sealing. The unserviceable not removed ducts have similar danger.

Figure 1. Density of public utilities in urban environment (Budapest Sewage Works)

Disturbed "urban soil" with high seepage capacity. In urban territories, the soil was moved several times as the built infrastructure developed and as the surface was transformed. In most cases there are so-called "culture layers" over the natural or original sediments or bedrock. These soils are composed by the remaining materials of buildings which were used for the foundations of roads, or simply filling materials. The filling materials can contain mainly pieces of bricks, blocks of stones, debris, sand, gravel, clay, humus and of course air. These ingredients are characteristically mixed in these layers and so the mechanical and hydraulic parameters of these layers spread unpredictably. At the planting holes of trees by the sidewalks and roads, which can cause problems during a flood similarly the before mentioned situations, by secondary flood phenomena frequently. If the groundwater pressure is significant, in a lower territory near a dyke, the "urban soil" layers can result in produce sand boiling and with flushing out of the fine ingredients of soil. These processes can occur problems in buildings from inundations of cellars to load-bearing decrease of the foundation.

Underpasses, subsurface premises. The subsurface premises, underpasses, subsurface railroads or other transport tunnels can conduit a high rate of water farther from the river. The underpasses can open the way for the flood to get to metro tunnels. The protection of these objects is highly important, since the flooding water can have an almost infinite amount of supply in several cases.

Continuous surface traffic, logistic difficulties of supply of flood protection works. The flood control activity presupposes a highly organised logistics, especially if the flooding is very intensive. As will be shown in the next section, there are several types of floods. Their character is different, depending on the flood level, the intensity of water level change, or the discharge of the flooding river or watercourse, the supply of the flooding water. The technique of flood protection work determines the logistical demand, as well. If there are steady built facilities – dykes, walls – with the specified necessary height, cross section and material, the logistic demand is limited to the supply tasks of interventions against the secondary flood phenomena, so i.e. the sand boil or leakage. These require fast intervention, but this is relatively rare in urban environments. The occurrence of secondary flood phenomena is random-like. It depends on the ever-changing urban environment. For example, the seeping water can break into a building site, or the ducts of public utility can be the source of a local inundation, the surface can collapse after the seeping water flushed the soil into the sewer on an unknown hole or discontinuity. The fight to repair these situations supposes a certain kind of "guerrilla fight"; the errors appear unexpectedly, in the living urban environment, which requires the fast reaction and fast transport of defence defence materials.

Typology of urban floods

Several types of floods can threaten the towns, depending on their geographic situation.

Every town is exposed to rainfall caused floods, these can occur even in a desert environment. The heavy rainfalls can cause flash floods if the topography is favourable for the concentrated runoff.

Towns were generally deployed near water, let it be sea or river or lake. The vicinity of waters has various advantages, ensuring potential source of drinking water, transport route or crossing point for interregional trade, technological water source for some industry, and mainly in ancient times it could have defence value as an obstacle. On the other hand, the proximity of water is a disadvantage to these towns, regarding the floods. Some types of floods can be the result of human activities, industrial processes.

The flood exposition depends on the geographic environment. The vicinity of sea and lake can be the source of coastal floods. The relevant relative relief in the urban areas or upstream of urban areas may cause flash flood exposition. Greater rivers can cause river floods with longer duration and large – quasi infinite – volume of water to retain between the levees. Artificial lakes, reservoirs can be the source of reservoir floods, which can be related to meteorological events, but also to structural malfunction or rupture, or structural fatigue.

Pluvial (surface water) flooding

Pluvial flooding occurs when an extremely heavy rain exceeds the capacity of the drainage systems. In this context, the drainage capacity covers the infiltrating capacity of the earth, the retention capacity of surface objects, ponds, and the hydrological capacity of the rainwater drainage system. The pluvial flood is a territorial threat, the flood protection is spatially distributed, while other types of floods can be characterised by lines (e.g. levees) where the inundation can be blocked.

The drainage capacity is variable in time, it depends on the present instantaneous state of the soil, the filling status of ponds, the wetness of the surface, and the free capacity of the drainage system (was its capacity exhausted during a previous rainfall, could debris jams block the drainage process in the drainage system). From the point of view of pluvial floods, the most relevant kind of precipitation are the cloudbursts, thunderstorms, when the intensity of rainfall can exceed multiple times the design intensity of the rainfall drainage systems. In well drained towns, the critical rainfall intensity of possible occurrence of pluvial flood exceeds 35–45 mm/h in a 10–20 minute time span (it means at least 97–125 l/(s.ha) average intensity). The critical intensity rainfalls are connected to convective atmospheric currents.

In the temperate climate, the duration of critical rainfalls is generally very limited. The convection is driven by the heating of the Sun or frontal situations, or the orography, so generally nights can block the convectional currents in the atmosphere, or the fronts are moving fast. There are exceptionally long-lasting thunderstorms, supercells which can remain active more than one day, but characteristically they move with the regional currents, so a certain territory can be covered by the storm cloud only for a few hours. In mountainous regions rainfall can be extremely intensive.

In tropical and subtropical territories, enormous rainfalls occur regularly, which can last for several hours or some days. The tropical storms are regularly driven by the heat stored by the sea, and so over the sea the convection can be continuous in the night hours as well. The sea continuously supplies the storm with water. This can result in extreme rainfalls in the hurricane, monsoon and typhoon zones, or in the archipelago of tropical or subtropical seas.

Table 1. Some extreme intensity values from the world [4]

Duration (min)	Precipitation (mm)	Intensity (mean) [l/(s.ha)]	Place	Date
1	38.1	6,365	Guadeloupe	26.11.1970
1	31.2	5,200	USA Unionville (MD)	04.07.1956
15	198	2,200	Jamaica	12.05.1916
20	205.7	1,714.2	Romania, Curtea de Argeş	07.07.1889
42	304.8	1,209.52	USA Holt (MN)	22.06.1947
60	401	1,113.9	China, Shangdi	03.07.1975

Duration (min)	Precipitation (mm)	Intensity (mean) [l/(s.ha)]	Place	Date
60	305	847.2	USA Holt (MN)	22.06.1947
60	305	847.2	USA Kilauea (HI)	24–25.01.1956
360	840	388.9	China, Mudocaidoang	01.08.1977

Table 2. Some Hungarian extremities [4]

Duration (min)	Precipitation (mm)	Intensity (mean) [l/(s.ha)]	Place	Date
10	64.2	10,700	Zirc	24.05.1915
60	120	333.3	Heves	23.08.1988

The character of pluvial floods depends on the topography of the territory. If the relative relief is higher, the pluvial flood becomes similar to flash floods with high velocity and locally concentrated damages. The plain territories suffer from inundation depending on the micro topography which can fall in the range of decimetres.

Rainfall is the most variable meteorological phenomena both in time and space. The spatial distribution of rainwater depends on the extent of the storm cloud, and the direction of its motion. In the temperate climate, storm clouds are typically a few kilometres in size, sometimes reaching or exceeding 10 km in diameter. Even under a storm cloud, rainfall is variable, so the heaviest rainfall is limited only for a 1–2 km zone. It means that some parts of the given urban area will be hit by the heaviest rainfall, and the damages will show a certain concentration. In the temperate continental climate, extreme rainfalls are not frequent, in a person's lifetime it occurs only 3–5 times in a certain place. The phenomenon takes place very fast, the forecast practically does not work, the so-called nowcasting can ensure the relevant data but only for a very short time, so the civil defence forces cannot be at the field in time, because the rainfall happens in short time scale. The mitigation of these factors is very difficult, the people (and civil defence forces) cannot intervene; in these cases, preparation, individual initiative and problem-solving capability is of primary importance.

Based on these characteristics, the defence tools of the pluvial floods can be determined. There are two stages of flood prevention:

1. Determination of rainwater courses (and its parameters as velocities, depth), and the probable extension and depth of inundation. These results can be determined by runoff models.

2. Determination of flood prevention tools for the present and future buildings and land use.

The flood prevention tools for pluvial floods have to be:

- quick to assemble, simple structures to be built up by civilians
- easily storing kits
- trained civilians for the mounting of basic protection tools and kits

During extreme rainfalls, preventing loss of lives is more important than the protection against inundation.

Combined Sewer Overflow (CSF), the sewage appears in cellars, or on surface

If the rainfall drainage is solved by a combined sewer, the CSF causes biological contamination on the terrain, or directly in flats where the water can get in. Although the sewage becomes highly diluted by rainwater, the polluted water can still cause diseases, and it can be source of epidemy. If the rainwater drainage system is separated from the sewage, the water can be contaminated by the flushed pollutants and waste, and can cause health problems similarly to diluted sewage. The CSFs can be prevented using:

- adequate capacity canals to take the water away (it is an economic and technical question)
- correct hydraulic design to diminish losses
- well prepared system to operate even under pressure, so the diluted sewage cannot get out from the sewer through manholes, gullies and household openings (rainwater should also not enter into the combined sewer until the pressure does not decrease into an adequate level; this problem has to be treated by storage tanks)
- preventing sewage backflow with automatic plugs, butterfly valves in household lines

Surface inundation can occur as a fast or a slow-moving current, or can be a temporary pond too. The inundation threatens the cellarage and low floors of buildings. The flooding can cause significant damages in these parts of the buildings, like underground parking floors, but even electric transformer units, surgery rooms of hospitals, archives in record offices, etc. can be affected in this way. The closing of these parts of the building can be managed with light metal beams, flood doors and gates. The height of the structure must be at least the probable highest level of water. The highest water level can be determined by modelling or empirically (based on experience or wise precaution).

Flash flooding

Flash floods are the result of heavy, extremely intensive rainfalls on the catchment area of smaller rivers or ditches. The phenomenon can occur in areas characterised by high relative relief, it supposes hillsides, mountainous territory. Because of the steep surface, the time of concentration is rather short: it takes a few tenth of minutes or a few hours in few 10 km^2 catchment areas exposed to the cloudburst, and as the exposed territory grows to 100 km^2 the time will be longer.

Generally, the flash floods can occur more frequently if the runoff coefficient is high, and beyond the high fall, the surface cover is scanty. However, flash flood occurs in forested area as well, since the water retention capacity of the vegetation, soil and terrain can run out if the rainfall intensity and volume exceeds a certain threshold. In this case the frequency of flash flood is rather low, and occurs only with quite extreme rainfalls.

Reduction of flash flood effects can be achieved if the runoff can be slowen significantly in the upstream valleys.

The characteristics of occurrence of flash floods are very similar to the pluvial flood event, since the flash flood is the consequence of extreme rainfalls, so the main considerations are related to this phenomenon. Flash flooding is related to convective rainfall activities (supercells and other cloudbursts). The extreme rainfall intensities cause the extreme peak of discharge. The possibilities of forecast of flash flooding are similar to the cloudbursts, these can be early warning signals from real time measurements or estimates, produced based on:

- remote sensing devices (radar, high spatial and temporal distribution “miniradar”)
- network of rainfall measurement devices (datalogging rain gauges with nearly real time data communication unit)
- water level detecting unit (ultrasound surface detecting, pressure detection in riverbed, etc.)
- automatised picture analysis to recognise the flood’ appearance in the riverbed

The earliest signal can be earned from the rainfall detection. The most direct solution is the water level detection. It can be operated as a part of an automatic early warning system. The success of detection must be independent of perpetuated circumstances of flash flood. The device must be mounted within a secure distance of the flooding water. This is important, because devices immersed into the flow can be teared off and taken away by the high quantity of transported materials. The nowcasting can be gained from real time measurement of rain depth and simultaneous high resolution radar observation of cloudbursts. The rainfall radar can give spatially continuous estimation of rainfall intensity distribution. The technical solutions of the radar estimation and the connecting technical and theoretical problems are discussed in several studies [5]. Here it is important to mention that the radar cannot measure rainfall intensity directly, it measures the reflection of the radar beam on raindrops or pieces of ice in the storm clouds, so there can be significant differences of estimated rainfall and the quantity of rain reaching the earth surface. This uncertainty can be eliminated by using field measurement units. However, there are several sources of errors of field rain gauges, the field measure of rainfall is an essential tool of the refining of radar data. If there are more rainfall gauges close to each other (within 2–5 km) the radar data can be used as an estimation of rainfall distribution between the two devices.

Flash floods can have a triggering effect on another dangerous phenomenon, the debris flow. Debris flow can occur if some particular conditions come together, such as a steep slope of the valley, the flushable material is in enormous volume (5,000–30,000 m^3/km^2), and the water wets this material and lubricates the moving mixture (precipitation and/or waterflow). In case of a debris flow, water has a special role, it works as a lubricant between the rocks, wood pieces and mud; sometimes it looks like the fresh wet mixture of concrete. The debris flow is not a Newtonian fluid, it has special characteristics. It can move very fast in steep valleys (0.5–40 m/s), but if the sloop of the valley diminishes under a threshold value (2%), the transported material stops and the mud and stone mixture can cover the bottom of the valley in several metres of height [6]. Debris flow can occur without atmospheric trigger effect, the root cause can be volcanic eruption with melting snow, or human activity such as reservoir catastrophe. The question of debris flow has been studied first of all in countries that are particularly vulnerable to this kind of disaster by their geographical and meteorological environment.

Defence against flash floods is very difficult. Due to the rapid character of the flash flood, the rapid runoff and unexpected water level rising, cause that operative flood control cannot be successful. There is no time to build defence lines to salve riverside territories.

The best solutions are the structures which have continuous defence capacity (structures), or those which can be deployed very fast, or which are well prepared, well designed, and well armed against the extreme effects of flash floods. These extreme effects are:

- extremely fast rising of the water level
- extremely high velocity
- presence of masses of sediment
- significant mass of transported debris

The quick to assemble structures can be applied for the defence of solitaire buildings, for closing gates, doors and windows of the given facility. There are several products for this aim, which use the following schemes:

- removable flood barriers
- flip-up flood barriers
- drop-down flood barriers
- self-closing flood barriers

The technical part of the prevention is some built flood prevention solution, such as a flood protection wall or some levee. These structures are always prepared to the flood protection, at least to their defence capacity (height, resistance of seepage, etc.). Over the usual construction rules of these structures related to the defence level and seepage, some further circumstances must be considered, as the extreme velocity of the currents and the effect of probable collision of heavy floating objects, tree trunks or other things of significant dimensions. Such flood prevention constructions become the part of the landscape. These structures will have a role only in cases of flash floods, so only for short times and rarely, during their expected lifetime. It is essential to find an optimal use and aesthetics for these structures for the time when there is a normal hydrological regime. The realisation of this aim can be achieved by careful design with the collaboration of landscape designers and architects.

The second part of the prevention is the forecast and early warning system of the local flood protection service. The problem is similar to the pluvial floods: the phenomenon is related to very intensive, extreme rainfalls which can be characterised by quickly changing rainfall intensities and fast moving rainfall fields.

Coastal flooding

Coastal flooding occurs in the vicinity of lakes and seas. This kind of flood is caused by extreme tidal conditions including high tides, storm surges, seiche and tsunamis (the duration is in dimension of hours or days).

The flood defence facilities of coastal cities are planned on the basis of water level observations of the given lake or sea. Due to the climate change, there are predictions of increasing number of storms and tempests. The importance of coastal floods is increasing with the rising and forecasted rising of sea level all over the world. The summary of these factors is that the frequency and seriousness of coastal floods will increase rapidly. There are several examples of more frequent coastal flood issues. Some of them happens parallelly with the sinking of the coastal territories, as in Venice, or in Jakarta. In other regions, the river' estuary areas are threatened, as in London. In the Netherlands, complete regions are under average sea level, where the stormy tide jeopardise the lives and economy. Similarly, in the case of New Orleans, the low coastal region was destroyed in the extreme hurricane Katrina, which caused the backwatering in the estuary region of the Mississippi, beyond the heavy rainfalls. Some of these issues can be handled with flood gates which can be closed during the high tide to save the cities and their outskirts at river mouths. These kinds of facilities can be found in Rotterdam [7] and London [8].

Figure 2. Maasdijk, tide gate at Rotterdam [7]

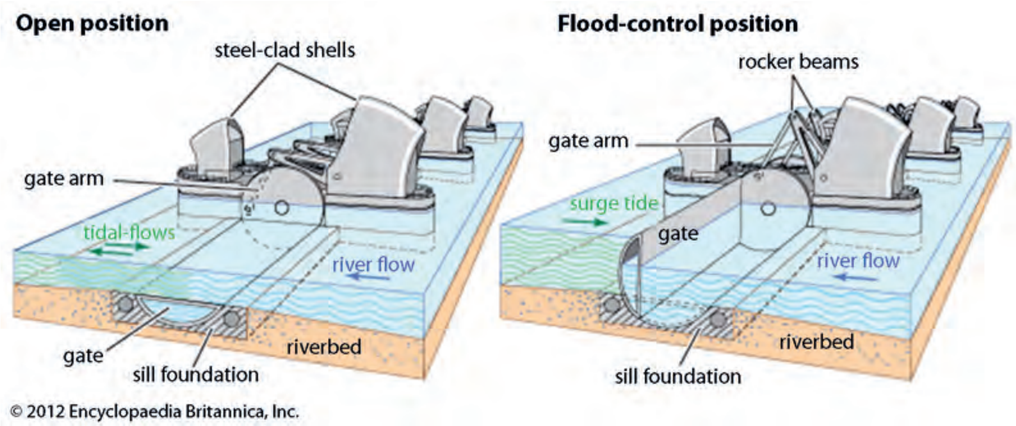

Figure 3. Thames barrier, London – Functional sketch [8]

Lakes show similar threats, only the rising level of lakes is not in direct relation with the global changes in all aspects, and – contrary to seas – the water level of lakes can be regulated in most of the cases. As the water level is regulable, if the meteorological regime changes, the regulation should be overviewed and reshaped. The motion of the waterbody and the flooding at shores of a lake can be caused by:

- surplus water
- wind
- geological processes

The duration and development of these phenomena is significantly different. The surplus water caused elevation is the function of the arriving water volume. The water level elevation depends on the intensity of appearing of surplus water, the free storage capacity of the lake, and the natural or artificial (sluice dams or sluice canals, tunnels, etc.) overflow capacity. The development of floods caused by surplus water in lakes depends on the characteristics of the catchment area and the free storage volume of the lake, over the normal water level. If the storage volume is pretty great comparing to the possible extreme runoff arriving the catchment area, the coastal flood develops relatively slow, the process can take days, or weeks. The success of intervention depends on the regulatory capacity. But, a stormy weather results in inconvenient surprises.

Floods caused by wind are the result of waves higher than the lakeshore and the phenomenon seiche. The harm of flood is heavier if the water level is higher from the beginning, in this case the wind is able to push more water out from the lake. The seiche is another group of – mainly – wind-caused floods. The seiche is a oscillating motion of lakes (e.g. Great Lakes, Leman Lake), semi-closed seas (e.g. Baltic Sea, Nord Sea, Adriatic Sea), harbour basins or smaller artificial basins. The seiche are related to winds, air pressure change, underwater internal waves, and sometimes to seismic activity.

