

ANALYSIS OF TORRENT PROTECTIVE STRUCTURES AS A BASIC ELEMENT OF THE HAZARD MAPPING PROCESS

DETERMINING THE AS-IS-STATE OF 100 YEARS OLD CHECK DAMS MADE OF NATURAL STONE MASONRY

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ABSTRACT

The natural hazard map for the villages of Schwanden, Hofstetten, Brienz and Brienzwiler (Bernese Alps, Switzerland) had to be revised as a consequence of large debris flows and flooding in August 2005, which caused huge damage. The more than 20 torrent control structures in the Lammbach gorge, constructed approximately 100 years ago, play a vital role for today's safety on the fan of the Lammbach with extended housing estate and small-scale industry. This paper presents the general hazard mapping process and main results of the assessment and rating for the as-is-state of the old check dams. For the visual inspection of the natural stone masonry of the check dams the "Lucerne Rating System" has been used. Simple 2D-models as well as 3D-FEM have been applied to check structural safety as well as effectiveness of the structures. The results of structure assessments have been directly integrated into the hazard mapping process.

Keywords: hazard, risk assessment, natural hazard map, torrent control structures, potential failure mechanism, natural stone masonry, as-is-state of structures, rating, 3D-FEM

INTRODUCTION

The torrents in the area of the villages Brienz, Schwanden and Hofstetten in the Bernese Alps, Switzerland (Fig. 1b), are well known for flooding and debris flows for hundreds of years.

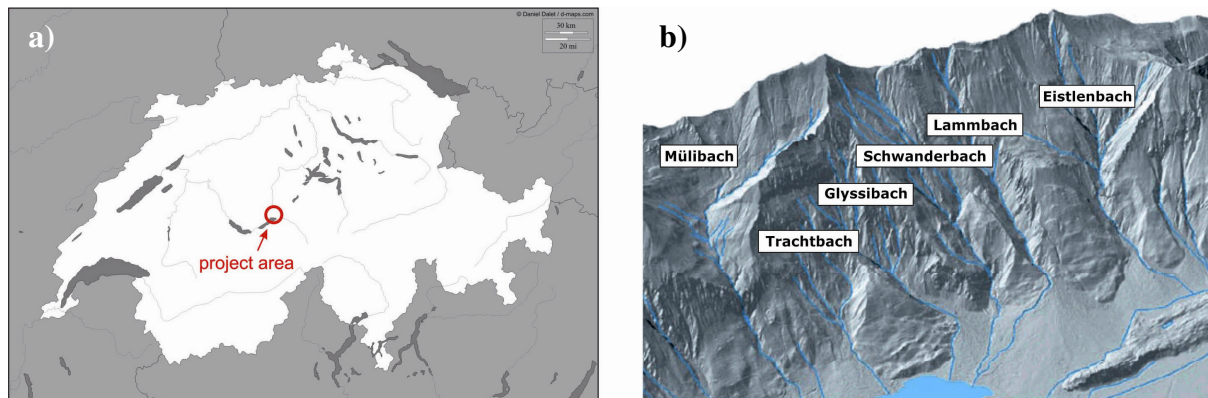


Fig. 1 a) geographical position of the project area, located north of the Lake Brienz, b) the mountain torrents of the Brienz area located at the southern slope of the Brienzgrat (Ryter, 2004).

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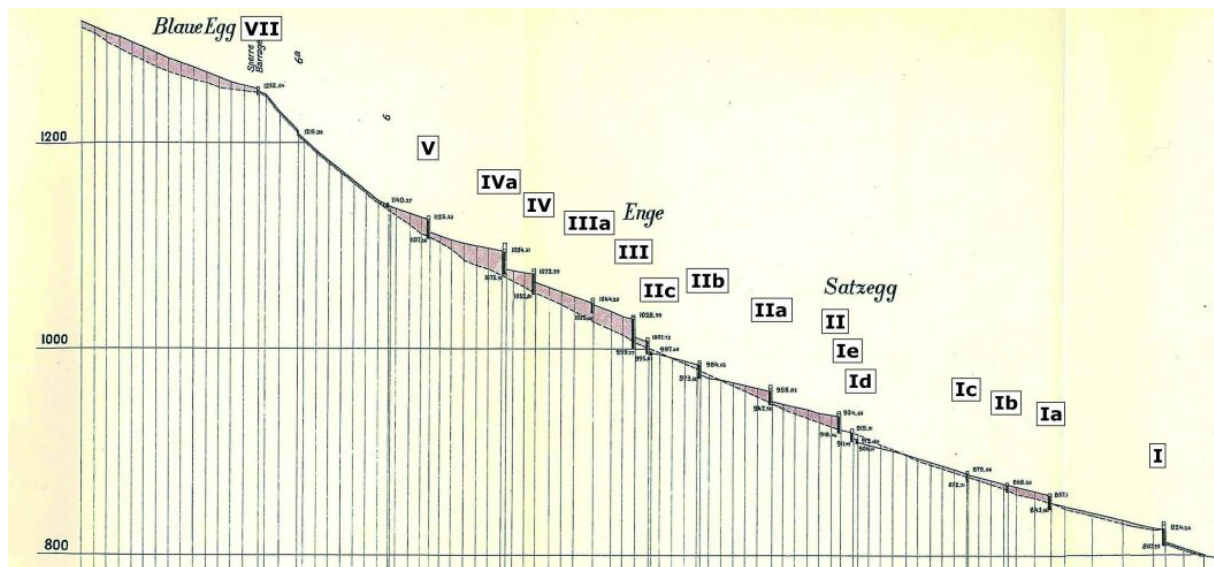


Fig. 2 Longitudinal section of the main part of the Lammbach gorge showing position and mode of action of the check dams, cut-out of a plan dated 1913 (OIK)

Because of this, a risk assessment for that area had been done already in the period 1997 - 1998. But due to large debris flows and flooding in August 2005 as well as the new discussion to take into account the impact of protection measures (PLANAT, 2008), the natural hazard map of that area had to be revised. In that revision process the Lammbach played a decisive role, because of the large uncertainties concerning the options of scenarios the hazard map is based on. Those large uncertainties traced back to the unknown as-is-state of the check dams which had been established in the Lammbach gorge approximately 100 years ago.

