Dam break analysis for Serra degli Ulivi dam

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Introduction

The new reservoir of Serra degli Ulivi is located in the Northwestern part of Italy, in the Piemonte Region, and its storage capacity of nearly 10 million cubic meters is designed to improve the efficiency of the existing Villanova irrigation scheme in the Cuneo Provence. The 56 meter high Serra degli Ulivi dam, as proposed in the preliminary design, is of asphalt-core earth-fill type with a lateral block in conventional concrete hosting the spillway and the intake tower. The dam break was modelled and analysed using USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS). The study was performed in compliance with the local authority regulation and following the recommendations of ICOLD Bulletin N.111. The regional directive establishes clear criteria to perform such studies, defining the extension of the downstream study area, the level of details of the study and the requirements for the flood hazard mapping. At present time no case history is available for the collapse of asphalt core dams and this study presents a conceptual model to predict the dam failure mechanism. Two different dam failure scenarios were investigated for the peak outflow prediction. For both cases the sensitivity of the main breach formation parameters was tested and compared with the empirical formulas available in literature. The downstream propagation of the flood wave and its interaction with the existing structures was analysed based on the available cartographic and topographic information, complemented with field surveys at any bridge, river crossing and singularity within the 12 km of the river stretch under study. The present study illustrates the methodology of the dam break analysis and presents the main results of the study. The generated inundation maps support dam owner, regulators and emergency agencies in the implementation of the warning systems, preparation of evacuation plans, and represent the basis for the hazard classification of affected areas.

1. Project description

1.1 Serra degli Ulivi dam and reservoir

Serra degli Ulivi (SdU) is an embankment dam with a central core in asphaltic concrete. The dam is 56 m high. The embankment is equipped with a concrete gravity structure on the right bank hosting the un-gated spillway and the intake facilities. The dam foundation is limestone bedrock dipping down from right to left. On the right bank the concrete plinth on which the asphalt diaphragm is connected sits directly on the grouted rock; on the left bank the plinth is connected to the bedrock by means of a cut-off wall in plastic concrete (Fig.1). The main parameters of the dam and reservoir are given in Table 1. The geometry of the dam is presented in Fig. 1, 2 and 3.

Dam Height	56	m
Crest Level	573.5	m a.s.l.
Crest Length	290	m
Full Supply Level	569.0	m a.s.l.
Reservoir Volume at the crest	11.6	Mm^3
Reservoir Volume at FSL	9.6	Mm^3
Reservoir max length	1.5	km
Spillway design discharge	152	m^3/s
Concrete Block Height	25	m