There are examples for deep lakes in the Alps, as the North Italian lakes, Lake Lemon in Switzerland, where seiche was studied for the first time [9]. An example for shallow lake is Lake Balaton, where seiche has been studied since the end of the 19th century [10], and seiche caused by wind has been making damages. The duration of a seiche depends on the meteorological situation and the damping of the pendulum-like motion of the water level; it can take some hours or several days. The range of water level change can reach the magnitude of meters, depending on the shape and volume of the lake, and the intensity of the triggering effect. .

The coastal floods of lakes can be caused by geological or geotechnical catastrophes in mountainous regions, where there are steep instable slopes of mounts over the lakeshore, which can slip or fall into the water. If the volume is extremely large compared to the volume of the lake, the waves can cause accidental inundation. Similar situation can occur if the sediment slips down under the water level if there are instable slopes. This kind of inundation happened in 509 AD in Lake Geneva. The trigger effect of these motions can be an earthquake, or the wet weather which is favourable for destabilising effects of slopes, as wetting of instable masses, decreasing the shear resistance and increasing the mobilising forces. These phenomena are very similar to tsunamis.

The last category of coastal floods is related clearly to tsunamis. The tsunami is a very long wave which appears when some impact constrains an enormous mass of water to a displacement. The impact can be for example an intensive horizontal or vertical motion of the seabed during an earthquake, a landslide on the seabed, an extreme volcanic eruption, collision of a significant size meteorite etc. The extent of the tsunami depends on the water volume moved by the impact and the intensity of the triggering phenomenon. The tsunami waves become extremely high when they reach the shallow water at the seashore. The height of the wave is also influenced by reflections taken place on the seashore or in narrow gulfs and river mouths. The height of tsunami waves can reach 10–15 metres. The character of a tsunami is on the one hand a rapid and extremely high inundation, on the other hand the high-speed currents of water in the flooded region. The damage potential is enlarged by the floating debris of shattered structures, vehicles, trees; it causes severe injuries and results in a chaotic landscape where finding surviving victims is rather difficult. The defence facilities of tsunamis are typically maritime engineering structures; further details can be found in relating books and papers. Some examples for well documented tsunamis of the past decades are the tsunami of the Indian Ocean on 25 December 2004, killing 230,000 people in fifteen countries, or the 11 March 2011 tsunami in Japan, causing severe damage also in a nuclear power station, and caused the death of more than 20,000 people.

Fluvial (river) flooding

Fluvial flooding occurs – as a general case – when water level of rivers exceeds or the water breaches the levees during a high-water period. Comparing the phenomenon to flash flooding, the flooding intensity of river flooding is characteristically lower, and the spatial extension of flash floods is mainly located to shorter river branches in narrow valleys, while the river flood affects a longer reach.

The cause of fluvial floods can be a sustained or intense rainfall or snow melting, or their combination, furthermore, a consequence of some catastrophe, e.g. a reservoir failure. In these cases, the flood depends on upstream conditions.

There is another group of river flooding, which occurs when the downstream reach of the river partially or completely loses its capacity to take the water away because of some reason, let it be the flooding of the recipient or tributary river, ice jam or jam of debris, riverbed sediments. The decreasing capacity of the riverbed can also occur as the consequence of some geological cause, as partial closing of riverbed by sliding of mass of material into the river. This kind of flood happens in mountainous regions, or in case of volcanic activity (lava flows).

Ice jam is characteristic in temperate and boreal climatic zones. Debris and sediment caused floods occur only if there is a huge volume of mobilizable sediment, floating debris (first of all wood) in the upstream; these conditions are given in mountainous regions, first of all. The danger of debris jam is more relevant in smaller rivers, where the arriving floating materials can fill up the cross section of the riverbed easily. The possibility of developing jams is increasing with the sinuosity of the river.

The forecasting of the above-mentioned floods differs on their types: the upstream water related floods have generally hydrologic causes, so their forecast can be ensured by hydrological-hydrometrical tools. The debris and ice caused floods are partially dependent of hydrological circumstances – however, these phenomena can occur characteristically parallelly with high water or floods –, the experience of previous cases is important in the preparation phase of the flood protection. In the further part of this section the upstream water related floods will be characterised.

The characteristic duration of river floods depends on a series of factors, such as:

- water supply and volume of flood wave
- slope of the river reach
- length of the river upstream

The water supply and the water volume of a flood wave increases the length – or duration – of the flood, the duration of the period of certain high water levels. A greater catchment area can be supposed if the upstream river reach is long, because a great basin supplies the flood waves. The rainfall origin floods of rivers are induced by intensive, long duration (magnitude of days) rainfall. The characteristic duration of flood inducing rainfall depends on the extension of the catchment area. If the catchment area is smaller, shorter and more intensive rainfalls cause floods; as the area is increasing the characteristic duration becomes longer.

In the case of River Danube, the long duration intensive rainfall of the upper part of the catchment area causes floods in the May–August period. This kind of floods caused extreme floods in the 2002–2013 period with relevant or extreme peak levels in several gauges of the Hungarian section Danube. Another kind of flood is induced by the snow melting in the Alps, in the March–April period. The snow melting combining with rain can cause extreme high water levels, as it happened in 2006. Earlier, the ice jam caused floods occured frequently on the Hungarian reach of the Danube, but it has not occurred since the 1956 spring flood as a result of the river control interventions, and the continuous ice breaking to prevent the formation of ice jams (Figure 4). On the downstream reach, in Serbia, Bulgaria and Romania the formation of ice jam can be a real danger even today, as it happened in the winter of 2017 [11]. The local effect of the global warming diminishes the occurrence of ice and the probability of ice jam caused floods.

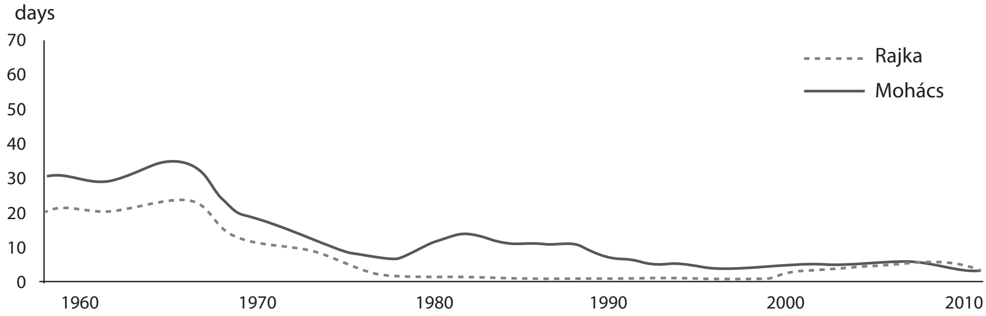

Figure 4. Changes of the average length icy period of the Danube between Rajka and Mohács in Hungary (10ys moving averages, 1958–2011) [11]

Reservoir flooding

Reservoir flooding occurs when a reservoir' dam failure takes place, and the stored water appears abruptly in the low-lying areas in the downstream of the dam. The flood takes places in a short time (few hours), but regarding the volume of the outflowing water, the intensity of the flood is often extreme, the effect of this kind of flood is devastating.

Hydrological, structural, geotechnical and geological factors can cause reservoir floods. During the lifetime of a reservoir, extreme hydrological events can occur which can overload the dam the spillway. Based on the short hydrological time series, the determination of the incoming discharge could not be exact enough, so for less frequent events the under-dimensioning of spillways could have happened. Also, the climate change can influence unfavourable the change of hydrological regime, which can favour the more frequent occurrence of extreme rainfalls. The dam failure can take place as a consequence of geological catastrophes, landslides, earthquakes. The Baldwin Dam collapsed in 1963 because of geological, geotechnical and structural problems and caused the death of 5 people (Figure 1).

Figure 5. Reservoir flood in Los Angeles, 1963 (http://damfailures.org/case-study/baldwin-hills-dam/)

The effect of a dam catastrophe can be aggravated if the stored liquid contains some industrial waste. There are technologies of the mining and ore processing industry which uses highly alkaline substances, or other poisoning materials for the extraction of metals from the ore, in an enormous volume. In the past decades, there were two devastating dam catastrophes related to liquid industrial waste storage reservoirs in Middle Europe. On 30 January 2000, 100,000 m^3 cianid containing sludge got into one of the tributaries of the Tisza River in Romania after a fast snow melting and rainfall which caused the flushing of the dam of the sludge reservoir near Baia Mare. On 4 October 2010, a poisoning liquid side material storing reservoir' dam collapsed in Kolontár, Hungary, because of the unusually much rain caused overload of the dam.

Flood protection facilities in urban flood control

The flood protection of the urban territories against the flood types mentioned in the previous sections is quite different and very similar at the same time regarding the defence methods. The difference can be determined in the time of development and the duration of the given flood threat, i.e. the possible timespan of operative flood protection. The timespan is important from the point of view of organisation of flood protection capacities (personnel and technical conditions).

The flood protection against pluvial floods concentrates mainly on the drainage capacity and inundation control. The process of pluvial flood takes several days, and a few hours, one or two days in extreme water level range. The direct inundation and seeping water can cause damage, and the seeping water can do it alone too. The water seeps or flows to the places where the water's energy level is lower. By seeping the groundwater's level rises, and threatens subsurface facilities, cellars and terrain depressions. A successful flood defence means a continuous control of groundwater and seeping water parallelly with the prevention of direct inundation. The same is true for rainwater control during floods, since the recipient of the rainwater is the flooding river, and the high water level of the river demands the rainwater to be pumped.

During rainfall, high volume of water appears in towns, inducing a runoff process. If the runoff exceeds the capacity of rain sewers, the water flows on the surface, and inundations occur. In case of combined sewers, it is extremely problematic, because it causes the overload of wastewater treatment plants, and in most of the cases, the overflowing water takes the pollution directly into the recipient. To prevent this, the separation of rainwater must have a priority, and the increase of sewer capacity or the retention or use of rainwater is to be solved. The most characteristic way of protection against floods caused by rainfall is rainwater drainage. The enlargement of sewer capacity is quite expensive. The paved surfaces of towns and the tendencies of climate change often provoke lack of water, therefore, rainwater harvesting must be provided, and only that part of water is to flow away, which cannot be caught, stored, infiltrated, used in some appropriate way. The time of utilization of water does not coincide with the rainfall often, so the storing of water is a critical issue.

Generally, the green solutions can manage the water of frequently occurring rainfall, but the extremely intensive rainfalls demand engineering solutions. The storage can be performed by significantly great subsurface tanks which can be filled up during the extreme rainfalls, and the water can be delivered later to irrigation systems, groundwater replenishing systems, etc.

The flash flood, similarly to the pluvial flood, is a very fast developing phenomenon, so if the forecasting (or nowcasting) can be managed, the flood protection systems have to be built up within a very short time span. Since, the success of operational protection work is low in a short reaction time, the preparation of inhabitants is important; in this way the people can learn the lifesaving and runaway strategy in the case of a flash flood, furthermore the tools of damage mitigation.

Coastal floods can be nowcasted and the protection is generally similar to the solutions of the flash flood control.

Generally, river floods can be predicted at least a day or two in advance, depending on the runoff characteristics of the given river, so there is a longer time window to the preparation of the flood defence system to the flood protection works. For example, in Budapest the preparation period for a flood are 4–5 days on the Danube, based on the data of Austrian water gauges. This is enough time to mobilise the needed materials, machinery and human resources for the operational flood protection.

The successful flood protection generally means two simultaneous tasks. First, the direct inundation must be prevented, secondly, the collected water must be placed into the recipient waterbody. The first part of the tasks can be divided into two groups, primarily defence of territories off inundation, and secondly, defence against the harm effects of seepage (Figure 6). These secondary phenomena take the danger of significant consequences, such as diminishing the shear resistance of soils, augmenting the probability of collapse of some structures, flood protection facility or other buildings. The possible consequences of urban flood are the inundation, rainfall drainage issues, infiltration of water because of harmful seepage into sewers, cellars, underground facilities.

In the case of a failure of flood protection, depending on the severity of the situation, the sav of lives and the localisation of the flooded area are tasks. A well elaborated emergency plan can help the rescue.

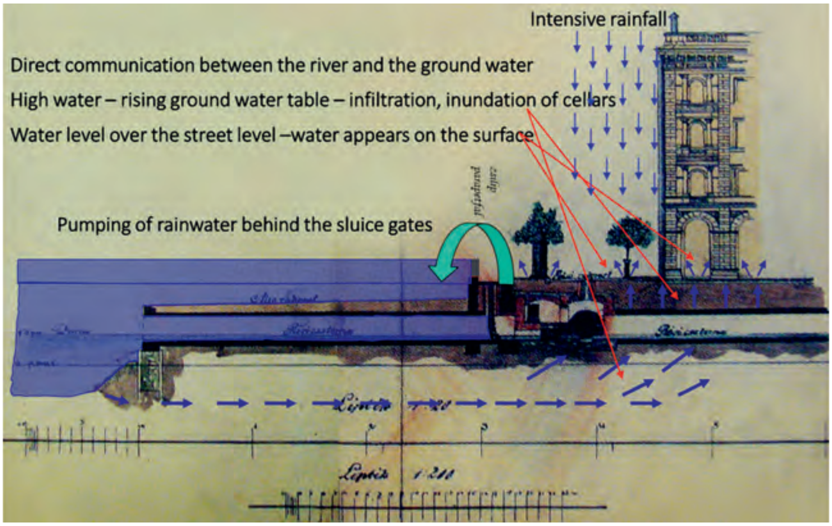

Figure 6. Characteristics of urban flooding [12]

In case of flood control of towns, prevention is most important. Prevention covers structural measures, building of dykes, floodgates, etc. On these structures operative flood protection work can be carried out. The operative work covers the blocking or control of secondary flood phenomena such as harmful seepage, the consequences of the wetting of soil (e.g. the slides of earth dykes or hazardous motion of structures, walls, or the direct burst of water in subsurface chambers, manholes or in the sewers).

The operative measures extend to the building of temporarily structures, as supplementary heightening of structures, building of localisation dykes and defence lines, if a catastrophe seems to be dangerously possible.

A classification of flood protection systems is shown in Figure 7. The basis of the classification is the steadiness of the given structures, determined or not determined building site [13].

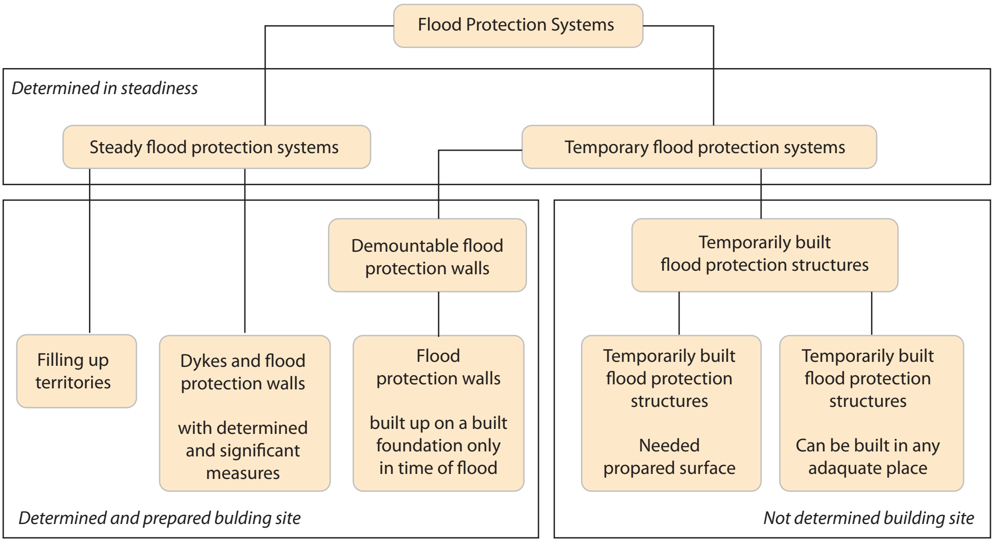

Figure 7. Classification of flood protection solutions (compiled by the author based on [13])

The upper rectangle shows two categories of flood protection solutions, if they are permanent and temporarily built facilities. The bottom left rectangle contains those solutions which are built as permanent facilities, so which effect their defence capacity in a precisely determined place; the demountable flood walls, which are have no permanent defence capacity are in this category since their foundation determines the defence line permanently. The bottom right rectangle contains the two categories of the temporarily flood protection solutions, accordingly to their terrain preparation demand.

The temporarily built flood protection structures are classified into two classes, into those ones which can be utilised in any place which is plain, quasi horizontal and where the load bearing of the earth is sufficient, and the surface is adequate for the secure build. The second class comprise those structures which does not need significant surface preparation before building, as the sandbag wall, for example.

The most secure solution is the rising of the terrain level high enough to prevent the inundation for longer time, hopefully.

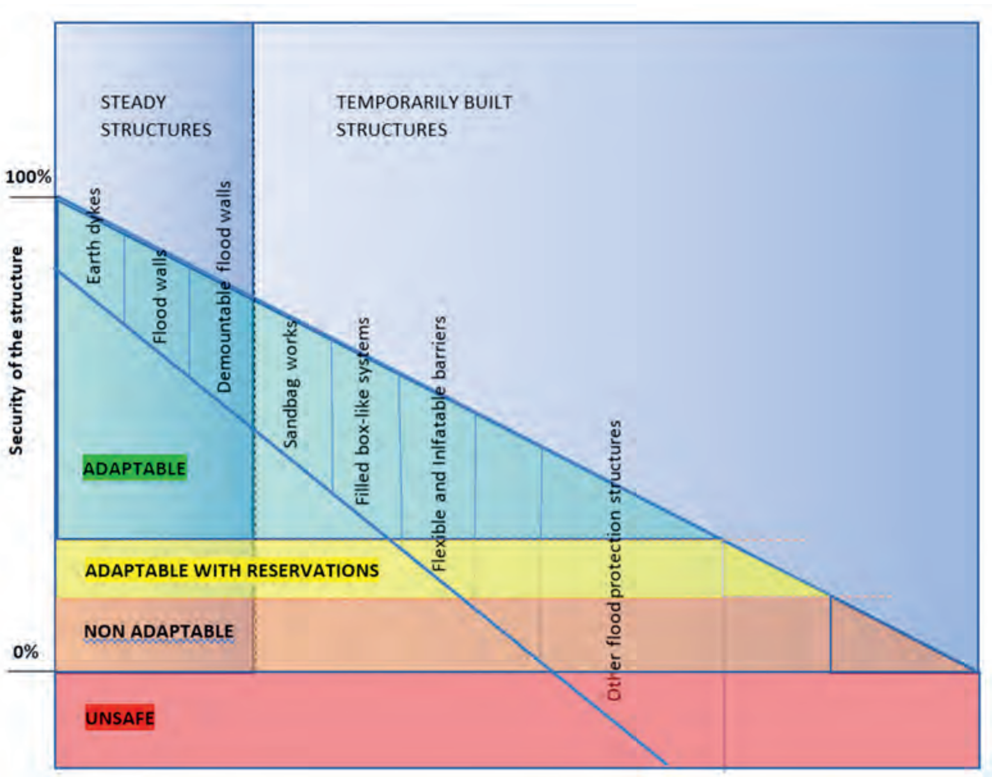

Figure 8. Security rate of constant and temporarily built flood protection structures (compiled by the author based on [13])

The security level of the constant and temporarily built flood protection structures can be characterised on the technical considerations. Figure 8 represents the security level of the widely used flood protection techniques referring to the security level of earth dykes. As Figure 8 shows, the new temporarily built flood protection products cannot reach also the security level of the traditional sandbag based operative flood protection solutions. The inflating barriers fall out the definitely applicable zone, because their security level is judged very low.

