5. Technical and hydrological effects across scales and thresholds of polders, dams and levees

Reinhard Pohl and Nejc Bezak

5.1 INTRODUCTION

Floods are a natural hazard that can result in large economic losses and endanger human lives all over the world. Global annual flood losses are estimated to exceed US$100 billion (Desai et al., 2015). Climate change together with other changes such as urbanization or economic growth is expected to additionally increase flood losses (Winsemius et al., 2016). A recent study has indicated that flood risk in Europe is increasing in north-western Europe and decreasing in southern and eastern Europe (Bloeschl et al., 2019). Therefore, society needs to adapt to a changing environment (Clark, 2006; Han, 2011).

There are multiple options available for flood risk management and these include both grey infrastructure measures, such as levees, and also blue or green infrastructure approaches such as the use of small ponds that have recently gained the attention of various scientific disciplines (Hartmann et al., 2019; Schanze, 2017; Simm et al., 2013; Bourke et al., Chapter 2 in this volume). Depending on the specific catchment properties such as size or topography, there are also multiple options as to where to locate the flood protection measures including the hinterland (Bourke et al., Chapter 2 in this volume), locations on the floodplains along rivers, and flood protection within resilient cities or urban areas (Popp-Walser, 2013; Rinnert et al., Chapter 8 in this volume). This chapter provides an overview of the hydrological, hydraulic and technical concepts of flood protection measures along rivers where traditional and nature-based flood protection measures are discussed and evaluated from the flood risk perspective.

5.2 MAIN HYDROLOGICAL CONCEPTS

The focus of this chapter is on flood water storage along rivers. Catchment headwaters tend to be more forested and hilly (Bourke et al., Chapter 2 in this volume) compared to mid-catchment reaches which often have lower slopes. Consequently, flow velocity is lower. This needs to be taken into consideration when planning flood protection measures. Additionally, these downstream catchment reaches are more frequently instrumented for flow stage and discharge and resultant modelling can be more confidently treated.

The basic concept used for the design of the flood protection measures is the so-called design discharge concept (Bornschein and Pohl, 2018). Every hydro-technical measure or structure is designed taking into consideration the design discharge value, which is a discharge with a given return period (e.g., 50 or 100 years). The return period for which the structure needs to be designed is usually determined by the national legislation or by national guidelines. Less important structures, such as culverts, are designed using smaller return periods. For example, in Slovenia the design discharge with a 100-year return period is used for culvert design in case of roads within the cities and roads with design speed higher than 60 km/h while a 20-year return period is used in all other cases. Moreover, objects that are more important are designed using higher return period values (i.e., low probability events, extreme event scenarios). For example, the levees used to protect the Krško Nuclear Power Plant are designed using the probable maximum flood (PMF) concept (LaRocque, 2013), which is the highest discharge that is expected to occur at a given location. Other concepts are risk-based (R), considering the flood probability (P) and the consequences (C) expressed by:

$$R = P \cdot C \tag{5.1}$$

The next step would be resilience-based design methods additionally including the time to recover (Pohl, 2020). Thus, a first and very important step in the design of any measure used for flood protection is the definition of the design discharge or water level. In cases where a relatively long series of measured discharge is available (>30 years; series should at least cover one-third of the length of the recurrence period) the most frequently conducted approach to defining the design discharge is the flood frequency approach (FFA) (Bezak et al., 2014). In the process of conducting the FFA based on the defined sample of discharge values (i.e., the most frequent annual maximum method is used, which is one of the hydrological concepts), one relates the return period concept with the discharge estimation (Bezak et al., 2014). Different distribution functions can be used for fitting to the observed values and multiple methods are available for the estimation of the distribution parameters (Bezak et al., 2014). It is desirable to test multiple distributions and select the one that gives the best fit to the measured data based on the results of statistical tests and goodness-of-fit criteria (Bezak et al., 2014). There are several sources of uncertainty in the design discharge estimation, such as uncertainty related to the rating (i.e. discharge-water level) curve extrapolation or uncertainty related to the limited accuracy of high-flow measurements. Therefore, uncertainty and sensitivity estimation should be included in the design discharge definition (Meylan et al., 2012).

In some cases, the information about the design discharge is not sufficient for the planning purpose and an entire design hydrograph is needed. One example would be the planning of a flood reservoir where the complete hydrograph needs to be routed through the reservoir to evaluate the impact of the changed hydrological conditions on flow along the river. In such cases, hydrological (i.e., rainfall-runoff) models are most frequently applied for the definition of the design hydrograph (Bezak et al., 2018; Sezen et al., 2019). There are numerous rainfall-runoff models available; each has its own parameters that need to be calibrated using measured data. Moreover, the model performance should be evaluated before further applications. The main hydrological processes that affect runoff generation, such as rainfall loss due to rainfall interception, infiltration into groundwater or transformation of effective rainfall into runoff, are incorporated into these types of models, which can be done semi-empirically, conceptually with the consideration of water storage concepts, or using a physical basis. There are considerable differences in the concepts used between models.

The most common input variables used by hydrological models are: precipitation, air temperature or potential evapotranspiration data (Sezen et al., 2019). Depending on the size of the catchment area, a sufficient network of meteorological stations should be used for the definition of the input data. In case of smaller catchments (<50 km^2), the most relevant station can be used. However, in larger catchments (>50 km^2) data needs to be interpolated. Another important component for the design hydrograph is the design hyetograph (i.e., distribution of the rainfall amount over time for the design rainfall event). The hyetograph can also be determined using a variety of methods (Bezak et al., 2018; Dolšak et al., 2016). Therefore, by using a calibrated model and a design hyetograph (i.e., rainfall event) one can determine the design hydrograph. There are also other approaches developed for the design hydrograph definition and these are mostly based on the regionalization methods using the similarity concept (Bloeschl et al., 2013).

