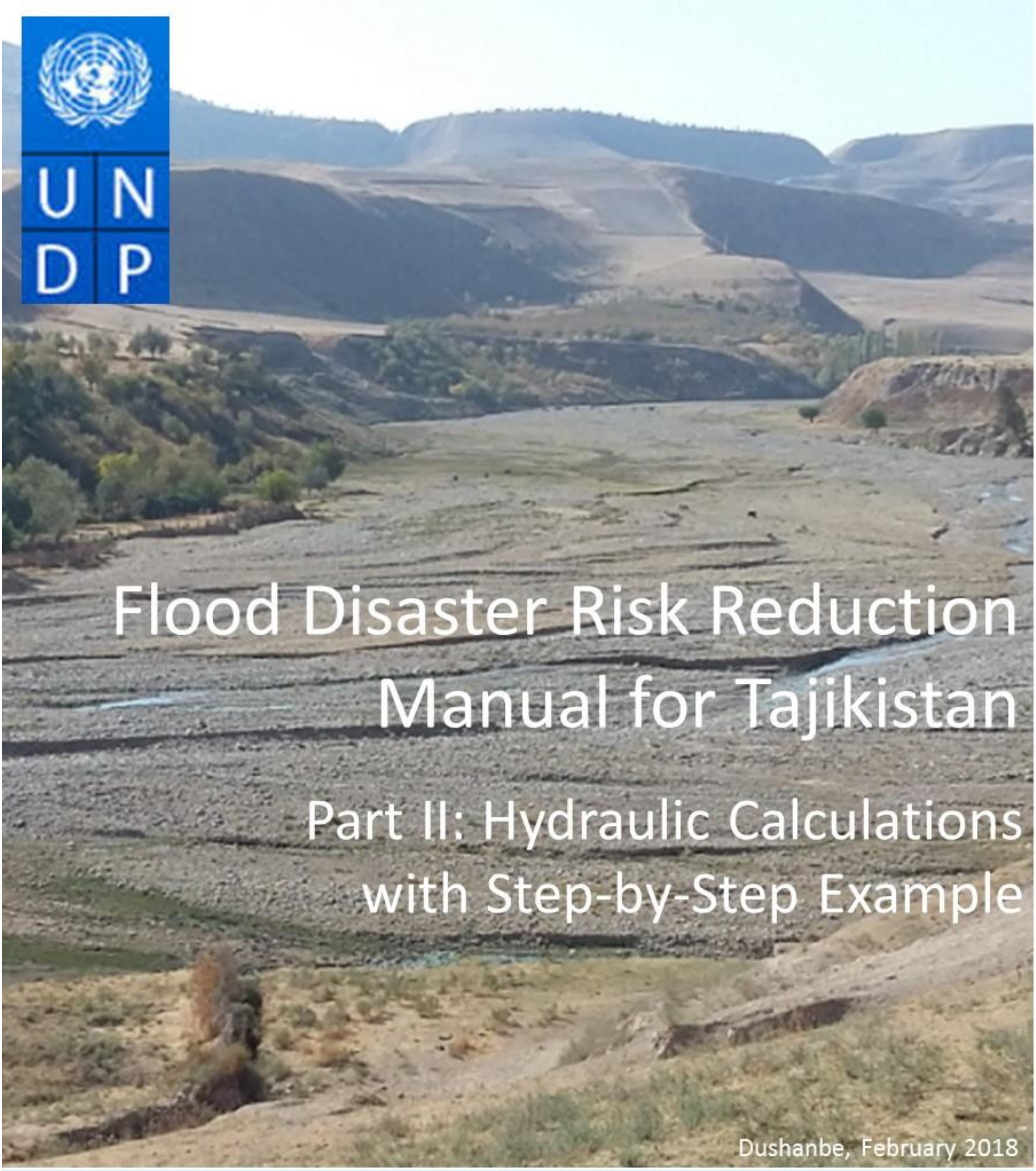

Flood Disaster Risk Reduction Manual for Tajikistan

Part II: Hydraulic Calculations with Step-by-Step Example

Dushanbe, February 2018

United Nations Development Program of Tajikistan

UNDP Disaster Risk Management Programme
Strengthening Disaster Risk Reduction and Response Capacities
Ecosystem based Disaster Risk Management

Acknowledgements

This document was prepared in order to support the Government of Tajikistan within the framework of the Disaster Risk Management Programme (DRMP) of United Nations Development Programme (UNDP). UNDP assists the country in their effort to conduct a nation-wide risk assessment, selecting and implementing risk reduction measures, improving early warning and disaster management planning, preparedness and response, as well as strengthening capacities of search and rescue teams.

The author wishes to thank the UNDP team in Tajikistan for their support, contribution and commitment. The discussions and meetings in Dushanbe and during the field trips were invaluable for preparing this guideline.

Particular acknowledgements go to Michihiro Tanabe, Ilhom Safarov and Firdavs Faizulloev from UNDP for their dedication in providing the author with information, data and for their assistance in making meetings possible and successful. Mr. Kelly, the Risk Governance Consultant for UNDP in Tajikistan, deserves a big thank you for his contribution regarding the organisational and institutional framework of flood management in the country. Credits belong also to the partners in the project for their help and contribution regarding best practice examples, in particular the people from ACTED, Camp Tabiat, CESVI, German Agro Aid (Welthungerhilfe) and GIZ.

Last but not least, credits belong also to Dr. Michael Bach from SYDRO Consult who supported the author in compiling appropriate hard and soft measures and for sharing his experience in conceptualising flood management with ecosystem-based measures.

Weinheim, March 6, 2018

Dr. Hubert Lohr

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Flood Management Guideline Tajikistan

Document Information

Project	Ecosystem based Disaster Risk Management
Project Countries	Tajikistan
Document	Flood Management Guideline Tajikistan
Date	05.02.2018
Consultant	Dr.-Ing. Hubert Lohr
Client	UNDP Tajikistan
Client Representative	Michihiro Tanabe, Firdavs Faizulloev
Financing Organisation	UNDP

1 STEP BY STEP EXAMPLE

This section provides a step-by-step example from site analysis over rainfall evaluation to flood calculation. A five steps approach can be taken:

1. GIS Analysis
2. Rainfall analysis
3. Discharge analysis
4. Assessment of flow paths
5. Selection of measures

1.1 The example site

The example site is located in the south of Tajikistan close to Muminabad.

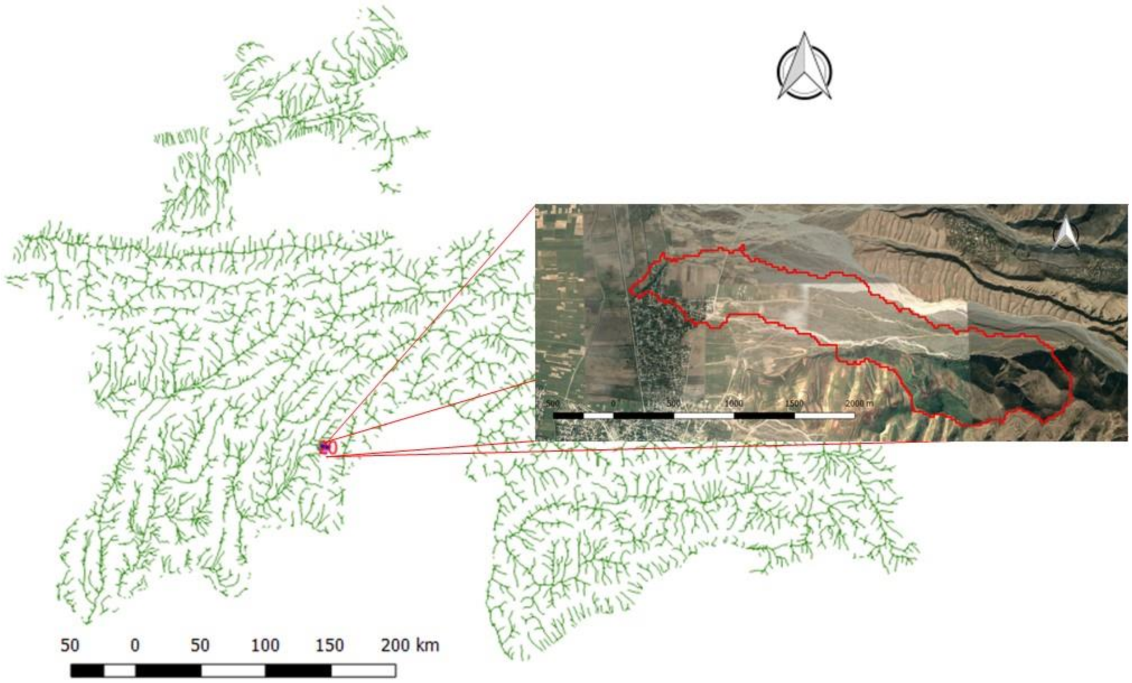


Figure 1: Location of the example site

The example site incorporates some aspects which might be representative for small tributaries and watersheds in Tajikistan. High and partly steep headwater areas, steep and narrow valleys which level off in a large alluvial fan. This area has only seasonal streams which bear water after rainfall. The catchment area comprises 378 hectares, the elevation ranges from 1213 m to 1886 m along a distance of approximately 5km. The headwater areas are mountainous with few agricultural activities. Most of the hills used as pasture show only sparse vegetation, if any. Some signs for erosion are visible. One hill is replanted with shallow plants, shrubs and cultivated with trees and vegetable which was done under the supervision of Caritas Switzerland (Caritas, Chukurak Watershed Activity Plan, 2012) and (Caritas, 2006) (see Part III). The transport reaches are short and steep forming incised ditches which carry water only for a short period of time. Downstream the transport reaches, the area widens and the gradient reduces. The narrow valleys open forming the deposition area with a wide braided river bed and a number of minor flow paths. Settlements are located further downstream and south of the main river bed where some buildings are erected in or very close to the minor flow paths.

The braided river bed in the background

Cultivated hill left with planting by Caritas

Tributary valley

New swales in the foreground, established swales with trees in the background

Figure 2: Pictures of the example site

1.2 GIS Analysis

The GIS analysis contains six steps from which parameters for calculating discharge have been derived.

Step 1: Obtain the digital elevation model for the project site

Select the area of concern and download the STRM30 (1-arc second).

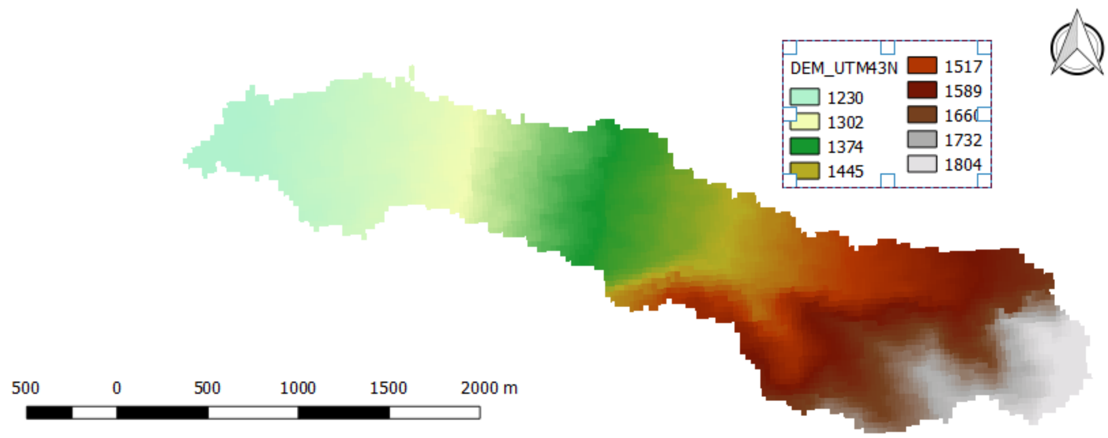


Figure 3: SRTM30 DEM for the surrounding of Muminabad

Step 2: Calculate the flow directions from the SRTM30

By using a GIS, the flow directions can be computed. This is a prerequisite for all subsequent actions.

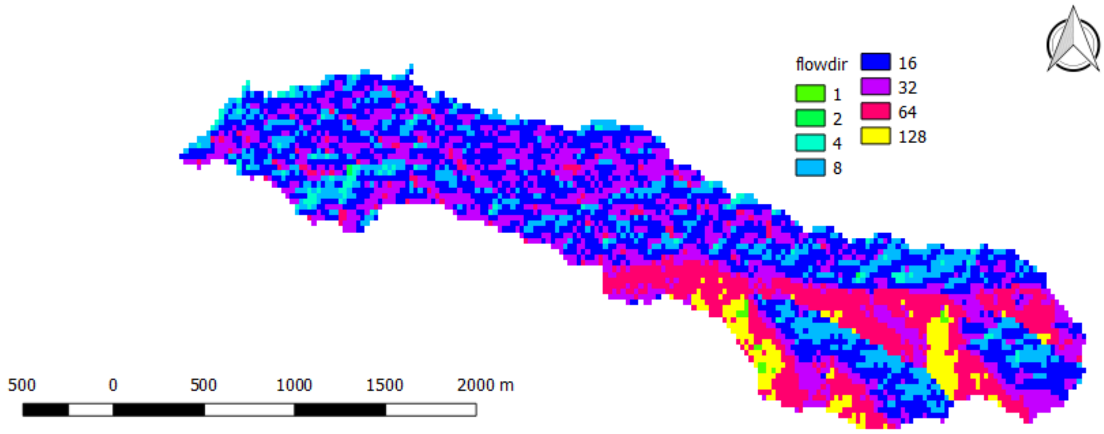
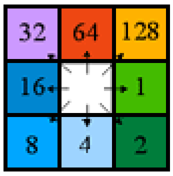


Figure 4: Flow directions

Each colour represents a flow direction. The number depends on the tool used. The principle can be shown by using the example from ArcGIS.



Direction coding

ArcGIS interpretation)

Step 3: Calculate the flow accumulation from the SRTM30

Flow accumulation is required to determine sub-basins in a subsequent step. The number of upstream cells is stored in each cell. This step is also used to ascertain the stream network.

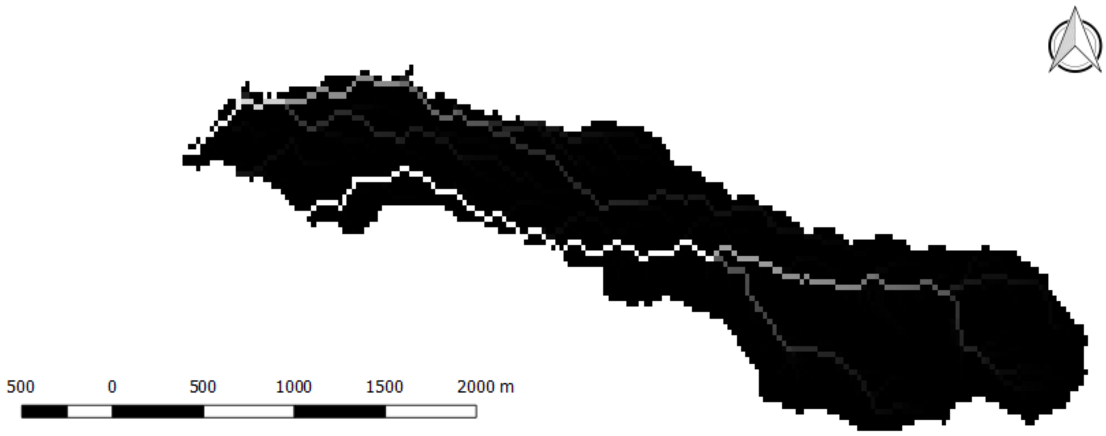


Figure 5: Flow accumulation

The grey shade indicates the number of upstream cells flowing through the respective cell. Black means no upstream cell.

Step 4: Generation of the stream network from the SRTM30

The stream network is relevant to obtain vectorised data about flow paths. The result does not necessarily follow real rivers, it indicates the steepest flow paths based on the analysis of flow direction derived from the DEM. A high number of upstream cells makes it very likely that a calculated stream from SRTM30 coincides with a real river.

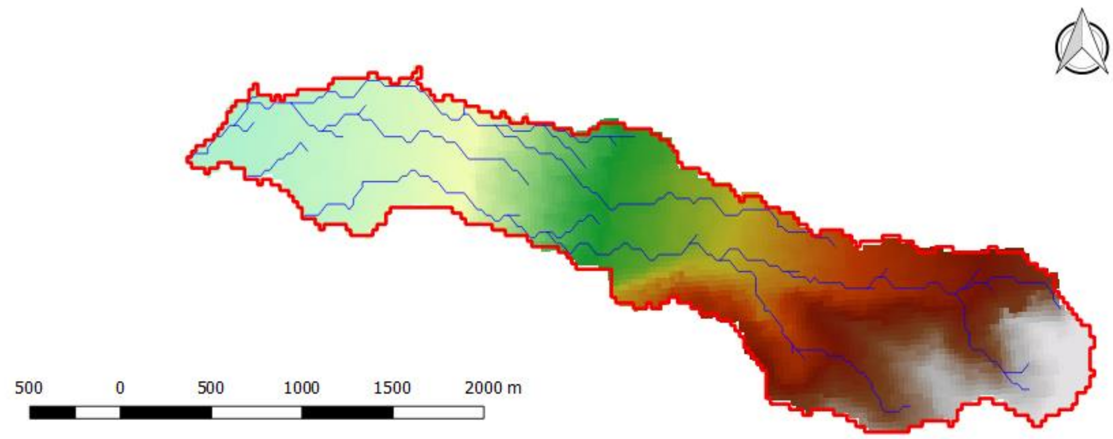


Figure 6: Stream network calculated

The stream network provides the means to identify possible flow paths and gives a direction for further measures. It is advisable to cross check and to support the stream network on site by means of a field visit and by making use of local knowledge.

Step 5: Determine sub-basins from the SRTM30

From the flow accumulation and stream network procedure, sub-basins can be derived by applying a threshold value for the number of cells forming one sub-basin. The larger the number the less sub-basins are created. Alternatively, pour points are created at which location a sub-basin is built.

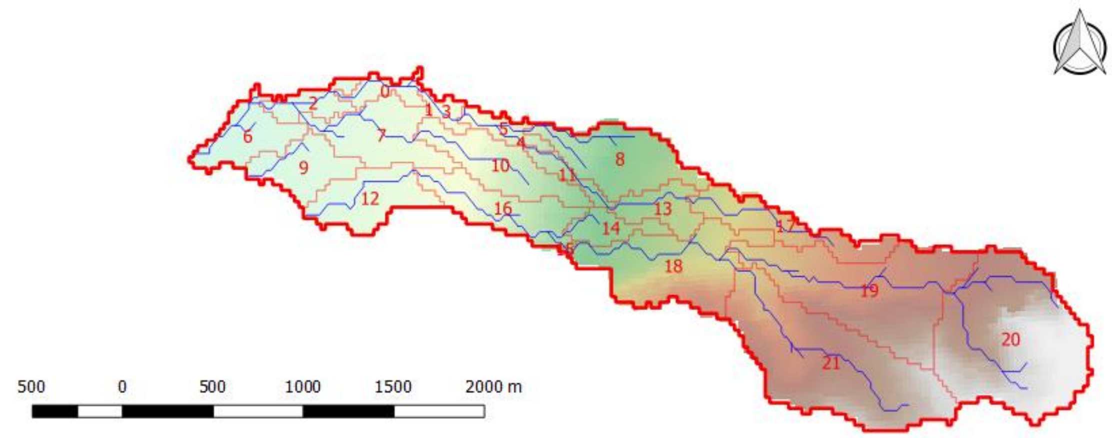


Figure 7: Sub-basins

Step 6: Generation of slopes

A very useful tool of GIS is to derive slopes from a DEM. The slope is a strong indicator for erosion proneness and is used in many applications, e.g. estimation of erosion, time of concentration, runoff, planning and siting of measure. Slope is also required to derive parameters for calculating discharge.