The Lammbach was not active during the 2005 events, but during the past centuries there have been a lot of examples for extremely dangerous debris flows and flooding. For example in 1499 the village Kienholz and the manor house Kien had been destroyed by large debris flows and debris and mud had been deposited up to a thickness of 10 m. The last large event occurred in the year 1896, when disastrous debris flows emerged in the Lammbach gorge which destroyed several houses in the village Kienholz, along with the track of the Swiss Federal Railway. The volumes of the last three incidents have been reported as approximately 300'000 m³ of material covering land along a shoreline length of 120 m at Lake Brienz with an average debris thickness of about 2.5 to 4.0 m. In consequence of those disastrous debris flows, a massive reforestation program had been started in the area and 20 torrent control structures (check dams) had been installed in the Lammbach gorge in the period 1896 to 1913 to raise the streambed and stabilize the slopes (Fig. 2). These barriers consist of natural stone masonry and have remarkable dimensions in part. The largest one, check dam IVa, has a span of 90 m and the visible height at the downstream face is still 13 m today (Fig. 5).

Since the construction of those check dams no disastrous debris flow with origin at the Lammbach valley has ever reached the villages. But on the other hand up to now by guess more than 500'000 m³ of debris has been accumulated behind the torrent control structures and at least the same quantity of debris are deposited at the slopes of the Lammbach gorge. For this reason there is a potential for large debris flows, especially if one or more of the check dams will fail to work. The structural safety of the old torrent control structures is therefore of utmost importance. An essential part of the revision process of the natural hazard map was therefore the assessment and rating of the old check dams made of natural stone masonry with regard to their structural safety and fitness for purpose.

TOPOGRAPHY AND GEOLOGY OF THE PROJECT AREA

The watershed of the Lammbach is placed at the southern slope of the Brienzgrat (Fig. 1b) and covers an area of approximately 3.2 km². The area ranges from approx. 2'200 m altitude down to the Lake Brienz at 578 m altitude.

The Brienergrat consists of the cretaceous rocks of the so-called Wildhorndecke. At the ridge siliceous limestone appear. Downhill the limestone is replaced by the marly layers of the Valangien. The dark siliceous limestone is normally stratulated and alternates with layers of marl. Compression, minor folds and buckling is very common in this geologic structure (Bauer, 1971). The dip direction (DD) of the bedding plane in the project area is SE and the dip (D) is 25° to 35°. Three main joint sets K1, K2 and K3 have been identified (Fig. 3, Tab. 1).

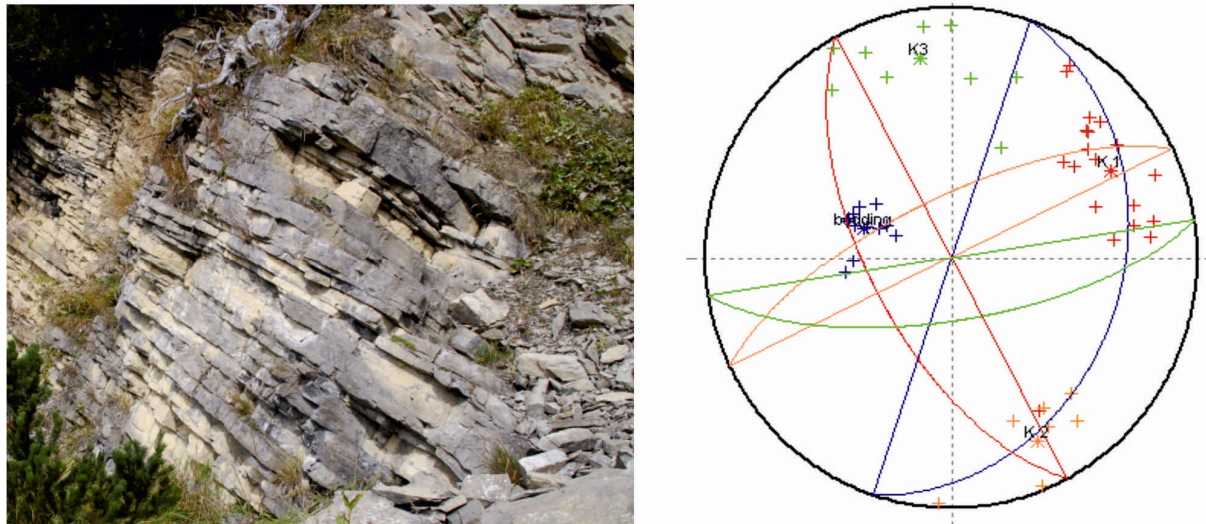


Fig. 3 Left: Example of thin to medium bedded limestone in the project area, right: Distribution of measured joints and great circles of joint sets

Tab. 1 Dip Direction (DD) and Dip (D) of the three main joint sets in the project area

Joint set	DD	D
K1	242°	63°
K2	334°	70°
K3	171°	69°

The bedding planes have slightly rough surfaces while the joint surfaces can be described as rough. The Geological Strength Index (GSI) is in the range of 35 to 45. The rock mass at the Lammbach valley is in general vulnerable for weathering and erosion. Due to the existing joint system the limestone degrades into cubical or block-shaped pieces with an edge length of 1 to 4 decimeters in the majority of cases. The marly parts of the rock mass decays into fine grained sediments. At the surface the fine-grained material is often eroded and only the blocky material is left. Therefore along the streambed and at the slopes as well as with depth the composition of the debris varies.