In case that measured discharge data is not available, which is common in many catchment headwaters (Bourke et al., Chapter 2 in this volume) and across semi-arid and arid regions, catchments are regarded as ungauged. In these cases, alternative approaches are used for the design discharge and hydrograph definition (Bloeschl et al., 2013). Different types of methods are developed for the prediction in case of ungauged catchments, from simple empirical equations for discharge estimation to more complex methods that can be based on the statistical methods and the similarity concept of the nearby catchments (Bloeschl et al., 2013 with more detailed information for further reading).

5.3 HYDRAULICS FOR FLOOD ROUTING AND FLOOD PROTECTION

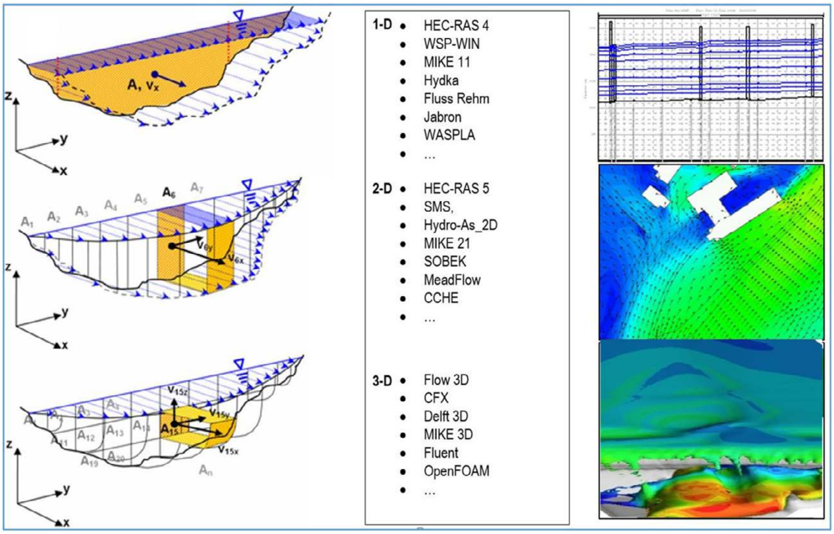
Finding the relation between water depth and flow rate is one of the basic issues in hydraulic engineering. Two questions arise from this issue: first, how much water can be conveyed at a given water level under certain boundary conditions and channel properties (channel and floodplain roughness, cross section, longitudinal slope) and second, which water level will result from a certain discharge at a certain point. The first answer enables the design of the dimensions of a channel and the second is important for flood protection and the estimation of inundation areas.

In the special case of a channel with constant cross section, roughness and longitudinal slope along the flow path (uniform flow) and constant discharge over time (steady flow), the so-called normal flow (depth) will be observed. In this case, the relation between the discharge and the water depth can be expressed, for example, using the Manning-Strickler formula (Chow, 1959):

$$Q = A \cdot \frac{1}{n} \cdot r_{hy}^{\frac{2}{3}} \cdot \sqrt{s_e} = A \cdot v \tag{5.2}$$

with A – flow cross-section area, n – Manning's roughness coefficient, r_{hy} – hydraulic radius (i.e., ratio between cross-section area and wetted perimeter), s_e – longitudinal energy slope, v – mean flow velocity. This also applies to very gradually changing time-dependent (i.e., quasi-steady) flows in prismatic channels (e.g., a trapezoidal-shaped channel). The roughness coefficient was originally derived for 1D-flow calculations. Its application in 2D hydro-numerical models requires experience and expert knowledge and can be confirmed by calibration and verification of hydraulic models.

In the more general case, the channel properties and, consequently, the flow velocity vary along the flow path. Furthermore, the discharge varies over time due to variations in rainfall, water utilization and snowmelt. This non-uniform, non-steady flow can be described by differential equations on the basis of the principles of conservation of energy (i.e., Bernoulli's equation) and mass (i.e., continuity equation) which were introduced by Saint-Venant (Graf, 1998). Especially in lowlands and areas with large floodplains (reclaimed land), a one-dimensional calculation using the above-mentioned approaches is not sufficient. For these applications, two- or three-dimensional numerical software programs have been developed in order to provide a realistic modelling of the flow characteristics. Figure 5.1 summarizes different methods of discretization and shows some examples of numerical software programs that can be used for hydraulic modelling.



Note: Names and abbreviations written in the middle panel represent hydraulic model software names.

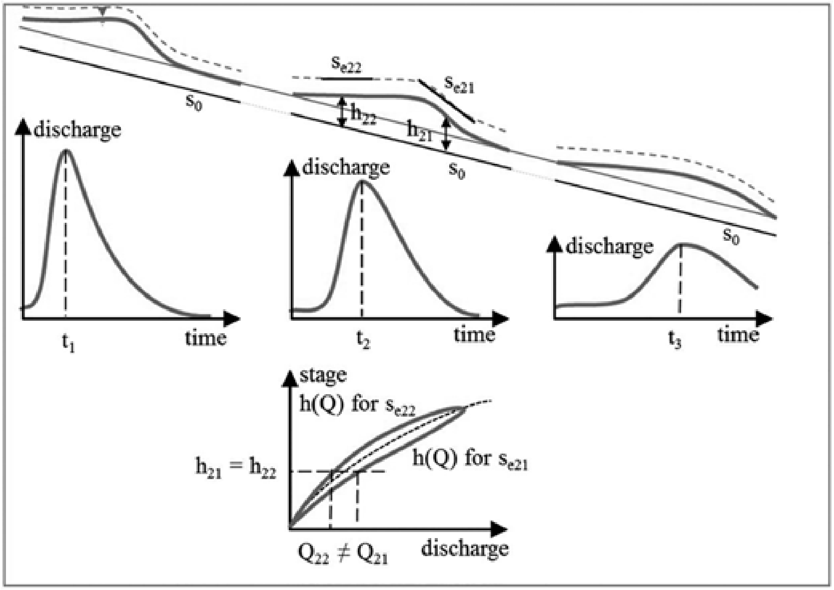
Figure 5.1 *Discretization of calculation elements for one-, two- and three-dimensional hydraulic models and some examples of such models*

The results of the hydro-numerical hydraulic models can be displayed using hydraulic profiles or inundation maps (Figure 5.2).