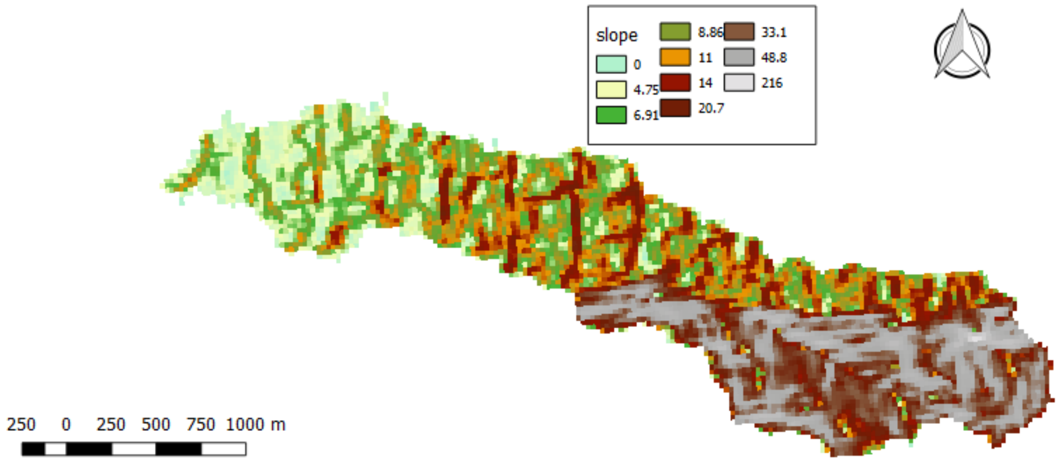


Figure 8: Slope calculated from SRTM30 DEM

It is not necessary to build categories but it makes it easier to read the map.

1.3 Rainfall analysis

A flood analysis requires a rainfall-runoff calculation to obtain flow and flood peaks in case no discharge observations and discharge statistics are available. This is probably the case for most of the tributaries and valleys which are not located close to one of the 89 hydropost stations in Tajikistan.

A prerequisite for rainfall-runoff calculations is the availability of information about rainfall depth/rainfall duration linked to return periods.

The assessment of rainfall yields the load for any subsequent computation. This process must be conducted carefully and effort should be made to obtain rainfall data which are relevant for the project site. This is especially challenging given the sparse data situation in Tajikistan.

There are two options:
1) an analysis of rainfall depth/duration/return periods is available
2) no analysis is available and it is necessary to derive the information

The first option is convenient and no further action is required. Here, option 2 is considered as standard so we concentrate on option 2.

For the example site of Muminabad, a rainfall time series located in the Khovaling district was used. Rainfall time series can be obtained from the Meteorological Department (see Part I) or this department conducts the analysis on request.

Additionally, this example uses data obtained from the Climate Forecast System Reanalysis (CFSR).

	Source	Request	Timeframe	Temporal resolution
1	Khovaling, Meteorological Department	Direct request	Jan. 79 – Dec. 2011	daily
2	Muminabad region, CFSR	download	Jan. 79 – Jul. 2014	daily

The platform for downloading data indicates if the area selected contains data or not depending on the number of points of the CFSR system which are incorporated in that area.

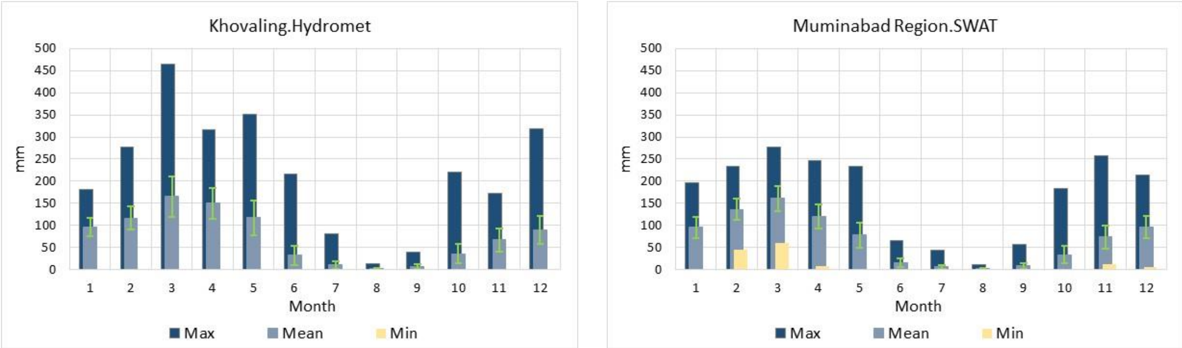


Figure 9: Monthly rainfall pattern from the Khovaling station and from CFSR

Step 1: Elimination of outliers

The time series (1) contains daily values of rainfall depth from 1979 to 2011. Before any analysis takes place, the series must be checked for outliers.

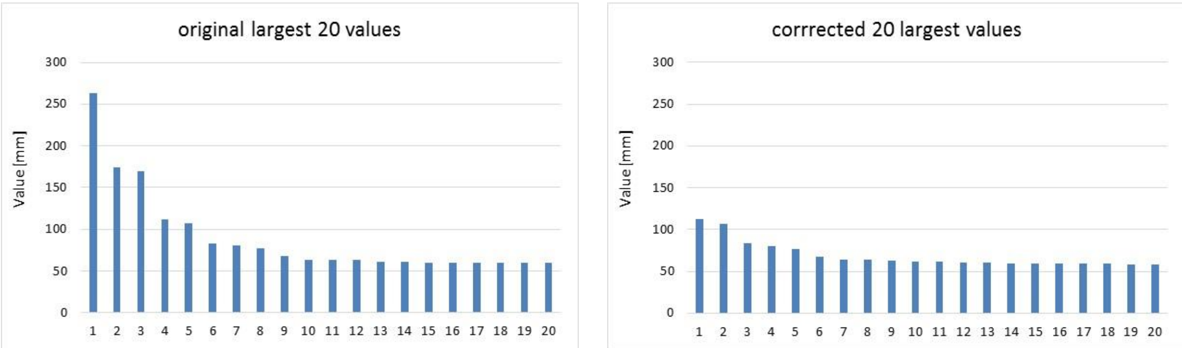


Figure 10: Elimination of daily rainfall outliers for Khovaling

The identification of outliers is difficult when values seem possible but still are beyond the expected range. It is recommended investigating in this matter as extreme rainfall depths are most important for the statistical analysis. Usually, extreme rainfall events are memorized and people in the region affected can remember it. It is worth obtaining the perception of local people if there is no other way of determining the reliability of outliers.

Step 2: Generation of Intensity-Duration-Frequency curves

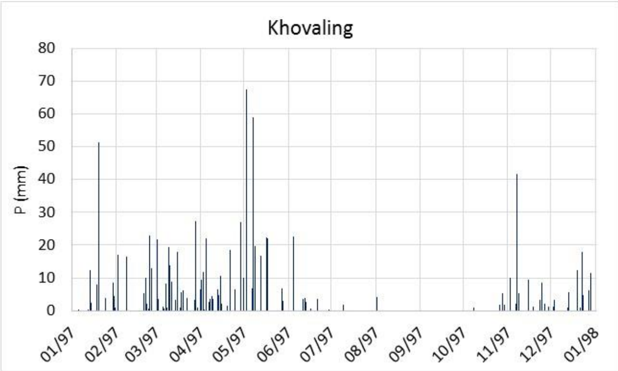
The background of Intensity-Duration-Frequency curves (IDF) of Depth-Duration-Frequency curves (DDF) is explained in Section 2.2. It is one of the most important analysis and constitutes a worldwide standard (Maidment, 1998).

An IDF analysis yields the necessary input for calculating flood events. The approach links rainfall depth/rainfall duration with return periods. Calculating discharge based on an IDF curves assumes that the peak discharge has the same return period as the rainfall event. This is a simplification but reflects common practice in deriving design floods.

The process of obtaining IDF or DDF curves requires some effort and background knowledge. It is recommended having the analysis carried out by the Hydro-Meteorological Centre in Dushanbe. A pragmatic way of deriving IDF curves is given below to familiarise those readers who haven’t been in touch with this kind of analysis.

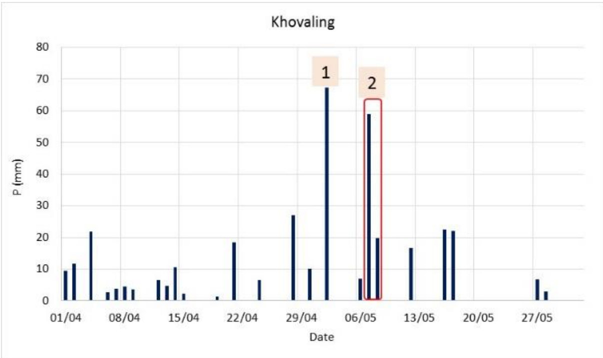
1 Select the maximum rainfall for each year. Start with 1 day maximum, then consecutive 2 days period, consecutive 3 days period up to consecutive 6 days period.

The maximum daily value is easily visible (right image = May), the cumulative sum of the other consecutive days is not that easy to identify. The analysis is shown below.



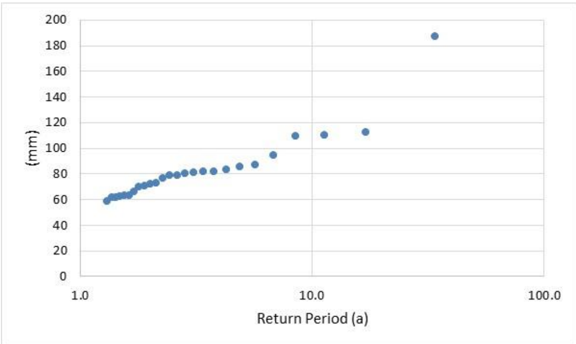
Max values			
1 day	2 day	3 day	4 day
1	2	3	4
P (mm)	P (mm)	P (mm)	P (mm)
67.3	78.6	85.5	85.5
02/05	07/05	06/05	05/05

Daily maximum (2nd of May) and 2-days maximum (7th of May) are illustrated in the right image.

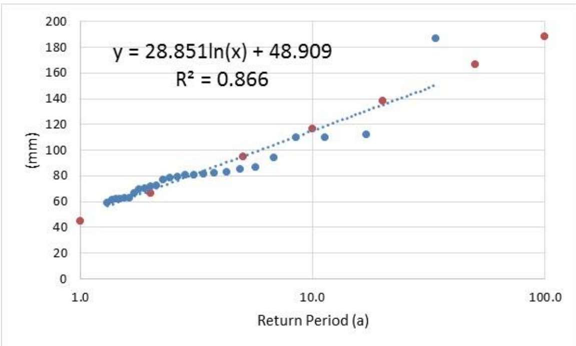


2 Sort all maximum values for all years and the 1 to 6 days and calculate the return period by means of a simple empirical function RP(a) = (N+1)/i where:
N = Number of years
i = rank (1 = largest, N = smallest)

The result is displayed for cumulative 2 days rainfall.



3 Create logarithmic trendlines in the form of RF (mm) = A x LN(x) + B for 1 day up to 6 days where x is the return period in years. Evaluate the parameters A and B. Having A and B at hand, the formulas for calculating rainfall depth depending on return periods for rainfall durations equal or longer than 24h are ready. For shorter rainfall durations continue with point 4.

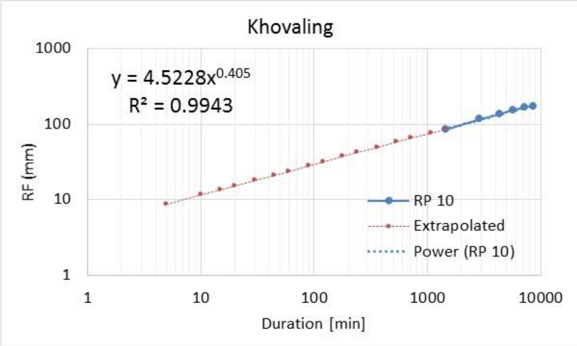


4 Select a return period and use all formulas for 1 day to 6 days to calculated the rainfall depth in mm. The table right shows the values for a 10 year return period.

		RP 10 a	RF (mm)
Rainfall duration		1 day	84
		2 day	116
		3 day	134
		4 day	153
		5 day	166
		6 day	173

5 The calucaled rainfall from point 4 is used to evaluate a trendline as a power function in the form of:
RF (mm) = A x D^B where:
D = Rainfall duration (min)

With both axis, duration in minutes and rainfall in mm, in logarithmic form the power function shows a strong linearity from which shorter rainfall durations can be extrapolated.



Calculating a set of return periods and displaying them in one graph, a complete Depth-Duration-Frequency curve is established. By referring the calculated rainfall depth to one hour, the Intensity-Duration-Frequency curve is accomplished. AEP is Annual Exceedance Probability and is the inverse of frequency or return periods.

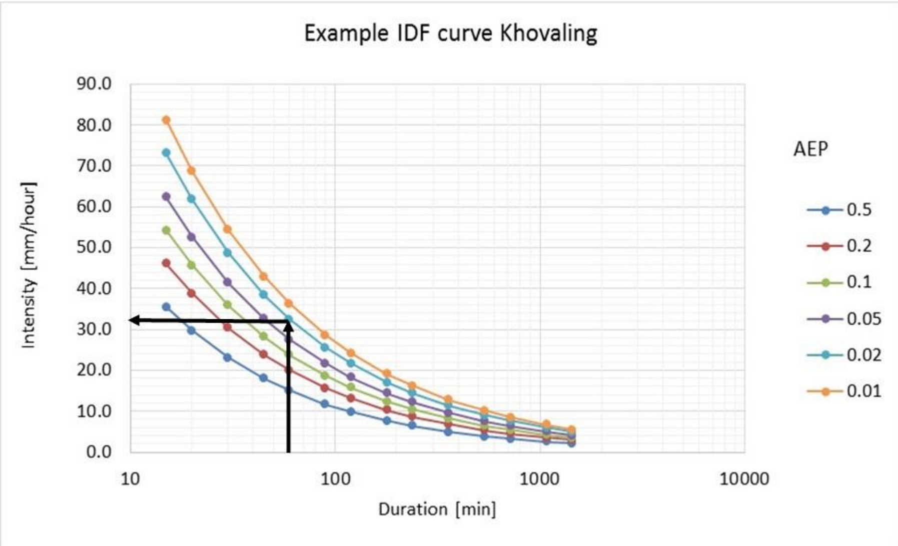


Figure 11: IDF curve derived with daily values for Khovaling rainfall station

In theory, any rainfall duration can be extrapolated. In practice and without any verification, rainfall duration shorter than 1 hour should not be used.

For the example a 50 year return period or 0.02 AEP and a 60 min rainfall duration was chosen. From Figure 11 a value of 32 mm is obtained. For the sake of simplicity, snow is not regarded. However, it could be embedded by evaluating rain and snow in combination.

Table 1: Matrix of rainfall Depth, Duration and Frequency. Cells >= 30mm indicated as blue

		Return period [a]							
		0.5	1	2	5	10	20	50	100
Rainfall duration	5 min	3.9	4.7	5.8	7.4	8.7	9.9	11.6	12.8
	10 min	4.8	6.0	7.6	9.8	11.5	13.2	15.4	17.2
	15 min	5.3	7.0	8.9	11.5	13.5	15.6	18.3	20.3
	20 min	5.8	7.7	9.9	12.9	15.2	17.5	20.6	22.9
	30 min	6.5	8.9	11.6	15.2	17.9	20.7	24.4	27.1
	45 min	7.3	10.3	13.5	17.8	21.1	24.4	28.8	32.2
	1 hour	7.9	11.4	15.1	20.0	23.7	27.5	32.5	36.3
	1.5 hour	8.9	13.1	17.6	23.5	28.0	32.5	38.4	43.0
	2 hours	9.7	14.6	19.6	26.3	31.4	36.5	43.3	48.4
	3 hours	10.9	16.8	22.9	30.9	37.0	43.2	51.2	57.4
	4 hours	11.8	18.6	25.5	34.7	41.6	48.6	57.7	64.7
	6 hours	13.2	21.5	29.8	40.8	49.1	57.4	68.3	76.6
	9 hours	14.9	24.8	34.8	47.9	57.8	67.7	80.8	90.8
	12 hours	16.1	27.5	38.8	53.7	65.0	76.2	91.1	102.4
	1 hours	18.1	31.7	45.3	63.1	76.5	90.0	107.8	121.2
	1 day	22.0	36.4	50.8	69.9	84.4	98.8	117.9	132.3
	2 day	23.5	45.0	66.5	95.0	116.5	138.0	166.5	188.0
	3 day	23.0	48.8	74.6	108.7	134.4	160.2	194.3	220.1
	4 day	23.8	53.7	83.7	123.2	153.2	183.1	222.7	252.6
	5 day	33.5	64.2	95.0	135.6	166.4	197.1	237.8	268.5
	6 day	39.8	70.5	101.2	141.8	172.5	203.2	243.8	274.5

1.4 Parameter of the sub-basins

In total 20 sub-basins were created by means of the GIS analysis. The parameters area, min/max elevation, flow length and slope were calculated with GIS features while land use was taken from Google Earth and verified during a field trip. If available, land use, soil and geological maps should be used for parameter evaluation.

The basin ID refers to the numbers given in Figure 7.

Table 2: Parameters of the sub-basins

BasinID	AREA [ha]	MIN [m]	MAX [m]	MEAN [m]	Max. Flow-length [m]	mean Slope [%]	Land cover
0	5.3	1251	1279	1265	554	5.1	Gravel, sand
2	7.2	1233	1260	1246	474	5.1	Cultivated to gravel
3	3.3	1276	1300	1287	294	6.9	Gravel, sand
5	5.4	1292	1342	1315	618	8.7	Gravel, sand
6	13.6	1213	1245	1231	770	5.3	Urban, green spots, gravel
7	19.7	1243	1298	1267	804	6.4	Gravel to sparse veg.
8	23.1	1324	1430	1377	1238	10.3	Gravel, sand
9	13.6	1231	1264	1246	772	5.5	Urban, dirt roads, gravel
10	18.8	1276	1373	1318	1164	9.5	Gravel, debris
11	4.3	1325	1382	1353	577	9.9	Gravel, sand
12	17.2	1244	1294	1265	799	7.1	Gravel, sand
13	9.4	1379	1429	1407	499	9.9	Gravel, sand
14	6.9	1357	1417	1382	660	10.1	Gravel, sand
16	18.8	1280	1372	1324	1110	9.9	Gravel, debris
17	16.4	1418	1547	1475	1362	10.6	Gravel, sand
18	32.3	1362	1550	1443	1171	26.4	Half gravel, half shrubs
19	47.3	1445	1737	1558	1442	28.5	Sparse veg. to bare soil
20	59.4	1564	1886	1709	1107	41.1	Sparse veg. to bare soil
21A	28.2	1451	1754	1610	1685	33.6	Shrubs, grass, cultivated
21B	28.2	1451	1754	1610	1685	33.6	sparse veg. to bare soil

1.5 Discharge analysis

The discharge analysis can be conducted in different ways. Here, three ways will be shown and compared to each other. The first option is a discharge analysis based on the application of the rational method. The second option is the application of the SCS approach and the third a hydrological model.

1.5.1 Rational method

The rational method is explained in Section 2.3.1. The approach computes peak discharges based on the size of the area, the rainfall intensity and a runoff coefficient. The latter was selected by distinguishing topography, soil permeability, vegetation and storage capacities. The values were taken from Table 8.

Table 3: Parameters for calculating peak discharge with the rational method

Name	BasinID	AREA [ha]	Cr [-]	Ci [-]	Cv [-]	Cs [-]	C [-]	Qp [m³/s]
Valley	0	5.3	0.14	0.08	0.14	0.1	0.46	0.19
Urban edge	2	7.2	0.14	0.08	0.1	0.08	0.4	0.23
Valley	3	3.3	0.14	0.08	0.14	0.1	0.46	0.12
Valley	5	5.4	0.16	0.08	0.14	0.1	0.48	0.21
Urban	6	13.6	0.14	0.08	0.08	0.07	0.37	0.40
Valley	7	19.7	0.14	0.08	0.1	0.1	0.42	0.66
Valley	8	23.1	0.2	0.08	0.14	0.1	0.52	0.97
Urban	9	13.6	0.14	0.08	0.08	0.07	0.37	0.40
Valley	10	18.8	0.19	0.08	0.14	0.1	0.51	0.77
Valley	11	4.3	0.19	0.08	0.14	0.1	0.51	0.18
Urban outskirts	12	17.2	0.16	0.08	0.14	0.1	0.48	0.66
Valley	13	9.4	0.19	0.08	0.14	0.1	0.51	0.39
Valley	14	6.9	0.2	0.08	0.14	0.1	0.52	0.29
Valley	16	18.8	0.19	0.08	0.14	0.1	0.51	0.77
Valley	17	16.4	0.2	0.08	0.14	0.1	0.52	0.69
Outflow Caritas site	18	32.3	0.2	0.1	0.08	0.1	0.48	1.25
Mountain+valley	19	47.3	0.28	0.1	0.12	0.1	0.6	2.28
Mountain	20	59.4	0.32	0.12	0.12	0.1	0.66	3.15
Caritas site	21A	28.2	0.32	0.1	0.1	0.07	0.59	1.34
not cultivated hill side	21B	28.2	0.32	0.12	0.14	0.1	0.68	1.54

1.5.2 SCS Approach

The SCS approach is explained in Section 2.3.2. The approach requires the Curve Number (CN) and time of concentration. The first table shows the values for computing time of concentration, the second table is the calculation of the peak discharge, all according to the formulas in Section 2.3.2.