IDENTIFICATION AND ASSESSMENT OF HAZARDS

Since more than 10 years the hazard mapping process in Switzerland has been done in a standardized way consisting of 3 individual steps (BWW, BRP & BUWAL, 1997):

1. identification of hazards,
2. assessment of hazards and
3. transfer of scenarios into a hazard map.

The identification and assessment of hazards is in general in the own field of action of the municipalities and the natural hazard map is part of the urban development. Normally the municipalities place an order to a consultancy firm for doing the executive work. The working process is attended by the cantonal specialist departments.

For the identification of hazards in a first step existing event registers are used. In Switzerland those inventories exist at many places and they contain information about events occurred in the last decades, sometimes in the last centuries. Especially events concerning torrents are very often well documented. Additional information about potential loss events will be given by the geologic-geomorphological analysis of the project area.

The essential parameters of a hazard process are the intensity and the return period. In Switzerland, a hazard rating system is used, which defines different danger levels according to a combination of the hazard probability (defined with a return period or frequency of occurrence) and the hazard intensity. The hazard intensity is strongly linked to the magnitude of an event. In the Swiss recommendations (BWW, BRP & BUWAL, 1997), the intensity for debris flows and related processes is expressed as a function of impact pressure, flow velocity and/or flow depth. As flow depth is difficult to determine in some cases, an estimated depth of deposition may be taken as an approximation. The relevant return periods are defined as 30, 100 and 300 years. Additionally an extraordinary event will be taken into account in the assessment of hazards.

During the hazard assessment process a detailed geomorphological analysis will be done including information about potential flow paths, flow velocities, range of coverage and transported volumes of material. Empirical methods as well as numerical simulations are used for that work. Nevertheless the uncertainties are high in the assessment of processes dealing with torrents (Zimmermann, 2006).

GEOMORPHOLOGICAL ANALYSIS

During the geomorphological analysis the circumstances in the catchment area of the Lammbach had been studied in detail. Especially slope stability, potential processes like debris flow, bedload transport and the concurrence of slope instabilities together with torrent processes have been examined.

As already mentioned before, today a large quantity of debris is deposited behind the check dams and along the slopes of the Lammbach gorge due to the geologic circumstances and weathering. All in all a total quantity of approximately 1.5 million m³ of erodible debris is deposited here. Especially in the middle part of the Lammbach gorge there is a chance that several 10'000 m³ of debris may move downward into the streambed by slope failure. This may result in an afflux, which may on his part be the origin of a large debris flow similar to those one of the Lammbach cataclysm of the year 1896.

A comparison of the gradient along the Lammbach gorge at different times shows also significant changes. Fig. 4 shows on the left that the gradient was significantly reduced as a result of the construction of the check dams in the years 1896 – 1913. This was one of the aims which had been connected with the construction of the check dams. But today due to sedimentation of debris along the Lammbach gorge the gradient is in many cases similar to that one before the construction of the check dams. In the case of the upstream area of check dam IVa the gradient has been even increased in comparison to the situation before dam construction (Fig. 4, right).

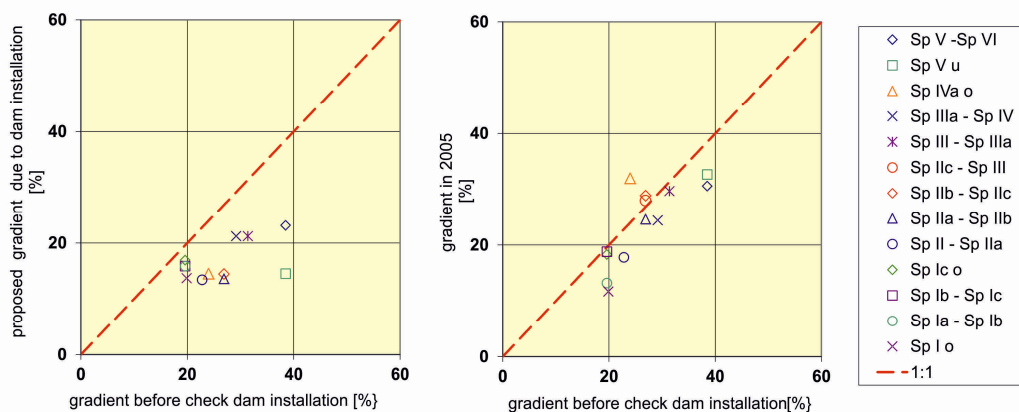


Fig. 4 Left: Comparison of the gradient before the installation of the check dams and the gradient expected due to dam installation (cp. Fig. 3), right: Comparison of the gradient before the installation of the check dams and the gradient measured by aerial survey in 2005 (legend: Sp = check dam, o = upstream, u = downstream)

The traces of former debris flows are only partial conserved on the alluvial fan of the Lammbach due to building development. But taking into account the remaining traces of debris flows, it can be suggested that the different events had been of large extend. Additionally it is recognizable, that the

major part of the masses of former debris flows are located at the upper part of the alluvial cone. At the lower part of the cone the thickness of former events is much less.