Due to the steeper energy slope on the flood wave front, there is a higher flow velocity than at the rear of the wave. The higher velocity of the advancing wave and the lower velocity behind the wave peak result in a flattened wave peak with a reduced peak discharge (Figure 5.3). This is particularly so where there is no confluence or surface water input to feed the flood wave entering the considered control section of the river.

Note: From left to right: terrain model, aerial photo, topographic map.

Figure 5.2 *Inundation maps displaying the water depth on different map layers*



Notes: Top: water level as a function of the flow path (profile h = h(x)). Mid: discharge as a function of time (hydrograph Q = Q(t)). Bottom: hysteresis of the stage (water level)-discharge-relation (different discharges at the same depth in front and behind the flood wave peak).
Source: Pohl (2012).

Figure 5.3 *Flood wave at three points of a river/open channel under the assumption of a prismatic channel without additional confluences or precipitation along the considered reach*

This retention effect can be intensified by large floodplains or other retention areas. However, for very long-duration floods (i.e., over several days or weeks), which may be common in perennial large river basins, this effect is almost not perceptible due to a very large water volume. This can result in a quasi-steady flow. In this case, very large storage volumes (S) are needed in or alongside the river to reduce the inflow Q_{in} into a considered river section.

The change of the water storage volume dS within a control section of a river reach, reservoir or polder equals the difference between the section/reservoir inflow Q_{in} and outflow Q_{out} during a period and can be written as a differential equation for small time steps dt:

$$\frac{dS}{dt} = Q_{in} - Q_{out} \tag{5.3}$$

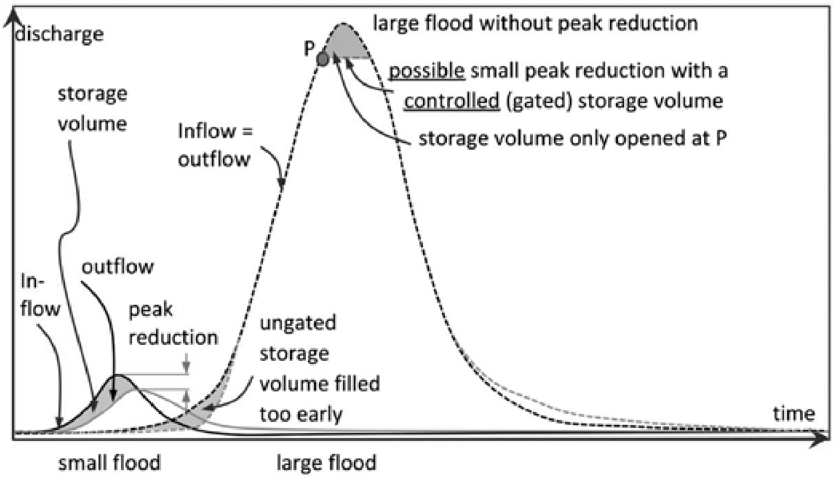
The inflow includes the upstream river inflow as well as the overland runoff from the sides as well as precipitation within the control section itself in the river channel. The outflow is the downstream discharge or the flow through outlets in the case of controlled polders or reservoirs.

The often-proposed levee setback is another flood risk management option that might also improve ecological parameters (Bozkurt et al., 2000). Filling the wider forelands with the arriving flood wave can retard the wave propagation and reduce the peak discharge. However, the latter does not work when the flood wave fills the retention volume before the peak has arrived (Figure 5.4).

In the case of large flood waves, only controlled polders or reservoirs can reduce the peak when their inlet structure is opened at the appropriate point in time shortly before the expected passage of the peak. This flood management requires a rather good flood prediction, which usually involves a combination of measurements and modelling to find the right moment for opening the gates (Acreman et al., 2002).

Another secondary effect of levee setback along relatively short flow paths is the development of the water level. Assuming subcritical flow conditions (i.e., Froude number smaller than 1, which indicates a flow with lower flow velocity and bigger depth compared to supercritical flow that is characterized by Froude number larger than 1 possessing higher flow velocities and lower depths) and applying the Bernoulli's-Theorem:

$$z + h + \alpha \cdot \frac{v^2}{2 \cdot g} = const. \tag{5.4}$$



Notes: The image shows a comparison of a small flood (solid lines) and a large flood (dotted lines). Inflow hydrograph: black line, outflow: grey lines. Hatched area: storage volume. Hydrographs of a small flood (i.e., 2-yr flood) and a large flood (i.e., 100-yr flood) are shown as an example. All areas in the chart indicate water volumes because the vertical diagram axis represents the discharge and the horizontal axis depicts time. For the small event, a peak reduction is possible; however, for the large event, controlled reservoir storage is required. Only when its inlet gates are opened at point P can the expected peak reduction be reached.
Source: Pohl (2019).

Figure 5.4 Effect of a small flood retention measure

with z = elevation above a datum (e.g., mean sea level), h = water depth, α = kinetic energy correction factor, v = mean flow velocity and g = acceleration due to gravity, together with the principle of continuity:

$$Q = v \cdot A = const. \tag{5.5}$$

This shows for the same discharge that a wider flow cross section after a levee setback causes a lower flow velocity and hence a slightly increasing water depth at least at the lower end of the setback reach upstream of a reduction of the flow cross section area. The requested minimum length of levee relocation to get a lower water level has been investigated by Gilli (2010).