Table 4: Parameters for calculating flood volume and peak discharge with the SCS approach

Name	BasinID	AREA_ha	CN [-]	S [mm]	Qv [mm]	Length [m	Min [m]	Max [m]	Slope [%]	S (ret)	tc [min]	Qp [m³/s]
Valley	0	5.3	88	34.6	9.225	554	1251	1279	5.08	1.36	0.17	0.20
Urban edge	2	7.2	75	84.7	1.747	474	1233	1260	5.1	3.33	0.23	0.05
Valley	3	3.3	88	34.6	9.225	294	1276	1300	6.9	1.36	0.09	0.13
Valley	5	5.4	88	34.6	9.225	618	1292	1342	8.67	1.36	0.14	0.20
Urban	6	13.6	70	108.9	0.578	770	1213	1245	5.28	4.29	0.39	0.03
Valley	7	19.7	88	34.6	9.225	804	1243	1298	6.39	1.36	0.21	0.75
Valley	8	23.1	88	34.6	9.225	1238	1324	1430	10.26	1.36	0.23	0.88
Urban	9	13.6	70	108.9	0.578	772	1231	1264	5.54	4.29	0.38	0.03
Valley	10	18.8	88	34.6	9.225	1164	1276	1373	9.53	1.36	0.23	0.72
Valley	11	4.3	88	34.6	9.225	577	1325	1382	9.91	1.36	0.13	0.16
Urban outskirts	12	17.2	75	84.7	1.747	799	1244	1294	7.09	3.33	0.30	0.12
Valley	13	9.4	88	34.6	9.225	499	1379	1429	9.94	1.36	0.11	0.36
Valley	14	6.9	88	34.6	9.225	660	1357	1417	10.05	1.36	0.14	0.26
Valley	16	18.8	88	34.6	9.225	1110	1280	1372	9.91	1.36	0.22	0.72
Valley	17	16.4	88	34.6	9.225	1362	1418	1547	10.6	1.36	0.25	0.63
Outflow Caritas site	18	32.3	88	34.6	9.225	1171	1362	1550	26.35	1.36	0.14	1.24
Mountain+valley	19	47.3	88	34.6	9.225	1442	1445	1737	28.52	1.36	0.16	1.81
Mountain	20	59.4	80	63.5	3.704	1107	1564	1886	41.14	2.50	0.14	0.91
Caritas site	21A	28.2	65	136.8	0.050	1685	1451	1754	33.61	5.38	0.33	0.01
not cultivated hill side	21B	28.2	80	63.5	3.704	1685	1451	1754	33.61	2.50	0.22	0.43

1.5.3 Hydrological modelling

Generally, modelling has become the state-of-the-art approach in hydrology, in flood management and in designing flood measures. Applying a model is advisable as it is able to better reflect the physical characteristic of a watershed. Provided that the concept of modelling is fully understood and parameters are available and wisely used, it results in higher accuracy. A higher accuracy is also a very relevant economic factor. The rational method and to a lesser extent the SCS approach incorporate safety factors to address the simplifications embedded in the approaches, which, of course, result in lager dimensions when it comes to designing measures. Economic viability is often a matter of balancing acceptable risk and provision of flood mitigation. A better understanding of processes and their interplay in combination with a higher accuracy foster viability and risk-informed decision-making. The modelling approach is explained in Section 2.9.1. The model Talsim-NG (www.sydro.de) is applied.

Step 1: Generating the flow network

The stream network and the locations of the sub-basins are used to compose the flow network of the model. Each model has its own approach but commonly sub-basins and river reaches are the elements used to construct the flow network.

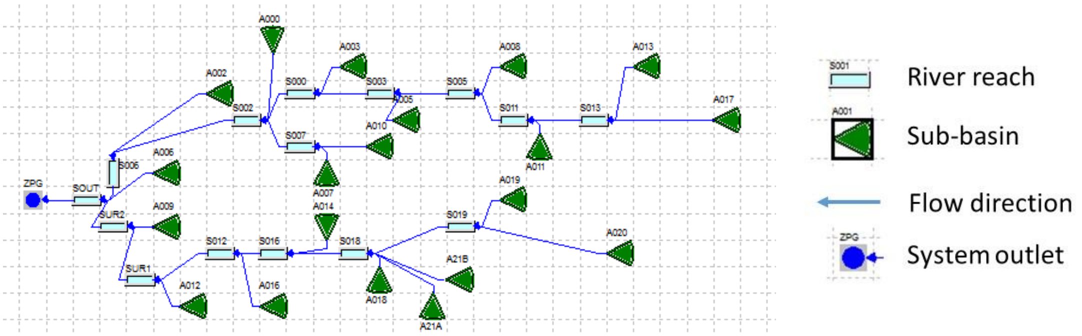
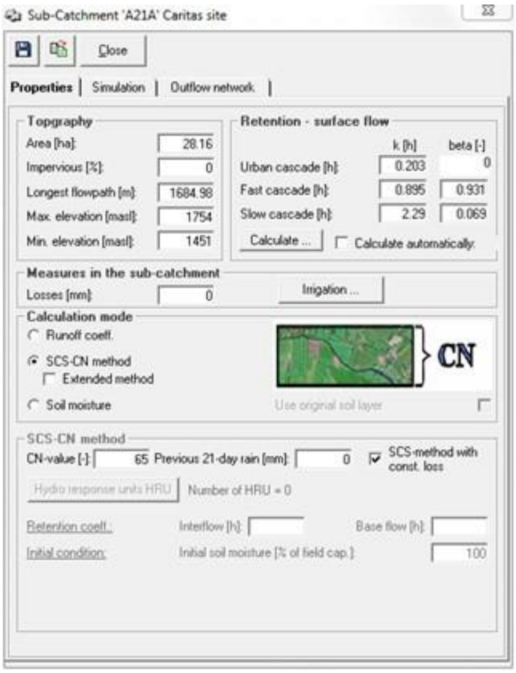
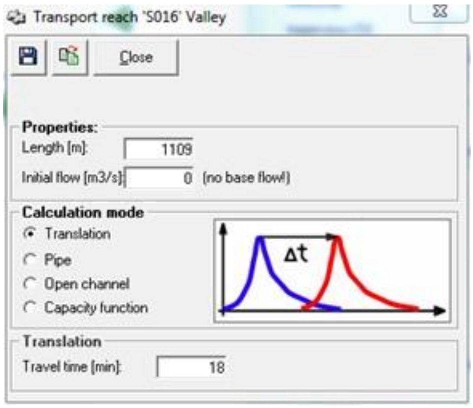


Figure 12: Flow network of the example site based on Talsim-NG

Step 2: Parameters

The user must enter the parameters for all elements. The Talsim-NG model is equipped with a graphical user interface which guides the user through the application. As the model is scalable, it offers different modes for computing sub-basins and river reaches, for example, a sub-basin can be modelled with a simple runoff coefficient, the SCS approach (as it was used here) and complex soil-moisture accounting.





River reaches can be modelled as:

- Translation
- Pipe
- Open channel
- Stage-discharge curve

Figure 13: Graphical user interface for sub-basins and river reaches - Talsim-NG

Step 3: Simulation

Simulation requires to setup the model stress in form of rainfall. In Section 1.3 a rainfall depth of 30 mm/h was selected. The storm profile, which is the distribution of the rain within the 60 min, must be determined.

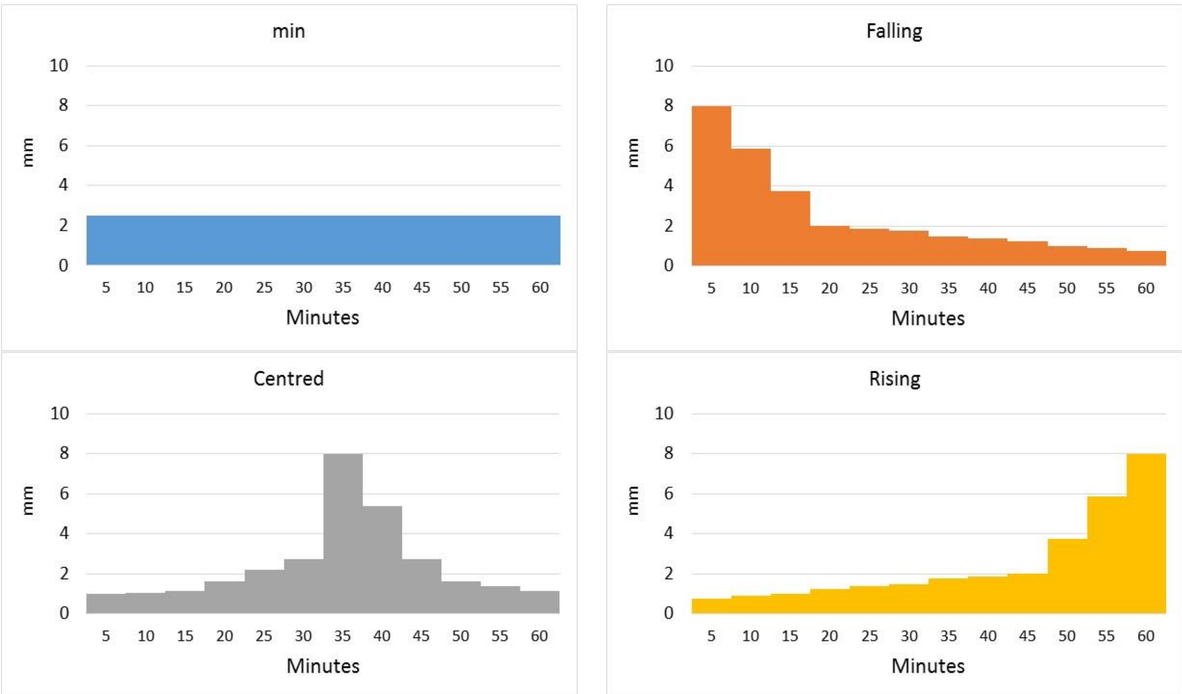


Figure 14: Distribution or storm profile of rainfall within the 60 min rainfall duration

A uniform distribution is applied. Results of the simulation are illustrated in Section 2.9.1.

1.5.4 Peak discharge and flood volume

All three above mentioned approaches yield the peak discharge. The values are shown below. As expected, the rational method owns the largest safety factors to compensate uncertainty and results in the highest values. SCS comes second and the model approach shows the smallest peak discharges. This is understandable as the hydrological model transforms not only rainfall in runoff but also allows for overland flow and transport in the river reaches. This slows down runoff as it happens in reality and gives smaller peak flows.

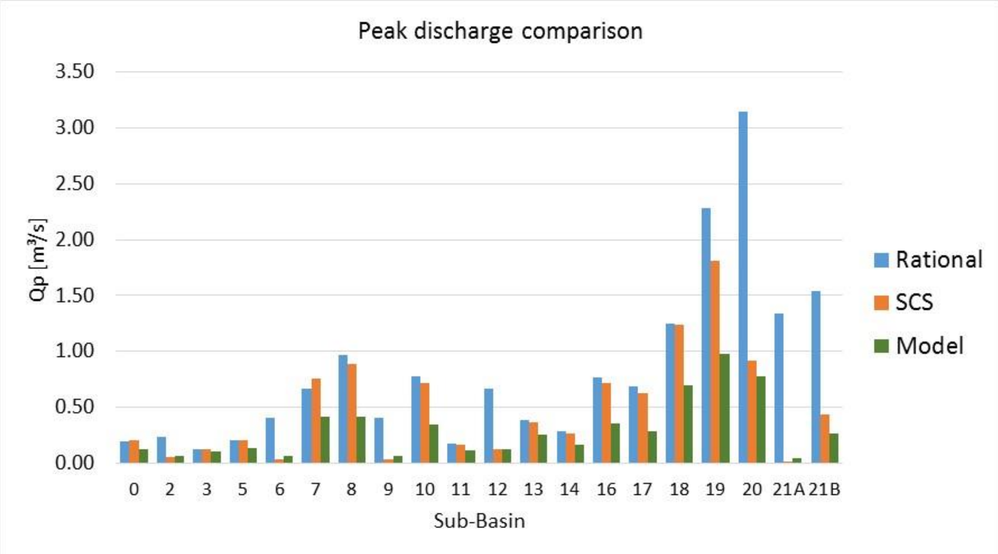


Figure 15: Comparison of peak discharge for the sub-basins

Table 5: Comparison of peak discharge

	Sub-Basin		Rational	SCS	Model
Name	BasinID	AREA_ha	Qp [m³/s]	Qp [m³/s]	Qp [m³/s]
Valley	0	5.3	0.19	0.20	0.12
Urban edge	2	7.2	0.23	0.05	0.07
Valley	3	3.3	0.12	0.13	0.10
Valley	5	5.4	0.21	0.20	0.13
Urban	6	13.6	0.40	0.03	0.07
Valley	7	19.7	0.66	0.75	0.42
Valley	8	23.1	0.97	0.88	0.41
Urban	9	13.6	0.40	0.03	0.07
Valley	10	18.8	0.77	0.72	0.34
Valley	11	4.3	0.18	0.16	0.11
Urban outskirts	12	17.2	0.66	0.12	0.13
Valley	13	9.4	0.39	0.36	0.25
Valley	14	6.9	0.29	0.26	0.17
Valley	16	18.8	0.77	0.72	0.35
Valley	17	16.4	0.69	0.63	0.29
Outflow Caritas site	18	32.3	1.25	1.24	0.70
Mountain+valley	19	47.3	2.28	1.81	0.98
Mountain	20	59.4	3.15	0.91	0.78
Caritas site	21A	28.2	1.34	0.01	0.04
not cultivated hill side	21B	28.2	1.54	0.43	0.26

The SCS method assumes a triangular flood hydrograph and the hydrological model computes a hydrograph according to the topography, soil and land cover parameters. The rational method gives no hydrograph at all.

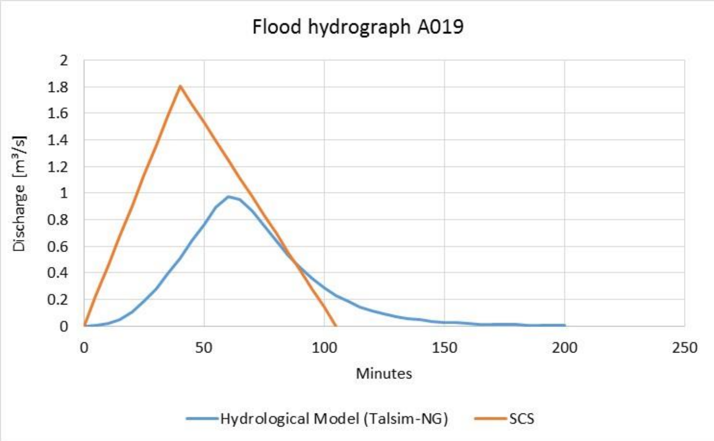


Figure 16: Flood hydrograph displayed for sub-basin 19

Considering that the flood hydrograph is not only relevant at one location, there is the need to overlay the flood hydrograph from different sub-basins and to assess peak discharges and flood volumes further downstream up to the settlement. Only the modelling approach propagates the flood from upstream to downstream automatically. The other approaches require assumptions with respect to the time of travel along the stream network.

A pragmatic way of propagating hydrographs along the stream network is to calculate the distance from the sub-basin up to the point of interest, to apply an appropriate formula for flow velocity from Section 2.6 or 2.10 for a mean cross-section of the stream and to calculate the flow velocity for the mean flow of the hydrograph. Subsequently, the time of travel can be calculated with the flow velocity and the distance from the sub-basin up to the point of interest. An example is given below for a distance of 300 m and a mean flow velocity of 0.5 m³/s.

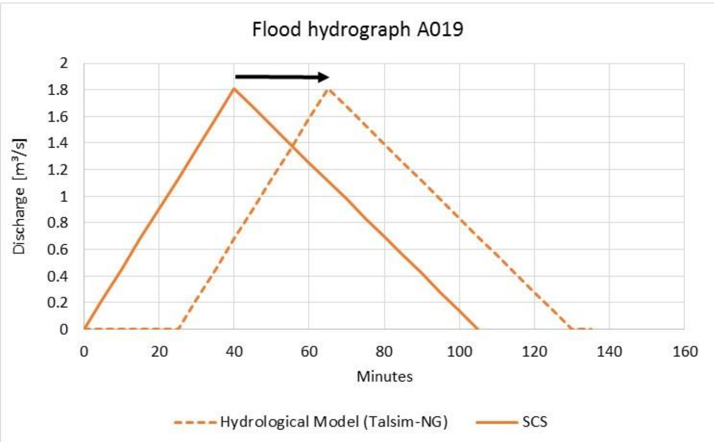


Figure 17: Simple translation of a flood hydrograph along a stream

Translation and retention through the watershed is given by the hydrological model automatically, depending on the calculation modes and parameters applied.

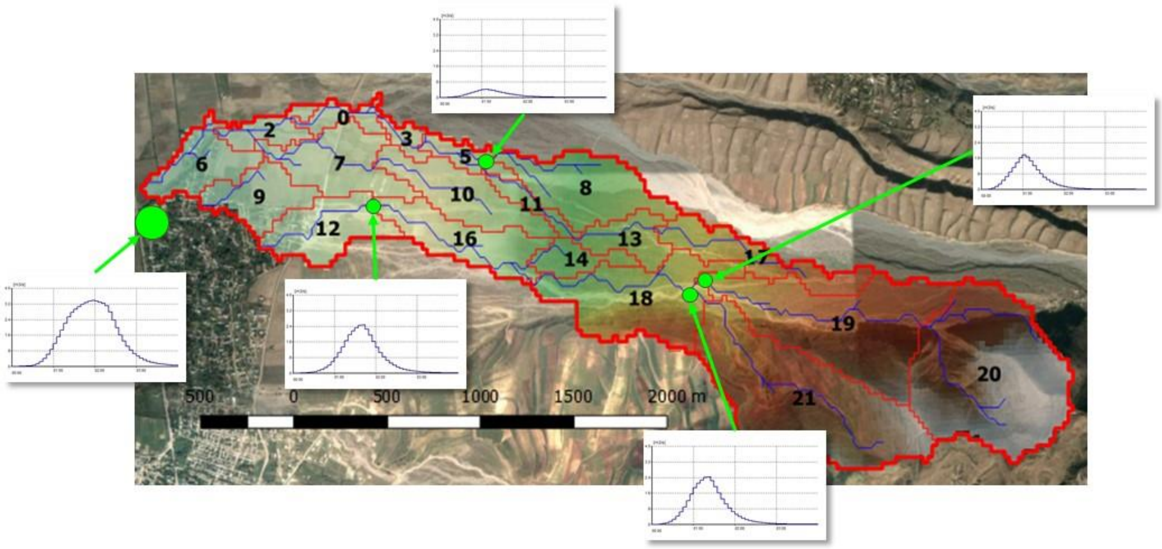


Figure 18: Flood hydrographs of the hydrological model at various nodes in the watershed

1.6 Flood inundation and risk map

Before any decision regarding measures can be made, it is necessary to identify the risk of flooding and to draw an inundation map from which informed decision-making can start.

There are two options:

1. Calculating water levels manually
2. Running a hydraulic model

1.6.1 Calculating water levels manually

Step 1: Identification of adequate cross sections

From the maps and stream network developed in GIS, the right locations for cross-sections can be identified. It makes sense to select locations which affect assets like settlements, vulnerable places of value etc.