The solutions judged less adaptable or less secure, in some cases are not designed prudently, and sometimes there are only promises about their functionality and in practice the characteristics of these products are not proven. The choice of new flood protection products has to be based on static and hydraulic considerations.

Filling low-lying urban areas

Figure 9. Raised street section in Budapest – Veres Pélné street (Google street view)

The inundation and even the secondary phenomena can be prevented by filling up the deep territories of the urban area. It can be simple if the territory which can be flooded is narrow. If the area of possible inundation is greater, the filling can be partial. There are several examples of areal fill up in Budapest. These interventions were made mainly after the 1876 flood when the street level was raised to the flood level, it meant 50-90 cm (Figure 9). After the devastating 1838 ice jam caused flood, the street level was raised at the Pest side of the Danube at the St. Rokus chapel in 1846 (Figure 10). This solution can be economic mainly if the flood threatened territory is narrow, the flood level is not significantly higher than the original surface level, the buildings and other assets of the questioned territory are not sensible to the filling up, their public utility connections remain operable. (A curiosity: the Saint Florian church in Budapest was lifted up by hydraulic jacks by a meter to put an end to the moisture caused damages after the filling up of surrounding territories. The walls at the basement were cut and built again as a supplemental basement structure in 1937 (Figure 12).

Figure 10. St. Rókus Church in Budapest – raised street level at the entrance of the church (Google street view)

Building of dykes

If there is no possibility to fill up the territories or if otherwise impossible, dykes can be built to prevent the inundation. The dykes have some basic disadvantages in urban environment, such as:

- the height of the dyke
- the width of dyke
- a leak in the dyke

Figure 11. St. Florian Church before its lifting of 1 m in 1937 – the street was raised 60–80 cm after the devastating floods of 1838 and 1876 [15]

The significant mass of a bare earth dyke (with its grass cover) can be strange in an urban environment, but if it is inserted into the green infrastructure, it can be an important urban space (Figure 12). The dykes in the urban environment have an aesthetic role as well, their design must be more than the simple water engineering task. Generally, earth dykes can be found generally in the outskirt areas of the towns. In the internal, or intensively used zones of the towns retaining walls, in the case of navigable rivers, wharfs are built. Often the vertical walls are built to gain terrain in the riverside where the lots are generally more valuable. Earth dykes generally disappear with the urbanistic development of the towns. In the past decades, a change in the urban planning resulted in the increasing extension of green zones, following the social demands. The riverside territories are the most available for the fulfilment of this aim, their increasing significance predictable. The application of earth dykes or greened engineering structures are going to be acceptable in these areas.

Figure 12. Earth dyke at the cross section change in Budapest, Pünkösdfürdő in the 2013 flood (Budapest Sewage Works Ltd.)

A general advantage of a dyke is the wide crown, which allows the works to be carried out if some operational intervention is needed during the floods. There is also space for temporary heightening if the circumstances demand this kind of intervention.

Flood protection walls

With the development of urban environment, a strengthening demand for an intensive use of rivers is emerging. This demand is a lot more explicit when the given river is navigable. The complex land use development aims at compelling multipurpose solutions, which can serve further targets beyond flood control. In the past centuries, in many cities the building of wharfs in the downtowns was an obvious demand for the continuous improvement of navigation and other industries. In the past decades, as merchant navigation and mainland logistics have been developed, the ships were directed more and more into well-equipped ports near the towns. In the beginning of this transformation process, the wide storage platforms of wharfs have become urban traffic zones, blocking the citizens from approaching the water, and after this period, the demand to give these sites back to the inhabitants has emerged. These places have been developed into mitigated traffic pedestrian zones, or revitalised riverside spots, serving the inhabitants as part of the green infrastructure. This process resulted in riverside embankment and/or flood protection facilities having new roles.

Flood protection walls are used if the size of the area at the assigned defence line and local land use do not let the building of the greater space-demand dykes. The flood protection walls are a more expensive than earth dykes. The basic expectations are the same, they have to ensure the necessary height, stability and seepage control. Their appearance can be a part of a complex structure, e.g. a 0.5–1 m high wall on a dyke, or the wall can be a stand-alone structure, with a height of 1–3 m. In this last case, there is no often way of further heightening if the water level could exceed its design value in an extreme flood. The walls can be reached from their defended side what can problematic if the height exceeds the 1.5 m. The difficulties are greater if seepage control interventions must be carried out, since in this case the place by the wall is occupied by the seepage control solutions, and the approach of the wall become impossible (Figure 13).

Figure 13. Low flood protection wall, Budapest (Budapest Sewage Works Ltd.)

If the wall is part of a dyke or a retaining wall (so that the flood protection wall serves as a strong heightening of the bottom structure), or the wall stands alone, but its height does not exceed 1.2 m, operative flood protection work can be done in a wider range (Figures 11, 12, 13).

Figure 14. Heightening of low wall with sandbags in Budapest (Budapest Sewage Works Ltd.)

Figure 15. Low flood protection wall as part of a wharf in Budapest – flood control operation in progress in 2013 (Budapest Sewage Works Ltd.)

Figure 16. High flood protection wall, seepage control by counterpressure basin, Budapest, 2013 (Budapest Sewage Works Ltd.)

Demountable flood protection walls

The use of demountable flood protection walls is favourable from point of use of land use. In the valuable urban spaces, the permanent flood protection walls causes continuous obstacle for the normal use, however, the floods generally occur only once or twice in a year, with a duration of some days. This issue can be managed using the demountable walls; their foundation is continuously present, but the structures over the terrain are to be mounted only in case of floods. The reliable forecast of flood is essential, since the construction work with the necessary logistics must be finished before the threshold of the wall would be reach by the flood. If the water is expected to rise faster than the length of the building process of the wall, this solution should not be used. Before the application of the demountable walls, an analysis of the building time to the possible water level rise is to be done (Figure 17) [14].

The success of building a flood protection wall in time presumes:

- a strong, well designed logistic system
- well maintained elements
- well maintained supporting pillars with their connection elements to fix the columns of the structure in any time, even when it is freezing

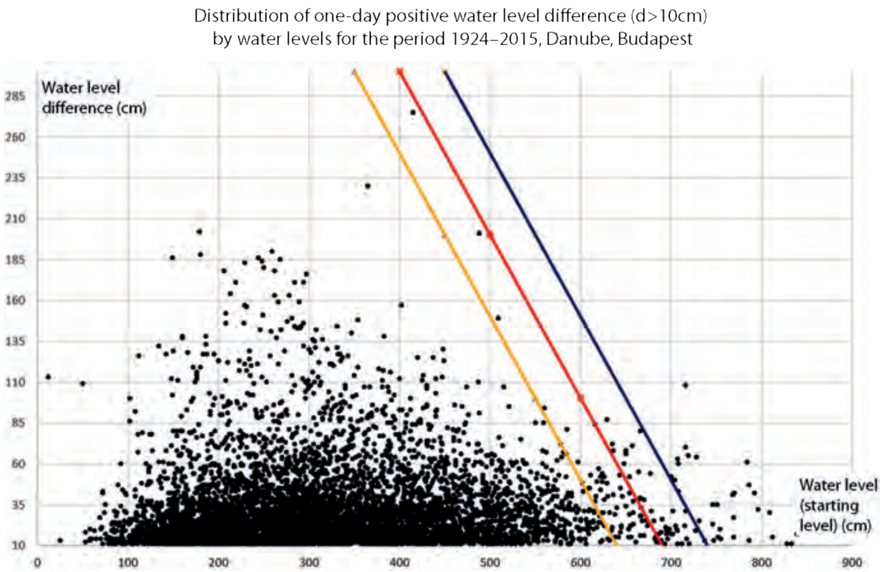

Figure 17. Water level change analysis for one day timespan [14]

This last condition concerns to secure connection possibility on the foundation; the base plates and the anchor bolts must be clean, intact, and defended from vandalism. The organization of mounting work must be planned step to step from storage to the needed spare parts and reserves. These steps are

- storage of the elements (where, how far, loading of elements on lorries, etc.),
- investigation of construction traffic;
- study of the roads' load bearing, width and height, are these adequate for the given lorries;
- how the unload of elements can be carried out;
- is there enough space to deposit the elements, ensure the construction traffic and adequacy of the space left for the building personnel;
- check of for bottlenecks of the transport to determine a realistic building time demand;
- determination of minimum number of installation personnel for the preparation of base plates (counter plates) of columns, mounting of elements;
- determination of the necessary number of tools, scaffolds, ladders, etc.

With the advantages of the demountable flood protection walls, there are of course some disadvantages, as

- sensitivity to river ice collisions, ice jam pressure if the ice phenomena are significant in the hydrological regime;
- the heighten of the structure is not possible, nor during flood, nor if the flood protection capacity is to be risen with the change of law;

- sensitivity to any kinds of vandalism;
- sensitivity to construction time before flood arrives;
- if some relevant elements of the structure do not work in time, or the construction work is delayed for any reason, the defence capacity will not be completed in the required time, and the defence capacity will not be adequate.

Drainage system

The drainage system has a key role to remove the water from the surface and from the upper layer of the earth. The capacity of the drainage system is the function of the current state of the recipient.

If the recipient can receive the water unlimitedly, the capacity depends on the hydraulic characteristics of the drainage sewer. These characteristics are related on the flow characteristics of the conduits (friction loss and local losses), and the water swallowing capacity of gullies. Generally, the gullies swallow the frequent rains, but for extreme rains these are not sufficient.

If the recipient is the flooding water body, the receiving capacity is often limited. In this case, the sewer's collected water must be pumped into the flooded water body. The collected water origins from the rainwater, infiltration, and local inundations. In this case, the pumping capacity has the key role. As the collected water cannot enter into its recipient by the gravity, the sewer must be closed by a sluice gate and a pumping facility has to be put into operation.

In greater sewers, the pumping capacity is ensured with built-in pumping stations, while in the case of smaller canals the pumping can be solved in temporary solutions, using fast deployable diving pumps in the manholes. If there is no built-in sluice gate, shutoff plugs can be applied to close out the flood from the sewer. The temporary pumping site is arranged similarly to the sewer bypass pumping. The pumping capacity has to be harmonised with the drainage system. Use of reservoirs helps the balanced use of the pumping capacity and serves also as a reserve. As more pump is applied in the pumping station, the shift can be more flexible, it can follow the change of the hydraulic load. The use of different performance pumps can ensure the flexibility, as well. Generally, it is better if the pumping station is equipped with some smaller pumps, some mid and high-capacity ones, with more turn on levels. The capacity of pumps can be regulated by frequency converters too. The modern pumping stations are equipped often with frequency converters. The determination of the applied pumps is the question of a complex analysis of hydrological, hydraulic, and energetical issues.

Seepage control in built-in areas

Seepage occurs between the two sides of the defence line; the energy level of water is higher on the flooding side, and the water seeps toward the lower energy level defended side. The groundwater level rises in the defended side because of the seeping from the flooding water body and because of those waters which keeps towards the recipient water body. These result in the wetting and inundation issues of subterrain facilities, as basements, public utilities; the seeping water can cause soil boiling, washing out of fine soil grains, etc. The water can appear in terrain depressions as well.

The appearing water can cause the buoyance on structures if their weight is lower than the power of hydraulic uplift.

The control of seepage can be ensured by:

- the total closure of water transmitting layer – it causes backwatering without groundwater control
- the partial closure of water transmitting layer (vertical lengthening of the seepage line) – reduces the rise of the groundwater level
- widening the foundation of the dyke (horizontal lengthening of the seepage line) – reduces the rise of the groundwater level
- frontal or core insulation – reduces the rise of the groundwater level
- the drain on the protected side (groundwater control) – reduces the rise of the groundwater level

The mentioned technical solution has relevance mainly in case of built structures, but some of them can be used for temporary (operative) flood protection facilities, the seeping line can be lengthened by impermeable structure, first of all nylon foil. As the seepage is pressure dependent, the height of temporary structures is limited by seepage.

A traditional procedure for the control of seeping in the operative flood protection is the counterpressure basin to control the seepage on surface, leakage or soil boiling (concentrated seepage) from the earth, or cracks of pavements or walls, etc. It is a widely used complementary tool for traditional operative flood control, practically for any structures. Traditionally, the counterpressure basin can be made of sandbags or if there is no other way, using any other available material. Building of counterpressure basin is a flexible structure depending on the building technology, it can be built following the terrain' level and the available place; it is very important in urban environment. The most flexible building material is the sandbag, it can be joined to almost any walls, earth structures, despite of its work demand. The well-built sandbag structures are largely impermeable if the bags are placed in a regular way, and the layers of sandbags are compacted. The counterpressure basins must have overflow to control the water level. The overflowing water must be taken into a recipient. In urban environment it can be a sewer.

Public utilities – blocking of secondary flood phenomena

Public utilities in the urban environment are excellent possibilities for water transmission, the directly or indirectly (seepage). As in case of seepage, the place's exposure to the appearance of water depends on the pressure circumstances, or in other words, the relative height position of the surface or the facilities, by the law of communicating vessels. If the defended object and/or its environment is lower than the flooding water's level, inundation can occur.

The water can be conducted directly in water utilities, or in the ducts of non-water utilities or defence ducts for water utilities. The abandoned and forgotten gas, hydrocarbon or heating water pipes do similarly as water or wastewater pipes, if the flood water gets into the pipe.

The operating water and waste water conduits can be simply blocked by valves. The same can be do with gas or other fluid-transporting conduits. In the case of the ducts of any kinds of utilities sealing can be applied. The sealing must be qualified for the possibly occurrent pressure level.

The abandoned, forgotten pipes can cause inundations, taking the water from the flooded area towards the defended sites; this issue can be managed temporarily by blocking the pipe, and after flood event, the conduit must be demolished. The malfunction of pressured public utilities can occur during floods, since the seeping water can cause decrease of load bearing capacity of the earth and uneven vertical movement; the result of this process can be the failure of public utilities. The gas pipes' failure can cause the danger of explosion, the water p'pe's breakdown results in washing out of earth. If it happens at the defence line, in the worst case the flood burst out towards the defended territory.

If a gravitational pipe suffers a failure, the water can get into the tube, washing away the soil through the pipe, and this causes the depression of the surface, collapse of pavements of roads, sidewalks, etc.

The water can provoke flooding somewhere else, in cellars for example. These errors can be prevented observing the related rules of approaching and crossing the defence lines by the public utilities.

Another category of threatening is the potential seeping around the public utilities. During the construction of the public utilities the earth has been disturbed, and changed to graduated material. This results in the increasing of seepage capacity of the earth around the pipes. The cause of this increasing can be: the

- inadequate compactness of the earth disturbed by the construction of public utility;
- grainy bedding layer below or around the pipes;
- seepage at the contour surface of the tubes in the earth.

The problem of seepage at the contour surface can be solved by closing the way of water, using cut-off-plates, or collars to block or lengthen the way of water. In reduction of the seeping, the compactness of the earth and the bedding layers around the pipes has importance. In some cases, the regulation of building ductile material pipe limits the compactness to secure the intactness of the pipe. This issue can be solved using a stronger pipe at least on critical places around, making a mechanical defence defence for the lighter tubes. On these strong pipes, the cut-off plates can be made, and the slot between the two pipes can be closed with some flexible sealing material. The cut-off plate must be wide enough to close the possible seeping in the bedding layer and in the zone around the pipe.

Pressure pipes for gases and liquids

Pressure pipes are equipped with valves which can be used as a closure for flood protection aims. These public utilities generally must work during floods, so the valves should be closable, operable during the floods as well.

Gravitation pipes, hollow pipes for periodical operation

Gravitation pipes are generally rainfall and sanitary sewers. The rainfall sewers have intake generally to the have flooding waterbody.

The sanitary sewers can be or separated or combined.

The flood water can get into the rainfall drainage sewers through the gullies, and by infiltration through manholes and pipe fittings. The free outflow is not possible if in the recipient water body’s level is high, so the earlier detailed closure and pumping is necessary. The same is true in the outflows of combined sewers, in this case the treatment plant must receive an greater volume of water. This last situation is true in separated sewers too. The flood water can get into the conduit, but in this case the increased discharge of water can get directly to the treatment plant, or it can cause combined sewer overflow, despite that originally these pipes were designed for the sewage runoff, exclusively.

Operative flood control tools

The operative flood protection covers any flood control intervention which is related to an ongoing flood event.

In urban territories, the flood protection facilities are generally existing, and only some defection must be managed, for example lack of drainage, inadequate height of dykes or flood walls, or some seepage issues. The aims are the ensure of inundation free state of the urban territories, exclusion of life losses, minimization of damages, maintenance of the operability of the town.

The traditional tools of the operative flood protection is the sandbag. However, the sandbag seems to be an archaic tool, its flexibility and favourable characteristics keep it in the focus. Positive, that only the sacks are necessary stored for the flood protection use, the sand can be transported from anywhere, mines, construction sites, depending on the urgency. If there is no available sand, as an ultimate possibility, almost any kinds of earth can be utilised. For the filling, civilians can be mobilised, since it does not demand professional knowledge. The filling can be mechanised to a certain level. Building a sand bag work demands a professional who can command a group of professionals or voluntaries; with a professional direction, any kinds of sand bag works can be constructed. The most important fields of the operative flood protection interventions are:

- heighten the defence lines where it is necessary, with temporary solutions
- counterweight to balance the buoyance of subterrain chambers, manholes, etc.

- control of sewer overflow by weighting the manhole covers
- construction of counterpressure basins
- strengthening the stability of structures, gates

As complementary materials, PVC foils can be used to diminish the wetting of sandbag works.