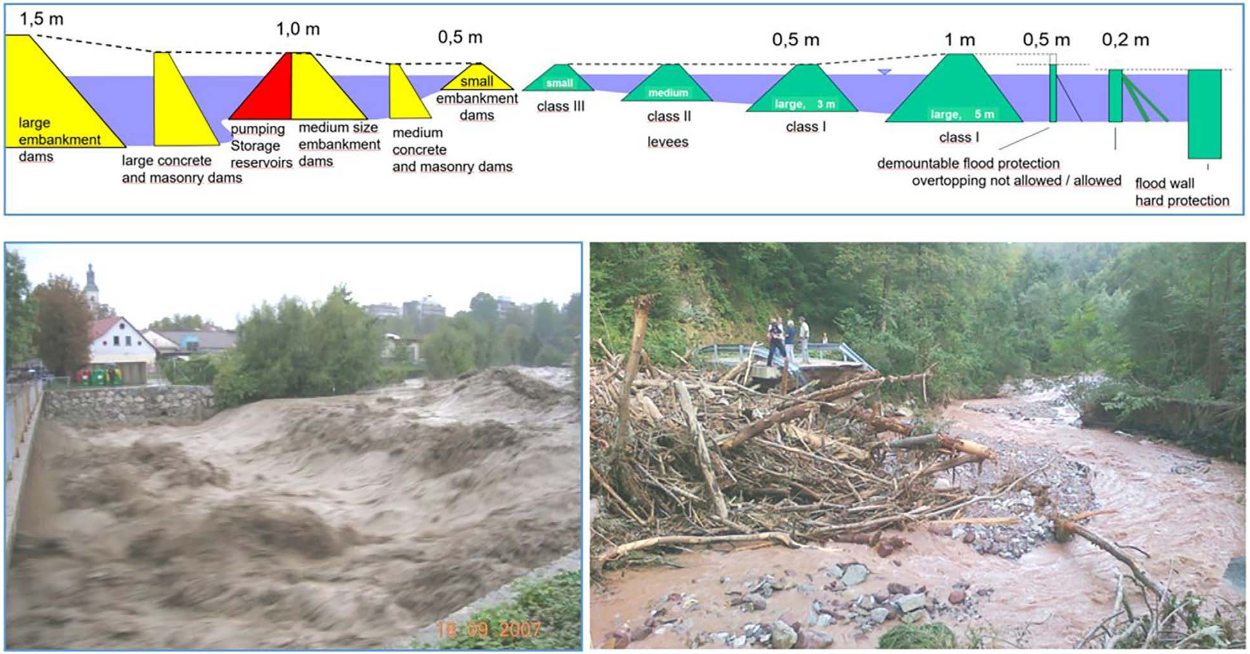
5.4 DESIGN WATER LEVELS

For effective planning of flood protection measures, the design water level must be defined along the entire flow path. As mentioned above, a design discharge can be determined based on the hydrological analysis (gauge observation series extrapolation or precipitation-runoff model). Settlements, industries and important infrastructure are often protected against a 100-year flood $(P \leq 0.01)$ according to the regulations and guidelines in many countries. Using the design discharge, the water level profile along the open channel is usually found by means of a hydro-numerical calculation. In very simple cases (approximately quasi-steady, uniform flow), at small streams the application of eq. (5.1) might be sufficient. For the majority of cases, a hydro-numerical model is recommended since the calculations can be done in this way more easily for larger river reaches and because these models use different computation methods that go beyond eq. (5.1). For valleys with clearly identifiable flow paths, a one-dimensional model (Figure 5.1) might be sufficient. For rivers in lowlands with unclear flow paths during floods, two-dimensional models are preferred (Figure 5.1). For very complicated flow situations, e.g., proximal to physical structures, abrupt flow changes, rapids or obstacles, three-dimensional hydro-numerical models can help to understand the flow pattern and to find the wetted perimeter of the channel (Figure 5.1). Furthermore, in the most critical examples, a physical model can be constructed to obtain required information for the optimal planning of structures such as outlets or gates (Bombač, 2012; Novak et al., 2016).

For the design of the bank, embankment or elevation of flood protection measures, a freeboard is usually designed to cope with waves (run-up height calculation), settlements and uncertainties. In some countries, a minimum freeboard allowance is given, whereas elsewhere only recommendations exist (Figure 5.5). These freeboard allowances are normally sufficient when the fetches (wind interaction length) are less than 100 metres, the water depth under five metres and the bank slopes lower than one in three (approx. 18°).

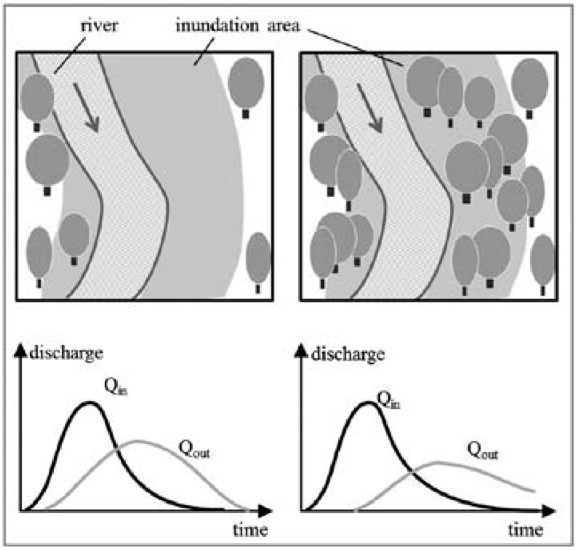
When a floodplain is covered with high or dense vegetation (resulting in a higher roughness coefficient, Mannings n), the downstream peak discharge can be reduced and the flood wave duration increased (i.e., compared to the upstream section) (Figure 5.6). Accordingly, water would remain longer within the inundation area and the flood water level may be higher (Gilli, 2010). Nevertheless, these effects have a lower impact on extreme and rare (i.e., catastrophic) floods than on smaller and more frequent floods. In addition, the roughness could be increased by the entrapment of large floating woody debris which is stuck between upright trees during a flood event. The floating (e.g., woody) debris can also enhance infrastructure damage (e.g., bridge openings or culverts) (Figure 5.5).