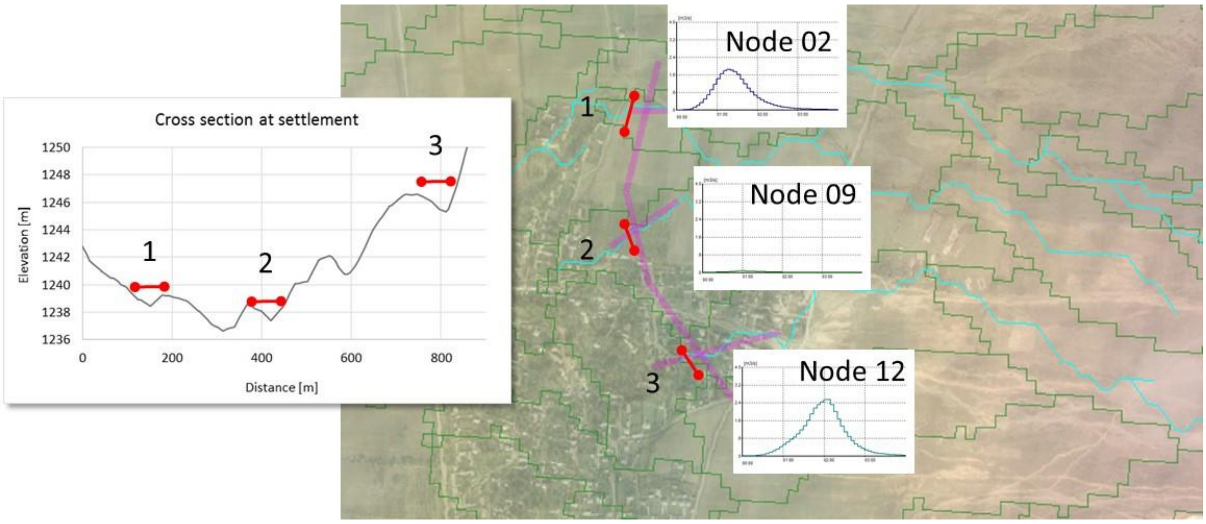


Figure 19: Identification of relevant cross sections for hydraulic computation

Three streams discharge into the settlement. According to the hydrological model, cross section 1 and 3 obtain higher flows compared to 2. Cross-section 1 is demonstrated.

Step 2: Calculating water levels

From the hydrological model, we obtain at node 02 a peak discharge of 1.8 m³/s. The stream discharge may not be mixed up with the outflow of a sub-basin. In contrast to sub-basins, the stream discharge at a node gives the accumulated flow from all upstream sub-basins.

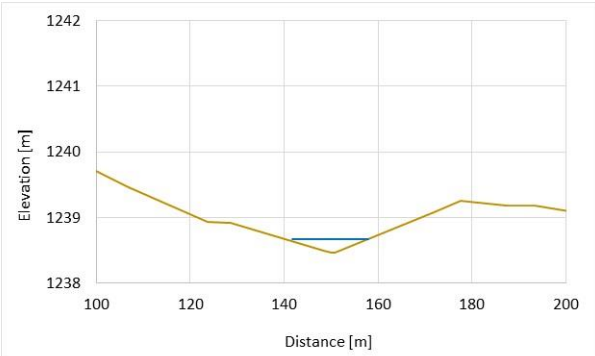
The water level is calculated by $v = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$. A roughness coefficient of n = 0.05 is used for natural channels with a stream bed consisting of gravel, cobbles and few boulders. The slope at this cross section was taken from the slope map and is Is = 0.033.

Left Bank		Right Bank	
Angle	0.02	Angle	0.03
W (m)	9.01	W (m)	7.36
WP (m)	9.02	WP (m)	7.36
A (m²)	0.97	A (m²)	0.80

Cross-Section	
A (m²)	1.77
WP (m)	16.37
R (m)	0.108062

Angle of the left and right bank refers to the gradient of the river banks. W is the horizontal width measures from the lowest point in the cross section and WP is the wetted perimeter. R is the hydraulic radius (A/WP).

Using the function is best done with Excel as the formula requires an iteration.
The resulting water level is 22 cm and the flow velocity is 1 m/s which is quite fast.



The result indicates that the water level itself is not very high but still can cause problems if it reaches doorsteps or when items block the drainage path and increase the water level. However, the flow velocity could give rise to problems. A speed of 1 m/s exerts enough energy to wash items away which are not fixed or cause a threat for children.

1.6.2 Running a hydraulic model

Step 1: Model setup

The model setup requires the determination of a suitable cell size which suffice the needs and is appropriate to obtain reliable and stable results. This is not always easy in steep terrain. For simplification a 10x10 m grid was chosen.

The image below shows the boundary of the hydraulic model and the maximum extent of inundation for a 50 year return period with 30mm or rain within one hour. Light blue indicate cells where either water level was over 10 cm or flow velocity was over 0.1 m/s.

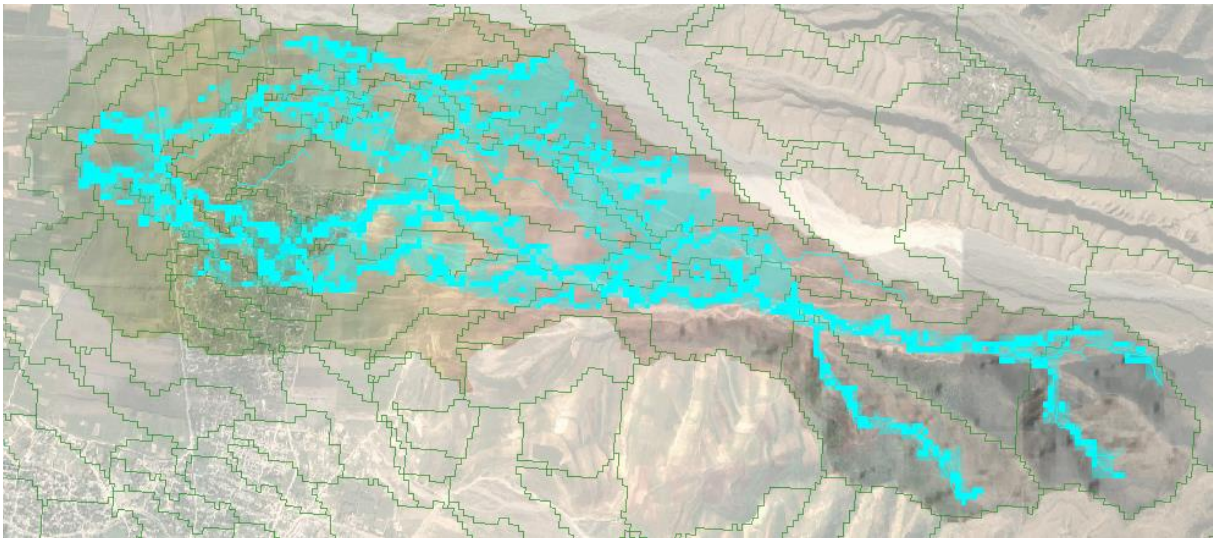


Figure 20: Inundation map of a 50 year return period rainfall with 30 mm within one hour (50a/1h)

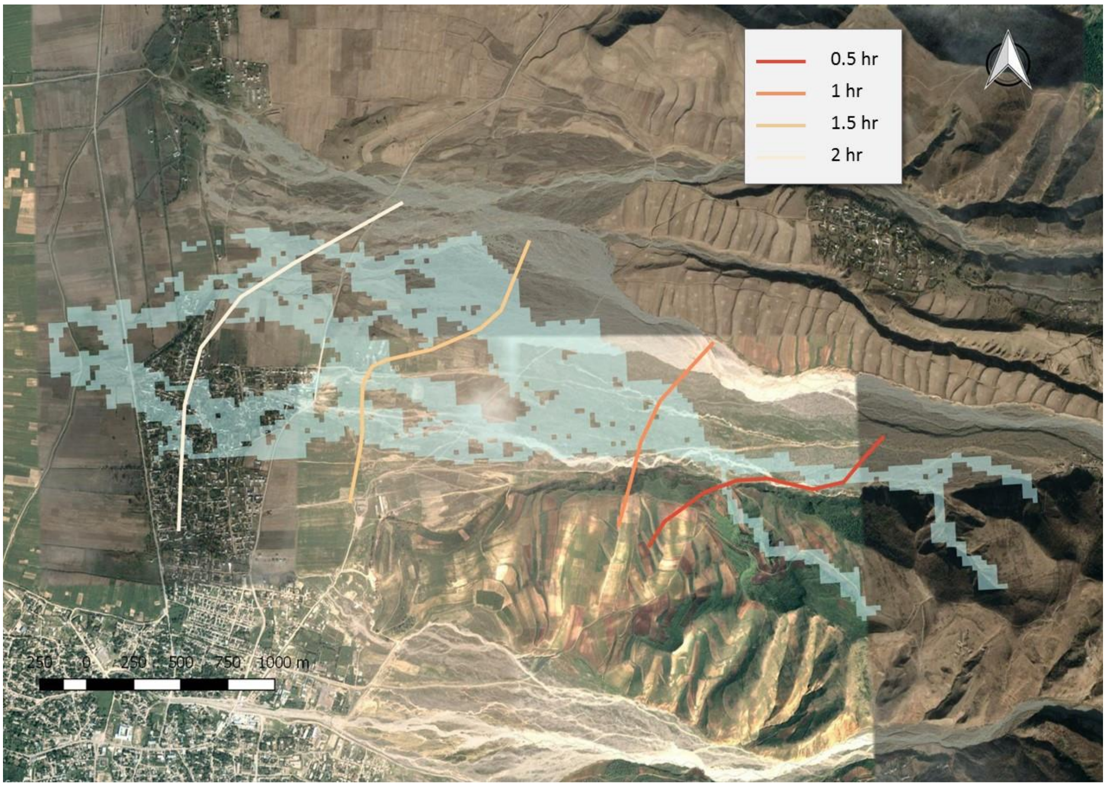
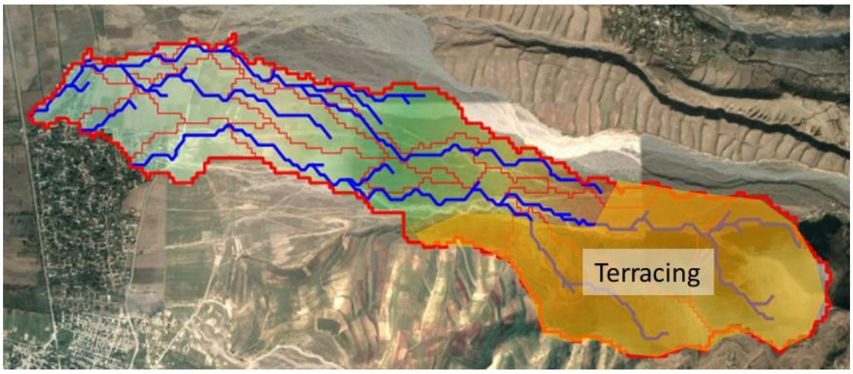


Figure 21: Arrival time of the flood peak after heavy rainfall (50a/1h)

1.7 Watershed management - terracing

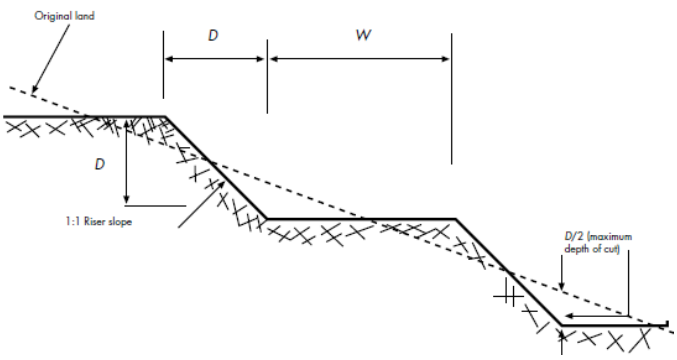
Terracing is the technique of converting a slope into a series of horizontal step-like structures. It is very effective and was applied in Tajikistan.



Typical areas for terracing are steep headwater regions where erosion largely originates.

Figure 22: Headwater area for watershed management measures

From the viewpoint of flood control, the aim of terracing is to slow down surface runoff and to convey it to a suitable outlet with non-erosive velocity. Depending on the form of terraces, additional effects are the trapping of soil in the terraces and the preparation of land suitable for cultivation.

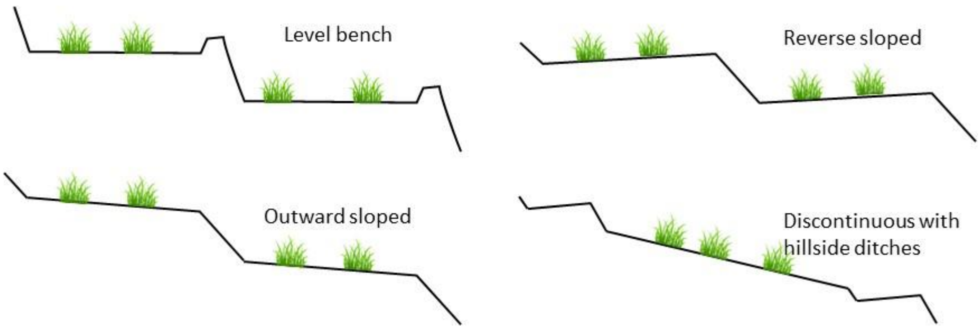


The design of terraces requires considerations in regard to the type, width, spacing between terraces, height of the riser, length (perpendicular to slope) and so on.
Design considerations should also include hydraulic calculation of runoff for safe drainage.

Source: (ICIMOD, 2012)

Not all aspects of terracing can be examined here. More information can be found in (FAO, Watershed management field manual: Slope treatment measures and practices, 2017).

The type of terraces are selected according to slope, soil and rainfall. Terraces can either strictly follow contour lines (contour terraces) or have a gradient perpendicular to the thalweg so that runoff runs along the terrace.



Rule of thumb:
Reverse slope = 5%
Outward slope = 3%
(FAO, 2017)

Figure 23: Type of terraces according to (FAO, Watershed management field manual: Slope treatment measures and practices, 2017)

Soil depth limits the width and thus the spacing of terraces. The shallower the soil layer, the smaller is the width of the terraces. According to (ICIMOD, 2012) and (FAO, Watershed management field manual: Slope treatment measures and practices, 2017) the following considerations and formulas assist in planning and designing terraces.

As a rule of thumb, level bench, reverse and outward sloped terraces are applicable with deep soils and slopes up to 25° while discontinuous terraces with hillside ditches may be feasible up to 30°. Level bench terraces are good for crops like rice which require flood irrigation and impounding water. Reverse sloped types are more suitable in humid regions and outward sloped types in arid or semi-arid regions. Discontinuous types are less labour-intensive.

Spacing of the terraces can be estimated according to (FAO, Watershed management field manual: Slope treatment measures and practices, 2017) by using the formula:

$$VI = \frac{S \cdot Wb}{100 - (S \cdot U)}$$

where:

VI:	Vertical Interval [m]
S:	Slope [%]
Wb:	Width of bench excluding the width of the riser [m] (is indicated above as W)
U:	Slope of riser (with 1 for machine-built terraces, 0.75 for hand-made earth risers and 0.5 for rock risers)

The volume required due to cut and fill is computed as

$$Vol = \left(\frac{Wb \cdot VI}{8} + DC \right) \cdot L$$

where

Vol:	Volume to be cut and filled [m³]
VI:	Vertical Interval [m]
DC:	Mean cross-section of the dyke along the length L [m²], if any
L:	Length of the terrace [m]

(FAO, 2017) recommends building the terraces from top of a hill and proceed downslope. If building has to start from the bottom, temporary protection measures are necessary to avoid soil is washed away in case of heavy rain.

Scheduling the work requires to estimate the effort of time needed. Generally speaking, a man can cut and fill 3 to 4 m³ of earth with eight hours of work. Supported by draught animals, FAO indicates 12 to 14 m³ within 8 hours what can be increase to 20 m³ or more when using small machinery.

When the layout of a terrace system is made, it is necessary to proof safe drainage in terms of hydraulic capacity and erosion. An example is given how to calculate runoff and to check the hydraulic capacity and erosion. The formulas given in Section 2 are applied.

Example:

The examples uses a bench terrace (reverse slope) with a length of 140 m with a hillside of 16% gradient. The width is set to 5 m. The riser has a height of 1 m and a slope of 0.75:1 (man-made earth riser). The traverse slope of the terrace is 5% and the soil type is loamy silt.

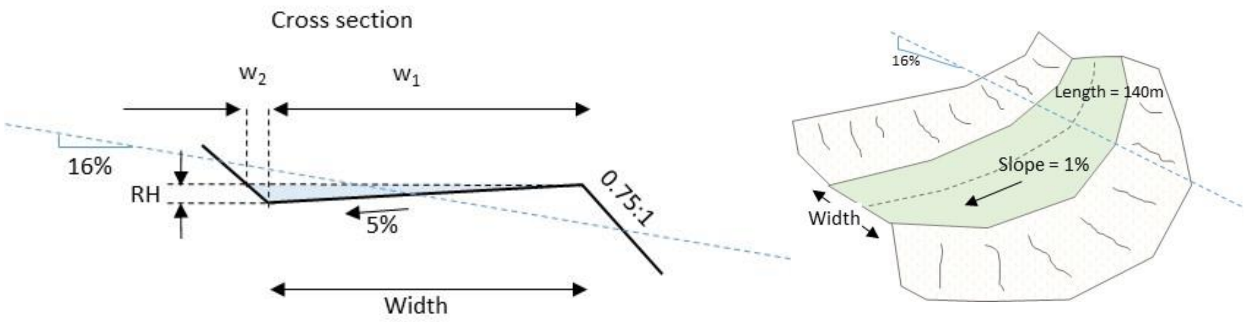


Figure 24: Example bench terrace

The reverse height is calculated to RH = Width x Traverse slope = 5 x (5/100) = 0.25 m

The drainage area of the terrace is A = Width x Length = 5 x 140 = 700 m² = 0.07 ha.

For the peak discharge, the SCS approach and the rational method is used. We assume rainfall of 50 mm within one hour as a 10 year storm. Two different stages are calculated.

- Stage 1: bare soil, not yet cultivated (CN=90, n = 0.02, runoff_coef = 0.1)
- Stage 2: vegetated with grass (CN=70, n = 0.035, runoff_coef = 0.05)

First, the peak discharge is computed. For the SCS method the potential retention S = 25.4*(1000/90-10) results in 28.2 mm. With 50 mm or rainfall the runoff volume Qv = (50 – 0.2*S)^2 / (50 + 0.8*S) is 27.1 mm. Time of concentration with the Kirpich formula gives tc = 0.066 hour. Applying the formula requires the conversation factor 0.3048 for the overflow length. The peak discharge Qact according to the SCS approach results in 0.007 m/s or 7 l/s.

When using the rational formula with a rainfall intensity of 50mm/hr and a runoff coefficient of 0.1, which represents an undeveloped plain area, the peak discharge amounts to 0.010 m³/s or 10 l/s. The results show that selecting the CN value or runoff coefficient are sensitive parameters.

SCS method: Qact = 0.007 m³/s; Rational Method: Qact = 0.01 m³/s

If the cross-section is large enough to carry the peak discharge can be answered by comparing the actual discharge with the maximum capacity. The Manning formula is applied to compute the flow velocity from which the maximum carrying capacity can be derived. The flow cross-section in Figure 24 is indicated as blue.

The maximum cross section is: Amx = (0.5 * w1 * RH) + (0.5 * w2 * RH) = 0.667 m²

with w1 = RH /5 m; w2 = RH / RHslope = 0.25/0.75 = 0.33 m

The maximum wetted perimeter $P = \sqrt{w1^2 + RH^2} + \sqrt{w2^2 + RH^2}$ = 5.42 m

With the Manning coefficient n = 0.02 (≈ earth channel) and
the hydraulic radius rhy = Amx/P = 0.667/5.42 = 0.123 m,
the maximum flow velocity is $v = (1/0.02) \cdot 0.123^{\frac{2}{3}} \cdot 0.5^{\frac{1}{2}}$ = 0.87 m/s.

The maximum carrying discharge capacity is now Qmx = v * Amx = 0.87 * 0.667 = 0.583m³/s

It can be concluded that Qmx > Qact and the cross section is large enough during stage 1.

Stage 1 is the phase with bare soil. Loamy silt has a critical flow velocity ranging from **0.1 to 0.2** m/s. In order to compare the critical flow velocity with the actual flow velocity, the actual flow cross section must be computed. This requires iteration with the flow depth h as h determines the cross-section.

The underlying formulas are:
$w2 = h/0.75$ (depth/slope of riser) and $w1 = h/0.05$ (depth/traverse slope) from which A can be calculated as $A = (0.5 * w1 * h) + (0.5 * w2 * h)$ and $P = (w1^2+h^2)^{0.5} + (w2^2+h^2)^{0.5}$ and $r_{hy} = A/P$.

The result is a depth h of 0.054 m. This results in v_{act} = 0.32 m/s and $v_{act} > v_{crit}$. Erosion would occur during stage 1 with a storm with 50 mm.