Further traditional materials are the clay, wood beams, boards, practically anything what can be available in mass, and can be utilised in a relatively simple way. Negative characteristic of traditional tools and materials that after use, a lot of them must be taken into landfills. The use of materials can be counted in hundreds of tons. Regarding sandbags, the reuse of plastic sacks has low possibility, because this material is not UV stable and the Sun ray decompose it within weeks. Originally the sandbags were made of jute; those sandbags were recycled, cleansed, and stored in for a next use; their application seemed to be uneconomic in the past decades. The wood materials must be cut in size; after flood utilisation is limited. But, traditionally, even beyond the sandbag, anything can be used to prevent the inundation, and gives hope to prevent the harms.

An old idea is to diminish the wastes of flood protection materials, the research for well-designed, multiple applicable solutions, which would be developed explicitly for flood protection.

Heighten the defence lines with temporary dykes or barriers

If the water level exceeds the level of the defence line, heightening of the defence facilities is a plausible solution.

It can be done using traditional sandbag wall, or applying modern products to reduce the working hours. The new solutions are generally more expensive, and in the most of the cases, demand professional personnel.

Control of buoyance

There are several underground structures in the urban areas which are sensitive to the hydraulic uplift. The uplift power can be balanced with weighting. These interventions can prevent damages of manholes, under surface chambers, basins, the connected pipes, and in some cases the harm of road pavements.

The weighting together with the other stabilisation forces must exceed the uplift power. The weighting can be made of sandbags, filled water tanks, deposed of earth on the endangered facility, or sometimes vehicles. (Figure 18). It is proposed to overweight the facility approximately 1.5 times, for security considerations.

Figure 18. Weighting of underground railway by vehicles (buses and garbage collecting lorries) in a highest ever flood, 2013 Budapest (Budapest Sewage Works)

Figure 19. Weighting of manhole during the 2013 Budapest flood (Budapest Sewage Works Ltd.)

In the case of the weighting of manholes and chambers under pressure of the flood, the weighting must exceed the direct uplift power awakening on the bottom surface of the manhole cover. This solution works if the weighting is at least 1.2 higher than the uplift power. Near the balance, the gaps around the manhole cover will leak. In this case an alternative of weighting can be the construction of a counterpressure basin (Figure 19). The drainage of the leaking water is to swallowed by the local rainwater sewer.

Figure 20. Counterpressure basin over a manhole, the overflow is not necessary in this case (Budapest Sewage Works Ltd.)

In the case of underground chambers, which are not water sensitive, but their structural secure is not enough against the hydraulic uplift power, another way of the counter-weighting is the flooding of the chamber. The chamber can be filled up from running water, pumped water from the flooding water body, or there can be built in wells on the bottom of the facility to fill up the internal volume to a specific level.

If the chamber or its installations cannot be flooded, the chamber should be weighted. The load can be anything which can increase the resistance force against the uplift force. In case of smaller manholes, chambers, sandbags, filled "big bags", earth or rock depos can be used. Sometimes the load bearing of the chamber's bottom plate is question marked. In this case, the chamber's bottom must be weighted. When the weighting is placed on the slab, and there are holes on the slab, the holes must be covered a load bearing plate or boards, etc., and a 1.5 mm thick nylon foil must be used under the mound of sand or earth. Loading of greater structures can be done by vehicles, as happened in 2013 in Budapest during the flood of the Danube, when the tunnel of the local train was loaded by unused trolleys, buses and even sanitary tracks (Figure 18).

Control of sewer overflow

A sewer overflow can occur if there is high water on the recipient, or an intensive rainfall causes overload, or because of the combination of these two causes. Less frequently, also extreme seepage can cause overload by infiltration or breakdown of sewer. The overflow is the consequence of a high energy, over the level of terrain. Theoretically, intervention can be the weighting and the counterpressure basins, if the pressure level is manageable. Generally, there is no time of intervention in the case of rainfall caused overflow; these phenomenon takes place within a short time, so a reaction cannot be performed. Since the manholes are mostly in the roads, the intervention could be obstacle by the traffic. In the case of the high water level of recipient, the interventions can be made more easily. If the overpressure lifts up the cover of the manhole, the possibility of success of the intervention is low, in this case the counterpressure basin can be a good solution, but the building of it can be a huge struggle because of the elevate water discharge. In this case the discharge of the outflowing water can be so great that the sandbags or other materials can be flushed away. In this case, the most practical is to drive the water towards a manhole of another sewer line, which can temporarily receive the outflowing water, or to greens.

Summarising, a sewer overflow can be managed if the pressure is not to high in the tubes. In this case, the solution can be the loading of covers, manholes or chambers or by making counterpressure basins around the manholes as it was showed earlier (Figure 19, Figure 20).

Using of new solutions of flood control barriers

As it was mentioned before, there are several newly introduced products for flood protection purposes which can be accepted with reservations regarding the security issues. The main aims of this development are:

- the undisturbed use of urban spaces in flood free periods
- the shortening of the building time of temporary flood protection works
- the diminishing of human work demand
- reusable solutions for economical flood protection methods

The most important characteristics of the temporarily built flood protection solutions and their deployment are the followings:

- structures should be mounted before the flood arrives
- if the building of temporary structure is in delay, the defence capacity is absent
- the applied product can be joined to other flood defence structures, they have to be flexibly joined to walls, dams
- resistant to the surface level depression
- minimises seepage between the terrain and the bottom of the structure
- strong and resistant to the punching or abrasion effects (these are often inevitable during the deployment of these structure)
- resistant to waves
- resistance to the water pressure without slipping
- well determined maximum water level

In the urban environment several dangers threaten the temporary flood protection structures. The urban environment is rather various, with earth surfaces, road curbs with 5–15 cm steps, unevenness, poles, rails, walls of buildings or flood protection walls, etc.

The applied product must be linkable to these objects simply and secure way. There must be sure way of fixing any probable leakage or breakdown with some complementary structure, as foils or sandbags.

There are several types of temporary flood protection solutions which can be joined to vertical surfaces with significant difficulties only, for example the water filled cylindric composite plastic-tissue tubes (Figure 21.). These products cannot be formed to angular, so in the angles of the terrain and wall the waterproof joining is impossible. These gaps can already cause a significant water discharge with some 10 cm pressure.

Figure 21. Inflatable barrier during the 2013 Budapest flood (Budapest Sewage Works Ltd.)

Similarly to the join to walls, the unevenness of terrain can cause flows, despite of the significant weight of the cylinder. Its curvature cannot mime the surface, the gaps let the water flow through below the cylinder causing potentially washing out. The materials of these products are quite strong, but some of them, despite of the really high strength of the composite plastic tissue cannot be resistant to abrasion. When in the construction phase the inflated cylinders are moved to their position, abrasion can occur if the tube touches the terrain. Some of these products have significant size, 10 m or more, so the moving of these elements demands a coordinated work of 6–8 persons. The elements can be built only in a certain direction, so the speed of construction is limited. The filling of the cylinders can be supplied from the flood water, according to the brochures. The flood water is not clear, so the internal surface of the tubes is going to be polluted. After use, the emptying of the cylinders can be solved one by one, since the tubes cannot be disjunct in filled state. The emptying of a 10 m long element takes several 10 minutes, so it can take more time than the construction did. Then remains the problem of cleansing, the internal parts are unreachable, the drying takes very long time. Another issue is the loadbearing which is the function of the inclination. The filled cylinder is prone to crawl downwards direction, and the several tons mass cannot be stopped. If the water pressure pushes the tube, the motion is unstoppable. There is no exact calculation about the real loadbearing, nor experimental verification. However, the material is strong, holes can be formed unnoticeably, the loss of water can be noticed only after filling up, when the change of the tube takes a lot of time. These issues influence the judgement of secure.

Similar questions can be raised concerning the use of flood control barriers with steel structures, and several other products. Without responding these issues, the utilisation of this kind of products can induce problems. A strong criticism is important to avoid a potential failure later, in a real flood protection situation.

The Figure 22 shows a flood barrier which kept 1–1.3 m water with success in 2013 Danube flood.

Figure 22. Steel structure based flood barrier, Budapest, 2013 (Budapest Sewage Works Ltd.)

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Mitja Brilly

Climate Change and Facts on Floods

Introduction

The problem of climate change has become an essential subject of political and, to a lesser extent, scientific and expert debates in recent decades. Discussions gained global significance by establishing the IPPC (The Intergovernmental Panel on Climate Change). WMO (the World Meteorological Organization) and UNEP (The United Nations Environment Program) established the IPPC in 1988. The Panel Secretariat is located at the headquarters of the WMO Management Office in Switzerland. To date, 195 members of the UN have joined the panel and appointed their representatives to panel. To date, the IPPC has composed five reports compiled from the elaborations of individual countries and working group reports. The problem of climate change has been highly popularised, so individual countries, including the EU, have invested relatively high research funds to respond to climate change issues, mainly caused by increased carbon dioxide emissions from the combustion of fossil fuels.

The functioning of the IPPC has been criticised for its working methods. For example, researchers who had comments on the work of the IPPC were systematically excluded from the work of the panel (see the documentary film entitled *The Great Global Warming Swindle (2007)* [20]. Scientific periodicals also systematically refrained from publishing articles that contradicted IPPC reports' views.

The enormous resources invested in climate change research have also given a deeper insight into our climate and its dynamism, nature and the processes that condition and influence all areas of our activity. Costly paleohydrological studies (e.g. removing ice samples in Polar Regions) and other research have brought us to a long-standing view.

Recent historical research has shown us the effects of our activity on the environment and, last but not least, the climate. As a result, Anthropocene is becoming a concept that describes today's modern geological environment.

Increased energy in the atmosphere is expected in the future, and thus a more frequent phenomenon of hydrological extremes: drought and floods. In this context, various methods and procedures have been developed for their determination and the search for suitable solutions.

What can we learn from the past?

Several studies have been carried out concerning the past geological history of temperature changes. In any case, these studies contain great uncertainty in presenting results. The temperature diagram in the past (Figure 1) shows interesting dynamics and large oscillations of a higher temperature in the past. In the last geological period, the Holocene witnessed very calm temperature dynamics. Temperatures show a nearly constant value for the last ten thousand years. The highly dynamic development of human society occurs in the Holocene's relatively uniform climate conditions, which have more mild dynamics (Figure 2). Indeed, after the increase, the relatively stable temperature state continues until today.

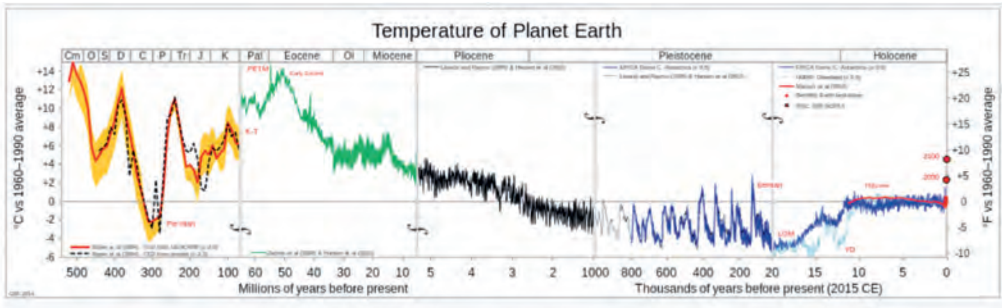


Figure 1. Changes in global temperature in past geological periods [22]

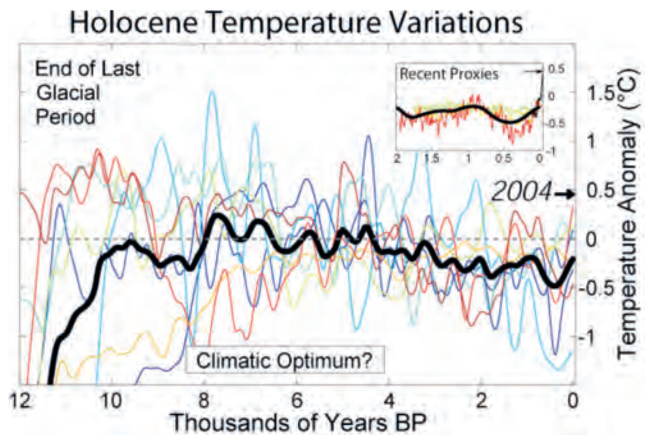


Figure 2. Global temperature changes in the Holocene [22]

Changes in temperature in the last 2,000 years are shown in Figure 3, where the warmer period is noticeable around 1000 and the so-called medieval cold period around 1600. The last 2,000 years are a historical period that can be studied based on many written sources. The dynamics of the measurement period, the last 200 years, are shown in Figure 4. For this period, we have temperature measurements at some places on the Earth. From the diagram, the temperature of the 19th century is very rapidly changing because of the operation of volcanoes. The chronicles of this era witness the pronounced climatic dynamics, problems in agriculture, hunger and political turmoil. The 20th century shows calmer dynamics, although, in the second half, there is a smaller, marked period of more cold weather in the sixties and seventies, Figures 5 and 6. The dynamics of temperature change show a marked increase in temperatures in the last quarter of the 20th century. Taking into account linear trends, the temperature rise is even more pronounced. If we take the average temperature of 1785–1800 for the baseline of the analysis, we get evidence of a marked increase in temperatures around the world (Figure 6).

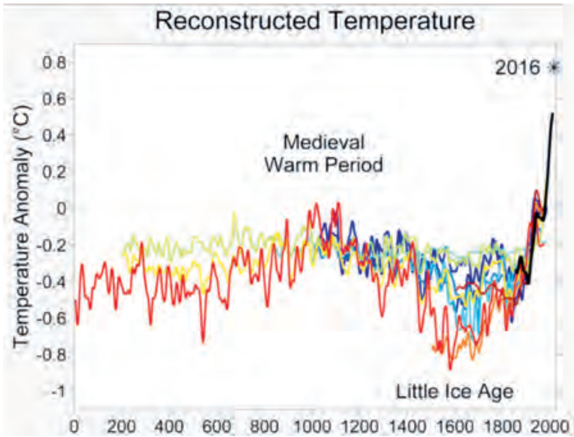


Figure 3. Changes in global temperature in the last historical period [22]

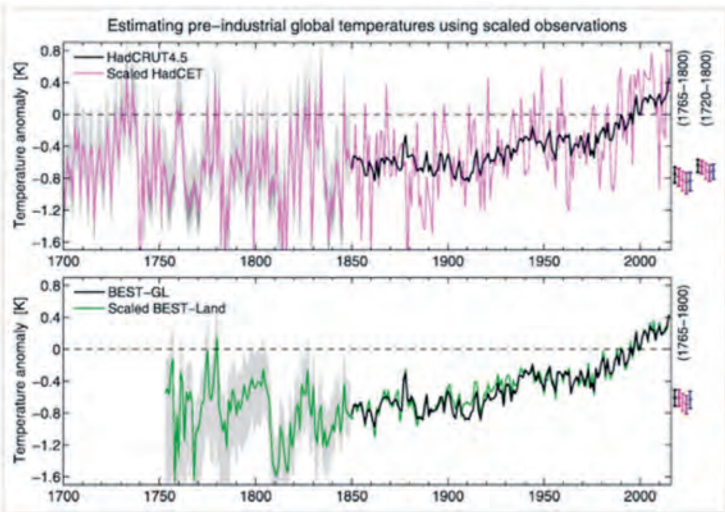


Figure 4. Changes in global temperature during the period of measurement [14]

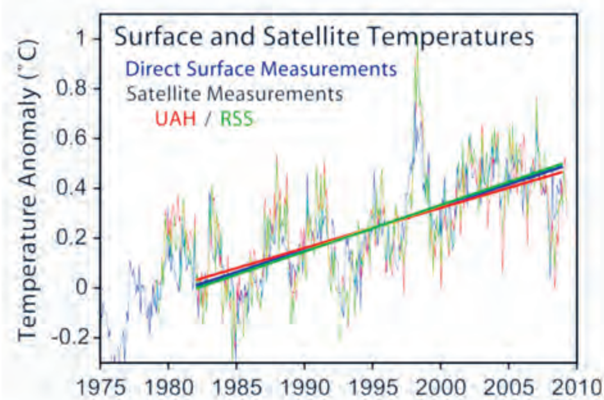


Figure 5. Changes in global temperature in the last historical period [22]

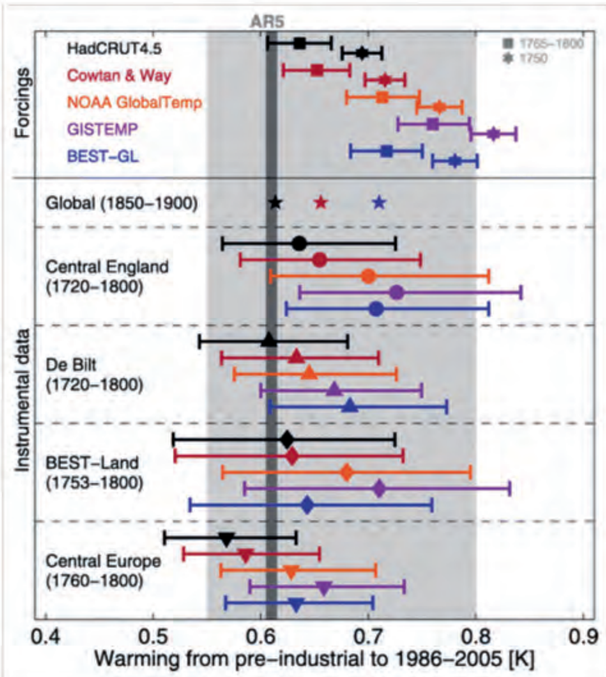
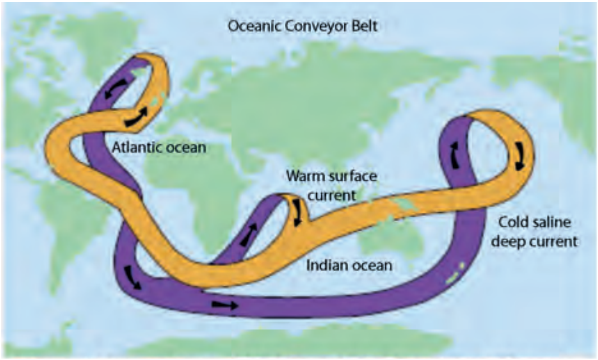


Figure 6. Increase in global temperatures in the last two hundred years [14]

Data analyses also point to other geophysical factors that affect the climate and the dynamics of global temperature. For example, the eruption of volcanoes introduces large quantities of ash into the atmosphere and changes climate at a global level for a shorter period. Further, we influence ocean currents (Figure 7). Due to ocean currents, we have today a relatively calm climate change of the Holocene. Any change or interruption of these flows will cause radical climatic changes.



Schematic representation of the thermohaline circulation of ocean
Source: Broecker (1991) Oceanography 4:79–89.