This relationship between floodplain roughness and flow hydrograph was confirmed by an experiment when modelling the flood control function of floodplain woodland in a 2.2 km-long reach of the River Perrett, United



Notes: The figures in the lower panel show a situation during a flood event (the left figure shows standing waves upstream of the bridge structure and was taken during the 2007 flood in Slovenia on the Sora river, while the right figure shows the destruction of a bridge due to the transport of woody debris during the 2014 flood in Slovenia on the Gradaščica river) where it is evident why consideration of freeboard is essential (i.e., processes and events can occur that cannot be predicted or prediction is very uncertain).

Figure 5.5 Proposed minimum freeboard allowance (upper panel)



Notes: Left figures: lower roughness. Right figures: higher roughness.
Source: Bornschein and Pohl (2018).

Figure 5.6 Flow hydrographs (Q_{out}) at the end of a river reach with the same inflow hydrograph Q_{in} but different land use

Kingdom (Nisbet, 2006). The setup of a 133 ha wet woodland within this area would increase the flood storage by 71 per cent and delay the flood peak arrival downstream by 140 min in case of a 100-year flood event. It is also noticeable that the influence of vegetation on flood propagation differs seasonally as vegetation in winter is often less dense and/or lower. Hydraulic models that were calibrated using a winter flood event should be modified before forecasting flood water levels for a summer flood event (Heyer et al., 2015).

5.5 FLOOD RISK MANAGEMENT CONCEPTS

Flood risk management concepts may have different objectives. One objective can be a lower peak water level. Another aim could be a later arrival of the flood wave. Sometimes upstream measures can be carried out to protect downstream people and property. In other cases, the measures can only be organized in the floodplain areas and not upstream. Sometimes the removal of downstream obstacles, blockages or sediments can help to avoid backwater effects (i.e., higher upstream water level).

From these issues, different measures and hydro-technical structures can be derived. As mentioned above, the retention capacity in the catchment area can be improved by rough terrain surfaces (crops, bushes, trees, forest). In addition, other measures include forming troughs and hollows in the landscape, by ploughing parallel to the elevation contours (and not downhill), by meandering streams and rivers and more groundwater recharge instead of surface runoff. These 'soft-engineered' structures use natural materials, but do involve design and construction. The more frequently deployed technical approach for regular flood protection is the construction of flood defences ('hard' engineering).

There are three main kinds of flood defences: levees, flood protection walls (so called 'hard' engineered structures) and demountable (i.e., temporary) elements. For individual building solutions, stop logs and plates are often used to seal doors and windows. Some examples are provided in Table 5.1 with possible objectives. Additional examples can be found in Chapter 8. Thus, it can be seen that specific measures in almost all cases can have some negative effect (e.g., negative effect of upstream levee on downstream flood conditions). Therefore, a holistic approach should be used when planning flood protection measures and planned measures should be evaluated from the upstream-downstream perspective (Rinnert et al., Chapter 8 in this volume). Furthermore, Figure 5.7 shows some examples of hard engineered flood protection measures.

5.6 FLOOD RISK AND RELIABILITY OF MEASURES

The objective of flood protection should be chosen in such a way that the overall financial benefit of the measure is greater than the investment. There are several guidelines and publications available about cost-benefit analysis in hydraulic engineering and water management (Dittrich et al., 2018; DVBU, 2008; DVWK-M10/1985; LAWA Leitlinien 1979; LAWA Grundzüge 1981; LTV Erstellung von Hochwasserschutzkonzepten, 2003). This approach can be applied using the flood risk management concept with the basic eq. (5.1) where the consequences are the (avoided) costs per year and the probability refers to an arbitrary year. However, this equation implicates the zero-times-infinity-problem with numerical instability for extreme values of both variables. Very rare events (i.e., catastrophic floods) with low probability normally cause very huge consequences and vice versa. When displaying the consequences (ordinate) as a function of the probability (abscissa) in a coordinate system (Figure 5.8), the area between the curve and the abscissa is the risk being represented by the integral over all possible events.

$$R = \int_0^1 C(P).dP \qquad (5.6)$$

Notes: From left top to right bottom: 1. Levee with flood wall at Jessnitz, Germany. 2. Flood wall under construction (2013) on the right Rhone bank in Arles, France. 3. Levee with flood wall and openings for demountable flood protection elements at the Elbe River in Dresden, Germany. 4. Leveed river with smoothed bed: Weisse Elster south of Leipzig, Germany. 5. Flood bypass cutting a meander of the river Elbe, Dresden, Germany. 6. Flood bypass using the artificial channel (i.e., Gruber channel that was constructed in 1780) that is used to protect the Ljubljana city centre.
Sources: First five photographs: R. Pohl. Final image: Atlas okolja (2021).

Figure 5.7 Six flood protection measures

Table 5.1 Examples of different hard-engineering flood protection objectives and measures with possible implications

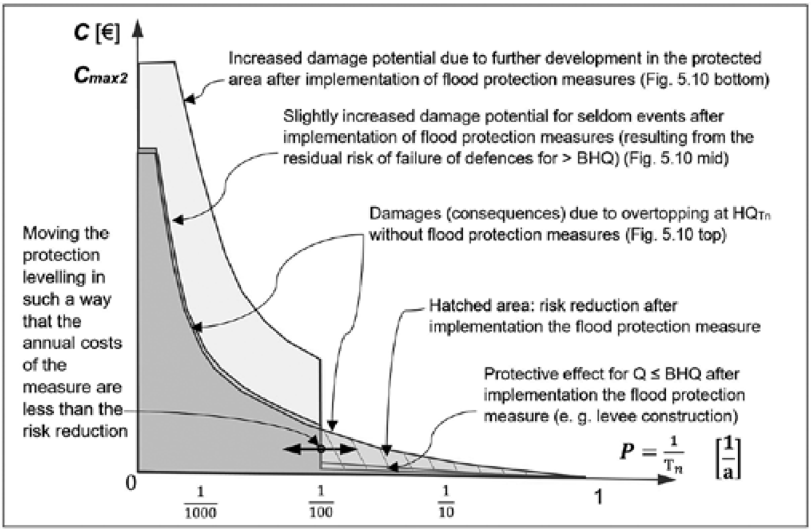
Objective	Measure	Issues and possible implications
Lower flood water level	Reducing channel roughness	Loss of natural river bed, erosion along the channel, earlier downstream arrival of flood peak
	Flood bypass cutting meanders	Increased hydraulic gradient, earlier downstream arrival of flood peak
	Removal of sediments	Disturbance of sediment balance, required bed load disposal, loss of aquatic habitat
	Deepen the river bed	Disturbance of sediment regime and groundwater flow, upstream erosion, downstream sedimentation, loss of aquatic habitat
	Upstream flood retention to cut the discharge peak	Upstream inundation or reservoir needed, large storage for relative small peak reduction downstream, inlet-, outlet structures for polders
Later flood peak arrival	Raise channel roughness	Higher water level, more upstream inundation
	Upstream flood retention	Upstream inundation or reservoir needed, outlet structures
Protection of people and properties	Structural flood defences (levees, walls, demountable flood protection elements	Structures in the landscape, reduction of retention area affecting downstream reach (i.e. higher water level downstream)
	Individual object protection, flood-adopted buildings	Flood-protected isles in an inundated area