In **stage 2** is the terrace developed with grass. The following parameters change:

CN value = 70, Manning's roughness n = 0.035, Runoff coefficient = 0.05, critical v = 1.5 m/s

The SCS method yields 0.002m³/s and the rational method 0.005 m³/s. The resulting actual depth is 0.05 m and gives an actual flow velocity of 0.17 m/s which is less than the critical 1.5 m/s velocity.

Once the terrace is fully developed the grass can withstand a rainfall event of 50mm within one hour.

1.8 Check Dams

In regions with heavy rains, watershed management alone will not suffice to control erosion, gullies and torrents. Additional slope stabilization, torrent and gully control measures, such as check dams, ground sills, bed ramps are needed. Check dams are typically sited in steep tributaries with high sediment loads.

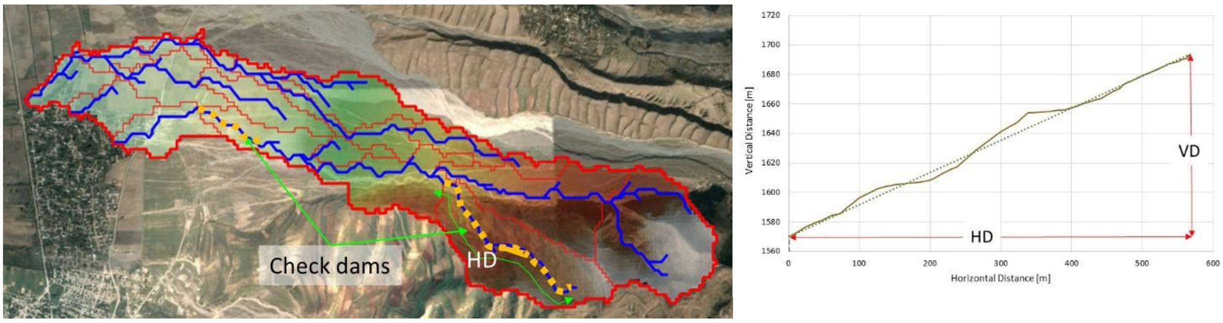


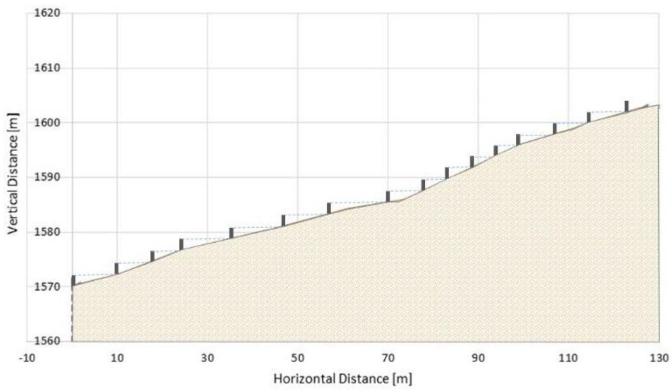
Figure 25: Transport reaches suitable for check dams

After the identification of suitable sections, the survey of the longitudinal profile starts. For developing this example, the reach indicated with HD is used.

A practical instruction regarding check dams is given in (FAO, Gully Control, 1986) from which basic concepts are adopted.

Spacing of check dams can be determined according to the compensation gradient and the effective height of the check dams. The compensation gradient between two adjacent check dams is the slope measured from the top of the lower check dam to the bottom of the adjacent upper one. This is considered as a slope which is formed when material carried by flowing water fills the check dams to spillway level and keeps a balance between erosion and sedimentation. Formulas have been developed to compute the compensation gradient. However, field experience has demonstrated that the compensation gradient of gullies is usually not more than 3 percent. For practical reasons, 3 percent are used for estimating the number of check dams along a gully (FAO, Gully Control, 1986).

The average gradient is calculated with $g_{avg} = VD / HD$. The number of check dams is then estimated by $N = \dfrac{HD \cdot g_{avg} - VD \cdot 0.03}{H}$ where H as is the effective height (excluding foundation) of the check dams. The taller the effective height is, the less is the number of check dams. A decision has to be made regarding more and smaller check dams, which are easier to be built, or less and taller check dams, which require more effort for construction.



The average gradient of the stretch is 22%, the compensation gradient is chosen to 3% and with an effective height of 1.5 m, the whole stretch of 570 m horizontal distance (HD) with 123 m vertical distance (VD) would require approximately 80 check dams. A section of 130 m is illustrated with the compensation gradient from which the number of check dams was estimated.

The first check dam should be constructed on a stable point in the gully such as a rock outcrop, the junction point of the gully to a road, the main stream or river. If there is no such stable point, a counter-dam must be constructed. The distance between the first dam and the counter-dam must be at least two times the effective height of the first check dam (FAO, Gully Control, 1986).

Example:

The hydrological parameters in the reach are derived from the sub-basin 21 A and B. The peak discharge is given to:

Sub-basin	ID	Area ha	Rational Qp m³/s	SCS Qp m³/s	Model Qp m³/s
Caritas site	21A	28.2	1.34	0.01	0.04
not cultivated hill side	21B	28.2	1.54	0.43	0.26
Peak discharge Qp			2.84	0.44	0.3

A cross section is selected for which the check dam is calculated.

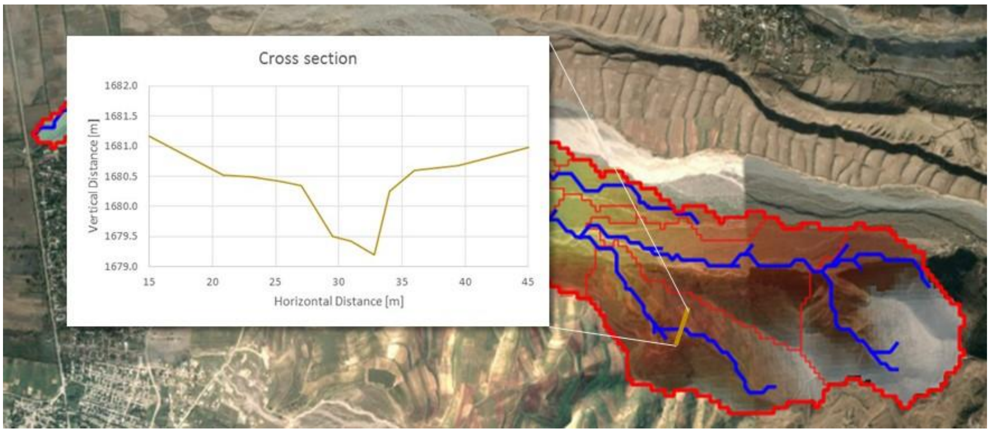


Figure 26: Cross-section in a river reach for developing check dams

Applying the criteria for check dams given in Part I, a cross-section with a check-dam could look like this:

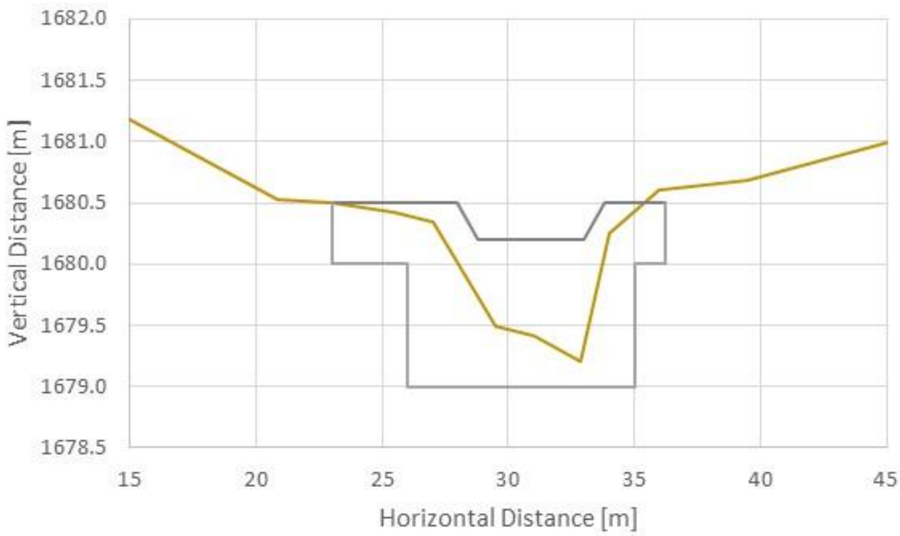


Figure 27: Cross-section of the check dams

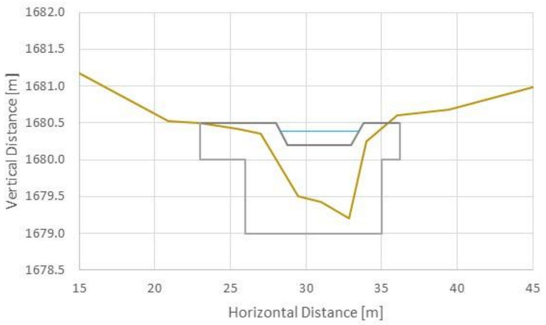
A gabion dam with one layer of 0.5 m height with three layers (1.5 m height) is chosen. The wings reach into the banks and the foundation is anchored one gabion deep into the stream bed. The spillway is considered as broad crested weir. With a peak discharge Qp of approximately 0.45 m³/s (= SCS approach), the necessary height of the spillway is calculated by using:

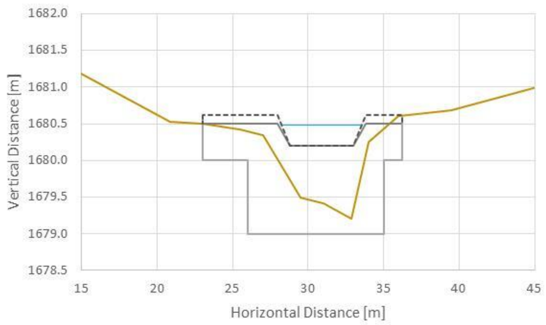
$$Q = \frac{2}{3} \cdot \mu \cdot c \cdot w \cdot \sqrt{2 \cdot g} \cdot h^{1.5}$$

where:

µ:	coefficient [-]
c:	factor for broad crested weir [-]
w:	width of the spillway (here assumed as rectangle) = 4.2 [m] (results as constraint due to the width of the cross-section)
g:	gravity [m/s²]
h:	overflow height [m]

With Q = 0.45 and w = 4.2 m the height results in 0.18 m. This gives enough freeboard for the selected design event. About 50% of the sub-basin was re-vegetated, terraced and developed due to the watershed management measures developed by Caritas Switzerland. What if no watershed management were in place? The resulting overflow height rises 10 cm to 0.28 m and is shown in the right picture below. No freeboard is left and the check dam had to be build higher to achieve the same freeboard. It is possible that elevating the crest level requires a new gabion layer as no standard size fits the change in height.





With the watershed management due to Caritas Without watershed management

Figure 28: Cross-section of the check dam with/without watershed management upstream

The positive effects of watershed management affects all check dams which are to be developed. In other words, without watershed management, all check dams, gabions or boulder check dams, had to be higher causing more material to be used for the structure, more effort for construction, more labour force and higher costs.

Many examples can be found demonstrating positive effects of watershed management. A lesson learnt is that watershed management measures always have to be developed, no matter which hard or soft measure is chosen downstream.

More than one check-dam needs to be developed but not all are gabion dams. The first check dam downstream will be developed as a gabion dam together with the counter check dam. Most of the other check dams upstream can be developed as boulder dams, ideally fortified with logs or other sturdy material.

1.9 Longitudinal structures and streambed stabilisation

Part of the settlement was erected in the direction of flow paths coming down from the catchments in the south of the example site. The flow paths are usually dry, but with heavy rain, flash floods can occur which come down the flow paths exerting destructive forces on buildings and other infrastructure.

The situation calls for the development of a diversion channel with longitudinal protection structure and stream bed stabilisation diverting a flash flood into the main channel to the right in direction of flow but mainly provides a protection against high sediment loads. The slope in the alluvial fan offers options for erecting an embankment. However, the measure may interfere with some paths used as access roads to get into the headwater area. This must be considered and dirt roads need adjustment.

This example measure was chosen to demonstrate both ramps for reducing gradients and embankments.

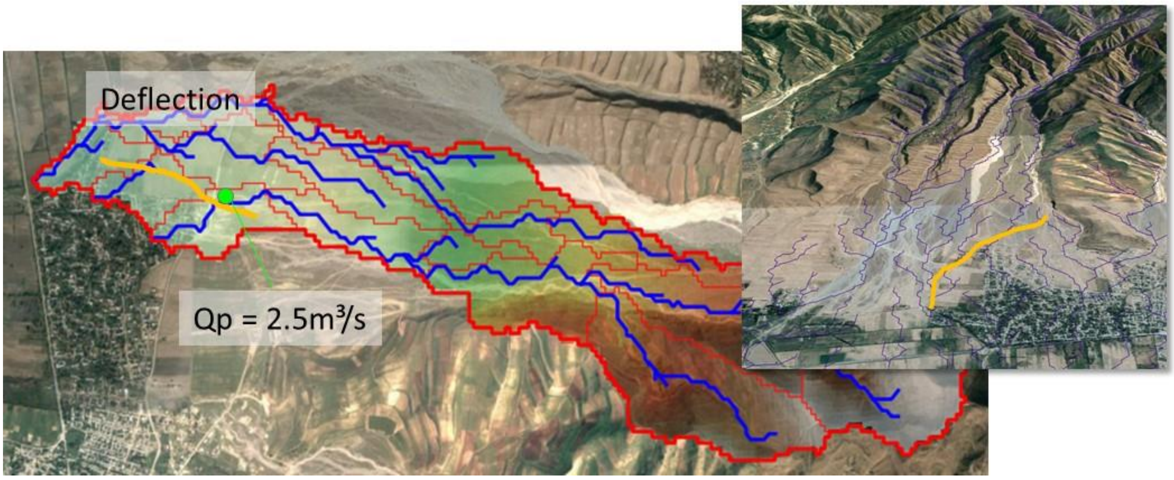
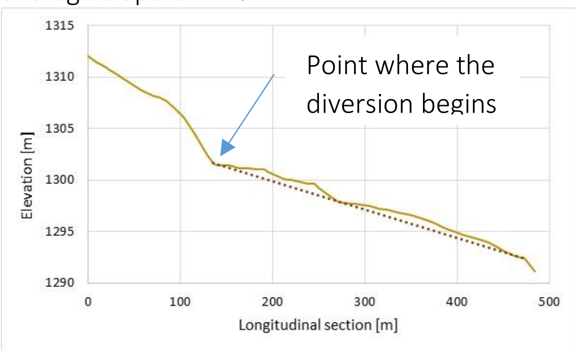


Figure 29: Area for developing a diversion channel

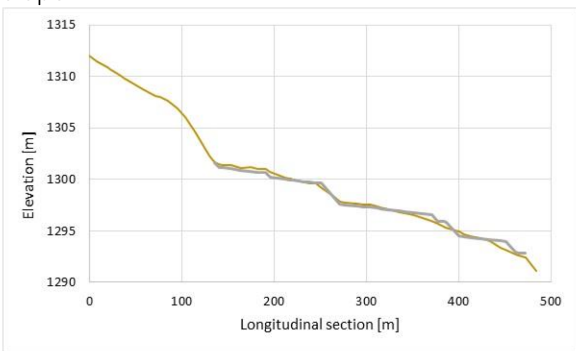
At the impact point, indicated with a green dot, the peak discharge of a 50a / 1hr rain results in 2.5 m^3/s derived from the hydrological model. The task is to develop a suitable longitudinal section with an adequate gradient and suitable cross-section for facilitating the peak discharge without erosion.

The natural soil characteristic along the suggested diversion channel is fine to coarse sand mixed with coarse gravel.

Diversion profile according to the terrain with an average slope of 2.7%



Diversion profile developed with ramps and 1% slope



Given the material of the underground, the terrain with an average slope of 2.7 % would result in erosion incising the diversion channel and destabilising the banks. A reduction in slope is needed by developing the new profile with ramps. Only the ramps require stone packages while the rest of the profile could be developed according to Figure 30.

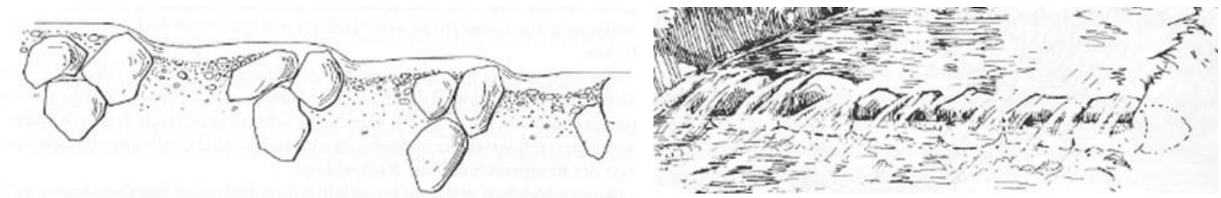


Figure 30: Natural river bed developed with cascade of boulders according to (Patt, 1998)

The water level should be developed with less than 1% with a boulder cascade resembling the diversion channel as a natural mountainous riverbed. In addition, seven ramps are necessary to bridge the vertical difference to the target riverbed north of the settlement. The profile of one ramp is illustrated as an example.

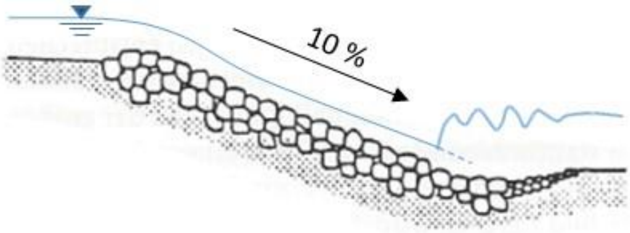


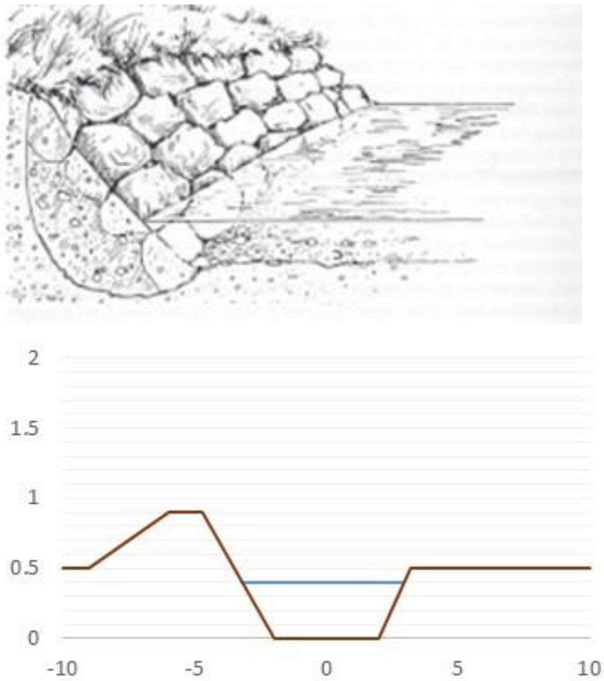
Figure 31: Profile of a streambed ramp given as an example taken from (Patt, 1998)

Alternatively, the implementation of ramps could be avoided with a longer diversion channel that meanders from A to B. However, the diversion must reach more than 1000 m length to result in a slope less than 1%, thus, this option is not further developed or illustrated in this example.

Measure:
Diversion channel with embankment left river bank.
Length: 336 m
Average slope: 2.7%
Developed with boulder cascade and 7 ramps
Water level < 1% gradient,
Ramps = 10%

Figure 32: Diversion channel with embankment on the left bank

The cross-section of the diversion channel



Riverbank protection according to (Patt, 1998)

Cross-section:
- Bed material:
  medium to coarse sand and gravel,
- boulders d>25 cm
- bottom width: 4 m
- bank slope: 1:3,
  Developed with stone packages d>25 cm

Figure 33: Cross-section of the diversion channel

The hydraulic calculations are as follows:

The calculation uses the Manning Equation (Section 2.6) and computes sheer stress according to Section 2.7. Critical sheer stress and critical flow velocity is taken from the tables in Section 2.7.

The discharge used to derive the geometry is 2.5 m^3/s (50 a return period, 30mm rain in 1 hour). The results require adaption if a larger return period or other rain events are applied.

1. Stability of the bed material and size of the boulders in the cascade

The bed material is assumed to consist of medium to coarse sand and gravel. According to Table 12 critical sheer stress is 1 (medium sand) to 45 (coarse gravel) and critical velocity ranges from 0.35 to 1.6 m/s.

