Freeboard analysis in river engineering and flood mapping – new recommendations

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ABSTRACT: The Swiss Commission for Flood Protection elaborated a unified concept to determine the freeboard in order to evaluate the discharge capacity of a stream. It consists of several partial freeboards. On one hand, the partial freeboards take the uncertainty of the calculation of the water level into consideration. On the other hand, they consider hydraulic processes like the formation of waves, the backwater effect at obstructions or the additional space needed to convey floating debris underneath bridges. The concept includes recommendations on how to account for freeboard in flood mapping. With its concept paper on freeboard, the Swiss Commission for Flood Protection wishes that the freeboard will be taken into account in flood mapping and hydraulic design in Switzerland on an unified and coherent basis.

1 INTRODUCTION

Design procedures for flood protection structures or the evaluation of the discharge capacity of a stream usually consider a freeboard. However, in Switzerland no best practice has been established so far. Some practitioners use a constant value (e.g. 1 m) others set the required freeboard equal to the velocity head of the current. Furthermore, different criteria and different approaches are being used in design procedures and hazard evaluations (KOHS, 2012). This multitude of approaches seeds doubts among design engineers and authorities about the use of the ”correct” freeboard and makes it difficult to compare different hazard analysis and flood protection projects. In order to overcome this weakness, the Swiss Commission for Flood Protection elaborated a unified concept for the determination of the freeboard. The concept is applicable to river courses. The freeboard requirements for dams and reservoirs in Switzerland are defined in the respective guidelines (BWG, 2001). The present publication is an extract of the recommendation published by KOHS (2013a) in German and KOHS (2013b) in French.

2 THE FREEBOARD CONCEPT

2.1 *Definitions*

The freeboard f denotes the vertical distance between the water level and the top edge of the bank or a hydraulic structure (Fig. 1) or the bottom edge of a bridge (Fig. 2). The water level can be observed or calculated. The required freeboard f_r denotes the freeboard that is necessary to guarantee a calculated discharge capacity.

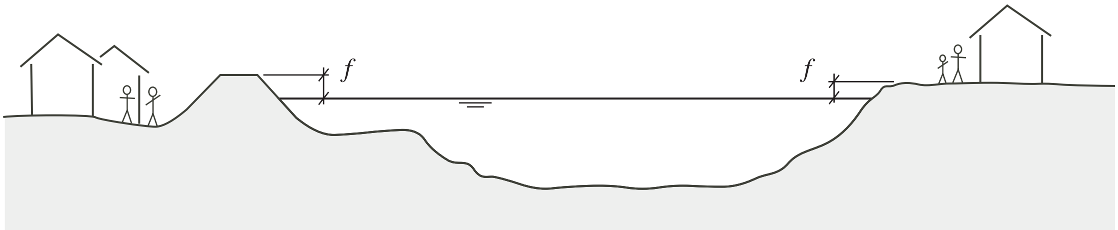

Figure 1. The freeboard *f* denotes the vertical distance between the water level and the top edge of the bank.

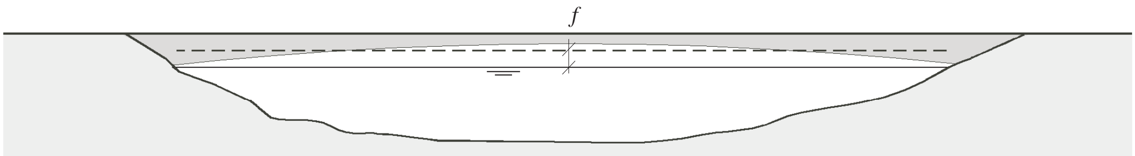

Figure 2. The freeboard *f* denotes the vertical clearance between the water level and the mean bottom edge of a bridge.

2.2 *Uncertainty, wave formation and back water effects*

On one hand, the required freeboard is understood as a parameter that describes the uncertainty in the calculation of the water level for a given cross section geometry. On the other hand it considers wave formation or local backwater effects that are not necessarily included in hydraulic calculations. The required freeboard is therefore conceived as a *hydraulic* parameter. It should not be used to cover uncertainties in hydrologic peak flood estimation nor to justify an elevated flood protection objective for high damage potentials.