Assuming full (100 per cent) protection against all events which occur more frequently than after periods of T_n years, the remaining risk could be reduced to the value:

$$R = \int_0^{1/Tn} C(P) \cdot dP \tag{5.7}$$

In case of additional building activities in the protected area after completing the flood risk management measures (Figure 5.10), the potential consequences would rise and therewith the risk. Furthermore, a higher risk than without protection might be thinkable, which can be read from the larger area below the revised curve in Figure 5.8. That such activities really occur, one can see in Figure 5.9.

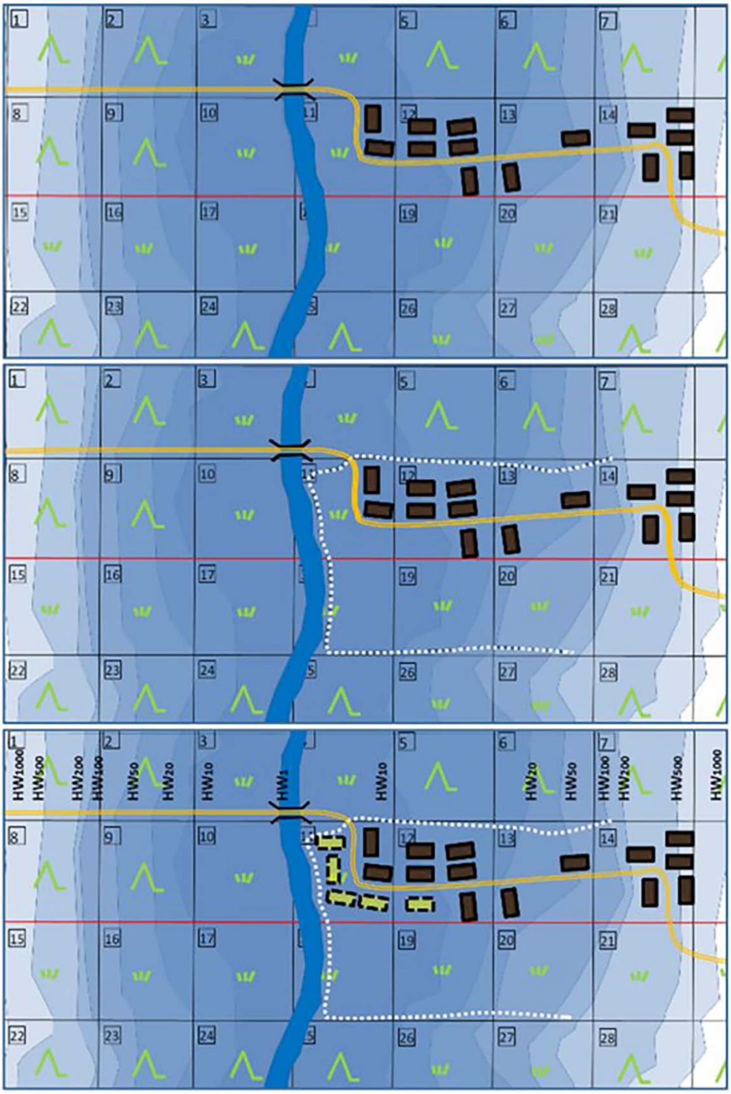
When the consequences of flood events can be expressed as monetary damage (e.g. €), the risk gets the unit €/a and represents the expected annual damage.



Notes: Schematic diagram not scaled: Q = discharge, C = consequences, costs, P = exceedance probability, BHQ = design flood, HQ = flood with a certain recurrence period.

Figure 5.8 *Risk of inundation of the area behind a flood defence compared with the situation without protection and with additional property after having built the defences*

Figure 5.9 *Real estate brokers are offering building lots directly behind the levee that has been refurbished after the hurricane Katrina 2005 (Highway 23, Belle Chasse, New Orleans, LA, USA)*



Notes: Originally without flood protection (top), then leveed (white dotted line, mid) and later further developed (light grey houses, bottom).