Table 1 – Serra degli Ulivi Dam, main parameters

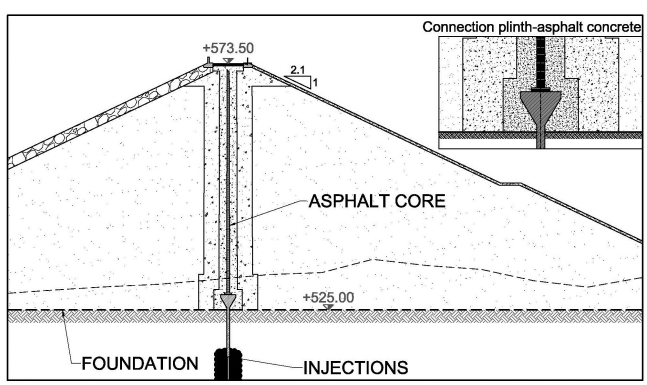

Fig. 1 – Serra degli Ulivi transversal section

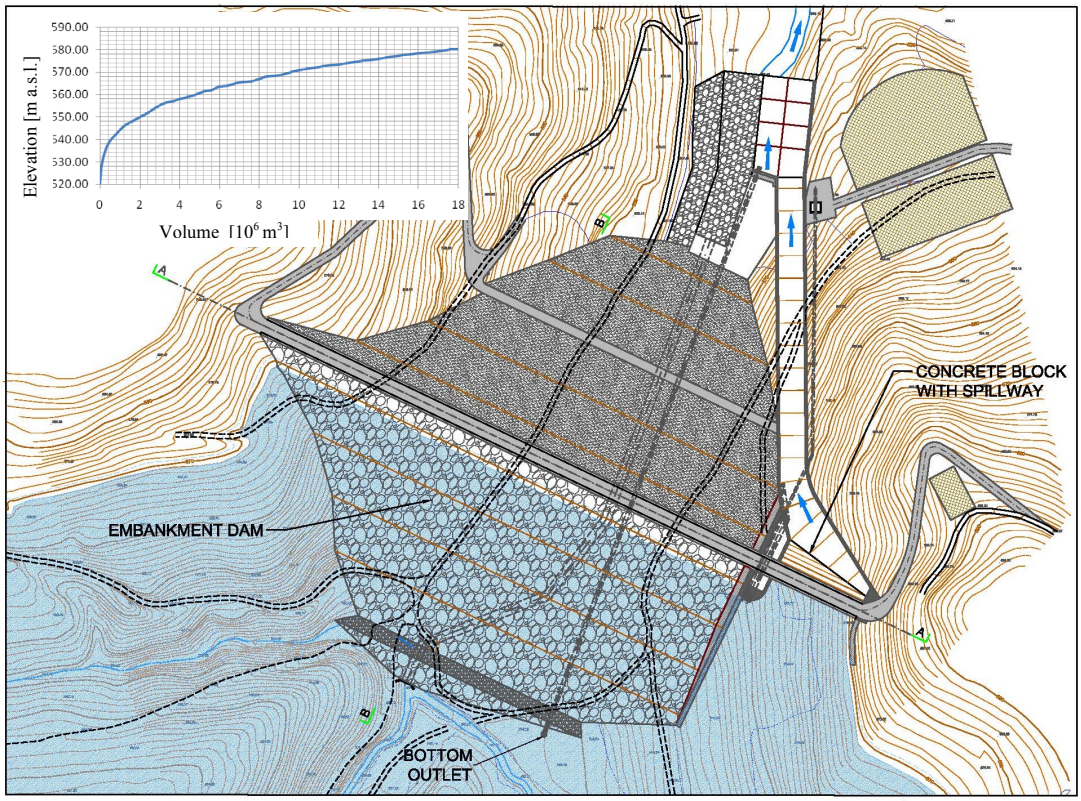

Fig. 2 – Plan view of Serra degli Ulivi dam

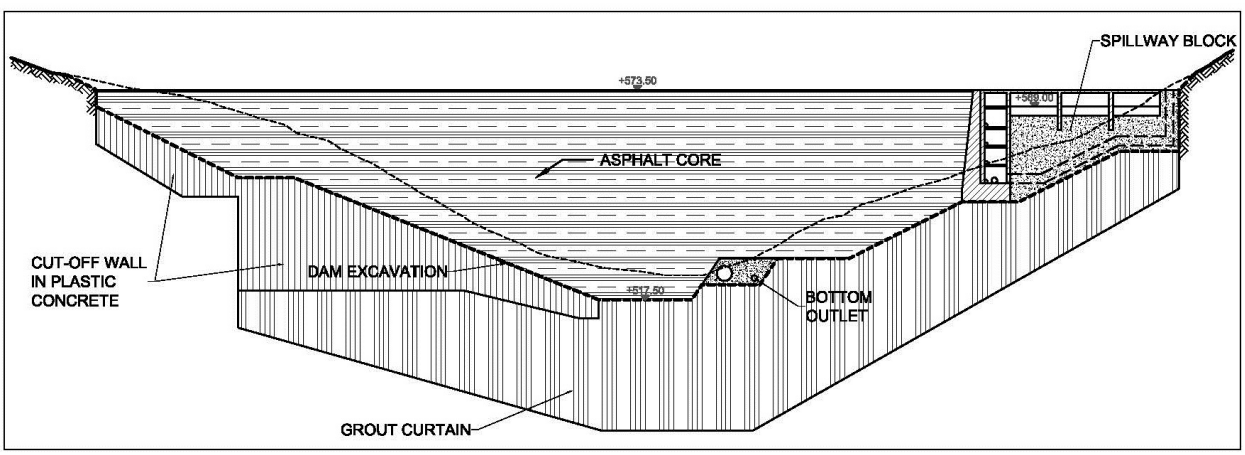

Fig. 3 – Longitudinal section of Serra degli Ulivi dam

The lateral overflow structure is made in conventional concrete and it is composed of three plots with vertical joints every 20 m. The latter hosts the spillway with a capacity of 152 m^3/s, corresponding to the 3000-yr flood, as per the Italian regulation requirements. The Serra degli Ulivi reservoir is filled by means of two pipelines coming from two water intakes located on the Pesio and the Ellero Rivers. The two conduits enter the reservoir in correspondence of the intake tower. The reservoir volume curve is given in Fig. 2.

1.2 The study area

The dam is located in a depression of the Pogliola creek, a right tributary of the Pesio River. Downstream of the dam, the average gradient of the riverbed is 2.2% in the upper part, 1.5% in the middle part and 1% in the lower part, before the confluence with the Pesio River. In the upstream 2.5 km the river is narrow and entrenched, then the hydraulic sections enlarge and the river runs along terraces and irrigated areas. The stretch of interest is crossed by 10 road bridges and 1 railway bridge. Nearly 5.5 km downstream of the dam axis an existing intake structure was built to supply an irrigation channel. All the singularities and infrastructures influencing the study were surveyed and modelled in Hec-Ras, as they represent restrictions and sections of hydraulic control. At the confluence of the

Pogliola into the Pesio a wider section is observed, nearly 500 m wide, presenting the possibility of a more important routing effect of the flood wave.

The natural flood peaks relevant to the study are provided in Table 2.

Return Period	POGLIOLA at Serra degli Ulivi Dam S=5.94 km^2	PESIO at the confluence with the Pogliola S=279 km^2
year	m^3/s	m^3/s
500	119	5585
1000	132	6195
3000	152	6857

Table 2 – Flood Peak values for the Pogliola and Pesio rivers

According to the *Circolare DSTN/2/22806* (i.e., the Regional Directive for the Dam Break Analysis) the downstream study area must be extended until the routed peak reaches a value that is lower than the Q_{500}; in case of a confluence into another river, the peak must be lower than the Q_{500} of the receiving river. In the present study the river cross sections are extended until the confluence with the Pesio River.