Figure 7. The main ocean currents that shape the climatic conditions on the Earth [18]

Last but not least, we have to mention that in a changing environment, measurements in the past were carried out with different instruments and were implemented with different procedures. Moreover, places with temperatures higher than in the natural environment are covered with vegetation in the urban environment. All this confuses the measurements and increases their uncertainty.

Important conclusions based on paleo research are [18]:

- significant switches in the Earth System functioning occurred on much shorter timescales than the glacial/interglacial cycles
- the recorded changes were often rapid and of high amplitude; in some cases, temperature over large regions changed by up to 10˚C in a decade or less
- although major, abrupt transitions, reflecting a reorganisation of the Earth System, are most evident in predominantly cold, glacial periods, they are not absent in the last 12,000 years, especially in lower latitudes
- the changes demonstrate widespread spatial coherence but are not always globally synchronous
- complex inter-hemispheric leads and lags occur that require feedback mechanisms for amplifying and propagating changes in both space and time

From a physical point of view, we must be aware that we are dealing with non-linear phenomena in a complex environment that we know very little about.

Anthropocene

With civilisation's development, humanity changed the environment and adapted it to its needs from prehistoric times. Until the Industrial Revolution, these changes were more of a local significance. However, these impacts already interfere with global geophysical processes with extremely dynamic industrial development.

In the last two centuries, the human population and the world's economic wealth have been overgrown. These two factors significantly increased the consumption of resources registered in agriculture and food production, forestry, industrial development, transport and international trade, energy production, urbanisation and even recreational activities (Figure 8).

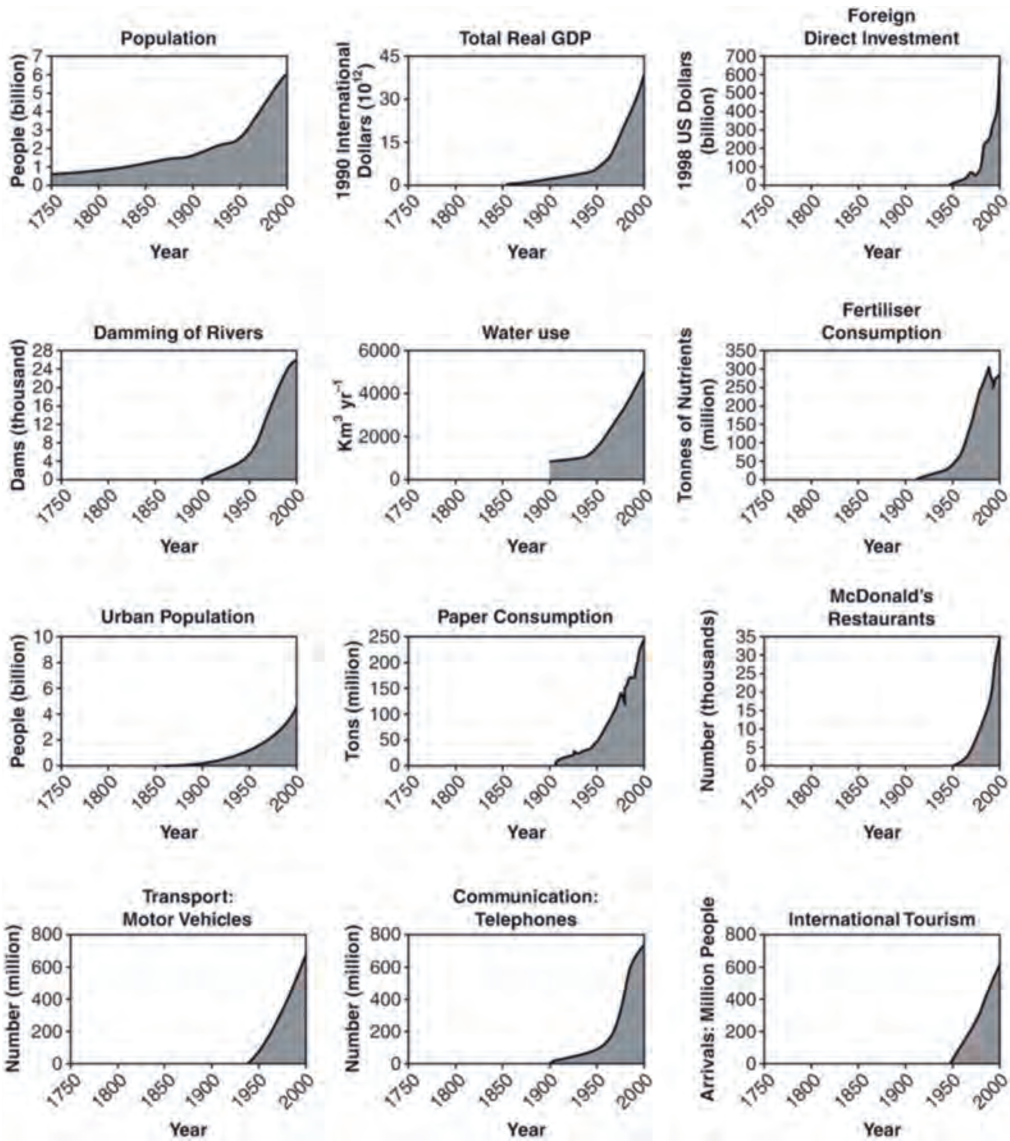


Figure 8. Increase in human activities [18]

Today, man has already subordinated 50% of the surface of the Earth to his needs. More than half of the people on Earth live in cities, and this trend is increasing. The exploitation of fossil fuels increases the amount of CO_2 in the air. More than half of the available fresh water is exploited for human needs (Figure 9). The consequences are global.

The extraordinary technological development of the last century has led to societal changes. The diagram of the proportion of employees in individual activities in the US is shown in Figure 10. The diagram shows the decline in the US labour force in the previous century. In the preindustrial society, the share of employees in agriculture was 80–90%. Even employees in other activities covered a large proportion of self-sustaining food. Country-specific figures are shown in Table 1.

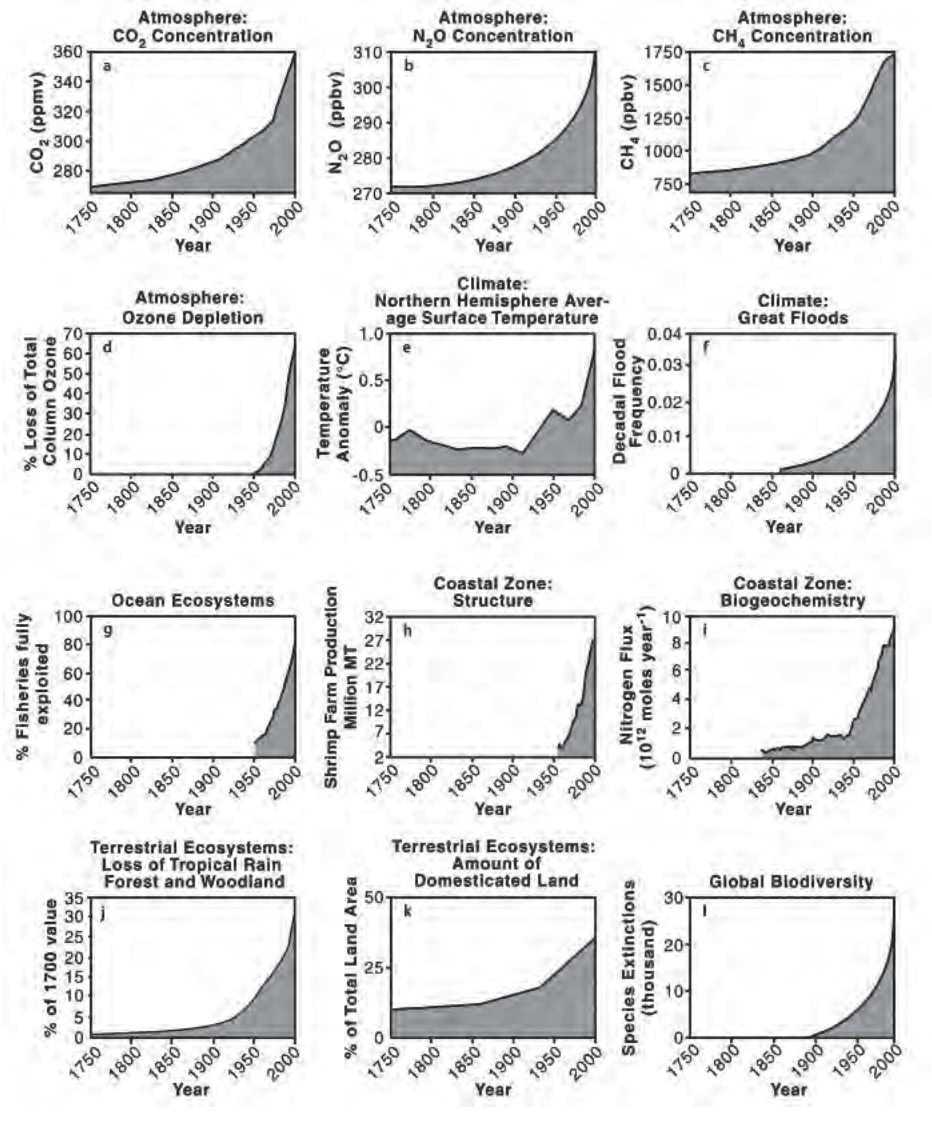


Figure 9. Global changes in the Earth System [18]

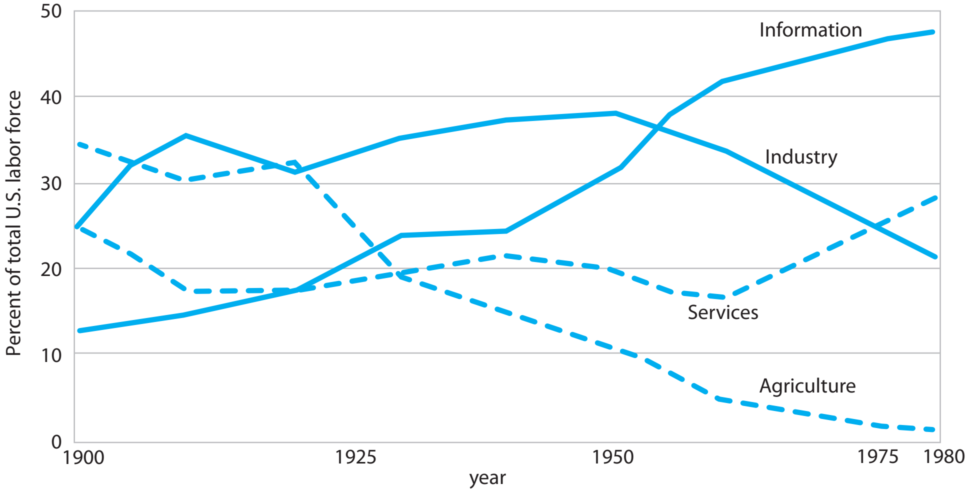


Figure 10. Dynamics of the labour force share in individual activities in the USA

Table 1. Labour force by activity in the period 2008–2012 [4]

Country	Agriculture	Industry	Management	Services	Sales and office
US	0,7	20,3	37,3	17,0	24,2
Germany	1,6	26,6	73,8		
France	3,8	24,3	71,8		
Sweden	1,1	28,2	70,7		
Swiss	3,4	23,4	73,2		
Greece	12,4	22,4	65,1		
Portugal	11,7	28,5	59,8		
Austria	5,5	26	68,5		
Hungary	7,1	29,7	63,2		
Slovenia	2,2	35	62,8		
Croatia	2,1	29	69		
Serbia	21,9	19,5	58,6		
BiH	20,5	32,9	47		
Montenegro	6,3	20,9	72,8		
Macedonija	16,7	27	57,3		
India	53	19	28		
Brazil	15,7	13,3	71		

Data in Table 1 and Figure 10 show that the share of labour in agriculture dropped in the United States from 35% in the middle of the 20th century to just 0.7%. In EU countries, this percentage of the workforce ranges from 1.6% in Germany to more than 10% in Greece and Portugal. In this context, we must draw attention to the considerable budgetary resources of the EU and individual countries in order to keep this share of the workforce actual. As a result, there are fewer people in rural and agricultural areas. The workforce in the industry has also been steadily falling from nearly 40% in the 1950s to 20% at the beginning of the 21st century. Most people in post-industrial societies today are employed in services, management and development. If the countryside is where agriculture works, the city is an environment for developing services and research activities.

Modelling climate change

The expected climate change results from increased CO_2 emissions into the atmosphere. This is the problem with which the IPPC deals and offers solutions. It is a fact that the amount of CO_2 in the atmosphere has been rising in recent decades (Figure 11). There are also noticeable oscillations for CO_2 in the air from trapped ice samples. Interesting is the phenomenon of sudden CO_2 increase and then feedback the biosphere's response, gradually reducing the amount of CO_2 in the next period.

Carbon dioxide is not released only from burning fossil fuels but also from various limestone rocks and organic soils in the decomposition process. The CO_2 balance on the Earth's surface is shown in Figure 12. The carbon balance on the Earth is dynamic and complex. The use of fossil fuels increases CO_2 emissions by 9 billion tonnes. Additionally, organic matter in the soil and carbonate rocks decompose faster due to higher temperatures.

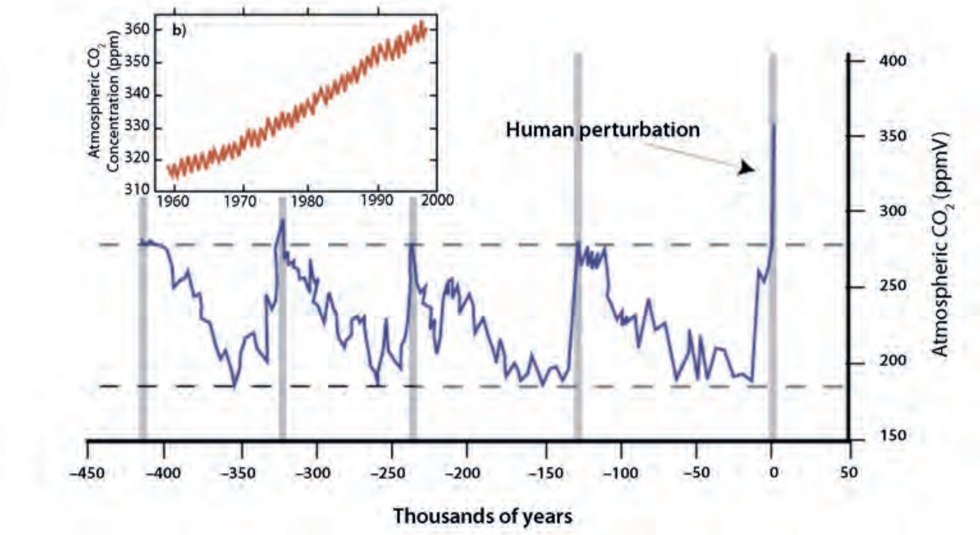


Figure 11. The dynamics of the change of CO_2 in the atmosphere [18]

The impact of increased CO_2 in the atmosphere on climate change is simulated by numerous Atmosphere-Ocean General Circulation Models (AOGCMs). The IPPC develops its views and guidelines policy based on this modelling.

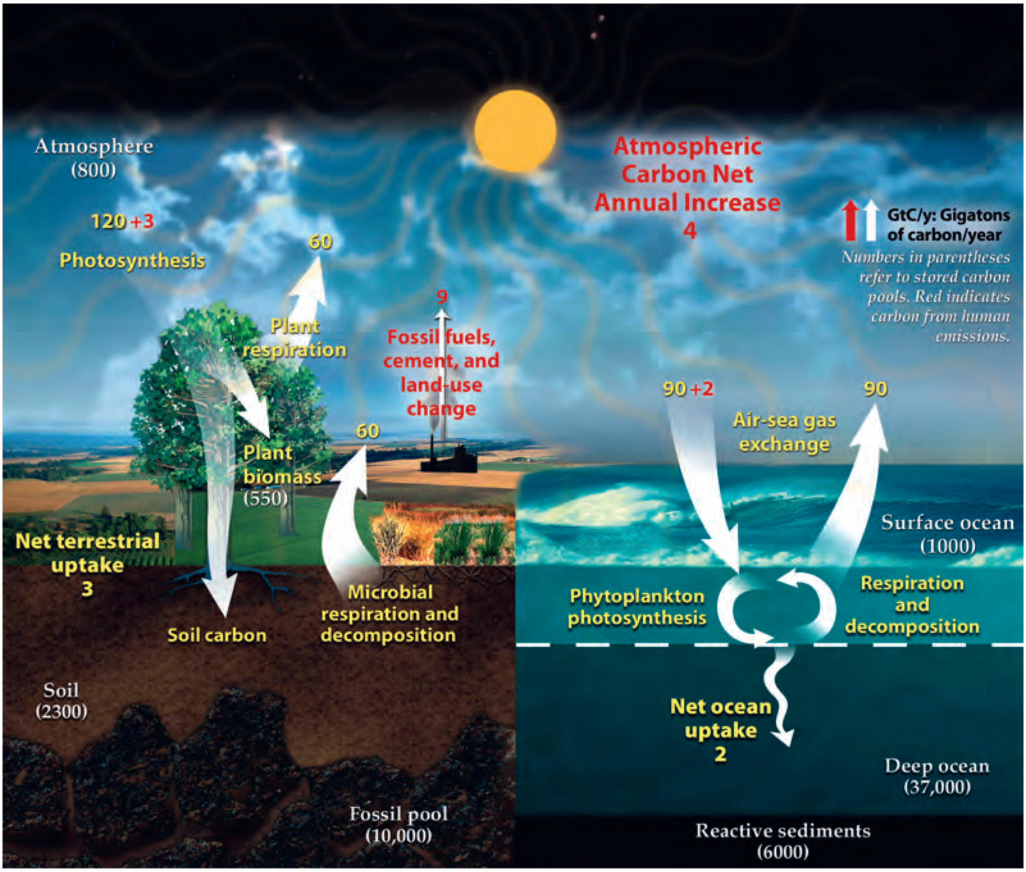


Figure 12. Carbon cycle [21]

By investing relatively large funds, different models have been developed, which more or less successfully try to simulate different scenarios of future climate development (Figure 13). Criticism of the results is more rarely published [8]. It is also noted in the IPPC reports that decisions to reduce CO_2 emissions were made based on not-so-successful models. Official predictions of the increase in global temperatures are given in Figure 14.