Figure 5.10 Example of a flood-prone housing development

5.7 EXAMPLE OF GOOD PRACTICE: DRY RETENTION BASINS

Dry retention basins are one of the examples of a water retention hydro-technical structure. In the case of this type of structure, the area inside the retention structure is flooded only during high-stage events. During low and mean-flow conditions, the area can be used for other purposes, e.g., agricultural production or recreational amenities (e.g., grass area or partly forested, although forest reduces the capacity). It should be noted that floods can have a negative effect on the soil properties if the area is used for agriculture (e.g., Glavan et al., 2020). Despite this, they are frequently used in Slovenia in order to improve flood safety. An example of such an object is the Prigorica dry-retention reservoir located on the Ribniščica river (southern part of Slovenia) in order to ensure flood safety of the nearby Ribnica settlement. The dry-reservoir was constructed more than 30 years ago and is in operation during high-flood conditions (Figure 5.11). The reservoir can retain up to 12×10^6 m^3 or, during catastrophic floods, up to around 15×10^6 m^3. When the reservoir is completely full, it covers an area of 270 ha. The hydro-technical elements include an embankment dam, outlet, gate and emergency spillway. During more than 30 years of operation, this basin has been in operation several times. However, it should be noted that regular maintenance is required to ensure optimal performance during high-flow events. Such measures are regarded as an example of good practice that combine elements of green (i.e., area inside basin can be used for agricultural production) and grey infrastructure and most importantly can be used to reduce the flood damage. However, as pointed out by Nester et al. (2017), the effect of such retention reservoirs decreases with scale. Thus, for large catchments the effect of multiple reservoirs can be small. For example, Nester et al. (2017) showed that use of 130 alpine retention measures with total volume of 21×10^6 m^3 located in the Inn river would only reduce flood peak by 2–3 per cent compared to the situation without these retention measures. Thus, it is clear that in such a case some other flood protection alternative should be used.

5.8 CONCLUSIONS

More space for flowing waters can help to reduce the flood peak and to delay the arrival of the flood peak downstream. Small measures can in some cases only affect small floods or have a local effect. Nevertheless, they can help to reduce the frequency of inundations downstream. The reduction depends on the scale. If a considerable cut of high flood peak is desired or needed, large storage volumes are required which should additionally be gated so that the

Notes: Upper two photos show the area inside the retention reservoir during high- (left) and low- (right) flow conditions. The lower image shows reservoir outlet flow during a high-flow event.
Source: Ribnica24 (2021).

Figure 5.11 Wet retention reservoir in operation

storage is not filled before the arrival of the flood peak. Therefore, some other measure could be more suitable in such cases.

Hydrological and hydraulic models can help to assess the effects of flood mitigation measures. Hydraulic models are applicable to extrapolate stage (i.e., water level)-discharge-curves for large floods that were not recorded in the past. Roughness coefficients like Manning's n or Strickler's k_{st} were originally introduced into 1D-flow calculations so that their application in 2D-hydro-numerical models needs experience and expert knowledge. That is why calibration and verification of hydraulic models are very important for the reliability of the modelling results. The same applies for the hydrological models where model calibration and evaluations should be done before further use of the models. Therefore, the modeller should also keep in mind the drawbacks and limitations of the model.

Often it is postulated that a couple of small protection measures is better than one large measure. As the economic and hydraulic efficiency depends on many factors, it cannot be said in general and without profound individual analysis whether one large or several smaller flood protection measures will bring the better effect. When speaking about nature-based solutions or non-structural methods, we must confess that also these projects need a lot of construction work at least during the phase of project implementation but in many cases also during their later lifetime (i.e., maintenance). Furthermore, these measures are structural measures too, including earthworks, excavation, reinforcement of embankments, building pathways and roads and in some cases also bridges, inlet/outlet structures, and flood defences. In the end, it is important that all flood risk management actions and protection measures should be evaluated from a cost-benefit perspective in order to ensure that public money is spent in an effective way to protect people, property and the environment.

ACKNOWLEDGEMENT

This research is an outcome of the COST Action No. CA16209 Natural flood retention on private land, LAND4FLOOD (www.land4flood.eu), supported by COST (European Cooperation in Science and Technology, www.cost.eu). Open access of this chapter is funded by LAND4FLOOD. N. Bezak would like to acknowledge the support of the Slovenian Research Agency through grant P2-0180 and his research activities were conducted in the scope of the UNESCO Chair on Water-related Disaster Risk Reduction.

REFERENCES

Acreman, M., Crook, S., Glenny C., Hewitt N., Lucas F., Packman J., Pepper A., et al. (2002). *Hydrological Impact Assessment: Modelling the Impacts of Floodplain Restoration*. The Royal Society for the Protection of Birds, Sandy, United Kingdom. http://www.floodplains.org/pdf/technical_reports/Hydrological%20Impact%20Assessment%20draft1.pdf.

Atlas okolja (2021). http://gis.arso.gov.si/atlasokolja/profile.aspx?id=Atlas_Okolja_AXL@Arso.

Bezak, N., Brilly, M., and Šraj, M. (2014). Comparison between the peaks-over-threshold method and the annual maximum method for flood frequency analysis. *Hydrological Sciences Journal*, 59(5), 959–977.

Bezak, N., Šraj, M., Rusjan, S., and Mikoš, M. (2018). Impact of the rainfall duration and temporal rainfall distribution defined using the Huff curves on the hydraulic flood modelling results. *Geosciences*, 8(2), 69. https://doi.org/10.3390/geosciences8020069.