The required freeboard covers uncertainties in the calculation of the water level that have their origin in the uncertainties of

- the measured cross section geometry,
- the calculated bed level during peak discharges,
- the determination of the channel roughness and
- the determination of the effective channel geometry in presence of growing vegetation.

The above-mentioned uncertainties must be displayed as a result of the hydraulic calculations. They should not be replaced by applying conservative values for channel roughness or the channel geometry.

The required freeboard is used to cover the following processes and it ensures that the discharge capacity is not exceeded despite of these phenomena:

- Waves that are formed by the current (namely at flow conditions near to critical flow).
- Drift wood and drift ice.
- Back water effects at local obstacles (e.g. trees or overhanging corners of walls).

Sediment deposits at the channel bottom during floods, the banking of the water level in bends or the accumulation of drift wood and drift ice at bridge piers and abutments raise the water level. These effects must be considered when calculating the water level. They may not be regarded as effects covered by the freeboard.

2.3 *Load, impact and capacity*

In a given river course, the discharge Q and the supply of bed load S_b, drift wood and other floating debris F_d may be considered as loads (Fig. 3). As impact parameters the water level z_w, the flow velocity U and a parameter d above the water level can be defined. The latter describes the space occupied by floating debris. The water level is a result of the (changing) bed level z_b and the flow depth h. The cross section area A, the channel slope S, its roughness k and the freeboard f determine the capacity of the cross section to convey the load parameters.

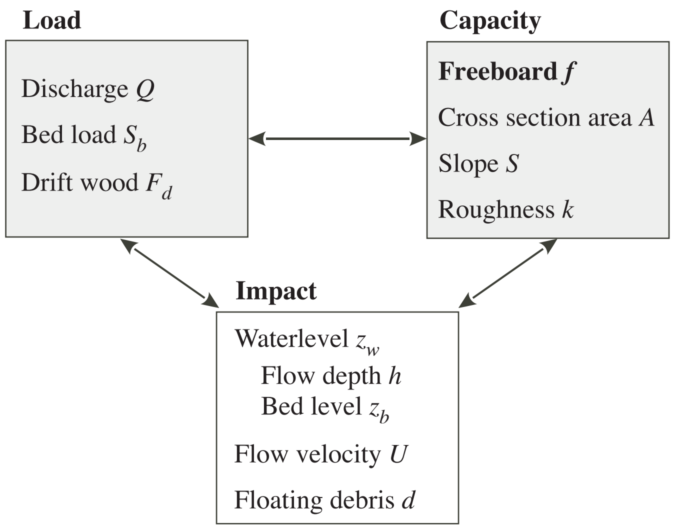

Figure 3. The system of load, impact and capacity sets the framework of the freeboard concept.

3 CALCULATION OF THE REQUIRED FREEBOARD

3.1 *Partial freeboards*

The required freeboard consists of three partial freeboards that are added geometrically. Each of the partial freeboards takes into account one of the above-mentioned impacts parameters.

$$f_{min} \leq f_r = \sqrt{f_w^2 + f_U^2 + f_d^2} \leq f_{max} \tag{1}$$

where f_{min} = minimal required freeboard, f_{max} = maximum required freeboard, f_w = required freeboard due to uncertainties in the calculation of the water level, f_U = required freeboard due to wave formation and back water effects caused by local obstacles and f_d = required freeboard due to additional space needed to convey drifting debris underneath bridges.

3.2 *Freeboard due to uncertainties in the calculation of the water level*

The partial freeboard f_w is set equal to the uncertainty of the water level calculation σ_w

$$f_w = \sigma_w = \sqrt{\sigma_{wb}^2 + \sigma_{wh}^2} \tag{2}$$

This uncertainty has two reasons (Fig. 4): first, the estimated bed level z_b during peak discharges may have an error and this error affects the water level calculation (σ_{wb}), second, the calculation of the flow depth h above the bed level may be imprecise because the cross section geometry may not represent the channel geometry properly or the roughness coefficients may be badly estimated (σ_{wh}). Both errors are added geometrically.