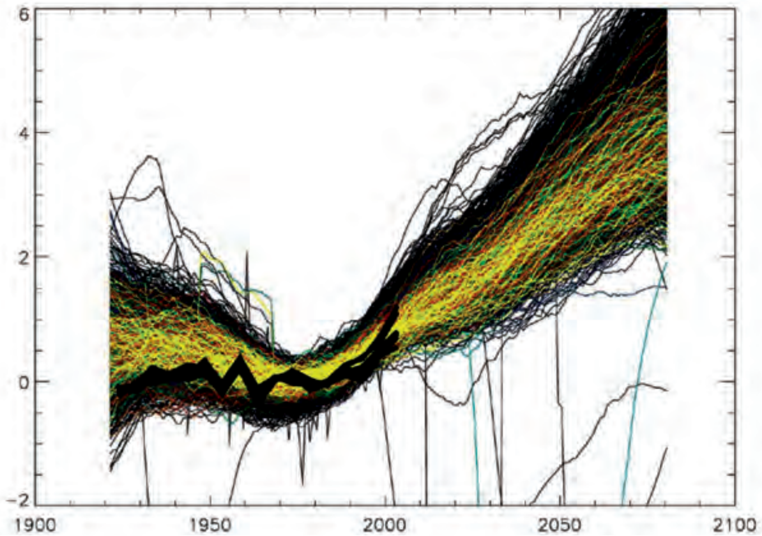


Figure 13. Simulation of temperature rise in the UK (the University of Oxford, climate prediction@net)

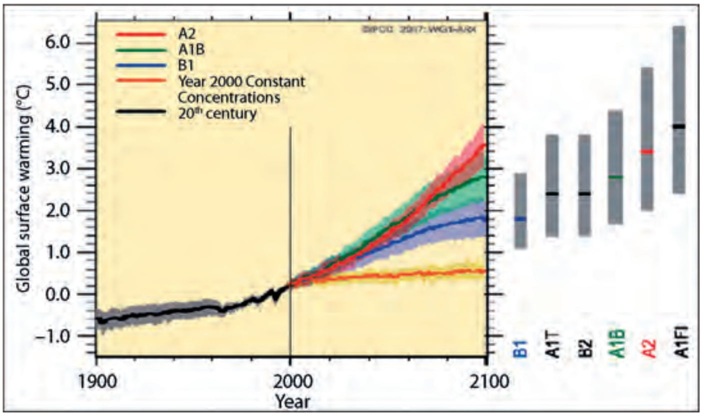


Figure 14. IPPC global temperature rise simulations [7]

The deficiency of the official IPPC projections is the relatively small uncertainty of results. Therefore, even the worst scenarios were pushed to the forefront in the forecasts and implementation of the policy.

Water and climate change

Water is the fundamental factor in the transfer of energy and the formation of climate on Earth. Water vapour is also the most important greenhouse gas. Unfortunately, the dynamics of water in the atmosphere meteorological models could not yet be accurately simulated, and there are high discrepancies between the forecast and the measurements in the order of magnitude of the phenomenon. The case is similar to the climatological models of the IPPC. However, various simulations and forecasts, especially extreme phenomena, were performed. [12] gave an overview of the achievements in this field for Europe. The position of the flood directive on the impact of climate change on floods is the following: "The scale and frequency of floods will likely increase in the future as a result of climate change, and inappropriate river management and construction in flood risk areas." Some studies have been done in Europe based on climatological and hydrological models. The results differ significantly from one another (Table 2).

Studies show remarkable differences between regions, as seen in the research carried out in Germany (Figure 15).

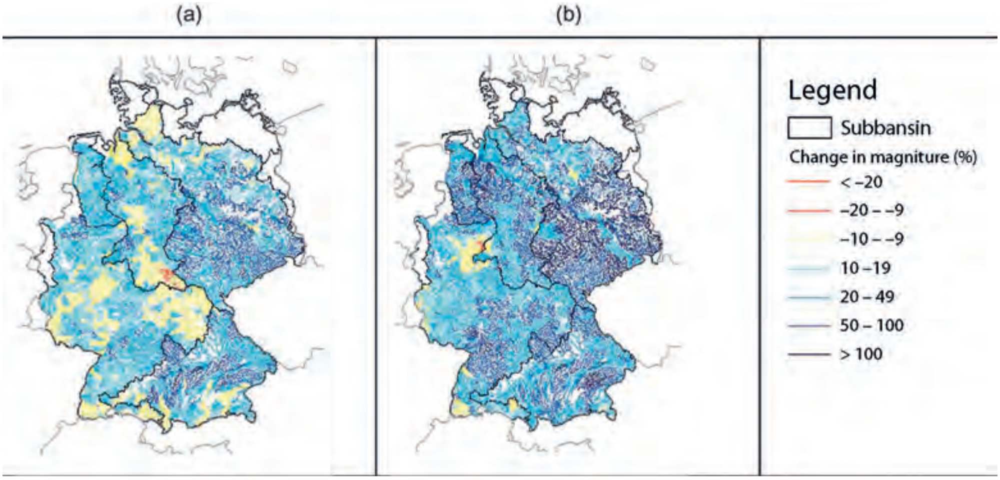


Figure 15. Predicted changes in flood flow with a 100-year-return period [12]

Differences result from different climate change scenarios, models for different levels of calibrated parameters, and so on. Unfortunately, in the analysis of larger areas, hydrological models are often used without any calibration. The problem is the inadequate knowledge of related disciplines that deal with hydrological simulations. Furthermore, floods are complex and depend not only on the increase of one-day precipitation, especially on more significant watercourses. In any case, the results of calculations on climate change by hydrological simulations present very high uncertainty.

The flood observations are not in line with the hydrological predictions of the climate change scenarios. This is because large watercourses have specific oscillations in their regimes, which are not related to each other or show remarkable trends of climate change impacts (Figures 16 and 17).

Table 2. Comparative analysis of hydrological studies [12]

Table 1. Comparison of studies on large-scale projections of changes in flood frequency and intensity.

Paper	Number of climate model scenarios	Number of hydrological models	Variable	Time period	Emissions scenario	Central Europe	WE: British Isles	EE: Eastern Europe	NE: Scandinavia, Finland	SE: Iberia	SE: Italy, Greece
European-scale studies											
Roudier *et al.* 2016	5 RCM/GCM combinations	3: LISFLOOD, E-HYPE, VIC	Q_{100}	+2°C*	RCP 2.6, 4.5, 8.5	⬆	⬆	⬇N ↑C, S	⬇↑	⬆	⬆
Alfieri *et al.* 2015	7 EURO-CORDEX	1: LISFLOOD	Q_{100}	2080s	RCP 8.5	⬆↓	⬆↓N	↓N ↑S	⬇↑	↑N ↓S	⬆
Rojas *et al.* 2012, 2011	1: HIRHAM5-ECHAM5	1: LISFLOOD	Q_{100}	2070–2099	A1B	⬆↓NE	⬆	⬇↑S	↑ ↓mix	↑ ↓mix	⬆↓
Dankers and Feyen 2009	5 RCMs	1: LISFLOOD	Q_{100}	2071–2100	A2, B2	↑ NW SE↓C	⬆	⬇↑	⬇↑	↑ ↓mix	↑ ↓mix
Lehner *et al.* 2006	2 GCMs	1: WATERGAP	Q_{100}	2070s	A1B	⬇↑	↑ ↓mix	⬇↑	↑	⬆↓	↑ ↓mix
Global-scale studies											
Giuntoli *et al.* 2015	5 GCMs	6 GHMs	Frequency of high-flow days	2066–2099	RCP 8.5	–	–	–	⬆	–	–
Dankers *et al.* 2014	5 GCMs	9 GHMs	Q_{30}	2070–2099	RCP 8.5	↑ W ↓E	⬆	⬇	⬇↑	⬇	⬇
Arnell and Gosling 2016	21 GCMs	1: Mac-PDM.09	Q_{100}	2050s	A1B	⬇↑W	⬆	⬇↑	↑ ↓mix	↑N ↓S	⬇↑
Hirabayashi *et al.* 2013	11 GCMs	11 AOGCMs	Q_{100}	2071–2100	RCP 8.5	⬆NW↓SE	⬆	⬇↑	⬇↑	⬇↑NW	⬇↑
Hirabayashi *et al.* 2008	1: MIROC	1: MATSIRO LSM	Q_{100}	2071–2100	A1B	⬆↓	⬆	⬆↓	⬇↑	↓ C ↑ SN	↑ ↓mix

⬆ mostly increase
↑ partly (in sub-areas) increase
⬇↑ mostly decrease, in sub-areas increase
⬇↑ mostly decrease, in some sub-areas increase
↓N ↑S decrease in N, increase in S
↑ ↓mix: mixed patterns
– no significant changes
*reference to time instant when the global warming reaches 2°C above the pre-industrial level

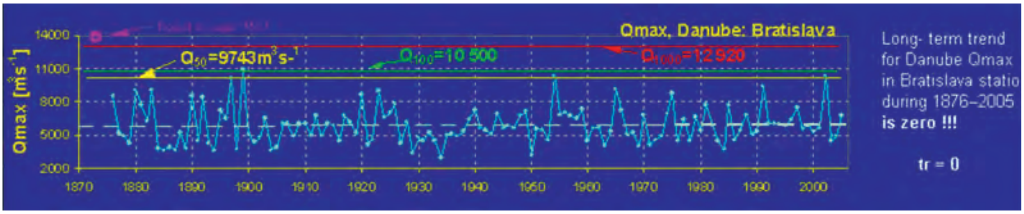


Figure 16. The result of the research on the maximum flows of the Danube River [16]

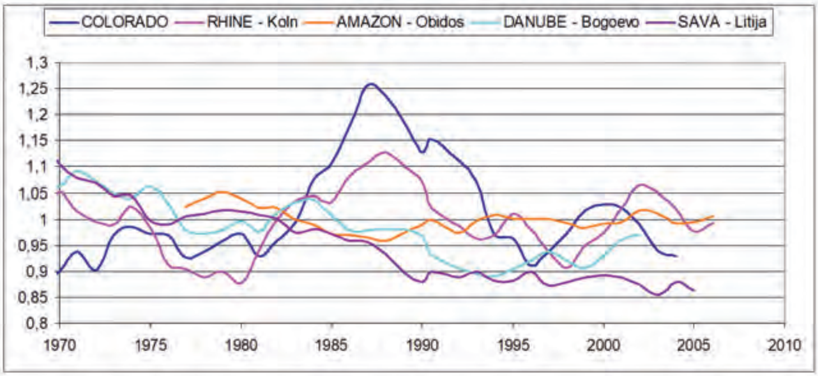


Figure 17. Ten-year moving average of discharge ratio (annual discharge/mean discharge from 1961–1990) [2]

In addition, fluctuation of the discharges is related to a differentiation between rivers, and there is no high difference between humid and arid climates. Also, there is no difference in amplitude in the past century and the past thirty years. The Sava and Colorado Rivers have long periods when their discharge is below the long-term average. This below-normal trend appears to be increasing during the past ten years. If this trend continues, it will significantly impact water supplies, hydropower generation, agriculture and the availability of potable water for municipal use.

The problem is best illustrated by the conclusion in the article [11]:

“[9]) claim that climate is not changing due to human activities (climate is “naturally trendy”:[5] and that climate models do not provide a reasonable basis for assessing possible future impacts. The first assertion is not supported by the evidence (as thoroughly reviewed by the IPCC [15]), and we have demonstrated that the second claim is also false here. Climate models are not used to make predictions but to make plausible projections of possible future change. Water management’s challenge is using this information on possible futures to help make adaptation decisions.”

There is also the opinion that [12]:

“For the time being, there is no conclusive and general proof of how climate change has affected flood behaviour. The conventional attribution framework struggles with the small signal-to-noise ratio and uncertain nature of the forced changes [19]. As a result, there is low confidence (due to limited evidence) that anthropogenic climate change has affected the magnitude/frequency of floods. However, changes in other components of the hydrological cycle (e.g. soil moisture) also play a role. As a result, there are considerable uncertainties in projecting future evapotranspiration. Non-climatic factors include changes (mostly anthropogenic) in rivers, such as modification of river channels (e.g. dikes and dams), and changes affecting runoff coefficient and available water storage capacity in catchments, such as urbanisation, deforestation and drainage of wetlands (see [13]). In some basins, non-climatic factors can be largely responsible for changes in the frequency of flood events (see [6] [1]). However, reliable determination of flood frequency trends requires a long time series of good quality river flow data. Often, time series of records are not long enough for trend detection, and hydrological networks have typically been shrinking for budget reasons. Scarcity of ground data of adequate quality and quantity is also a reason for uncertainty in projections because the material for calibration and validation is unsatisfactory. There has never been stationarity in flood frequency—except in the minds of hydrologists. Nonstationary means that a present-day design flood (e.g. Q100) for a particular location, established from historical observations in the reference period, can be dramatically different from a design flood value projected for a future horizon of importance for adaptation.”

Study of the climate change impacts on the Sava River

A study of the impact of climate change on floods in the Sava River Basin was made in 2012 on behalf of the Sava River Commission. The study consists of three parts: the meteorological report, the hydrological report and the proposed measures. A comprehensive overview of the work done was presented in the article [3]. Regional Climate Models (RCMs) carried out the meteorological basics for the A1B scenario and E-OBS – European observation – European daily high-resolution gridded data set. As a result, we obtained data on daily precipitation in a network with a resolution of 0.25o. In addition, there are data for maximum daily precipitation with a return period of 20 and 100 years by samples from 1961 to 2010. The data were processed for climate periods of the year: summer, autumn, winter and spring. We also received forecasts of maximum daily rainfall for 2011–2040, 2041–2070 and 2071–2100. In the exact resolution and period, we also received data on temperature. In the first period, the temperature should increase by almost one degree, in the second by two, and in the third by almost three degrees.

For hydrological analysis, a hydrological model of a basin was developed with the software tool HBV. The Sava River Basin is divided into 13 sub-basins (Figure 18). The model covers all main tributaries: The Kolpa River, The Una River, The Vrbas River, the Bosna River and the Drina River.

The model incorporated three altitudes: up to 700 metres, 700–1,400 metres and above 1,400 metres, and two types of biological coverage – forest and other.

The following input data are required to calibrate/run the model:

- precipitation (32 measurement stations)
- temperature (8 measurement stations)
- discharge data (12 measurement stations)
- potential evapotranspiration (8 measurement stations)

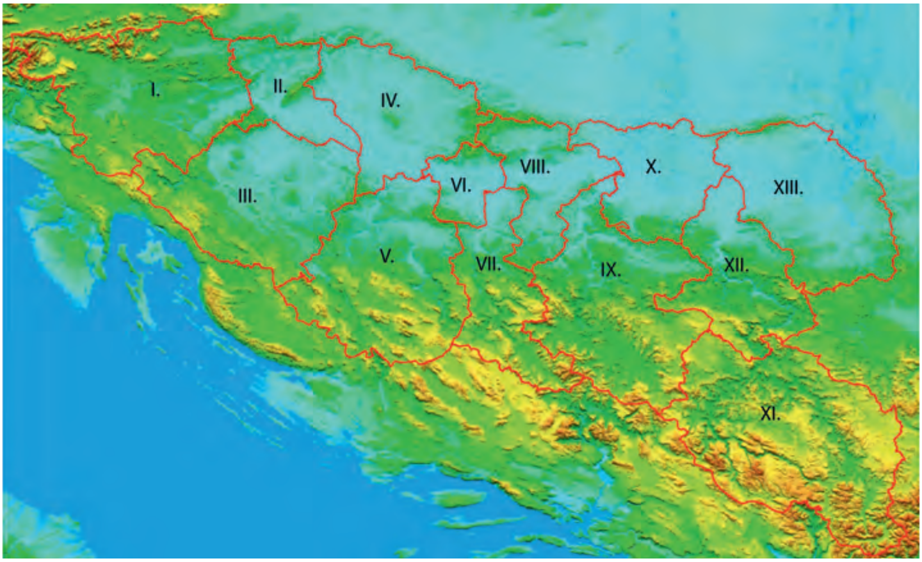


Figure 18. Modelled Sava River watershed – from its source to its confluence with the Danube – with orographic sub-basin and watershed borders [3]

Flood events from 1 September to 30 December 1974 were used for calibration and 1 September – 30 November 1978 for validation. The calibration and validation results were not impressive, but the peaks were quite well simulated.

The precipitation and temperature data from the meteorological report were taken from the raster data set based on the position of rain gauge stations and used for the hydrological model. Summer daily precipitation is slightly higher than autumn precipitation. However, the runoff in the autumn season is much higher due to lower evaporation, so we chose the autumn values for further calculations and analysis. Hydrological model run with E-OBS data for precipitation and evapotranspiration.

The same input data for the calibrated model for the 1974 flood were used for modelling climate change's impact. However, we only changed the rainfall data for the day with maximum precipitation and increasing temperature. Instead of using the measured maximum daily precipitation on rainfall stations, we used E-OBS data with

20- and 100-year-return periods. The model also calculated discharges for E-OBS20 and E-OBS100 events for predicted values.

The probability analysis was derived from the analysis presented in Prohaska's previous report [17]. The probability analysis in the report was derived from data collected from 1926–1965. The analysis does not consider the impact of flood protection measures in Central Posavina, as they developed later. The data about 10, 1 and 0.1 percentage of probability was used as the fundamental relations for water stations. The probability of discharge values calculated for the E-OBS data with the 20- and 100-year-return periods was estimated based on probability for each station. We assumed that predicted discharges calculated by the model and by predicted maximum EOBS precipitation have the same probability as today's discharges, table 3 and Figures 19 and 20.

Table 3. Probability of peak discharges on WS Čatež (m3/s) [3]

	E-OBS_20	E-OBS_100		
Probability	26%	3.05%	1%	0.1%
Observed data	2,308	2,780	3,027	3,400
2011–2040	2,551	3,296	3,694	4,056
2041–2070	2,859	3,770	4,248	4,627
2071–2100	3,072	4,133	4,687	5,060

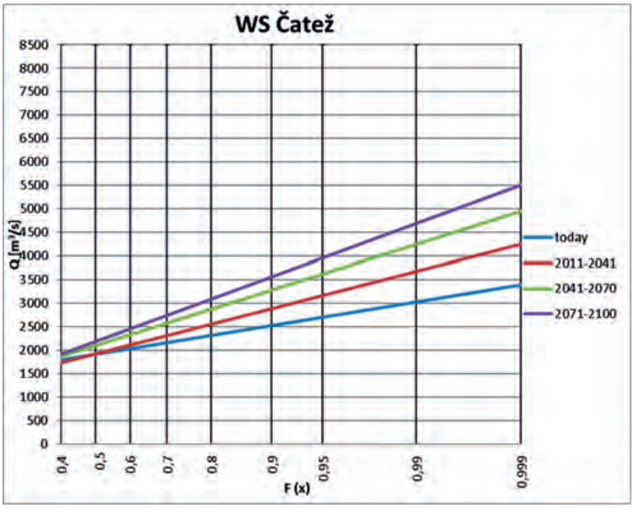


Figure 19. Climate change influences discharge probability on WS Čatež (compiled by the author)

The hydrological system of large rivers is always quite complex. However, the more enormous the system is, its response is more robust. For example, the upstream WS Čatež (10.173 square kilometres) will increase discharge by more than 50% due to climate change until 2100. However, on the downstream station, Županja (62.220 square kilometres) will increase discharge by only 25% in the same period.