For the flood-wave propagation analysis the *Circolare DSTN/2/22806* also prescribes the use of maps in scale 1 to 5000, whenever available, or alternatively the adoption of the official regional cartography in scale 1 to 10000 (i.e, Carta Tecnica Regionale-CTR) in combination with field surveys of the existing infrastructures and all areas potentially affected by the inundation. As recommended by the directive, cross sections were chosen perpendicular to the flow direction, with a density sufficient to fully describe the variability of the geometry and the hydraulic behaviour of the valley. Sections are shown in Fig. 8.

2. Dam break analysis

Although a large number of dam failures occurred in the past, no failure of asphalt core embankment dam has been observed. Such technology was developed over the last 30 years and therefore fairly recent compared to others; for this reason a clear failure mechanism is still unknown and a prediction must be based on conceptual assumptions.

Modern dam-building using asphalt as a central core inside the embankment dam (ACED) was first introduced in 1962 in Germany and over the following 15 years a large number of similar dams were built, mostly in Germany. The Chinese developed their knowledge in such structures and they built their first asphalt core dams in the 1970s. To date 13 dams of this type have been completed in China. In Norway until the 1970s, the large majority of embankment dams were rock-fill dams with a central moraine/clay core; however, as these materials became increasingly difficult to find, asphalt was used to replace the core material. In Norway the first dam with an asphalt core was completed in 1978, and since then nearly all large Norwegian embankment dams have been built using this technique. In the world, more than one hundred asphalt core dams have been constructed since 1964, under all climatic conditions.

The dam break analysis was carried out using HEC RAS, a mono-dimensional model developed by the US Army Corps of Engineers. The use of Hec-Ras is contemplated in the local regulation and it can successfully be applied to the present study due to the narrowness of the valley and the lack of floodplains.