Bloeschl, G. et al. (2019). Changing climate both increases and decreases European river floods. *Nature*, 573, 108–111.
Bloeschl, G., Sivapalan, M., Wagener, T., Viglione, A., and Savenije, H. (eds.) (2013). *Runoff Prediction in Ungauged Basins: Synthesis across Processes, Places and Scales*. Boca Raton, FL: CRC Press.
Bombač, M. (2012). Hydraulic research of the construction pit of HPP Brežice on a physical model. *Acta Hydrotechnica*, 25(42), 1–17. https://actahydrotechnica.fgg.uni-lj.si/paper/a42mb.pdf.
Bornschein, A. and Pohl, R. (2018). Land use influence on flood routing and retention from the viewpoint of hydromechanics. *Journal of Flood Risk Management*, 11(1), 6–14.
Bozkurt, S., Dekens, P., Gartland, R., Gragg, J., Lawyer, J., and McGoogan, M. (2000). Evaluation of setback levees on sacramento river. http://www.bren.ucsb.edu/research/2000Group_Projects/Levees/levees_final.pdf.
Chow, V. T. (1959). *Open Channel Hydraulics*. New York: McGraw-Hill.
Clark, C. (2006). Planning for floods: Will we ever learn? *International Water Power & Dam Construction*, 58(6), 20–27.
Desai, B., Maskrey, A., Peduzzi, P., De Bono, A., and Herold, C. (2015). *Making Development Sustainable: The Future of Disaster Risk Management*. Global Assessment Report on Disaster Risk Reduction. http://archive-ouverte.unige.ch/unige:78299.
Dittrich, R., Ball, T., Wreford, A., Moran, D., and Spray, C. J. (2018). A cost-benefit analysis of afforestation as a climate change adaptation measure to reduce flood risk. *Journal of Flood Risk Management*, 12, e12482.
Dolšak, D., Bezak, N., and Šraj, M. (2016). Temporal characteristics of rainfall events under three climate types in Slovenia. *Journal of Hydrology*, 541(B), 1395–1405.
DVBU (2008). Dritte Rhonekorrektion – Sicherheit für die Zukunft. Synthesebericht des Generellen Projektes, Kanton Wallis, Departement für Verkehr, Bau und Umwelt, Sitten, Switzerland. https://www.vs.ch/web/rhone.
DVWK Merkblatt (10/1985). Ökonomische Bewertung von Hochwasserschutzwirkungen.
Gilli, S. (2010). *Die Wirkung von Flussaufweitungen auf Hochwasserwellen – Parameterstudie einer Deichrückverlegung im Flussmittellauf*. Wasserbauliche Mitteilungen, Technische Universität Dresden, Germany, Heft 43.
Glavan, M., Cvejić, R., Zupanc, V., Knapič, M., and Pintar, M. (2020). Agricultural production and flood control dry detention reservoirs: Example from Lower Savinja Valley, Slovenia. *Environmental Science & Policy*, 114, 394–402.
Graf, W. H. (1998). *Fluvial Hydraulics: Flow and Transport Processes in Channels of Simple Geometry*. Hoboken, NJ: Wiley.
Han, D. (2011). *Flood Risk Assessment and Management*. Bristol: Betham Science Publisher, Kindle Edition.
Hartmann, T., Slavíková, L., and McCarthy, S. (eds.) (2019). *Nature-Based Flood Risk Management on Private Land: Disciplinary Perspectives on a Multidisciplinary Challenge*. Cham: Springer.
Heyer, T., Scholz, R., and Goreczka, F. (2015). Die Aktualisierung der 2D-HN-Modelle der Saale bei Halle und der Elbe bei Magdeburg auf Grundlage von Messwerten des Hochwassers 2013, Heft 53. Dresden: Wasserbauliche Mitteilungen, Technische Universität Dresden, 119–130.

LaRocque, A. (2013). Probable Maximum Flood (PMF). In P. T. Bobrowsky (ed.), *Encyclopedia of Natural Hazards*. Dordrecht: Springer. https://doi.org/10.1007/978-1-4020-4399-4_276.

LAWA Grundzüge der Kosten-Nutzen-Untersuchungen.- Länderarbeitsgemeinschaft Wasser, Bremen, 1981.

LAWA Leitlinien zur Durchführung von Kosten-Nutzen-Analysen.- Länderarbeitsgemeinschaft Wasser, Stuttgart, 1979.

LTV Erstellung von Hochwasserschutzkonzepten.- Landestalsperrenverwaltung des Freistaates Sachsen, Pirna, 2003.

Meylan, P., Favre, A. C., and Musy, A. (2012). *Predictive Hydrology: A Frequency Analysis Approach*. Boca Raton, FL: CRC Press.

Nester, T., Komma, J., Salinas, J. L., and Boeschl, G. (2017). Monte Carlo simulations to evaluate the potential of alpine retention measures. XXVII Conference of the Danubian countries on hydrological forecasting and hydrological bases of water management. https://www.danubeconference2017.org/.

Nisbet, T. (2006). *Interactions between Floodplain Woodland and the Freshwater Environment. Annual Report and Accounts 2004–2005*. Farnham: Forest Research.

Novak, G., Mlačnik, J., Bombač, M., and Vošnjak, S. (2016). Hydrodynamic forces during the operation of a model radial gate. *Journal of Applied Water Engineering and Research*, 5(1), 70–77.

Pohl, R. (2012). *Applied Hydromechanics*. Lecture notes. Technische Universität Dresden.

Pohl, R. (2019). Flood retention in urban floodplains: A Plzen case study from the viewpoint of a hydraulic engineer. In T. Hartmann, L. Slavíková, and S. McCarthy (eds.), *Nature-Based Flood Risk Management on Private Land: Disciplinary Perspectives on a Multidisciplinary Challenge* (pp. 135–138). Cham: Springer.

Pohl, R. (2020). Quantifying resilience in hydraulic engineering: Floods, flood records, and resilience in urban areas. *WIREs Water*. https://doi.org/10.1002/wat2.1431.

Popp-Walser, C. (2013). Development of flood risk considering vulnerability and resilience: The example of the Rhone in Valais from 1850 to 2015. Diplomarbeit, Institute of Hydraulic Engineering and Applied Hydromechanics, Technische Universität Dresden (in German).

Ribnica24 (2021). https://www.ribnica24.eu/.

Schanze, J. (2017). Nature-based solutions in flood risk management: Buzzword or innovation? *Journal of Flood Risk Management*, 10(3), 281–282.

Sezen, C., Bezak, N., Bai, Y., and Šraj, M. (2019). Hydrological modelling of karst catchment using lumped conceptual and data mining models. *Journal of Hydrology*, 576, 98–110.

Simm, J., Sharp, M., Hemert, H., Igigabel, M., Pohl, R., Tourment, R., Wallis, M., Drake, C., Banks, J., Bramley, M., Chassé, P., Conforti, T., Jenkins, O., Mallet, T., Matheu, E., Royet, P., and Verigin, S. (Editorial Steering Board) (2013). *The International Levee Handbook*. London: CIRIA.

Winsemius, H. C., Aerts, J., van Beek, L., et al. (2016). Global drivers of future river flood risk. *Naure Climate Change*, 6, 381–385.