Today, the cone is incised up to 16 m deep, starting from the apex and making a strong bend towards the west. Having a look on old maps it is clear, that this incision is a result of the events in 1896. Before, the Lammbach was flowing more or less on the surface of the cone. Still in 1886 there was a debris flow in eastward direction towards the village of Hofstetten. However, nowadays a large debris flow might be able to fill up the deep cut and travel straight (south). Approximately 150'000 m³ of debris are necessary for such change. The volume of debris for this process is available, particularly in the middle part of the Lammbach, as described above. A failure of one or more check dams may make this quantity of debris disposable.

ASSESSMENT OF THE LAMMBACH CHECK DAMS

The Lammbach check dams, constructed during the period 1896 to 1913, had been made of natural stone masonry. Natural stone masonry is of course a durable construction material but even such a material underlies weathering and aging. The lifespan of such constructions made of natural stonework in general is specified with 60 to 80 years, maximum 100 years (e.g. Rudolf-Miklau & Agerer, 2007). But that does not mean the construction will lose its structural safety and the fitness for purpose automatically when a structure attained such an age. However the structural safety and the fitness for purpose of the check dams had to be verified.

Additional to weathering and aging of the masonry, due to old plans there are some specifics concerning the construction of the check dams:

- footing with wooden sleepers (check dams Ia, Ib, IIIa)
- wooden piles at the footing (check dam II)
- arch constructions within the masonry (check dams Ie, II, III, V).

Since the construction of the check dams, at some of them modifications had been done. So the check dams I, Ic, II, IV, V, VII and VIII had been increased in height. The check dams III and IVa got partly a concrete slab on downstream face with pre-stressed anchors (Fig. 5). Those modifications of the dams are attended with changes of the applied loads as well as changes in the structural system.



Fig. 5 Check dam IVa with rehabilitation measures, middle part fixed by a concrete plate and pre-stressed anchors in 1976, buttresses and pre-stressed anchors installed at the barrier wings in 2000

The Swiss National Platform for Natural Hazards (PLANAT) defines a 5 step procedure concerning the assessment and the effect of protective structures (PLANAT, 2008). The steps are named as follows: preliminary work, crude appraisal, arrangement assessment, outcome assessment and preliminary work for urban and regional planning. Analogous to that, the assessment of the Lammbach check dams and the preparation of the hazard map had been done also in a 5 step procedure with the following steps:

1. evaluation of the basics
2. process assessment
3. arrangement assessment
4. outcome assessment
5. realization of the hazard map

EVALUATION OF THE BASICS

In the first step the available documentation has been analyzed, a geodetic survey has been done and the barriers' as-is state has been mapped. The last one has been done by using a special rating system for natural stonework, which has been developed at the Lucerne University of Applied Sciences and Arts in the last years (Kister et al, 2008). The rating system for natural stone masonry has been developed based on our experience with rating systems in rock mechanics on one hand and our experience with natural stonework on the other. Combining both and additionally taking into account aspects of the surrounding area as topography, geology, groundwater and plant-cover, the assessment and rating of old natural stonework has been put on a less subjective base.

Some adaptations had to be made for the check dams because the original rating system was developed for retaining walls placed at traffic infrastructure. Nevertheless an assessment of the check dams made of natural stone masonry has to keep in mind the same 3 general aspects:

- as-is state of the construction itself and the construction elements,
- constructiveness and
- properties of the surrounding area.

The as-is state of the construction implies the as-is state of the construction itself, as for example break-outs in the masonry or damage due to settlement, but also the as-is state of the components stone and mortar and the status of the drainage. The constructiveness sums up data concerning the design and composition of the structure, such as ratio of masonry wall thickness to construction height, size of stones and thickness of joints (Fig. 6). Plant cover, potholes, indication of slope instabilities or indication of erosion for example are properties of the surrounding area. Fig. 7 shows as an example for the summary of the rating of a Lammbach check dam the results for check dam I. Maximum value for a rating value is 1.0, values less than 0.6 show damages, a value less than 0.5 show significant damages. In the case of the basic data low values show missing information, as for example no information present on the anchoring depth.

basic data				
structure		elements (natural stones + joints)		drainage
thickness ratio (wall thickness at the base / max. height)	RKB	natural stones:	RKG	weepholes
		variation in size		
inclination of bearing area	RKF	position of largest stones in the wall	RKE	drainage gutters
		relation of layer thickness to stone length		
anchoring depth of the wall	RKF	orientation of foliation cleavage	RKE	drainage gutters
		execution of transverse bracing		
type of masonry	Rtyp	joints:	RKE	erosion
		cross joints		
		transversal joints crossing several layers		
		properties of joints in relation to type of masonry		