The program to mitigate the impact of climate change was developed based on country reports and basin vulnerability analysis:

1. Institutional strengthening of the organisations responsible for the collection and exchange of hydrological data; updating equipment for water level measuring; purchase of new state-of-the-art equipment (meteorological radars, snow cover water content and infiltration rate); use of satellite images for hydrological monitoring; development of models for the prediction of rainfall and runoff; the installation of additional water stations on the Sava River and their transboundary tributaries. Institutional strengthening is fundamental for developing an up-to-date hydrological forecast and warning system.

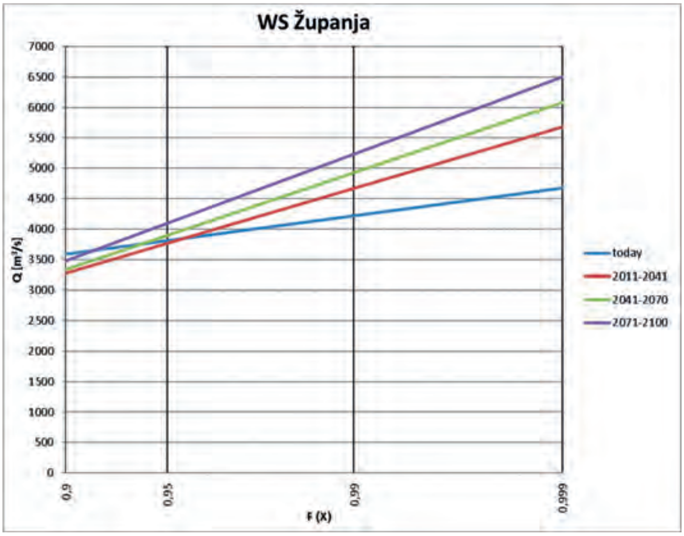


Figure 20. Climate change influences discharge probability on WS Županja (compiled by the author)

2. Determination of cross sections for monitoring changes in the morphology of the riverbed along the main stream of the Sava River and the tributaries on the border: The Kolpa/Kupa River, the Sotla/Sutla River, the Una River, the Bosut River and the Drina River. Particular attention should be on the sections of the intake of significant tributaries in the Sava River mainstream. The profiles should be labelled with permanent geodetic points on the ground, and the measurements should be repeated every two or at least ten years. Erosion or sedimentation could have a significant impact on the water level. For example, the riverbed erosion of the Sava mainstream in Ljubljana and Zagreb municipalities has significantly increased flood protection. Conversely, the sedimentation of the riverbed will increase inundation and decrease flood protection. Morphology data are also essential for successful hydraulic model calibration and validation.

3. The development of hydrologic models for predicting flood flows, including assessment of land use and changes in the biosphere—the development of hydraulic models for calculating water levels and determining the effects of various flood protection interventions. A well-calibrated and maintained hydraulic model is essential for good flood forecasting and determination of flood measures on flood protection along the Sava River. The model will be developed using the results of tasks 1 and 2.

4. Increase the level of protection of significant cities along the Sava River: Belgrade, Zagreb and Ljubljana. Hydraulic models should determine the impact of provided solutions. Similar protection should be developed for critical infrastructures: highways, railroads, industrial and health care buildings.

5. Protecting other cities and populated areas along the Sava River depends on long-term spatial planning and development. Therefore, zoning should be integrated with spatial planning. For example, giving more space to rivers by deepening and widening the river channel; increasing the floodplains by lowering the surface and the movement of dams; removing structures that impede water flow with particular attention to riverfront development.

6. The protection of agricultural areas should be kept up-to-date to mitigate the effects of additional protection of urban areas. Those areas should be equipped with proper warning systems.

7. Integration of flood protection measures with water management, Water Framework Directive and sustainable development.

What to do – Conclusions

The problem of the impact of climate change on floods is complex and still open. From the toy model to the real world [10] suggests:

In comparison to our simple toy model, a natural system (e.g. the atmosphere, a river basin, etc.):

- is extremely complex
- has time-varying inputs and outputs
- has a spatial extent, variability and dependence (in addition to temporal)
- has greater dimensionality (virtually infinite)
- has dynamics that are largely unknown and difficult or impossible
- has unknown parameters

Hence, uncertainty and unpredictability are even more prominent in a natural system.

The role of stochastics is even more crucial:

- to infer dynamics (laws) from past data
- to formulate the system equations
- to estimate the involved parameters
- to test any hypothesis about the dynamics

Data offer the only solid grounds for all these tasks, and the failure of evidence analysis of these data renders the hypothesised dynamics worthless.

The question is what to do in such huge uncertainty of hydrological analysis of climate change. However, at the same time, we should understand the uncertainty of hydrological analysis without climate change impacts under the influence of anthropogenic activities. In practice, we are often surprised by high events like the flood on the lower Sava River in

2014. Therefore, design floods should be calculated with a more extended return period, like in Holland, with a return period of 10,000 years for urban areas. Alternatively, we could take discharge values on the upper uncertainty limit up to 5% or even 1%. Eventually, maximum flood flows should also be calculated. Otherwise, we do not know where flood limits are.

The economic value of damage to agricultural areas is relatively low, and there is no need to increase the level of protection. Such areas could be treated as green solutions to protect urban areas that will be more densely populated in the future. Efficient water management solutions are needed for the future.

Institutional strengthening of hydrological forecast and observation services is an essential governmental issue. However, unfortunately, today, we have fewer hydrological observation stations than years ago in a situation when the need for data and its importance increases.

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Momir Paunović

Environmental Aspects of Floods

The objective of this document is to shortly present the current up-to-date knowledge in relation to floods and environment and thus provide basic information for course attendants on this complex issue.

The course covers general considerations on the topic, discussion on the collection of data on the influence of flooding to the environment, relation of flooding events and aquatic and riparian biota, relation of flooding and pollution and genetoxicological aspect of floods. Basic terms related to the environment are considered, as well. Having in mind that we discussed the collection of data on floods, which comprise field work activities, the document comprises the discussion on safety measures on field.

Terms and definitions

Here we explain the meaning of terms used in the text, but also the terms that should be properly understood by the academic community in order to be able to fully address this important topic.

Biota means aquatic organisms in general.

Ecology is a scientific discipline; the term originates from the Greek root Oikos, "at home", and ology, "the study of"; Haeckle (1870): "By ecology we mean the body of knowledge concerning the economy of Nature – the investigation of the total relations of the animal to its inorganic and organic environment;" Andrewartha (1961): "The scientific study of the distribution and abundance of organisms;" Odum (1963): "The structure and function of Nature." Ecology is also a biological discipline which involves the scientific study of mutual relations of organisms and their interactions with the environment. It is extremely important to use this term in its right meaning and not to mix it with "environmentally friendly" meaning. At least in the academic community, we have to be precise and use the right terminology.

Environment – in general, it means: "The surroundings or conditions in which a person, animal, or plant lives or operates." Here, we consider the environment as: "The natural world, as a whole or in a particular geographical area." The term is often wrongly used as synonym, or surrogate for ecology, or vice versa. Thus, it is important to use this term in its right meaning. Repeatedly, you can hear statements such as "ecological products", or "eco-product", which is the wrong use of the term ecology. It simply means that the product is not harmful to the environment, or that it is produced with the best available technology that reduces harmful effects to the environment.

Ecosystem functioning – is defined as "the joint effects of all processes (fluxes of energy and matter) that sustain an ecosystem" over time and space through biological activities [25].

Ecosystem services – are usually defined as "the benefits that humans receive from nature", our work shows that most services are actually co-produced by a mixture of natural capital and various forms of social, human, financial and technological capital.

Community – here the term is used in sense of biological assemblages.

Flood risk is defined as the combination of the probability of a flood event and of the potential adverse consequences for human health, the environment, cultural heritage and economic activity associated with a flood event [3].

General consideration of the topic

The influence of floods to the environment are diverse and could be generally considered "negative" and harmful (to native ecosystems, ecosystem functioning and native biodiversity, human health and society in general), or natural and "positive" (to ecosystem functioning, native biodiversity) – Figure 1.

Floods are one of the most devastating disasters and they can seriously endanger human life, damage living infrastructure, destroy industrial facilities and agricultural production. Flooding is also connected to the occurrence of different diseases that can often spread rapidly, even becoming an epidemic. Flooding accounts for one-third of natural disasters and affects more people than any other type of disaster [21].

In addition to well-known physical destruction, harmful effects are connected to different other stressors. Interactions of flow regime, including so-called "key hydrological events" (flooding and draughts), and environment (which is also a complex concept in itself) is a complex topic that involves many items that are connected in diverse ways.

Flooding, as a natural event, is an important phenomenon for the normal functioning of aquatic ecosystems and this influence is considered native and "positive".

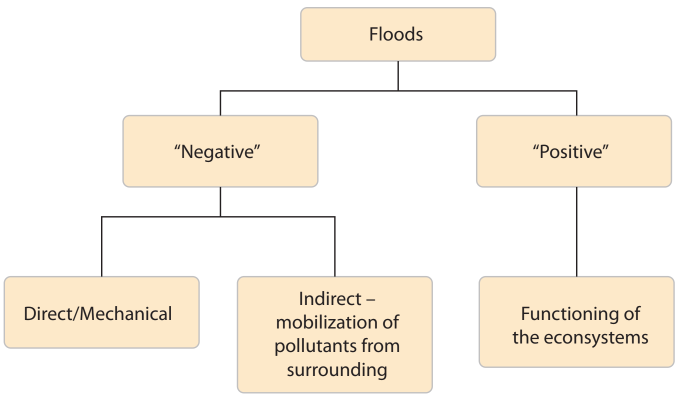

Figure 1. Schematic expression of influence of flooding to aquatic ecosystems (compiled by the author)

The topic of the relations of floods and the environment is even more complex taking into consideration that human influence has altered the interaction between floods and ecosystems. Human activities alter flooding characteristics – change the frequency, intensity and other features of floods. It is well known that even slight modifications to the historic natural flow regime had significant consequences for the aquatic and riparian ecosystems [24].

Our knowledge on the interaction of floods (and generally key hydrological events) and the environment is still limited and we still cannot properly assess the relation of loss and benefit of floods to the e.g. ecosystems [13]. Thus, the assessment of the influence of future extreme hydrological events, like floods and droughts on the environment includes high uncertainty. Traditionally, much attention has been focused on the hazards associated with flooding and floodplains, while less attention has been directed towards presenting the natural, economic and cultural importance of flooding and natural "breathing" of rivers.

The importance of floods to biodiversity

Flooding is a natural event and it plays an important role in maintaining ecosystem functioning and consequently significantly influence native biodiversity [17]. Rivers need floods to create unique habitat and support biological productivity and biodiversity. Dynamic (normal, typical) flow regimes, including flooding events, are important for the "normal" functioning of aquatic and associated ecosystems [15] – riparian forests, wetlands, etc.

Predictable seasonal floods are beneficial for riverine systems and can influence biotic composition, nutrient transport and sediment distribution but unpredictable floods may be disruptive for aquatic organisms [8]. This postulate is the basis of the "flood pulse concept for river–floodplain systems". The natural fluctuation of water level is crucial for the existence of ecosystems and habitats that depends on water. For example, riverine fish need floods for completing their life cycle [19] and many fish species find spawning areas in floodplains. Here we intentionally mention the importance of flooding for fish, since fish are considered a "flag" group for rising public attention. Flooding is also important to all other groups of aquatic organisms. Not only the frequency and intensity of flooding, but also the period of the year when high water levels usually occur is extremely important for aquatic biota, due to the seasonality of many life characteristics and processes in water ecosystems.

Additionally, many aquatic ecosystems have reduced resilience to extreme events including flooding due to diverse human activities (huge urban development, intensive farming on floodplains, river flow disruptions caused by different hydrotechnical constructions and pollution). These activities increase the likelihood that floods become catastrophic events especially from the perspective of "benefits" obtained from ecosystems [22]. The specific effects of flooding on aquatic ecosystems and their services are not well understood, but the importance of flooding for maintaining ecological functions in rivers has been recognised [15].

The negative influence of flooding on the environment

There is general consensus that extreme hydrological events will occur more often and will be more intense. Following the future increase in air temperature, water temperature will most likely increase in the temperate regions. Due to changes to all temperature-dependent chemical and biological processes, as well as increasing flood and drought events, the pressure on water quality in rivers and lakes will increase.

Floods have the potential to cause fatalities, displacement of people and damage to the environment, to severely compromise economic development and to undermine the economic activities of the Community.

Legal framework

There are many directives and strategic documents that regulate the matter of flooding at the EU level. Here, we address two umbrella documents: the Water Framework Directive in 2000 (Directive 2000/60/EC) and the EU Floods Directive (2007/60/EC).

The adoption of the Water Framework Directive in 2000 (Directive 2000/60/EC) set a new framework for the management of European river basins. The main goal of the WFD is to ensure the achieving of the environmental objective of good ecological status/good ecological potential and good chemical status for all water bodies in the European Union (the initial target year was 2015). In that respect, the river basin approach was introduced, requiring Member States to manage water bodies not within administrative/political units but for a river catchment. The environmental objective of the WFD in Art. 4.4, besides the requirements for the achievement of a good status (good ecological potential for artificial and heavily modifies water bodies), also addresses the issue of the preservation of water status in the future. Each river basin district was required to analyse the main pressures and impacts on water bodies, analyse the economic aspect of uses of water bodies and how it affects the natural environment. Programmes of measures have to be developed to ensure that water bodies achieve the environmental objectives. Thus, the WFD should be considered the umbrella document that, between other issues, regulates the relations of flooding and flood management and environment.

The EU Floods Directive (2007/60/EC) establishes a legal framework for the assessment and management of flood risks, and aims at reducing the adverse consequences of floods to human health, the environment, cultural heritage and economic activity.

The EU Floods Directive promotes that it is feasible and desirable to reduce the risk of adverse consequences, especially for human health and life, the environment, cultural heritage, economic activity and infrastructure associated with floods. The purpose of this Directive is to establish a framework for the assessment and management of flood risks, aiming at the reduction of the adverse consequences for human health, the environment, cultural heritage and economic activity associated with floods in the Community.

Flood risk management plans should focus on prevention, protection and preparedness. With a view to giving rivers more space, they should consider where possible the maintenance and/or restoration of floodplains, as well as measures to prevent and reduce damage to human health, the environment, cultural heritage and economic activity. In particular, it seeks to promote the integration into Community policies of a high level of environmental protection in accordance with the principle of sustainable development as laid down in Article 37 of the Charter of Fundamental Rights of the European Union.

Two Directives are interconnected and regulate the relation between flood protection and general environmental objectives established by the WFD. Those documents are reflected in the national legislative of EU member states, but are also incorporated in the regulatory system of many other non EU countries, thus the variety of agreements and initiatives having legal influence beyond the Union. Requirements regulated in the WFD and Floods Directive provide the frame for better organised flood protection that is compliant with environmental protection principles. Good balance between flood protection and environmental protection, as well as careful planning in the future should minimise the influence of flood protection measures on the environment, including mitigation of the negative consequences to biodiversity.

There are a number of reasons why better coordination is required [4]. The integrated and coordinated planning under the WFD and Floods Directive has the potential to identify measures that can deliver on the objectives of both policies. Natural Water Retention Measures are viewed as one of such win–win measures. Those measures can address major causes of not achieving good ecological status, for example through river and floodplain restoration measures that re-establish flows. Natural flow regulation can significantly contribute to a reduction of flood risk.

The effects of floods to aquatic organisms, community and ecosystems

Although many studies have been written, the effects of floods to aquatic organisms are not yet properly addressed. Flooding is a major disturbance that impacts aquatic ecosystems and the ecosystem services that they provide.

There are two general types of influence – mechanical, direct and indirect, through mobilisation of pollutants. In practice, the influence is often mixed, characterised as "multistressor" influence [13].

Direct, mechanical influence of flooding to the environment

Floods mechanically disturb communities, affect the behaviour of organisms, feeding, breeding, etc. There are many research gaps in knowledge on the direct influence of extreme hydrological events to aquatic communities [16]. The same authors concluded that in case of macroinvertebrate and fish communities, it was demonstrated that the abundance, density, richness and diversity experienced statistically significant decreases following extreme events.

There are many separate conclusions about the direct influence of flooding to the particular aquatic communities. Thus, [20] discussed the issue of direct effects of flooding on the fish community. During flooding and high water levels in 2014, pelagic fish species were sampled in greater proportion than at lower water levels in 2015, when benthic fish species were more abundant. The pelagic fish species are more resistant to the stressful effect of flooding than benthic species [20].

Influence through mobilised pollutants and multiple stressors – indirect influence

The other aspect, the effect of mobilised pollutants on aquatic biota during floods, especially in the case of large rivers, is still not properly addressed and remains an open issue. Pollutants in river water and river bed sediments, in particular in highly urbanised or industrialised regions, are still a concern in Europe. During the flooding event, water mobilises the bottom sediment and material from the flooded riparian ground and with this material different pollutants are mobilised, which is clearly illustrated at Figures 2 and 3. During the "regular" (e.g. mean water flow/level) water level, rivers carry a certain amount of sediment (Figure 2), while during flooding (Figure 3) the amount of mobilised sediment in water is considerably higher, which is visible based on lower transparency and higher turbidity. Urban pollutants such as polycyclic aromatic hydrocarbons (PAHs), polychlorinated biphenyls (PCBs) and heavy metals may enter the rivers untreated via stormwater sewers or combined sewer overflows during intense rain events [13] [18].

Figure 2. River during "typical" medium flow. Photo – Paunović, Momir – the Sava River, upper stretch in Slovenia, 2015; Collection of the Institute for Biological Research "Siniša Stanković", National Institute of the Republic of Serbia, University of Belgrade

Suspended sediments represent a means of transport for particle related pollutants within river reaches and may represent a suitable proxy for average pollutant concentration estimation in a river reach or catchment [18]. Floods play an important role in the transport of pollutants associated with particulate matter. Generally, concentrations of suspended particulate matter and pollutant contents increase with increasing discharge, particularly in the early stage of floods. Based on our study on mussels, aquatic worms and two fish species, flooding had diverse effects on the level of DNA damage. DNA damage in the blood cells of fish specimens increased in summer 2014, one month after the flooding event [9].