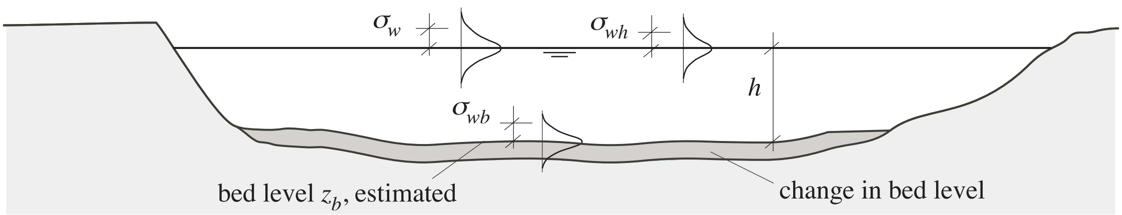

Figure 4. Uncertainty in water level calculation. The flow depth h is calculated on an estimated bed level z_b, that varies with discharge and time. Both the estimation of the bed level and the flow depth calculation have errors.

In Figure 4 σ_{wb} marks the error of the bed level estimation. It is set equal to its contribution to the water level calculation. The value of σ_{wb} must be estimated case by case. Values between 0.1 m in large low land rivers and 1.0 m in torrents are typical.

In order to determine the uncertainties σ_{wh}, flow depth was calculated in 18 rivers in Switzerland using the Manning-Strickler-formula and assuming errors of the input parameters as follows: channel width ±10 % (max. ±1 m); measured bed level ±0.1 m, roughness coefficient ±10 %, longitudinal slope ±10 %, bank slope ±3°. The errors of the independent input parameters were propagated to the dependent variable flow depth. In total 52 flow depths at different discharges were calculated. In Figure 5 a strong dependency of the uncertainty from the flow depth itself can be observed. The error σ_{wh} of the water level calculation caused by uncertainties of the flow depth calculation can therefore be estimated using the following equation:

$$\sigma_{wh} = 0.06 + 0.06\,h \tag{3}$$

Given certain circumstances it might be worth calculating σ_{wh} case by case using an estimation of the errors of the input parameters instead of applying equation 3.

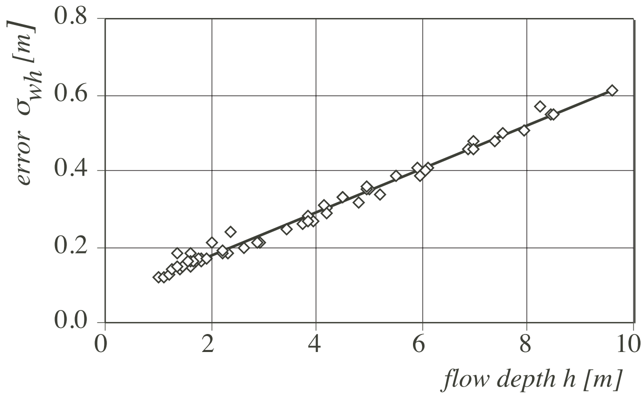

Figure 5. Error σ_{wh} of the flow depth calculation depending on the flow depth h.

3.3 *Freeboard due to wave formation and backwater effects*

Obstacles in the cross section (e.g. bridge piers, abutments, overhanging corners of walls) have a local backwater effect. The water level may rise to the level of the energy line. It is therefore in maximum $U^2/2g$ above the water level.

At flow conditions close to critical, waves appear where flow velocity is at maximum. That is in straight channels in the middle of the cross section. The wave crest lies at most to the extent of $U^2/2g$ above the mean water level.

The partial freeboard due to wave formation and backwater effects is therefore given by

$$f_U = \frac{U^2}{2g} \tag{4}$$

where U = flow velocity and g = acceleration due to gravity.

3.4 *Freeboard due to additional space needed underneath bridges*

Flow underneath a bridge needs additional space to convey floating debris (drift wood, drift ice etc.) without clogging. In order to determine the partial freeboard f_d for wooden debris a system of classes is proposed. f_d has a value between 0.3 m and 1.0 m and depends on the characteristics of the drift wood and on the construction type of the bridge (Tab. 1). In rivers with other floating debris than wood (e.g. drift ice), f_d must be defined accordingly.

Table 1. Partial freeboard f_d for wooden debris.

	f_d at bridges with a smooth bottom view m	f_d at bridges with a rough bottom view m
Small wooden debris (branches only)	0.3	0.5
Tree trunks, drifting individually	0.5	1.0
Rootstocks	1.0	1.0
Tree trunks, drifting as a carpet	1.0	1.0

3.5 *Selection of partial freeboards*

The calculation of the required freeboard can be adapted to the river reach of interest by selecting the relevant partial freeboards f_w, f_U and f_d. According to the given situation one or two of the partial freeboards can be set to zero. The criteria given in Table 2 should be applied.