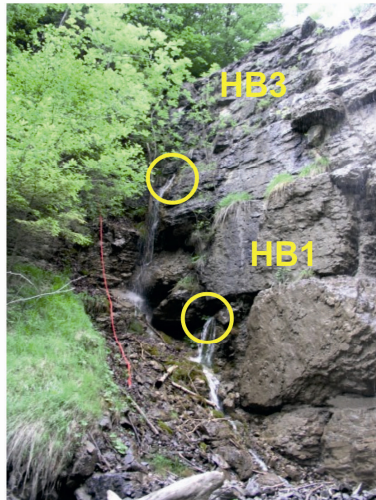
as-is state				
structure		elements (natural stones + joints)		drainage
partly or complete collapse of masonry	RZN	natural stones:	RZM	weepholes
		degree of weathering		
bulge / debonding of front layer	RZN	compressive strength	RZM	drainage gutters
		damage due to overstraining		
shift of inclination overhang	RZS	thrust of single stones	RZE	erosion
		settlement		
as-is state of masonry bond	RZV	joints: drystone walls: properties of stone surfaces		
		joints filled with mortar: as-is state of mortar		

Fig. 6 Left: Basic data for the assessment of the constructiveness of a retaining wall made of natural stonework, right: Assessment criteria of the as-is state of a retaining wall (Kister et al., 2008)

Constructions made of natural stonework are partly large-sized and sometimes there is a change in stone size or even in bond type within a construction. During the life cycle of a structure large-scale

changes may have happened, done with materials different to the original construction materials. In all these cases one has to divide the structure in so-called homogeneous segments. Homogeneous segments are characterized by the following features for example:

- same type of bond exists
- the structure of the masonry varies only marginal
- no significant changes in geometry



	Rating	BW	HB 1	HB 2	HB 3
structure :	q_{Mk}	0.47			
slenderness ratio	RKB	0.9			
anchoring depth	RKF	0.9			
constructiveness	RKG		0.58	0.61	0.64
as-is state structure :	q_{Mz}	0.5			
structure as a whole	RZN	1.0			
settlement	RZS	1.0			
bond	RZV	1.0			
discharge section	RZA	1.0			
drains / water	RZE	0.5			
as-is state elements :	q_{Bs_z}	0.89			
rock blocs + mortar	RZM		0.89	0.89	0.89
surrounding area :	q_{TG_z}	0.42			
plant cover	RZB	0.85			
subsoil	RZU	0.5			

Fig. 7 Example for the rating system: missing information about the transverse bracing and large variation in bloc size of check dam I induced a low value for rating RKG, check dam I shows significant water outlets at the western part (circles), this results in a low value for the rating RZE (drains / water), indication for an undercutting of the check dam led to a reduction of the rating RZU, BW: structure, HB: homogeneous segment

The evaluation of the basics includes also the determination of material parameters of the check dams as well as of the rock mass. To do this, it had to be taken into account, that the access to the check dams is only possible via hiking trail or by helicopter flight. Therefore easily manageable methods and non-destructive methods have been preferred for this work. So for the determination of the uniaxial compressive strength of the rock as well as of the mortar a Schmidt-hammer, type L, has been used in the field. A small-sized drilling equipment has been transported by helicopter to check dams IV and IVa and at each of both check dams a horizontal drilling was executed to get information about the interior of the dams. Additionally the drillings have been inspected via a borehole camera. Another part of evaluation of the basics was to check the masonry status at the upstream face of some of the check dams and to verify some structural elements. This was done by excavating trial pits as spot samples at check dams IV and IVa (Fig. 8a). A walking excavator, the only construction machine which was able to reach the check dams IV and IVa on its own, has been used to do this work. The essential results of that work can be summarized as follows:

The status of the masonry at the upstream dam face was found better than that one at the downstream face (Fig. 8, b and c). The decomposition of mortar in the masonry joints on the upstream face was predominantly less than 10 cm. At the same check dam the decomposition of mortar in joints at the downstream face often shows a greater depth, the maximum found at check dam IVa was 50 cm. The excavation of the trial pits eliminated also some lack of clarity concerning the thickness of the construction and the shape of the cross section.

During the drilling work water outlet at the masonry downstream side was detected approximately 1 m before the drilling bit reached the structure surface. Therefore it can be stated, that the depth of weathering of the masonry is approximately 1 m.

The mortar found in the drillings can be described as a tamped concrete with coarse grained aggregates and large porosity. This “mortar” is very much different from that found at the structure surfaces. Two specimen for uniaxial compressive strength tests were obtained with the drillings. The test result of the sample of check dam IVa was $\sigma_{d,R} = 6 \text{ MN/m}^2$, the test result of the sample of check dam IV was $\sigma_{d,R} = 15 \text{ MN/m}^2$. In both cases the uniaxial compressive strength was very much smaller than the values determined with the Schmidt-hammer on mortar at the check dam surfaces



Fig. 8 Check dam IV: a) trial pit at the upstream side, b) joints and decomposition of mortar at the downstream face, c) joints and decomposition of mortar at the upstream face.

For the determination of the uniaxial compressive strength of the rock with the Schmidt-hammer the formulas of Deere & Miller (1966) have been used. To confirm the results produced with the Schmidt-hammer some compressive tests on drill cores have also been done. Comparing these test results, it can be mentioned, that there is a difference in the compressive strength of the rock blocs used for the check dams. So the compressive strength of the rock blocs used for barriers IV and IVa was found to be less than the compressive strength of the rock blocs used for the barriers Ia, Ib and Ic. Fig. 9 shows the results of Schmidt-hammer tests in comparison to results of uniaxial compression tests on rock material. The compressive strength $f_{d,M}$ of the natural stone masonry was evaluated to approximately 6 MN/m^2 by using the formula of Berndt (1996).