With a longitudinal slope of 1%, a cross-section with a bottom width of 4m, slopes 1:3, manning roughness 0.03, which represents a mountain stream with gravel, cobbles and few boulders at the bottom, and a discharge of 2.5 m^3/s results in a flow depth of 0.37 m and a flow velocity of 1.3 m/s. This means that sand is eroded and gradually washed out while gravel remains stable. The boulder cascade would be stable with stones of 10cm diameter reducing the energy line to less than 1% and increasing flow depth to 0.45 m. The boulders have a specific density of approx. 2650 kg/m^3 and should be hard with a lower coefficient of abrasion. Placing them into the stream bed will cause small scours downstream the stones. The stones will dig into the bottom until the underground provides enough support. From a practical viewpoint, the diameter of the cascade boulders should be 5-times the diameter of the surrounding natural bed material. Therefore, the boulders are chosen to have a minimum diameter of 25 cm.

2. Stone package of the ramps

The ramp will be developed with a slope of 10%. A diameter of 20 cm should be chosen according to the equation in Section 2.8. To be prepared for larger discharges, a minimum diameter of 30 cm is suggested. All recommendation in Section 2.8 must be regarded.

3. River bank stone package

The left river bank in direction of flow is developed as an embankment with enough freeboard to protect the settlement. The right river bank, however, is open for flooding. As such, higher discharges can be facilitated without overtopping the embankment.

Along the sections with 1% slope with boulder cascades, riprap 63/90 would suffice according to the equation for tractive force on bank material in Section 2.8. Along the ramps riprap with the same diameter like the stones for the ramp itself should be chosen.

1.10 Summary of the step by step example

The aim of this step-by-step example is to enable the reader to identify the steps required, to become aware of the different tools and hydrological and hydraulic concepts and – most importantly – to realise to which extent experts with experience should be asked for advice.

To understand the example completely requires at least basic knowledge about hydrology and hydraulics. The explanations in Section 2 help and provide some useful background knowledge but, of course, they do not replace training and experience.

It must be beard in mind that the example with the measures shown were chosen to go through all steps and to demonstrate them rather than providing a detailed solution for the particular site.

As such, selecting another return period, rainfall event or bringing up different measure and to develop them at other locations is truly possible.

2 HANDS-ON HYDROLOGY AND FLOOD MANAGEMENT

This section provides simple hydrological knowledge, approaches and formulas to enable readers to make their own calculations.

There is a need to understand underlying hydrological and hydraulic principles to identify root causes, to select adequate short-, medium- and long-term measures and to design them. The principles of torrent control and streambed stabilisation plays a crucial role. This is why a set of approaches is provided to support considerations with respect to risk assessment, planning, designing and siting of flood mitigation measures.

2.1 Runoff process and flood formation in a watershed

Hydrologic features in a watershed are interconnected and changing one usually impacts on others. To understand the formation of floods in a watershed, it is important to comprehend the runoff process and to know how human-activities affect flood volume and peak.

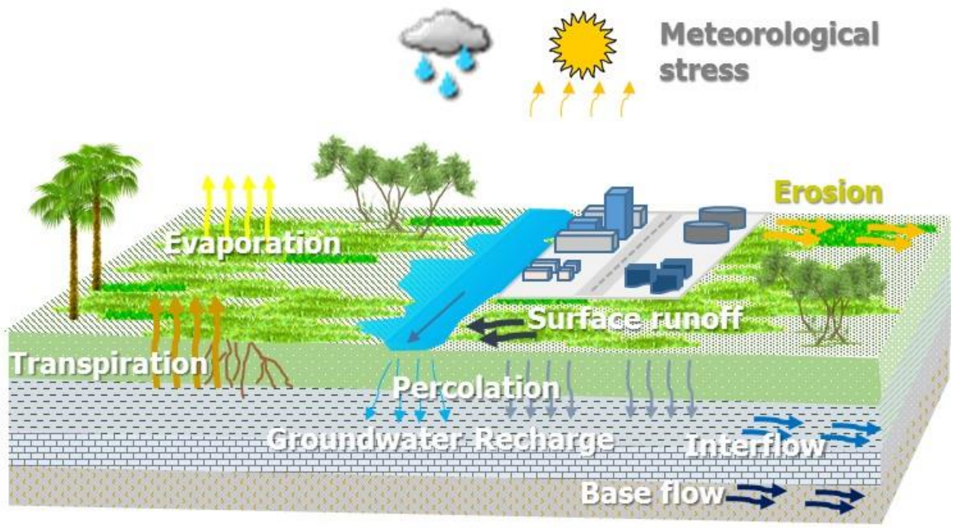


Figure 34: Hydrological processes related to runoff

The following table links hydrological features to runoff generation.

Table 6: Hydrological features impacting on runoff formation (adopted from (Maidment, 1998))

Feature	Characteristic	Runoff
Natural factors		
Topography	Steep slopes > 10°	
	Gradients > 1° and < 10°	
	Plain	
Soil	Texture with large pores and less adhesion are permeable (gravel, coarse to fine sand, silty sand)	
	Texture with small pores and medium adhesion are less permeable (silt)	
	Texture with small pores and high adhesion are nearly impermeable (loam, clay)	
	Deep soil or soil without a horizon with loam or clay	
	Shallow soil depth or soil with a horizon with loam or clay	

Feature	Characteristic	Runoff
Natural factors		
Land cover	Dense vegetation canopy with a deep root structure	↘
	Ground covered with vegetation	↘
	Ground sparsely covered with vegetation	↗
	Bare soil	↗
Human-made factors		
Urban areas	Paved surfaces (roads with tarmac or concrete, roofs)	↗
	Stones, bricks with impermeable joints	↗
	Compacted surfaces (dirt roads with car traffic)	↗
	Stones, bricks with permeable joints	→
	Planted surfaces	↘
Road drainage	No road drainage	↗
	Roads drainage with check dams	↗
	Road drainage diverted into vegetated and permeable areas	→

Apart from natural factors, land-use changes are often major drivers for an increase of runoff. Land use alterations can be understood as a root cause for increasing flood peaks, erosion, landslides and mudflows, if infiltration is impaired,. Table 7 provides a summary of hydrological impacts associated with land-use changes.

Table 7: Hydrological effects of land-use changes (adopted from (Maidment, 1998))

Land-use change	Component affected	Hydrological processes involved	Geographical scale and likely magnitude of effect
Afforestation (Deforestation has converse effects)	Annual flow	Increased interception in wet periods	Basin scale; magnitude proportional to forest cover
		Increased transpiration in dry periods through increased water availability to deep root systems	
	Seasonal flow	Increased interception and increased dry period transpiration will increase soil moisture deficit and reduce dry season flow	Basin scale; can be of significant magnitude to reduce dry season flow
		Drainage activities associated with planting my increase dry season flow	Basin scale; drainage activities my increase dry season flow
	Floods	Interception reduces floods by removing a proportion of the storm rainfall; build up of moisture storage	Basin scale; effect is generally small but greatest for small storm events
	Erosion	High infiltration rates in natural, mixed forests reduce surface runoff and erosion	Basin scale; reduces erosion
		Slope stability is enhanced by reducing soil pore water pressure and binding of forest roots	Basin scale; reduces erosion
		Windthrow of trees and weight of tree crop reduces slope stability	Basin scale; increases erosion
		Soil erosion through splash detachment is	Basin scale; increases erosion

Land-use change	Component affected	Hydrological processes involved	Geographical scale and likely magnitude of effect
		increased without understory of shrubs or grass	
		Management activities: cultivation, drainage, road construction, felling, all increase erosion	Basin scale; management activities are often more important than the direct effect of the forest
	Climate	Increased evaporation	Micro and meso scale
Agricultural intensification	Water quantity	Alteration of transpiration rates affects runoff	Basin scale; effect is marginal
		Timing of storm runoff altered through land drainage	Basin scale; significant effect
	Erosion	Cultivation without proper soil conservation measures and uncontrolled grazing on steep slopes increases erosion	Basin scale; effects are site-dependent
Draining wetlands	Floods	Drainage method, soil type, channel improvement, all effects flood response	Basin scale; open drains increase flood peak
Urbanisation	Flood volume	Impervious surfaces such as paved roads, parking lots, roofs prevent rainfall from infiltrating into the ground	Basin scale; increase of flood volume is proportional to impervious areas
	Flood peak	Surface runoff in urban areas has a higher flow velocity	Basin scale; increase in velocity, along with the increase of runoff volume and the concentration of the runoff in pipes and channels increases flood peaks significantly

The table indicates both positive and negative effects on water availability due to afforestation. This should not guide the reader into a wrong direction. It is worth noting that positive effects outstrip negative by far.

2.2 Intensity-Duration-Frequency curves (IDF curve)

An IDF curve illustrates the combination of rainfall Intensity in (mm/hr), rainfall duration and rainfall frequency. These three parameters make up the axes of the graph of an IDF curve. An IDF curve is ideally derived from long term rainfall records.

Rainfall is the driver for all discharge and design flood computations. The difficulty is to overcome the gap with respect to available rainfall records. Ground observation stations are sparse and often lack data, especially the availability of high temporal resolution less than one day is a problem. Another challenge in Tajikistan is that extreme events of rainfall often coincide with snowmelt.

A feasible way to achieve precipitation relevant to flood management are time series provided by satellite observations verified with data from ground stations. Except for TRMM or GPM data (see Part I), time series come as daily values. Even though daily values bear the risk to underestimate rain events with shorter durations than one day, they can be used for a statistical analysis from which IDF or Depth-Duration-Frequency (DDF) curves are extrapolated. However, results should be taken with care as long as no verification with observed records could be carried out.

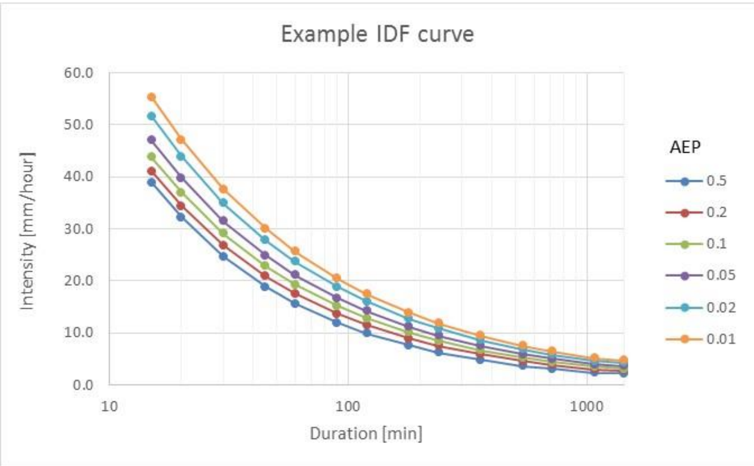
According to (Maidment, 1998), IDF curves can be described mathematically to facilitate calculations in the form:

$$i = \frac{c}{t^{c} + f}$$

where:

i:	design rainfall intensity [mm/hour]
t:	duration in minutes
c:	coefficient which depends on the exceedance probability
e, f:	coefficients depending on the location

It is recommended that coefficients for Tajikistan for various locations are developed homogeneously and to make them available for the public so that flood managers are able to apply them. The distinct advantage of a generalised approach covering the whole country is that the basis for design purposes is harmonised according to a standardised approach.



The curve is most likely underestimating the situation for durations less than 1 day due to the data base.

Figure 35: Example of an IDF-curve, developed with daily values from Khaburabad

Generally, IDF curves plotted on logarithmic scales show a strong linearity so that values equal and larger than one day can give an orientation for extrapolation towards shorter rainfall durations.

Applying an IDF curve without considering snowmelt results in an underestimation so that a safety factor should come on top. From Figure 35 a rainfall intensity of 20 mm/hr for a 60 min rainfall and a 10 year return period can be derived.

The values from Khaburabad were taken from (WB, 2017). Rainfall data can also be downloaded up to 1991 at: https://geographic.org/global_weather/tajikistan/khaburabad_853.html

2.3 Calculating runoff

Calculating runoff and deriving flood peaks and hydrographs are the first features needed to design flood mitigation measures. There are a number of approaches many of which entail sophisticated calculations and data requirements. Two widely used methods are introduced. Both need only a few parameters and are supported by a vast amount of literature and sources from where coefficients can be taken.

2.3.1 Rational Method

The simplest approach is the Rational Method which was originally developed for urban hydrology. It is a widely used approach and applies a relationship between the drainage area, rainfall intensity and a runoff coefficient representing land cover, soil types and sub-catchment slope. Its application is simple and data needs are low. The accuracy is inferior to more sophisticated and physically-based approaches and underlying assumptions and limitations must be observed.

The rational method is appropriate for estimating peak discharges for small drainage areas. The method provides the designer with a peak discharge value, but does neither provide a time series of flow nor flow volume.

The Rational method predicts the peak runoff according to the formula:

$$Q = c \cdot i \cdot A \cdot 0.00268$$

where:

Q: peak flow [m³/s]

C: runoff coefficient [-] (c is a function of the land cover, soil type and sub-catchment slope)

I: rainfall intensity [mm/hr] (the rainfall intensity is the average rainfall rate in mm/hr for a specific rainfall duration and a selected frequency. The duration is assumed to be equal to the time of concentration.)

A: sub-catchment area [ha]

Units must be taken with care and require conversion factors. The equation above calculates the peak discharge with i in [mm/hr] and area in [ha] and the factor reflects the conversion into m³/s. The runoff coefficient changes if applied to rural and mixed-use watersheds and is calculated based on four runoff components

Table 8: Runoff Coefficients for Rural Watersheds (adopted from (TxDOT, 2016)

Watershed characteristic	Extreme	High	Normal	Low
Relief - C_r	0.28-0.35	0.20-0.28	0.14-0.20	0.08-0.14
	Steep, rugged terrain with average slopes above 30%	Hilly, with average slopes of 10-30%	Rolling, with average slopes of 5-10%	Relatively flat land, with average slopes of 0-5%
Soil infiltration - C_i	0.12-0.16	0.08-0.12	0.06-0.08	0.04-0.06
	No effective soil cover; either rock or thin soil mantle of negligible infiltration capacity	Slow to take up water, clay or shallow loam soils of low infiltration capacity or poorly drained	Normal; well drained light or medium textured soils, sandy loams	Deep sand or other soil that takes up water readily; very light, well-drained soils
Vegetal cover - C_v	0.12-0.16	0.08-0.12	0.06-0.08	0.04-0.06
	No effective plant cover, bare or very sparse cover	Poor to fair; clean cultivation, crops or poor natural cover, less than 20% of drainage area has good cover	Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	Good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover
Surface Storage - C_s	0.10-0.12	0.08-0.10	0.06-0.08	0.04-0.06

	Negligible; surface depressions few and shallow, drainageways steep and small, no marshes	Well-defined system of small drainageways, no ponds or marshes	Normal; considerable surface depression, e.g., storage lakes and ponds and marshes	Much surface storage, drainage system not sharply defined; large floodplain storage, large number of ponds or marshes

The final coefficient is: $C = Cr + Ci + Cv + Cs$

For areas with a mixture of land uses, a composite runoff coefficient should be used. The composite runoff coefficient is weighted based on the area of each respective land use and can be calculated as:

$$C = \frac{\sum_{i=1}^{n} Ci \cdot Ai}{\sum_{i=1}^{n} Ai}$$

Assumptions and limitations are:

- The method is applicable if time of concentration for the drainage area is less than the duration of peak rainfall intensity.
- The calculated runoff is directly proportional to the rainfall intensity.
- Rainfall intensity is uniform throughout the duration of the storm.
- The frequency of occurrence for the peak discharge is the same as the frequency of the rainfall producing that event.
- Rainfall is distributed uniformly over the drainage area.
- The minimum duration to be used for computation of rainfall intensity is 10 minutes. If the time of concentration computed for the drainage area is less than 10 minutes, then 10 minutes should be adopted for rainfall intensity computations.
- The rational method does not account for storage in the drainage area. Available storage is assumed to be filled.

The table and the calculation of coefficients for rural and mixture of land use stems from the Hydraulic Manual – Texas Department of Transportation (see (TxDOT, 2016)).

The major drawback of this method is the poor physical representation of catchment characteristics and the absence of hydrographs.

2.3.2 SCS-Method

The SCS-Method was developed by the National Resources Conservation Service, Department of Agriculture, USA. The approach utilises physical parameters of a catchment area like soil type, land use, slope from which a so-called Curve Number (CN) is deduced. The CN-value represents the runoff characteristic and ranges from 20 (very high retention characteristic, almost no runoff) to 100 (no retention, no losses, precipitation results in runoff). It was developed as an event-based approach using accumulated rainfall from which the flood volume is calculated. The peak discharge is derived with the lag time, this is the time to rise to the peak of the hydrograph. A triangular hydrograph is assumed.

The approach requires more effort than the Rational method but considers physical characteristics. This makes the approach more transparent. In addition, the data base of CN-values is large, countless publications supply tables with CN-values. Derivatives of the approach include event-based losses and allow for antecedent soil moisture prior to an event. This is important as soil moisture conditions have a major effect. Without introducing antecedent soil moisture, best results can be expected for bare soil or sparse vegetation.

A list of CN-values for different hydrological soil groups and land cover can be found here: https://en.wikipedia.org/wiki/Runoff_curve_number.

The potential retention S in [mm] is calculated by: $S = 25.4 \cdot \left(\frac{1000}{CN} - 10 \right)$

where:

S: potential retention [mm]

CN: curve number [-]

The runoff volume Q is given by: $Qv = \frac{(P - 0.2 \cdot S)^2}{(P + 0.8 \cdot S)}$

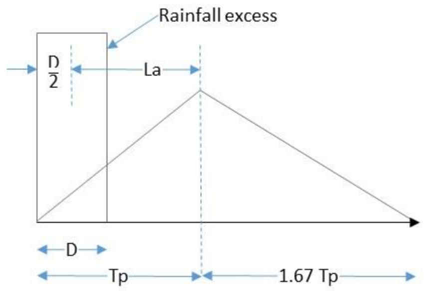
where:

Qv: runoff volume or depth of runoff [mm]

P: accumulated rainfall [mm]

The peak discharge is derived with the assumption of a triangular hydrograph given by:

$$Qp = \frac{0.208 \cdot A \cdot Qv}{0.5 \cdot D + 0.6 \cdot tc}$$



where:

Qp: peak discharge [m³/s]
A: catchment area [km²]
D: rainfall duration [hr]
Tc: time of concentration [hr]
Tp: time of rise [h]

2.3.3 Time of Concentration

The time of concentration tc is the time after commencement of rainfall excess when all portions of drainage basin are contributing simultaneously to flow at the outlet. It is also referred to a longest length of overland flow from the remotest point of the drainage area to the outlet while remoteness relates to travel time rather than distance. There are many formulas describing tc. Three are given:

Kirpich:

tc*: time of concentration [min]
tc: tc*/60 [hour]
L: L` [m]/0.3048, where L` is length of overland flow
So: slope [-]

$$tc^* = 0.0078 \cdot L^{0.77} \cdot So^{-0.385}$$

Kerby:

tc*: time of concentration [min]
tc: tc*/60 [hour]
L: L` [m]/0.3048, where L` is length of overland flow
n: manning coefficient [s/m$^{1/3}$]
So: slope [-]

$$tc^* = 0.83 \cdot \left(L \cdot n \cdot So^{-0.5} \right)^{0.467}$$

SCS lag:

tc*: time of concentration [min]

tc: tc*/60 [hour]