2.1 Dam failure mechanisms

Asphalt is a viscoelastic-plastic material with a self-healing ability and all of the existing dams have an excellent performance record with negligible leakage. For these reasons a piping failure mechanism is unlikely to happen in such structures. Singular cracks of small openings may only originate seepage of local and modest scale. Piping is more likely to occur in clay-core embankment or rock-fill dam left unprotected during construction. Even during construction, an asphalt-core embankment dam is not exposed to piping in case of cofferdam overtopping.

It is the authors' opinion that the only reasonable failure mechanism for ACED could be the overtopping due to exceptional flood. Due to the large size of the un-gated spillway and the low magnitude of the Pogliola floods, the overtopping could only occur after a structural failure of the dam, for instance due to a seismic loading. A large settlement of the crest followed by an exceptional flood may create a typical overtopping failure mechanism in which the embankment erosion progresses from downstream to the crest and causes the inception of the breach (Fig.4). The Italian Regulation (2009) for an embankment dam of H = 56m imposes a minimum freeboard of 2.6 m. This height must be counted from the 3000yr flood level incremented of the wind wave height and the run-up. Such large freeboard is designed to accommodate large settlements of the dam crest during an exceptional seismic loading

or exceptional flood event (i.e. PMF). The un-gated spillway of Serra degli Ulivi is able to evacuate the unrouted PMF peak with a hydraulic head of 2.8 m above the spillway sill. For the hydraulic simulations, it is therefore assumed that the crest level would have to drop to 571 m a.s.l. to provoke an overtopping that could compromise the global stability of the embankment.

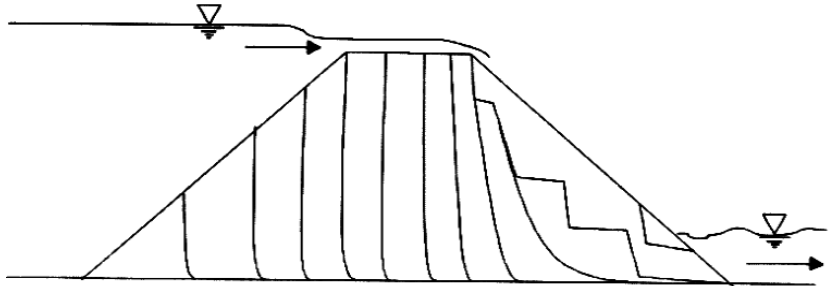

Fig. 4 – Development of headcut mechanism (Hanson 2000)

For Serra degli Ulivi two possible failure mechanisms were identified:

1. Structural failure of the embankment sections with drop in the crest elevation, overtopping of the embankment dam, followed by headcut erosion and breach opening in the central part of the embankment.
2. Structural failure of the concrete block adjacent to the embankment (e.g. sliding of a block) due to uplift or intense leakage through the construction joints, followed by the collapse of a second concrete block and the lateral erosion of the unsupported embankment.

2.2 Hec-Ras model

The software USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS) is a mono-dimensional program that directly integrates the open channel flow equations. In the present study an unsteady simulation was used. For the Serra degli Ulivi study two models were used to conduct the dam break analysis: the first one was used to simulate the breach formation and conduct a sensitivity analysis of the most influential parameters in order to finally obtain the hydrograph produced by the dam collapse; the second one was used to study the wave propagation along the river until the confluence with the Pesio River.

2.3 Dam breach parameters

Since no case history was found in literature for failure of ACED, some hypothesis had to be made based on conceptual assumptions. For the failure of the embankment part it was assumed that when the asphalt core is exposed due to the headcut erosion, then an over-folding in the downstream direction of the asphalt diaphragm embedded laterally into the embankment occurs until it cannot accommodate the deformation and a tear forms in the upper part and initiates the breach; after that, the cut in the asphalt fast develops and extend down to the concrete plinth and when it reaches the plinth, the breach enlarges laterally progressing more slowly. The shape of the breach would be triangular or trapezoidal with narrow bottom width. It is also reasonable to believe that the side slope of the trapezoid would be steeper than that in which develops in homogeneous embankment. For the latter Von Thun and Gillette (1990) suggests a lateral slope of 1:1, however for very cohesive materials (e.g. the case of clay core dams), values of 0.5:1 and 0.33:1 could be appropriate. For the purpose of the present analysis, a slope of 0.3:1 was assumed.