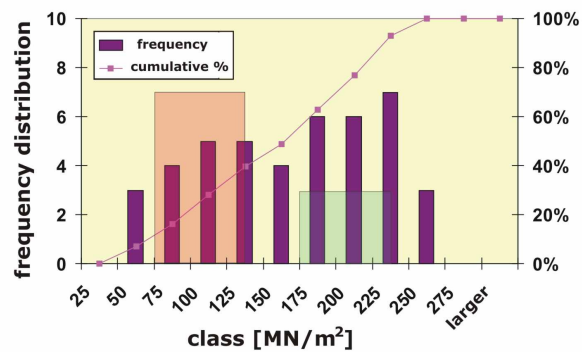


Fig. 9 Uniaxial compressive strength of the siliceous limestone, dark bars: frequency distribution of Schmidt-hammer results; left-hand side: band of test results of uniaxial compression tests, cores derived from check dams IV, IVa and rock blocs of the area between check dam IV and check dam IVa; right-hand side: band of test results of uniaxial compression tests, cores derived from rock blocs of the area check dam I to check dam Ia

PROCESS ASSESSMENT

In the second step the potential failure mechanisms and hazard scenarios had to be detected and the forces acting on the constructions had to be determined. For example at check dam I a water outlet has been detected at the interstice between the former spillway and the part which has been erected at a later date. The erected part has a length of about 23 m and the fixed support at the side may be incomplete. In case of large debris flows with latitude larger than the today spillway dimensions a debris flow impact may occur and this part of the construction may fail by toppling. A 2D-approach in this case is sufficient to describe the problem. Fig. 10 shows the cross section as well as the loads acting on the upper part of the check dam.

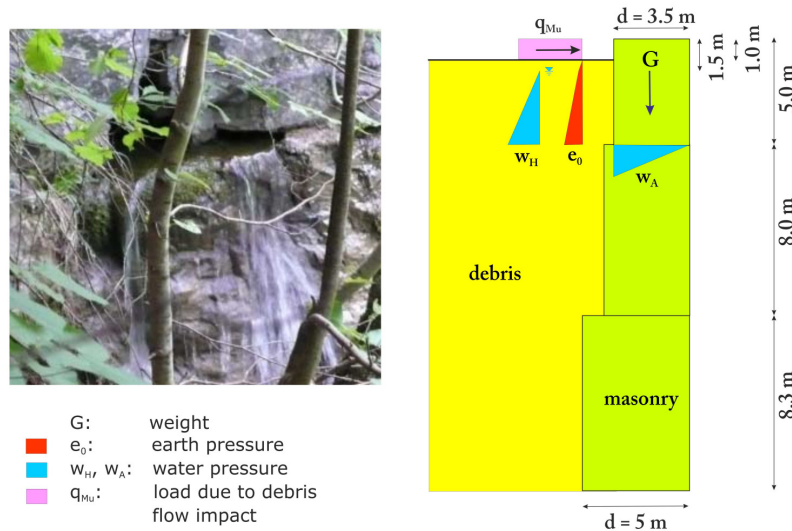


Fig. 10 water outlet and cross section of check dam I, position of the water outlet at 5 m below the capstone

In the literature different approaches can be found for the debris flow impact. In general one can distinguish between hydrostatic and hydrodynamic approaches. The hydrostatic approach of Lichtenhahn

$$q_{Mu} = k \times \rho_{Mu} \times g \times h_{Mu} \quad (1)$$

has been used as well as two hydrodynamic approaches (e.g. in Egli, 2006):

$$q_{Mu} = 0.5 \times c_d \times \rho_{Mu} \times v_{Mu}^2 \quad (2)$$

$$q_{Mu} = a \times \rho_{Mu} \times v_{Mu}^2 \quad (3)$$

Hereby q_{Mu} is the load due to debris flow impact, ρ_{Mu} , h_{Mu} and v_{Mu} are the density, the height and the velocity of the debris flow. The parameters k , c_d and a may vary according to the properties of the debris flow (coarse or fine grained material, low or high velocity of the debris flow, etc.).

In the case of the Lammbach check dams the area behind the spillway is completely filled up with debris and a debris flow impact force as described before will therefore only act at the dam's wings where the backfilling is not completed up to the crest. At the spillway an overtopping of the check dam by the debris flow will occur and the stress due to fluid shear acts on the check dam in this case. Two different models have been studied in the project to estimate the fluid shear stress of a debris flow. The first one is based on normal bedload transport (LfU, 2002):

$$\tau_{sh} = \rho_{Mu} \times g \times h_{Mu} \times \sin \Theta_{bed} \quad (4)$$

where Θ_{bed} is the mean channel gradient. A second model was developed by Iverson (2005):

$$\tau_{sh} = (\rho_{Mu} \times g \times h_{Mu} \times \cos^2 \Theta_{bed} - p_{bed}) \times \tan \varphi_{bed} \quad (5)$$

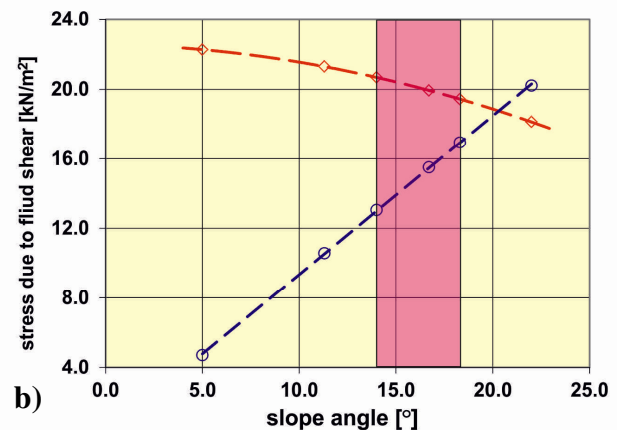


Fig. 11 a) overtopping of a check dam by a debris flow (WSL, 2000), b) correlation of stress due to fluid shear and slope angle, comparison of 2 models, marked area is decisive for the Lammbach check dams.