Table 2. Criteria to apply the partial freeboards.

Partial freeboard	Criteria
f_w	In all river reaches
f_U	At bridge cross sections; In reaches with flood protection dikes or walls that may collapse as they are overtopped; In reaches where slopping the banks results in a considerable water outlet; On alluvial fans; In paved torrent trenches.
f_d	At bridge cross sections where floating debris are relevant.

3.6 *Minimum and maximum required freeboard*

The required freeboard should be calculated cross-section-by-cross-section and should be unified along river reaches. A minimum value of the required freeboard of 0.3 m should be used. This gives more weight to the uncertainty of the calculated water level in small, slowly flowing rivers. A maximum value of the required freeboard prevents unrealistic high values. In watercourses with fluvial bed load transport, a maximum value of 1.5 m is proposed. In torrents with potential debris flow, the maximum value could be higher.

4 EFFECTS OF THE EXCESS OF CONVEYANCE

4.1 *Freeboard and weak point analysis*

The freeboard is used to determine weak points in the framework of the design of flood protection measures or in the framework of a hazard analysis. The weak point analysis gives answers to the following questions:

Where does water overtop the banks at a given discharge?
Why does overbank flow occur?
Which amount of water overflows?

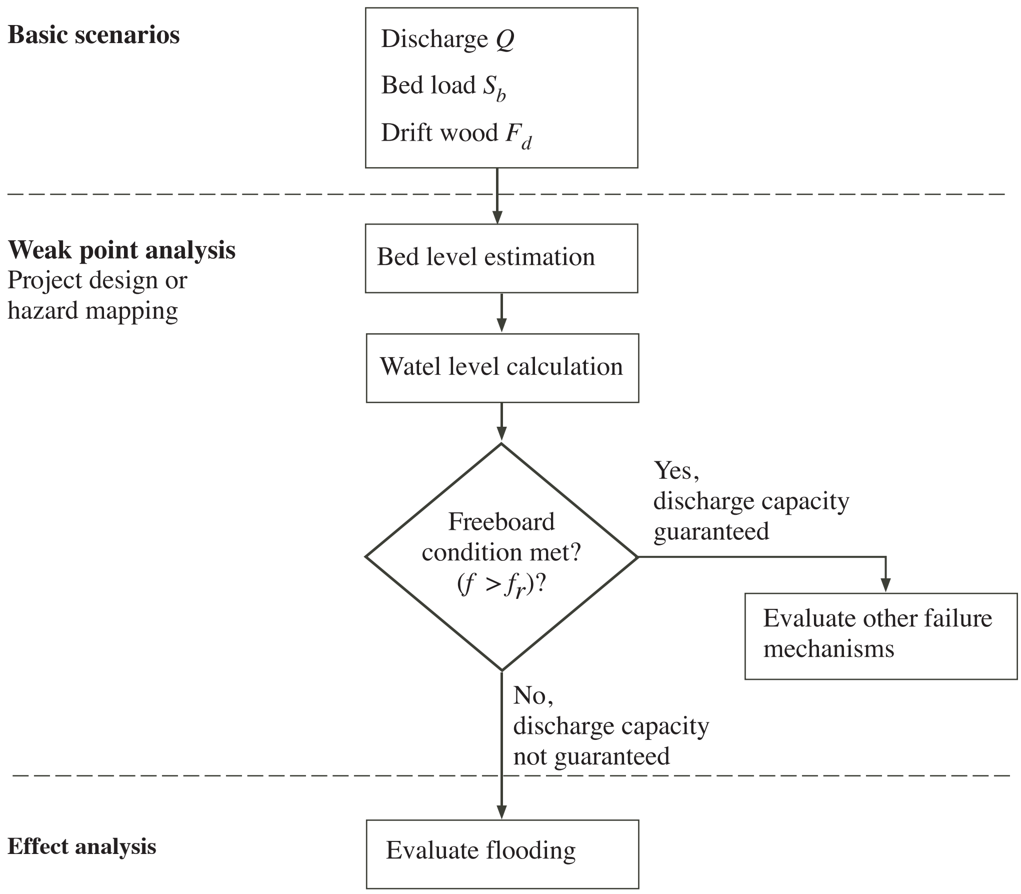

Figure 6. The freeboard calculation is part of the weak point analysis.