L: L` [m]/0.3048, where L` is length of overland flow

S: potential retention S = 1000/CN – 10

CN: curve number

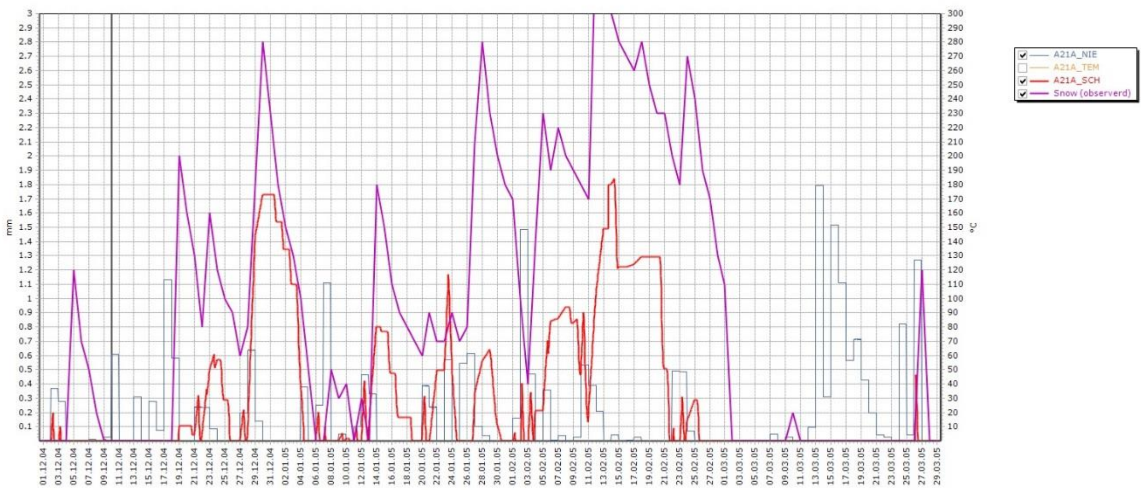
So: slope [%]

$$tc^* = L^{0.8} \cdot \frac{(S+1)^{0.7}}{1900 \cdot So^{0.5}}$$

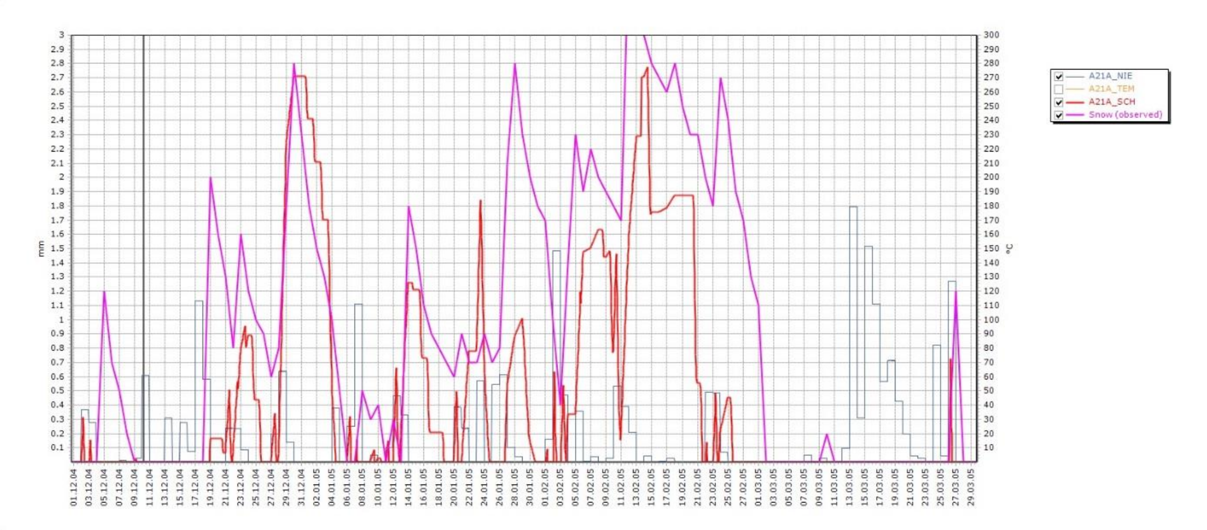
Kirpich considers slope and overland flow length but does not account for land cover. The disadvantage of the Kirpich formula is that tc would not change even if land use changes occurred in the drainage basin. Kerby introduces the manning coefficient reflecting land cover and is able to cope with land use alterations. The SCS lag formula uses the CN-value and yields longer tc compared to Kirpich and Kerby. Applying the SCS lag formula gives better results compared to model applications considering losses and sophisticated approaches like isochrones of travel time, cascades with different travel times and different flow components.

2.4 Snow computation

Computing snow accumulation, compaction and water equivalent is crucial in Tajikistan for any hydrological question. A short example is demonstrated with data from Khaburabad computed with the Snow Compaction approach according to (Bertle, 1966) and (Knauf, 1980). The method is based on field tests conducted by the US-Bureau of Reclamation.



(without any calibration)



(with only one calibration step)

The pink line indicates the observed snow, the red thin line shows the computed values. The model used was Talsim-NG (www.sydro.de). The model applies the Snow-Compaction approach as described in (Knauf, 1980), (Bertle, 1966). Input parameters are:

Table 9: Parameter of snow compaction method adopted from (Knauf, 1980), (Bertle, 1966)

Key	Parameter	Default
Tsnow	Temperature threshold when snow is accumulated [°C]	0
Mp	Rate of snowmelt [mm/(day Kelvin)]	4 - 5
Dmax	Threshold pack density at which compaction ceases and drainage from the snowpack begins [%]	40 - 45
Dfr	Initial dry snow density of snow pack in [%]	10

The approach is rather simple and data requirements are low compared to other methods. Calibration can be conducted based on observed snow depths.

2.5 Estimating erosion

Land erosion is an important parameter to identify adverse conditions which might come along with flood events, e.g. mudflows. Erosion is a very complex process and estimating it requires parameters which are difficult to assess. The universal soil loss equation (USLE) is one of the mostly used approaches. The equation is:

$$A = R \cdot K \cdot K \cdot S \cdot C \cdot P$$

Parameters and their dimensions are:

A	long-term average annual soil loss	$ML^{-2}T^{-1}$ (ML^{-2})*
R	rainfall erosivity factor	MLT^{-4} (MLT^{-3})*
K	soil erodibility factor	$L^{-3}T^{3}$
L	topographic factor of length	M L T
S	topographic factor of slope	M L T
C	Land management factor (C = C1 · C2)	
	C1: cropping management factor of vegetal cover	M L T
	C2: cropping management factor of tillage	M L T
P	conservation practices factor	M L T

M = mass, L = length,T = time

Each of the parameters has its own set of assumptions and coefficients which are often unknown and require a guess. Still, USLE is an accepted approach and provides a good overview to establish a map about erosion-prone areas. A disadvantage of the equation is the result as annual soil loss. This means it is not event-based. Event-based approaches have been developed known as modified USLE (MUSLE) replacing the rainfall erosivity factor by an event-based erosivity factor.

2.5.1 Rainfall erosivity factor R

The rainfall runoff erosivity is calculated as a product of storm kinetic energy (E) and the maximum 30-minute storm depth (I30) summed for storms in a year. Rainfall erosivity is calculated based on annual rainfall or monthly rainfall.

Formula based on monthly and annual rainfall

An approach which considers inner-annual rainfall uses:

$$MFI = \sum_{i=1}^{12} \frac{PM_i^{\,2}}{P}$$

PM: Monthly rainfall

P: Annual rainfall

Each month is weighed with its long-term average. To obtain the factor R two equations are applied:

$$R = [0.07397 \cdot MFI^{1.847} / 1.72], \text{ when } MFI < 55 \text{ mm}$$

$$R = [95.77 - 6.081 \cdot MFI + 0.4770 \cdot MFI^2 / 17.2], \text{ when } MFI > 55 \text{ mm}$$

2.5.2 Event-based soil erosion

The modified USLE (MUSLE) replaces the rainfall erosivity factor R with the product of rainfall amount and runoff amount with the aim to predict soil erosion for a single water erosion events.

Examples of formulas are:

$$S' = 95\left(Qp_p\right)^{0.56} \cdot K \cdot L \cdot S \cdot C \cdot P$$

where:

S’: sediment yield for a single event in tons [t]

Q: total event runoff in [ft^3]

p_p: event peak discharge [ft^3/s]

The parameters K, L, S, C and P are identical to the USLE.

A transformation into metric units requires a factor for converting feet^3 into m^3 so that the result is S = S’ 0.0283168.

2.5.3 Soil erodibility (K factor)

K reflects the susceptibility of soils to erosion. According to a study conducted Faizabad in Tajikistan, K factors ranged from 0.37 to 0.42 (Bühlmann, et al., 2010). This study applied the nomograph derived by (Wischmeier & Smith, 1978).

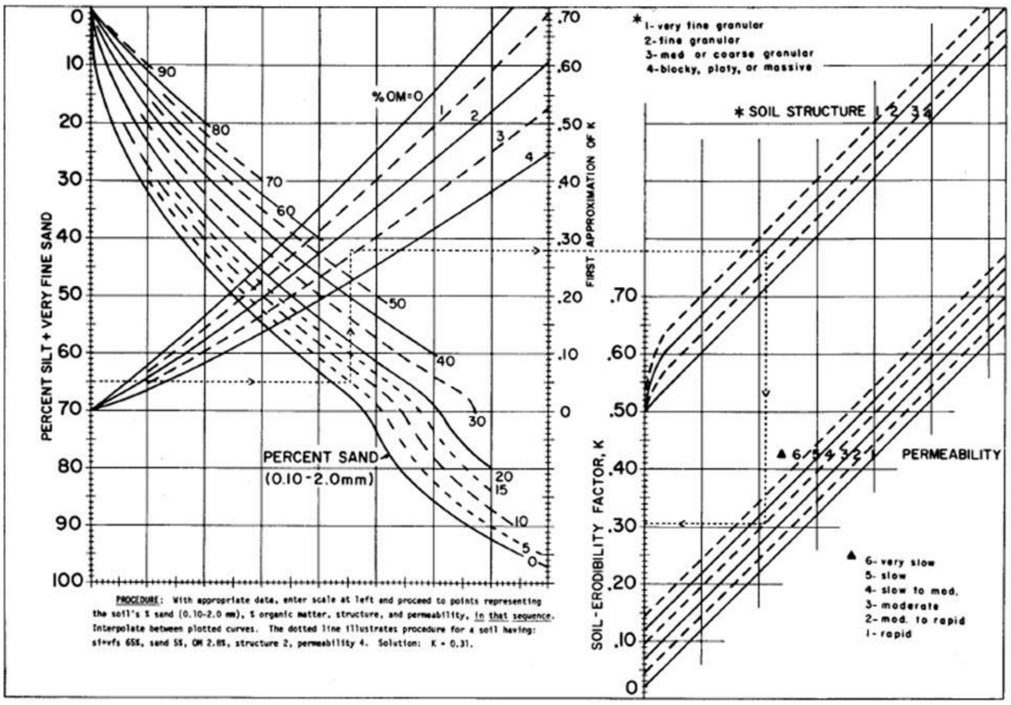


Figure 36: Nomograph for estimating K-factor (Wischmeier & Smith, 1978)

2.5.4 Slope length (L factor)

The L factor in the USLE is the distance from the point of origin of overland flow to the point where either the slope gradient decreases enough that deposition begins, or to where the flow connects to a river system.

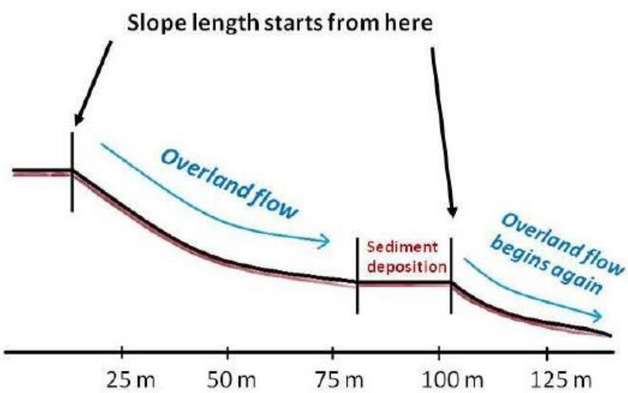
$$L = (\lambda / 22.13)^m$$

where

λ: Average slope length of single fields in [m]

m: variable slope length exponent that depends on slope steepness

- m = 0.5 for slopes greater than 5%,
- m = 0.4 for slopes between 3% and 5%,
- m = 0.3 for slopes less than 3%



(all adopted from (Wischmeier & Smith, 1978))

For practical use, average values can be determined by means of a GIS or by using a map and estimating mean conditions. This factor is linearly connected with the annual erosion losses. That means that an error of 10% in estimating this parameter results in a 10% change of the result.

2.5.5 Slope steepness (S factor)

Calculating slope, which is required to calculate the S-factor, is a standard procedure in GIS applications with a digital elevation model. The approach to estimate the S factor is according to (Renard, Foster, Weesies, McCool, & Yoder, 1997):

$$S = 65.41 \sin^2\theta + 4.56 \sin\theta + 0.065$$

Θ = mean slope angle in degrees

or

$S = 10.8 \sin\theta + 0.03$, gradient < 9%

$S = (\sin\theta / 0.0896)^{0.6}$, gradient ≥ 9%

2.5.6 Cover management or land cover (C factor)

The C factor is defined as the ratio of soil loss from land cropped under specific conditions to the corresponding soil loss from a continuously tilled fallow area.

Example values of C

Land Cover Class	C-Factor	Location	Author/Source
Dense forest	0.001	Sumatra	KOOIMAN (1987)
Open forest	0.001	Sumatra	KOOIMAN (1987)
Shrubs and bush vegetation	0.1	Java	HAMER (1981), quoted in KOOIMAN (1987)
Low cover vegetation (fallow)	0.2	Java	HAMER (1981), quoted in KOOIMAN (1987)
Bare soil	1	Sumatra	KOOIMAN (1987)
Residential areas and home gardens	0.14	Sumatra	KOOIMAN (1987)

More information on how to assess the parameter in detail provides (Renard, Foster, Weesies, McCool, & Yoder, 1997).

Literature about C and P values in Tajikistan is sparse. (Bühlmann, et al., 2010) has obtained a C value of 0.2 for vegetable which is in the range of mixed agriculture in the table below.

2.5.7 Conservation support practice (P factor)

P factor is the soil loss ratio with a specific support practice to the corresponding soil loss with up and down slope tillage.

The P factor value will reduce when there are more effective supporting mechanical practices such as contouring, strip cropping, terracing and retention ditches. When there are no conservation support practices in the area of interest, maximum values of 1 will be assigned, meaning no land use influence.

2.6 Hydraulic calculations

Hydraulic calculations are needed for transforming discharge from hydrological considerations into flow velocity, flow depth and to calculate tractive forces exerting on movable bed particles.

Given a flow cross-section, the mean velocity can be derived by using the continuity equation: $v = Q / A$ where Q = discharge [m^3/s] and A is the flow cross-sectional area [m^2].

Assessing a channels capacity, the use of the Manning Equation for uniform flow is commonly applied.

$$v = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

where:

v: velocity in m^3/s

n: Manning's roughness coefficient (= 1/kst where kst=Strickler coefficient)

R: hydraulic radius [m] = A / WP

A: flow cross-sectional area [m²]

WP: wetted perimeter of flow [m]

S: slope of the energy gradeline [m/m]. For uniform, steady flow, S is the channel slope.

Iteration is required because the water level is needed to compute WP and A. With A and the resulting flow velocity the discharge must be checked with v = Q/A. A result is achieved when the estimated water level results in a flow cross section from which v = Q/A and v from Manning Equation give the same flow velocity.

It is common practice to assume stationary, uniform flow and to use the channel bed slope. It is necessary to bear in mind that during a flood event, flow is neither stationary nor uniform so that the result incorporates uncertainties. This must be reflected with safety factors during design. Suggested Manning roughness coefficients are given in Table 10. These coefficients are subject to change in steep terrain.

Table 10: Manning roughness coefficients (adopted from (TxDOT, 2016))

Natural Channels	Minimum	Normal	Maximum
Minor Streams (top width at flood stage <30 meters)			
Streams on plain:			
Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
Same as above, but more stones and weeds	0.030	0.035	0.040
Clean, winding, some pools and shoals	0.033	0.040	0.045
Same as above, but some stones and weeds	0.035	0.045	0.050
Same as above, but lower stages, more ineffective slopes and sections	0.040	0.048	0.055
Clean, winding, some pools and shoals, some weeds and many stones	0.045	0.050	0.060
Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages:			
Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
Bottom: cobbles with large boulders	0.040	0.050	0.070
Flood Plains			
Pasture, no brush:			
Short grass	0.025	0.030	0.035

Natural Channels	Minimum	Normal	Maximum
High grass	0.030	0.035	0.050
Cultivated areas:			
No crop	0.020	0.030	0.040
Mature row crops	0.025	0.035	0.045
Mature field crops	0.030	0.040	0.050
Brush:			
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush and trees, in winter	0.035	0.050	0.060
Light brush and trees, in summer	0.040	0.060	0.080
Medium to dense brush, in winter	0.045	0.070	0.110
Medium to dense brush, in summer	0.070	0.100	0.160
Trees:			
Dense willows, summer, straight	0.110	0.150	0.200
Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Same as above, but flood stage reaching branches	0.100	0.120	0.160
Major Streams (top width at flood stage >30 meters)			
Regular section with no boulders or brush	0.025	--	0.060
Irregular and rough section	0.035	--	0.100
Lined Channels			
Concrete-lined	0.012	--	0.018
Concrete rubble	0.017	--	0.030
Unlined Channels			
Earth, straight and uniform	0.017	--	0.025
Winding and sluggish	0.022	--	0.030
Rocky beds, weeds on bank	0.025	--	0.040
Earth bottom, rubble sides	0.028	--	0.035
Rock cuts	0.025	--	0.045

An alternative to Manning's equation provides the formula of Darcy-Weisbach.

$$v = \sqrt{\frac{1}{\lambda} \cdot 8 \cdot g \cdot r_{hy} \cdot I_E}$$

where:

v: average velocity [m/s]

λ: Coefficient of resistance [-]

r_{hy}: hydraulic radius [m] = A / WP

IE: slope of the energy gradeline [m/m]. For uniform, steady flow, S is the channel slope.

The coefficient of resistance can be expressed as:

$$\frac{1}{\sqrt{\lambda}} = -2.03 \cdot \lg\left(12.27 \cdot \frac{r_{hy}}{k_s}\right)$$

where k is the equivalent sand roughness. The approach is more complex than Manning's formula but gains wide acceptance due to a better approximation of flow processes. However, applying the formula requires iteration.

Table 11: Equivalent sand roughness coefficients (adopted from (Patt, 1998))

River bed structure	Ks [mm]
Rock:	
Machined, smoothed	220 – 350
coarse	450 – 700
Earth channels:	
regular	15 – 60
Good conditions, no vegetation	6 – 10
Bed and banks muddy, regular	25 – 50
Gravel bed, sparse vegetation	80 – 140
Medium vegetation	190 – 270
Poorly maintained	300 – 500
With bed load	100 – 200
Flow strongly impaired by weeds	500 – 1500
Stones and gravel (not transport):	
Coarse gravel	50 – 54
Coarse gravel mixed with sand and mud	30 – 40
Sand and gravel (< 6 cm)	20 – 55
Regular machined stones (10-20 cm) in bulk, plain river bed	16 - 18

2.7 Sediment transport

Measures for torrent control aim at reducing typical effects of torrential flows, erosion and transport/deposition of eroded material. In contrast to Section 2.5 where erosion is understood as land erosion and loss of soil, this section deals with erosion, deposition and stabilisation processes in open channels, river beds and stream banks. Streamflow causes the tractive force that detaches and transports materials either as bed load or suspended solids. This document concentrates on bed load. The tractive force follows the equation:

$$\tau = \rho_w \cdot g \cdot r_{hy} \cdot I_E$$

where:

τ: tractive force or sheer stress [N/m²]

ρ_w: density of water [kg/m³], ρ_w =1000 km/m³

g: gravity [m/s²]

r_{hy}: hydraulic radius [m] = A / WP

I_E: slope of the energy gradeline [m/m]. For uniform, steady flow, S is the channel slope.

The tractive force is countered by the resistance of materials to detachment and transport through weight, inertia and friction. The threshold when mass movement begins is called the critical sheer stress, boundary sheer stress or critical tractive force τ_{crit}. The torrentiality of a stream may be assessed by comparing τ with τ_{crit} to see whether t > τ_{crit}. If so, there will be erosion and/or sediment transport.

The tractive force on bank material can be calculated with:

$$\tau_{bank} = \tau_{bed} \cdot \left(\cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\varphi}} \right)$$

where:

τ_{bed}: tractive force or sheer stress stream bed (see above) [N/m²]

Θ: angle of bank slope above the horizontal

φ : angle of internal friction of bank material

Values for angle of internal friction are:

Rock	30
Sand	30 - 40
Gravel	35
Silt	34
Clay	20
Loose sand	30 – 35
Medium sand	40
Dense sand	35 - 45
Gravel with some sand	34 - 48
Silt	26 - 35

Because the angle of internal friction, is typically around 25 to 35, the coefficient of internal friction (tan) is 0.5 to 0.7.

The core of torrent control is the identification of the balance between actual sheer stress caused by streamflow and critical sheer stress due to the material's properties. Any form of hydrological intervention in the watershed that reduces the drivers for tractive force or increases boundary sheer stress contributes to improving torrential control. There are structural, engineered and nature-based measures as well as biological methods like watershed improvement, land conservation and soil stabilisation. All three components must complement each other to obtain a sustainable solution. Doing nothing in the watershed management but engaging in structural measures is like combating symptoms only without curing root causes. For example, fixing soil erosion will reduce the quantity of suspended sediments and decreases turbidity and density which, in turn, reduces the specific gravity γ and weakens the tractive force.