This model takes into account the friction angle φ_{bed} as well as the pore pressure p_{bed} in the boundary layer between debris flow and subsoil. The pore pressure p_{bed} was assumed to be 15 kN/m^2 and was taken from data measured by McArdell et al. (2007) at the Illgraben in Switzerland.

For a debris flow with a density $\rho_{\text{Mu}} = 1800 \text{ kg/m}^3$ and a height $h_{\text{Mu}} = 3 \text{ m}$ the correlation of stress due to fluid shear and channel gradient respective slope angle is shown in Fig. 11. The red marked area is the representative interval for the gradient in the Lammbach gorge.

ARRANGEMENT ASSESSMENT

The 3rd step deals with the estimation of the functional capability of the adopted measure. The structural safety as well as the fitness for purpose of the structures has to be checked. An example for loss of effectiveness due to aggradation is check dam V. Today this barrier is covered by debris except for a very small part at the eastern barrier wing (Fig. 12c). At the spillway the thickness of the debris layer today is several meters. Therefore check dam V has lost its fitness for purpose according to the primary design (cp. Fig. 2). But even in this status the check dam is of high importance for the complete torrent control system because in case of a large debris flow it limits the erosion of material to the level of the barrier's crown.

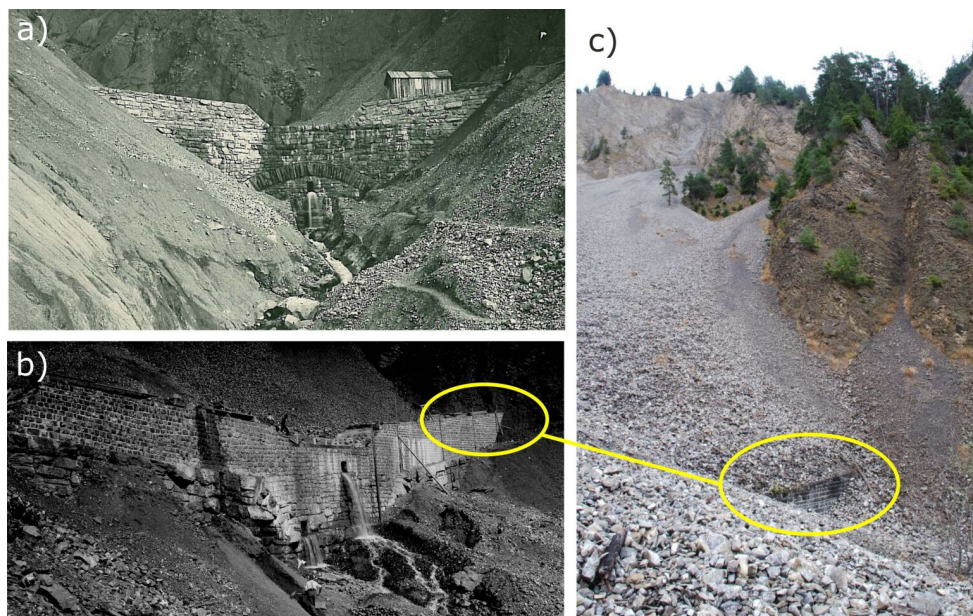


Fig. 12 Changes at check dam V: a) when finished in 1904, b) increased in height in 1936, c) the only visible part of check dam V today (OIK).

Check dams III and IVa are examples for an increasing load at the upstream side of the barriers due to aggradation. The results of the photogrammetric survey in the Lammbach gorge show for example the steady rise of material behind check dam IVa within a period of the last 65 years. Additional on both check dams changes had been done at the supporting structure in the context of rehabilitation measures. Because of the complexity in geometry, load cases and material behavior of those 2 large check dams III and IVa, 3D-Finite-Element calculations had been chosen for the analysis of the structural safety. Fig. 13a shows the 3D-FE-mesh of check dam IVa used for the calculations. It consists of 25'160 3D-elements, 49 discrete anchor elements and 1'393 interface elements. The calculations have been done using linear elastic and elastic-plastic material behavior for the check dam. Mohr-Coulomb failure criterion and a tension-cut-off model as well as the multi-laminate model have been used to simulate the masonry. For the subsoil and the securing mean (concrete slab, buttresses and anchors) elastic behavior has been chosen.

The calculations for check dam IVa show a deformation which was less than 10 cm in maximum. The anchor load show a value which is nearby, but lower than the tolerable value. Therefore even if the water level rises up to the level of the spillway and a debris flow occur additionally the calculations show no failure of check dam IVa.