The required freeboard f_r in a river reach is determined for a given discharge Q, bed load S_b and load of floating debris F_d. It does not matter whether the discharge Q corresponds to the design discharge of a hydraulic structure or the overload discharge of a flood protection measure or whether it corresponds to a flood scenario with a given return period.

If the calculated water level z_w (calculated with discharge Q) results in a freeboard f that is larger than the required freeboard f_r, the capacity of the river reach is sufficient to convey the loads without overbank flow. Using the terminology of hazard assessments the considered river reach is no weak point. No flooding is expected. Other types of failures must eventually be considered.

If the calculated water level z_w (calculated with discharge Q) results in a freeboard f that is lower than the required freeboard f_r, the capacity of the river reach is insufficient to convey the discharge loads without overbank flow. Using the terminology of hazard assessments the river reach in consideration is a weak point and flooding may occur.

4.2 *Effects in river courses with overflow resistant banks*

If the discharge capacity is considered insufficient in a river course that is delimited by naturally grown terrain or by a dike or wall that remains stable even if it is overtopped, an overbank flow scenario as a function of the water level can be defined. The relevant water level z_w' corresponds to the calculated water level including its error (Equation 5):

$$z_w' = z_w + \sigma_w = z_w + \sqrt{\sigma_{wb}^2 + \sigma_{wh}^2} \tag{5}$$

This approach allows defining a flooding scenario whenever the discharge capacity is considered insufficient (Figure 7, to the right). An alternative approach that is often used in flood mapping using 2d-simulations defines the calculated (best estimate) water level z_w as the relevant water level. However, as shown in the example of Figure 8 this approach would neglect overbank flow although the discharge capacity is declared insufficient because the freeboard condition is not fulfilled.

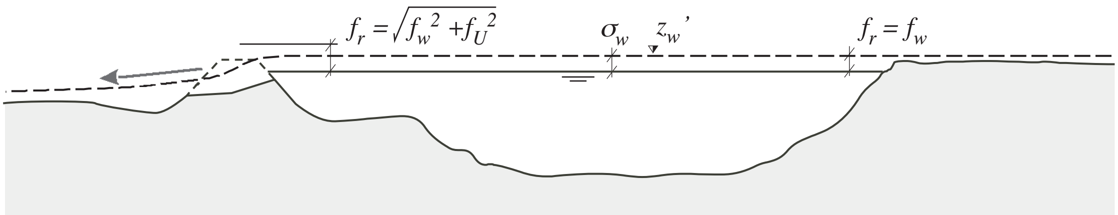

Figure 7. Flooding scenario in case of excess of discharge capacity ($f < f_r$). To the left: dike break. To the right: flooding in case of a bank resistant to overflow. The required freeboard f_r along the dike (left) differs from that along the naturally grown bank (right) according to Table 2.

4.3 *Effects in river courses with banks not resistant to overflow*

If the discharge capacity is considered insufficient in a river course that is delimited by a dike or a wall that does not resist overtopping, a failure scenario for the dike or wall (Figure 7 to the left) is defined. In order to calculate the outflow the water level, z_w' according to equation 5 or the best estimate of the water level z_w may be used. Usually, the outflow depends rather on the size of the dike breech, at the time of collapse or on sediment deposit in the channel than on the selection of the relevant water level.

5 CONCLUSIONS

The presented method for the determination of the required freeboard has to be considered as a recommendation by the Commission for Flood Protection of the Swiss Association for Water Management. It addresses hydraulic engineers of the private sector and of authorities. The approaches have been developed in an effort to be transparent, coherent and generally applicable. Nevertheless, the engineer is encouraged to adapt the method to the specific conditions of the river under consideration and hence to improve the approach.

The freeboard calculation is one element of the assessment of flood hazards or the design of hydraulic structures. Other elements like the definition of flood protection objectives, the definition of design scenarios or the concept to deal with overload scenarios must be examined separately.

6 ACKNOWLEDGEMENTS

The following members of the Commission for Flood Protection elaborated the recommendation on the freeboard: Lukas Hunzinger, Martin Jäggi, Jean-Pierre Jordan, Jürg Speerli, Heinz Weiss and Benno Zarn. The Swiss Office for Environment funded the elaboration process.

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