Table 12: Critical sheer stress for different material

Soil	d mm	Tau-crit	v crit	kst
Silt	0.02 … 0.063	-	0.1 … 0.2	40 .. 50
fine sand	0.063 … 0.2	0.5 .. 1.0	0.2 … 0.35	40 … 50
medium sand	0.2 … 0.63	1.0 … 2.0	0.35 … 0.45	40 … 50
coarse sand	0.63 … 2.0	3.0 … 6.0	0.45 … 0.6	40 … 50
fine gravel	2.0 … 6.3	8.0 .. 12.0	0.6 … 0.8	40 … 50
medium gravel	6.3 … 20	15	0.8 … 1.25	40 … 60

coarse gravel	20 … 63	45	1.25 … 1.6	35
stones, boulders 50 … 75	50 … 75	-	1.7 … 1.8	30
stones, boulders 75 … 100	75 … 100	-	1.9 … 2.0	28

Table 13: Critical sheer stress for bank revetments

Stabilization	d mm	Tau-crit	v crit	kst
riprap 32/63	32/63	30 … 58	-	20 … 30
riprap 63/90	63/90	40 … 75	-	20 … 30
riprap 63/125	63/125	75 … 100	-	20 … 30
riprap 100 … 150	100 … 150	-	1.9 … 3.4	20 … 30
stone packing	150 … 200	53 … 73	2.6 … 3.8	-
cobble-stone pavement	200 … 300	73 … 160	-	-
grass (short, well-rooted), average	-	15 … 18	1.5	-
grass (short, well-rooted), peak	-	20 … 30	1.8	-
concrete grid panels with grass	-	108	-	40 … 50
concrete grid panels with sand	-	40 … 50	-	40 … 50
concrete grid panels with gravel	-	50 … 100	-	40 … 50
concrete without sediment	-	-	4	-
concrete with sediment	-	-	2.5	-
vegetated gabions	-	30 … 40	-	-
well-rooted shrubs	-	100 … 140	-	-
quarrystone, fortified	-	-	5	-

(Schillinger, 2001) has evaluated field tests and laboratory tests to compile critical sheer stress of bioengineering measures.

Table 14: Critical sheer stress (adopted from (Schillinger, 2001)

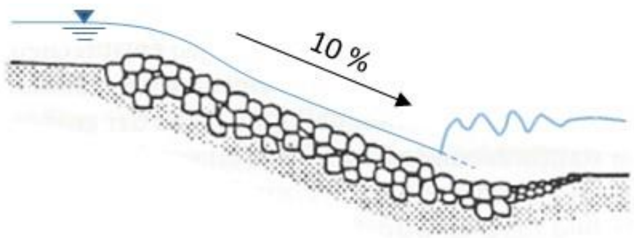
Measure	Literature / Author	Age [Month]	vm [m/s]	ISo [‰]	h [m]	bSo [m]	Bank slope	τ_{crit} [N/m²]	Comments
willow brush mattress	FLORINETH (1982)	15	-	30,0	1,20	16,0	4:5	218	Zangenbach
		15	-	30,0	1,15	8,0	4:5	195	Lasankenbach
	FLORINETH (1995)	7	-	18,0	3,00	36,0	2:3	309	Passer
		7	-	30,0	1,20	16,0	4:5	312	Zangenbach
		7	-	30,0	1,15	8,0	4:5	292	Lasankenbach
		7	-	18,0	3,00	36,0	2:3	480	Passer
	LACHAT (1994)							300	
	ZEH (1990)		3,5						
	BEGEMANN/SCHIECHTL (1994)							50 bis >300	
	GERSTGRASER (2000)	3 bzw. 7	3,2 - 3,5					200 - 300	
wattle fence	STEIGER (1918)			2,0			1:2	50	
	BORKENSTEIN (1976)							50	
	ZEH (1990)		3,5						
	RÖSSERT (1994)							50	
	GERSTGRASER (2000)	15	3,2 - 3,5					100 - 120	

Measure	Literature / Author	Age	vm	ISo	h	bSo	Bank slope	τ_{crit}	Comments
		[Month]	[m/s]	[‰]	[m]	[m]		[N/m²]	
fascine	BEGEMANN/SCHIECHTL (1994)							60	
	RÖSSERT (1994)							70	
	LACHAT (1994)							250	
	LfU (1996)		2,5 - 3,0	0,6 - 0,9				70 - 100	dead wood fascine
			3,0 - 3,5	0,6 - 0,9				100 - 150	Live fascine
	ZEH (1990)		3,5						
	GERSTGRASER (2000)	15	3,5 - 4,0					180 - 240	with planting stakes
		15	2,0 - 2,5					120 - 150	on brushlayer
		15	3,3 - 3,8					150 - 200	array of fascines
	STEIGER (1918)			7,0				180	Piles with fascines
	SCHOKLITSCH (1930)							70	
willow cuttings	WITZIG (1970)			5,5	3,00	28,0	2:3	165	Joint planting with concrete blocls
	EVED (1982)							> 140	Joint planting with bolder reventments
	BEGEMANN/SCHIECHTL (1994)	0 - 3 Jahre						50 - 250	With piles and stone packing
		0 - 3 Jahre						75 bis > 350	with array of blocks
	LfU (1996)		3,0 - 3,5	0,6 - 0,9				100 - 150	with riprap
	GERSTGRASER (2000)	15	2,2 - 2,8					80 - 120	Coconut fibre rolls
willow shrubs	WITZIG (1970)						2:3	100	elastic
	EASF (1973)							100 - 140	
	ANSELM (1976)	1-2 Jahre						50 - 70	
		> 2 Jahre						100 - 140	
		20 Jahre						800	
riprap	GERSTGRASER (2000)	15	3,0 - 3,5					120 - 160	Geotextile with brushlayers
	STEIGER (1918)			7,0				170	
	LfU (1996)		3,5 - 4,0					bis 150	
	ZEH (1990)		3,5						
	BEGEMANN/SCHIECHTL (1994)	0 - 3 Jahre						100 bis > 300	With joint planing
Reeds /	ZEH (1990)		2,0						
brush mattress construction with reeds	LfU (1996)		2,0 - 2,5					55 - 65	
grass	WITZIG (1970)						1:2 bis 2:3	50 - 100	
	EASF (1973)						1:2 bis 2:3	50 - 80	
	RÖSSERT (1988)							15 - 18	Long-term
								20 - 30	Short-term
	BEGEMANN/SCHIECHTL (1994)							15 - 18	Long-term
								20 - 30	Short-term
	LfU (1996)		1,5					30	With crushed stones
			1,8					40	seedings
			> 3,5					> 60	strip of turf
	ZEH (1990)		1,8						Dry seeds
			1,8					30	Seedings with geotextiles

Measure	Literature / Author	Age [Month]	vm [m/s]	ISo [‰]	h [m]	bSo [m]	Bank slope	τ_{crit} [N/m²]	Comments
			3,7						Strip of turf
Revetments	LfU (1996)		2,5 - 3,2					70 - 100	gravel (0 - 40mm)
			3,5 - 4,0					100 - 150	boulders
			> 4,0					> 150	Large boulders
	BOLLRICH (1992)		1,9 - 3,4						riprap
			2,6 - 3,8					53 - 73	bouldars (15 - 20 cm)
								73 - 160	boulders (20 - 30 cm)

2.8 Development of streambed stabilisation

Ramps are often used to bridge large slopes so that the rest of a longitudinal profile can be developed with less slope and as such with less tractive forces.



A formula to estimate the diameter needed for implementing a ramp was developed by (Whittaker, 1986):

$$d_{crit} = 1.225 \cdot \left(\frac{\rho_s - \rho_w}{\rho_w} \right)^{-\frac{1}{3}} \cdot I^{\frac{7}{9}} \cdot q_{crit} \quad \text{or asking for } q_{crit}\text{:} \quad q_{crit} = 0.235 \cdot \sqrt{\frac{\rho_s - \rho_w}{\rho_w}} \cdot \sqrt{g} \cdot I^{-\frac{7}{6}} \cdot d_s^{\frac{3}{2}}$$

where:

g:	Gravity [m/s²]
ρ_s:	Density of the stones used for the ramp [kg/m³]
ρ_w:	Density of water [kg/m³]
I:	Slope of the ramp [m/m]
q_{crit}:	critical discharge per m width at which movement of the ramp would start [m³/(s m)]

With a given discharge, the minimum diameter can be estimated or with given stones the critical discharge at which movement of the stones would begin. The stones used for developing a ramp should be very hard to resist abrasion. The stones need to be tightly placed or ideally fixed with cement or mortar. It is obvious that larger diameter of the stones provide more robustness.

Developing a ramp with diameter less than 40 cm, it is possible to raise the ramp still as a loose stone package saving time and labour force. Larger diameters require an excavator with a gripper arm to place each stone carefully. The development of a plain underground can be combined when an excavator is used. The material beneath the ramp should fulfil a rule of thumb in the way that

d_{85} (substrate) · 5 < d_{stones}

Is the substrate smaller, a filter must be laid. Generally, a ramp should be developed as a plain along the whole the cross-section to avoid flow concentration in the middle. The stone package is to continue into the river banks.

2.9 Advanced methods

2.9.1 Hydrological modelling

A hydrologic model is a simplification of the real world and distinguishes between different hydrological processes like precipitation-runoff, soil water and soil moisture, overland flow, flow in open channels or pipes, lakes and reservoirs, groundwater, etc. It depends on the model which methods are implemented and how complex they are. As a rule of thumb, more complex methods usually require more parameters and thus more data and observations for calibration. Hydraulic methods for weirs, spillways and diversion are often incorporated. A watershed can be modelled by composing the processes to a hydrological system.

The model approach starts with the delineation of sub-basins and river reaches,, followed by acquiring the parameters needed for each sub-basin and river reach. All elements are then combined to represent the flow network. The comparison of the GIS sub-basins and a screenshot of Talsim-NG (www.sydro.de) as hydrological model is shown in

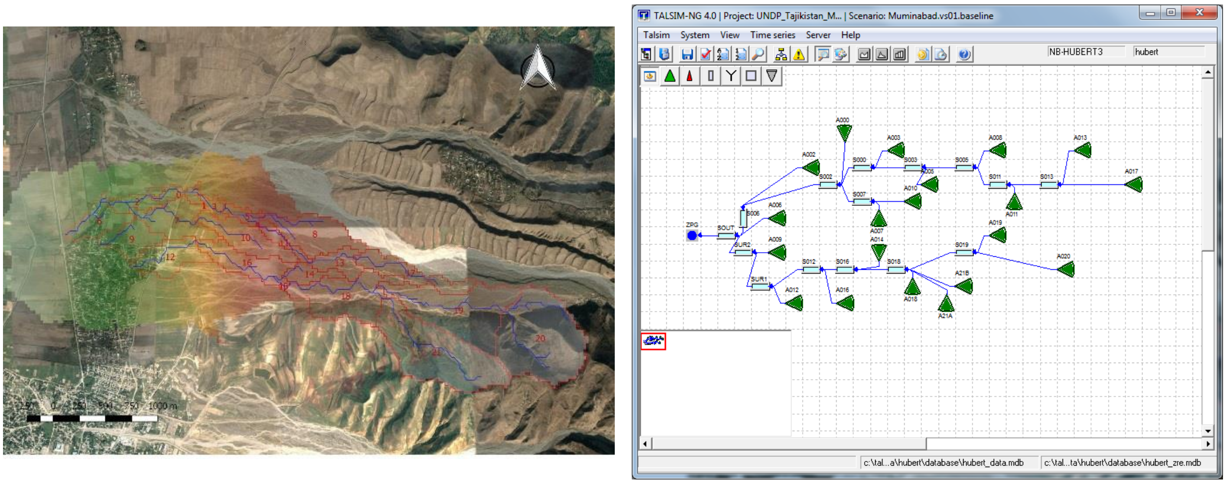
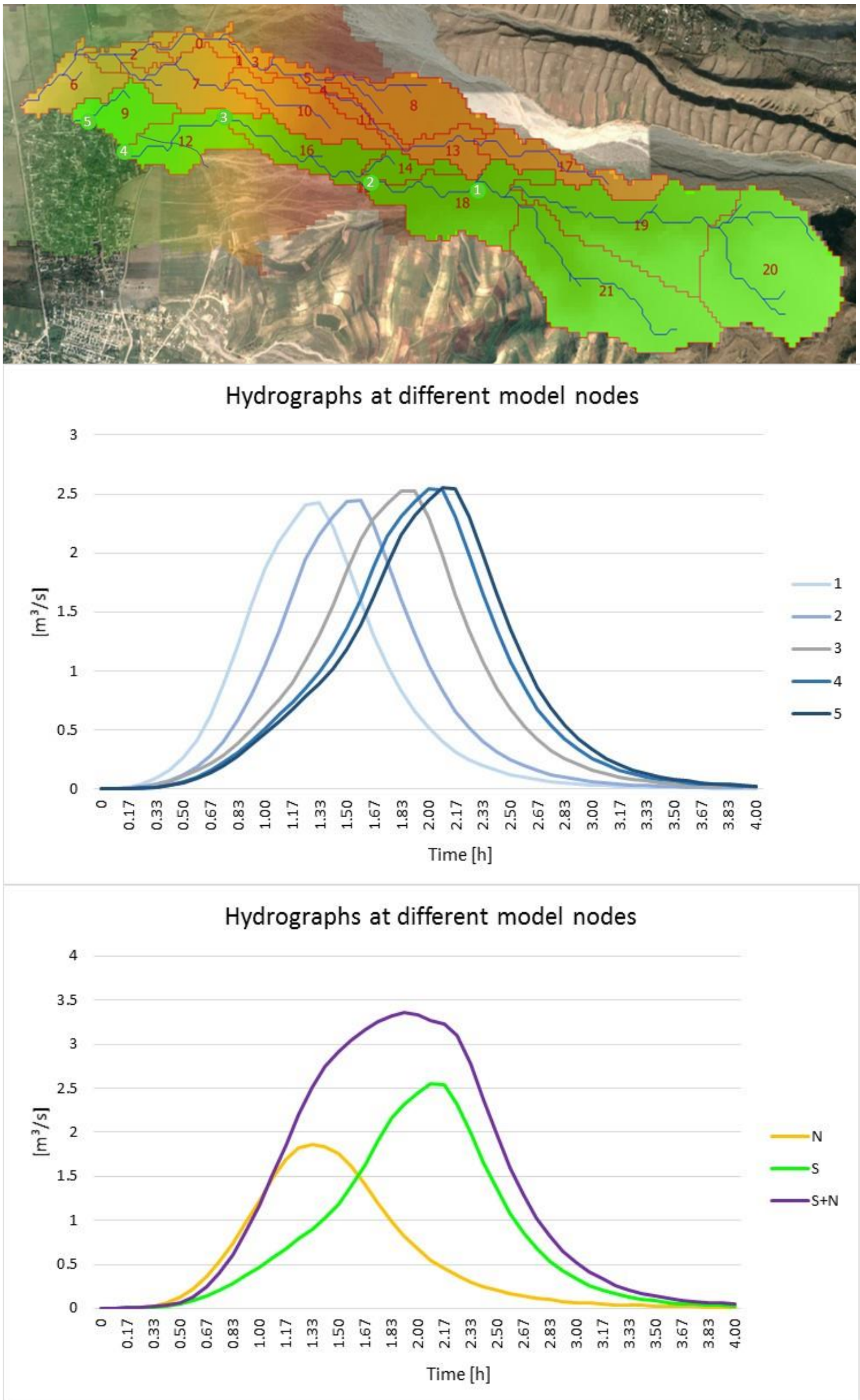


Figure 37: Hydrological model – from GIS to flow network (QGIS and Talsim-NG)

Hydrological models usually embed sub-basins, river reaches, diversions, weirs, reservoirs, consumers, point-discharge elements and sometime groundwater elements. Additionally, the Talsim-NG model allows for incorporating operating rules for controllable structures like reservoirs, gates, pumps and turbines.

Basically, hydrological models are state-of-the-art in computing runoff, propagating water through rivers and generating hydrographs at given points in a watershed. The capacity to allow for losses in the runoff generation, to consider time of concentration according to the topography and land use parameters and above all, the ability to overlay flow from different sub-basins and to transport water in a stream network are the major advantages.

The propagation of flow is demonstrated through the model nodes indicated as green as shown in the figure below.



The flow along the green area shows the time lag water needs to flow from one node to the next.

Figure 38: Overlay of flow for different catchments

Due to the different travel time in the watershed, the resulting maximum peak flow is not a simple addition of peak discharge from the green and orange area.

It is recommended to use hydrological models while assessing a watershed for flood management. Free models are available here:

http://www.hec.usace.army.mil/software/hec-hms/ (for beginners) http://www.bluemodel.org/ (for advanced users)

www.sydro.de (upon request) (for beginners up to experts)

http://swat.tamu.edu/software/ (for experts)

2.9.2 Hydraulic modelling

Flood modelling comprises of two components, hydrological simulation, which quantifies the size, duration and probability of a flood event and hydraulic simulation providing the means to compute water depth from which inundated areas can be derived.

Hydraulic modelling comes in two ways: 1D and 2D modelling. 3D is not considered here. 1D employs the longitudinal direction along the channel. A stream network composed by a 1D model is a linear system of river reaches where traverse flow is more or less ignored and vertical differences in a cross section are averaged. In contrast, a 2D model serves the longitudinal and lateral directions and consists of a regular or irregular mesh of cells, connected to each other and flow can cross the edges of all cells.

The preferred field of work for 1D and 2D are:

1D	Narrow valleys Steep gradients Modelling of hydraulic structure like gates, weirs, pipes, etc. No retention areas Steady or unsteady flow possible	2D	Floodplains River with large river banks Unclear and changing flow paths Flow with distinct traverse flow components Flow direction less predictable

Figure 39: Typical 1D and 2D hydraulic schemes

The question needs to be answered whether more advanced modelling like a fully developed 2D approach supported by detailed spatial information is actually more advantageous than simplified 1D modelling. In fact, the more sophisticated approaches become extremely demanding in terms of data and computational resources. This imposes substantial barriers on the utilisation of 2D models.

As a rule of thumb, the use of 1D models will suffice the requirements in a typical terrain with steep and narrow valleys. There is no need to apply 2D models, unless urban settlements are affected in an area where multiple possible flow paths exist which changing channels from flood event to flood events. Typical examples are large deposition areas and alluvial debris cones.

Figure 40: A case for a 2D hydraulic application

2.10 Flow in steep terrain and estimation of sediment load

The Manning roughness is not a constant value. It decreases with a smaller wetted perimeter while the cross section remains constant. In other words, the decrease in roughness is not only expressed in a smaller hydraulic radius but also in a decrease of the Manning roughness (Bergmeister K. S.-M., 2009). The range of validity for the Manning roughness is considered to be not more than 4% slope. A correction factor for n is given as (Bergmeister K. S.-M., 2009):

$$\frac{1}{n} = \frac{26}{d_{90}^{1.6}}$$

where:

n: Manning roughness

d90: 90% of the grading curve of the river bed material

Alternatively, the flow velocity for steep torrential streams was evaluated by (Rickenmann, 1996) as:

$$v = \frac{0.37 \cdot I^{0.2} Q^{0.34} \div g^{0.33}}{d_{90}^{0.35}}$$

Considering the load of sediment in steep torrents, the discharge itself requires an adaptation and the sediment load must be included. This can be accounted for by multiplying the discharge with an intensity factor, representing the additional load in the water-sediment mixture. (Bergmeister K. S.-M., 2009) suggests the following intensity factors:

Table 15: Increase of discharge due to sediment load (Bergmeister K. S.-M., 2009)

Process	Proportion of sediment	Intensity factor IF
Flood (low sediment)	0 – 5%	1 – 1.05
Fluviatile sediment load	5 – 20%	1.05 – 1.4
Mudflows	20 – 40&	1.4 – 3.5
Debris flow	50 – 80%	3.5 - 100

For estimating the sediment load or amount of material during mudflows, several empirical formulas were developed. These formulas contain a high degree of uncertainty and serve only as rough estimates in the absence of any other reliable information.

$M = 27000 \cdot A^{0.78}$	Zeller [219], Rickenmann [174]
$M = L_c \cdot (110 - 250 \cdot J_f - 3)$	Rickenmann/Zimmermann [178]

All empirical formulas stem from field investigation in the Alpine region.

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