The surface water quality at any given moment reflects the impact of both anthropogenic and natural pollution. Besides, extreme hydrological events which are related to a particular season, such as water scarcity and flooding, may further impair the already vulnerable state of freshwater bodies [1] [9].

It is proven that pollution related to flooding depends on many characteristics of the particular region, river type, historical pollution, distribution of point sources of pollution, etc. [18].

Recently, we worked intensively on genotoxical studies on the influence of flooding to aquatic biota [2] [1] [9].

Monitoring of the surface water quality based solely on the analysis of a limited number of xenobiotics cannot be considered reliable due to the presence of a large number of pollutants and lack of knowledge on their role in the environment. Generally, a mixture of different compounds (often in low concentrations) is the main reason for many harmful effects in aquatic biota [13]. In addition to the toxic influence, these agents can exert genotoxic effects, inducing damage in the DNA molecule, which, if not repaired, could lead to mutations and alterations in cells, tissues, organism of the whole population and the ecosystem. Biomarkers attract increasing attention in environmental studies, as a tool for detection of exposure and effects of pollution [2]. When examining surface water quality in situ approach and the use of aquatic biota are particularly valuable, since they provide a realistic insight into the consequences of exposure and bioavailability of a number of xenobiotics [2]. Fish may be exposed to harmful substances through water, sediment and food.

Figure 3. River during flooding. Paunović, Momir – the Sava River, upper stretch in Slovenia, 2014; Collection of the Institute for Biological Research "Siniša Stanković", National Institute of the Republic of Serbia, University of Belgrade

The level of DNA damage in specimens of mussels, fish and worms collected from the site situated on the Sava River was investigated in respect to the flooding event [1]. The selected site was found to be under the impact of two major sources of pollution: the coal processing power plant, with related fly ash disposal fields, and the wastewaters originating from the town Obrenovac. Extreme floods in May 2014 resulted in the evacuation of the entire town of Obrenovac, which resulted in the decrease of the amount and discharge rate of urban wastewaters. This was especially evident by the sudden decrease in the concentration of all indicators of faecal pollution. The results of correlation analyses indicated a negative correlation between the water level and faecal indicators, but on the other hand, a positive correlation was detected between the water level and concentrations of Ni, Cd, Co, Mn and Pb. In May and June 2014, with the peak of the flood wave, the highest concentrations of Mn, Pb, Cd, Co, As and B were recorded. It could be assumed that flooding of the fly ash disposal field and its intensive rinsing by rainfall could be the reason for increased concentrations of metals and metalloids and observed correlations with water level [1].

As a continuation of the study of [1], effects of water level fluctuation and related pollution to fish have been investigated by [9]. Authors applied the alkaline comet assay and histopathological alterations (biomarker of effect), as well as concentrations of metals and metalloids in gills, liver and muscle of selected fish species – freshwater bream Abramis brama (Linnaeus, 1758) to determine relations. Sampling of fish tissues was performed in 2014, during winter (January and February), spring (March and early June) and summer (late June, July and August). Significant seasonal difference in DNA damage was observed in analysed tissues. During spring and summer, the level of DNA damage in gills was significantly higher when compared to the liver. Histopathological analyses showed higher frequency of alterations in gills during spring and in liver during summer, but without a significant seasonal difference. Gills had the highest concentration of metals and metalloids during the spring and summer, and liver during winter. Muscle was the least affected tissue during all three seasons. This study highlighted the importance of the multiple biomarker approach and the use of different fish tissues in assessment of surface water pollution.

The conclusions on influence of pollutants carried along the river and water level change, including extreme events, could be drawn based on the investigation of heavy metals in riparian soil. Thus, [14] worked on the assessment the spatial distribution of arsenic and heavy metals (Cd, Cr, Cu Hg, Ni, Pb and Zn) in a riparian area influenced by periodical flooding along a considerable stretch of the Danube River (Figure 4).

This survey comprised analyses of soil and plant samples collected during the international Joint Danube Survey 3 expedition [10] from 43 sites along 2,386 km of the Danube River. The study revealed a significant correlation between the concentrations of analysed trace elements and three datasets (river sediment, riparian soil and riparian vegetation), which point to a close relationship between riparian wetland areas and adjacent waterways.

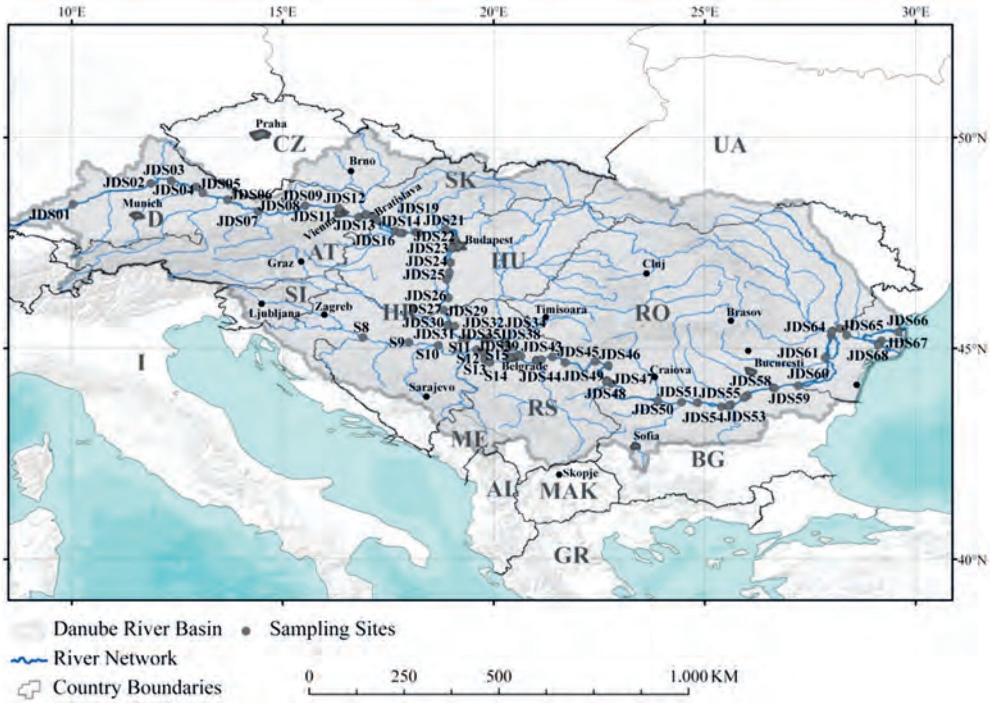

Figure 4. The area covered by the Joint Danube Survey. Paunović Momir – prepared for this publication

This is related to the presence of naturally occurring elements found in metal ore deposits in the Danube River Basin, and anthropogenic metals released by mining and processing of metal ores, industrial facilities and other anthropogenic sources/activities. In general, elevated levels of trace metals were characteristic for the Middle and Lower Danube stretches, with Cd, Cr, Cu, Hg and Ni measured at sites along the Middle Danube sector (in Hungary and Serbia), whereas the Lower Danube (Romanian and Bulgarian) sector was particularly polluted with Pb and As. Our findings point to the necessity of further analyses of the physical and chemical characteristics of the soil and metal accumulation patterns in plant tissues. The obvious correlation between the metal content in the sediment and soil as compared to the correlation between the two datasets and measured metal contents in plant tissue shows that river sediments and riparian soils are influenced by similar environmental factors, whereas the distribution and accumulation of the same elements in different plant species is more complex and species-specific.

Riparian zones are unique and dynamic systems proportional to the main water body size and site topography, which can play a key role in the functioning of an aquatic ecosystem, affecting its chemical, physical and biological processes. Vegetated riverine riparian areas influence the chemical loads from diffuse industrial and agricultural sources, and reduce in-river pollution during floods, with the riparian soil acting as an important sink for pollutants, especially heavy metals [14].

Anthropogenic heavy metals are usually deposited in top soil, therefore riparian soil is a complex and dynamic component and an excellent medium for monitoring heavy metal pollution.

The influence of flooding to ecosystem services

As previously emphasised, because of their dramatic influence on human society, the effects of flooding on aquatic ecosystems are often viewed as negative; however, this is not always the case. Flooding can also provide considerable benefits, including creating conditions for normal ecosystem functioning, providing necessary water for wetlands, rising fertility of soil, recharging groundwater, increasing fish production, etc. [8]. Since the effects of flooding on aquatic ecosystems can be both negative and positive, the assessment of influence of flooding to ecosystem services should also include the study of a mix of negative and positive outcomes [23].

Flood events could be characterised based on magnitude, frequency, duration and volume and these characteristics are important for determining the effects of floods on both aquatic ecosystems and the people who benefit from them. For example, flood magnitude can determine the amount of groundwater recharge or the extent of home and infrastructure damage during flooding. Flood magnitude is only one aspect of predicting flood impacts on aquatic ecosystems and ecosystem services. Ecosystem conditions prior to flooding are potentially equally as important as flood characteristics for determining ecosystem response to a flood event [22].

[22] analysed the effects of flooding to the following ecosystem services addressed by the Millennium Ecosystem Assessment framework [11]: 1. Supporting services (primary production, soil formation); 2. Regulating services (water regulation, water quality, disease regulation, climate regulation); 3. Provisioning services (drinking water, food supply); and 4. Cultural services (aesthetic value, recreation and tourism). Authors find out that:

- the influence of flooding on ecosystem services depends on the flood size and service type
- extreme floods are more likely to be associated with a decline in ecosystem services
- small floods could provoke the decline of ecosystem services, but they also enhance many ecosystem services
- although the trends in ecosystem service availability following flooding were detected, many services responded in complicated ways
- the ratio of gains and losses of ecosystem services related to flooding depends on initial aquatic ecosystem conditions
- the ratio of gains and losses of ecosystem services related to flooding depends on the time of the year

Flood protection strategies should take into the consideration basic requirements that provide normal functioning of aquatic ecosystems. Aquatic ecosystems require flood protection strategies designed to dampen the undesired effects of extreme floods and enhance smaller beneficial floods to maximise ecosystem service provision [22].

The effects of flood protection measures on the environment

Flood protection measures are found to be one of the significant triggers that negatively influence the environment. Technical constructions built for flood protection may disrupt lateral and longitudinal connectivity of river systems, change basic characteristics of natural habitats (including those that depend on water), influence the hydrological character and sediment transport, etc.

In case of the Danube River Basin, a large number of surface water bodies are failing good ecological status, largely due to pressures altering hydrological and morphological conditions and interrupting river continuity, which subsequently impact the aquatic fauna and flora [6]. Structural flood protection measures were identified to be one of the key drivers causing the failure of good ecological status/good ecological potential in river water bodies and new projects impacting water bodies are expected in the Danube River Basin by 2021 [7] [26]. The drivers causing hydromorphological alterations are in particular water supply, navigation (e.g. channelisation to improve ship ways), hydropower (e.g. dams interrupting river connectivity, ponding of rivers, changing flow regime in case of water abstraction or hydropeaking) and flood protection measures changing bed and bank structures. For example, the main key driving forces causing continuity interruption are hydropower generation (50%), flood protection (18%) and water supply (12%) [6]. The impacts of these activities on surface water bodies resulted in the designation of many European rivers as heavily modified according to Art. 4(3) WFD. Heavily modified water bodies (HMWBs) are considered being significantly changed in hydromorphological character due to specific uses.

Flood protection measures can cause a change in groundwater level which may threaten lowland forests, that are among the most complex, dense species-rich ecosystems, but also globally endangered ones.

The WFD include measures to ensure that the hydromorphological conditions provide circumstances within water bodies for the achievement of the good ecological status for water bodies, or good ecological potential in the case of artificial and heavily modified water bodies.

Safety measures

For the proper understanding of the influence of flooding to the environment, confident data is needed. Moreover, to increase the certainty of syntheses, large datasets are needed. In order to collect the data, often field activities are required. Collection of the field data always involves specific safety risk. It is specifically true if the field work is realised during floods. Conditions on field are often difficult and require specific skills and attention during the work (Figures 5 and 6).

Figure 5. Field work during floods. Paunović, Momir – the Sava River, upper stretch in Slovenia, 2014; Collection of the Institute for Biological Research "Siniša Stanković", National Institute of the Republic of Serbia, University of Belgrade

Figure 6. Field work during floods. Stefan Anđus – the Sava River, upper stretch in Slovenia, 2014; Collection of the Institute for Biological Research "Siniša Stanković", National Institute of the Republic of Serbia, University of Belgrade

Here we list same basic measures aimed to minimise safety risk during field work:

- never go alone to the field work
- you should always wear life jackets during the work on the river, or nearby the river
- it is desirable to wear a helmet during the field work
- in case of strong water current, use a rope to stay more stable in the water and to be secure
- in case you use a motorboat, be extremely careful in handling the boat, it is of specific importance if the water is not transparent
- it is necessary to provide the possibility of cell phones or radio communication for the field team

Conclusions

It is crucial to gain sound scientific information on interactions of flooding with other stressors in freshwater ecosystems, in order to understand its environmental and socio-economic consequences, and to convey this information to managers, stakeholders and policymakers, in order to minimise impacts, to adapt to oncoming changes, and to improve our management and policies.

The influence of floods to the environment could be generally considered "negative" and harmful or natural and "positive" (to ecosystem functioning and native biodiversity).

As a natural event, regular and seasonal flooding is important for the normal functioning of aquatic ecosystems and this influence is considered native and "positive".

There are two general types of "negative", "harmful" influence – mechanical, direct and indirect, through the mobilisation of pollutants. In practice, the influence is often mixed, characterised as "multistressor" influence.

Floods mechanically disturb communities, affect the behaviour of organisms, feeding, breeding, etc.

The effect of mobilised pollutants on aquatic biota during floods that could be considered an "indirect" influence is a complex topic. It is still an open issue.

The effect of mobilised pollutants could be assessed based on the measurement of selected parameters, or by using biomarkers.

In both cases, either if the measurements of physical and chemical parameters is applied, or if the biomarker approach is used, it is necessary to use the combination of several indicative parameters to be able to properly assess the influence of flooding to the environment.

Multiple biomarker approach and the use of different indicator organisms and tissues in assessment of the influence of pollution to aquatic ecosystems is required in order to gain a confident synthesis.

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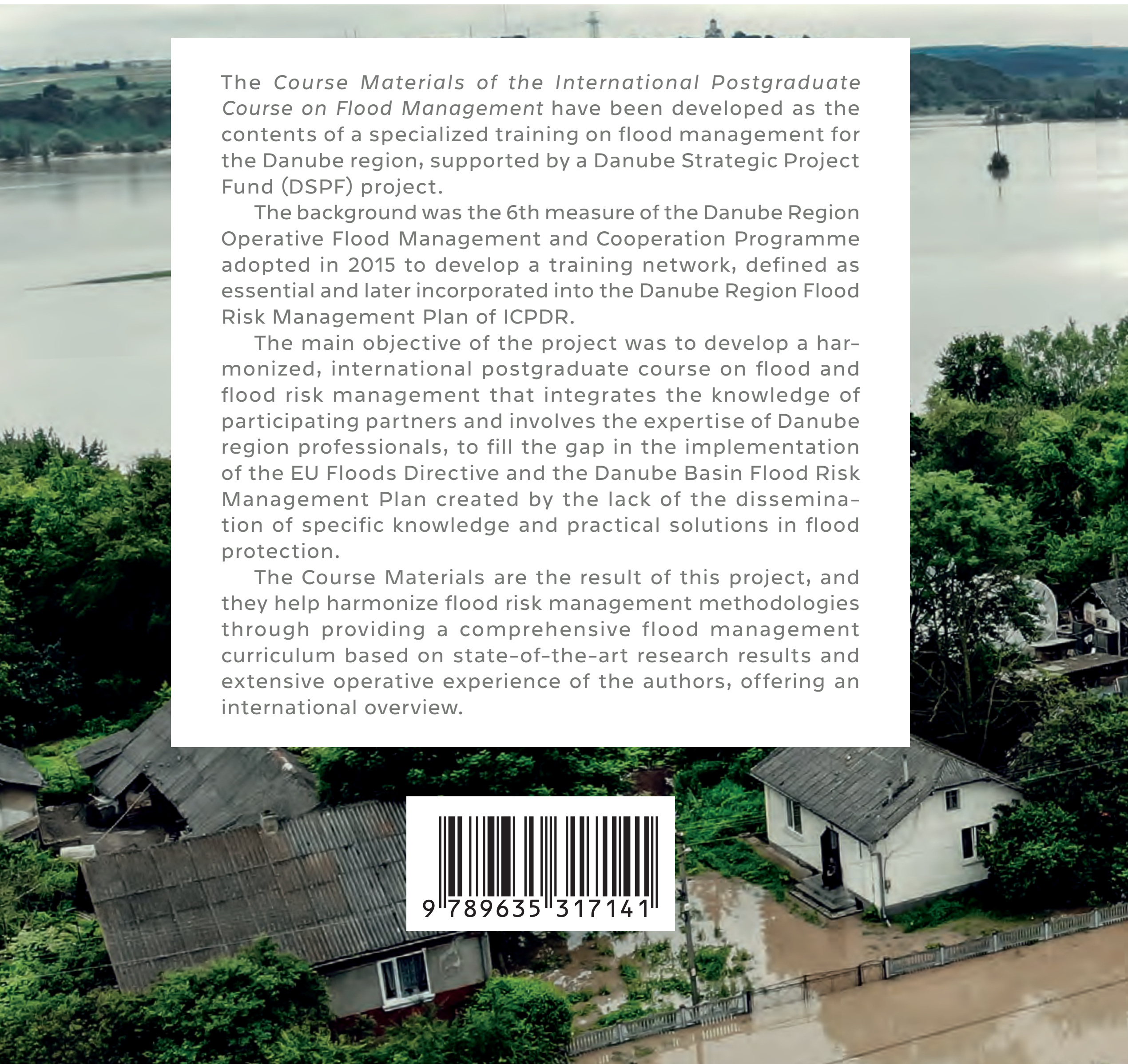

The *Course Materials of the International Postgraduate Course on Flood Management* have been developed as the contents of a specialized training on flood management for the Danube region, supported by a Danube Strategic Project Fund (DSPF) project.

The background was the 6th measure of the Danube Region Operative Flood Management and Cooperation Programme adopted in 2015 to develop a training network, defined as essential and later incorporated into the Danube Region Flood Risk Management Plan of ICPDR.

The main objective of the project was to develop a harmonized, international postgraduate course on flood and flood risk management that integrates the knowledge of participating partners and involves the expertise of Danube region professionals, to fill the gap in the implementation of the EU Floods Directive and the Danube Basin Flood Risk Management Plan created by the lack of the dissemination of specific knowledge and practical solutions in flood protection.

The Course Materials are the result of this project, and they help harmonize flood risk management methodologies through providing a comprehensive flood management curriculum based on state-of-the-art research results and extensive operative experience of the authors, offering an international overview.

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