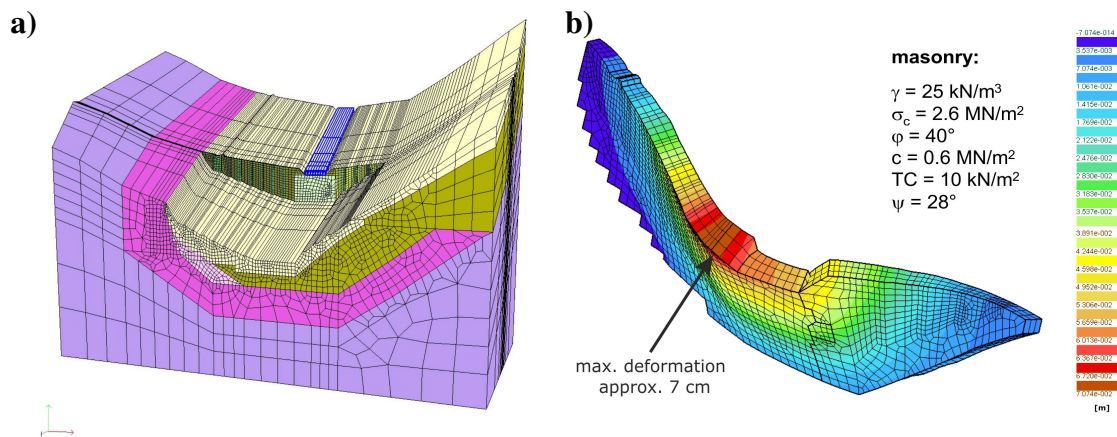


Fig. 13 a) 3D-FE-mesh of check dam IVa and the subsoil, b) deformed mesh of check dam IVa for a calculation with a tension cut-off model $\tau_{TC} = 10 \text{ kN/m}^2$, shown is the last load case: overtopping by debris flow.

OUTCOME ASSESSMENT

The outcome assessment, the 4th step of the procedure, handles the evaluation of uncertainties and the resulting risks. Uncertainties are given for example by incomplete project documentation of the constructions and by variation in material parameters.

REALIZATION OF THE HAZARD MAP

Based on the results of the previous 4 steps different scenarios had been analyzed. The most likely scenario acts on the assumption of the failure respectively partial failure of several dams, but check dam IVa will remain stable. The scenario starts with a large landslide in the middle of the Lammbach gorge. Approximately $50'000 \text{ m}^3$ material of this landslide will form a debris flow running down the valley and eroding further debris masses. This will result in

- an overtopping of check dam IVa by approximately $95'000 \text{ m}^3$ of debris, but the dam itself will remain stable,
- a debris flow of approximately $220'000 \text{ m}^3$, which will reach the top of the fan.

This will be a scenario with a return period of 300 years. For an extraordinary event it is assumed that all check dams will fail, i.e. check dam IVa too. In this case approximately $790'000 \text{ m}^3$ of debris will reach the top of the fan.

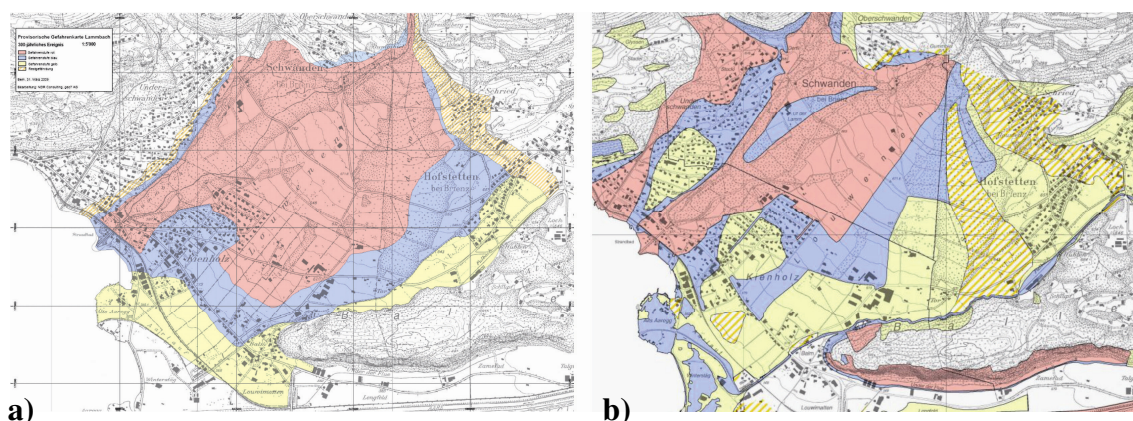


Fig. 14 Hazard maps obtained with different scenarios: a) failure of check dam IVa, debris flow volume at the fan apex will be approx. $790'000 \text{ m}^3$, b) check dam IVa remains stable, a debris flow volume of approx. $95'000 \text{ m}^3$ is overtopping check dam IVa, a volume of $220'000 \text{ m}^3$ debris will reach the fan apex.

In comparison to a scenario taking into account the failure of all dams, the scenario of a 300 year event shows a significant smaller so called “red area” in the hazard map, which is the area where the construction of new buildings is prohibited (Fig. 14). To choose this scenario as the most likely

scenario was only possible due to the engineering assessment of the as-is-state of the 100 years old Lammbach check dams.

CONCLUSIONS

The assessment of the 100-years old torrent control structures in the Lammbach gorge is a very complex and ambitious task which necessitate the close collaboration of experts of different disciplines. As shown before not all barriers in the Lammbach gorge show the fitness for purpose and the structural safety after a period of 100 years. But with the reconditioning done in the past the analysis of check dam IVa showed the structural safety of that barrier. And this results in a significant reduction of the mass of a potential debris flow, which may reach the inhabited area at Lake Brienz in case of a 300 year event.

Uncertainties are given especially by missing or ambivalent data concerning the barriers' geometry and composition. Not all of those uncertainties can be eliminated within reasonable operating expense by exploration and investigation. Additionally there is an incomplete knowledge concerning debris flows. Further research is needed to get a better understanding of the debris flow load cases and to reduce uncertainties in this domain.

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