Case 1 (Embankment erosion) – When overtopping occurs in an embankment dam, the formation of a breach is the most common cause of failure. To numerically reproduce the progression, two main parameters have to be determined: breach geometry and failure time. Due to the high number of parameters involved, the formation of a breach is a complex process, hardly describable with mathematical tools (Froehlich 2008). In literature many formulas based on empirical models have been proposed for earth-fill dams to estimate both breach length and failure time. A non-exhaustive list is presented in Table 2. Most formulas allow to predict both breach geometry and collapse time as a function of water height above breach bottom (H_w), breach height (H_b) and the volume of the reservoir (V_w). For the present study a final height of 41 m (from 571 to 530 m a.s.l.) and an average width of 39 m were chosen; the geometry of the final breach in the embankment is presented in Figure 5 (left). The difference between the predicted and the chosen values is due to the fact that most formulas were derived for earthen dams and not for asphalt core dams. The peak discharge strongly depends on the failure time: short failure times correspond to high peaks. A distinction has to be made between breach inception time, collapse time and total failure time. Froehlich's references (1995) and (2008) both count the failure time from the rapid growth of the breach to the moment when the significant lateral erosion of the embankment dam has stopped. Singh and Snorrason (1982) consider a total failure time from breach inception to completion. For the present study a total failure time of 1 hour

was chosen, with an inception period and a collapse time of 20 minutes. The collapse time is consistent with the failure time provided by both Froehlich formulas. The final breach progression plot is composed of three steps, respectively, from 0 to 20%, from 20% to 80% and from 80% to 100% of the final breach size, lasting 20 min each.

AUTHOR	AVERAGE BREACH LENGTH [m]		FAILURE TIME	
Bureau of Reclamation (1988)	$3H_w$	145.5	$0.011\cdot(3H_w)$	96 min
Von Thun & Gillette (1990)	$2.5\cdot H_w+C_b$ $Cb = 54.9$ for $V_w > 1.23\cdot 10^7$	164	$0.02\cdot H_w+0.25$ (erosion resistant) $0.015\cdot H_w$ (easily erodible)	73 min 44 min
DAMCAT Catalog	$3H_w$	174	$\frac{H_w}{10}$	5 min
Froehlich (1995)	$0.1803\cdot K_0\cdot V_w^{0.32}\cdot H_b^{0.19}$ With: $K_0 = 1.4$ for overtopping	91	$0.00254\cdot V_w^{0.53}\cdot H_b^{-0.90}$	23 min
Froehlich (2008)	$0.27\cdot K_0\cdot V_w^{0.32}\cdot H_b^{0.04}$ With: $K_0 = 1.3$ for overtopping	70	$63.2\cdot\sqrt{\frac{V_w}{g\cdot H_b^2}}$	20 min
Marche (2008)	$0.657\cdot K_0\cdot\sqrt[4]{V_w\cdot H_w}$ With: $K_0 = 1$ for overtopping	97	$7.14\cdot 10^{-3}\cdot\frac{V_w^{0.4}}{H_w^{0.9}}$	25 min

Table 2 – Literature review of existing formulas to predict breach geometry

Case 2 (collapse of concrete spillway). For this scenario the sudden collapse of a concrete block creates a rectangular breach on the right, modelled as recommended by ICOLD Bulletin N. 111. The failure of a second, adjacent block was also assumed. Leaving the embankment non-sustained, a triangular breach would rapidly develop on the left side, with the same characteristics previously described for the Case 1 (slope H:V = 0.3:1). The failure time would be shorter than in Case 1, as the first concrete block would fail in few minutes and the second one may follow few minutes later. The initial rapid collapse of the concrete blocks was imposed in the model, followed by the slower development of the erosion in the embankment section. For this case a total failure time 30 minutes was chosen, with a bi-linear behaviour, opening the breach from 0 to 70% of the final size in 12 min and from 70% to 100% in the remaining 20 min.

The assumed breach geometry for both scenarios is presented in Figure 5 whereas the breach progression assumptions are given in Table 3

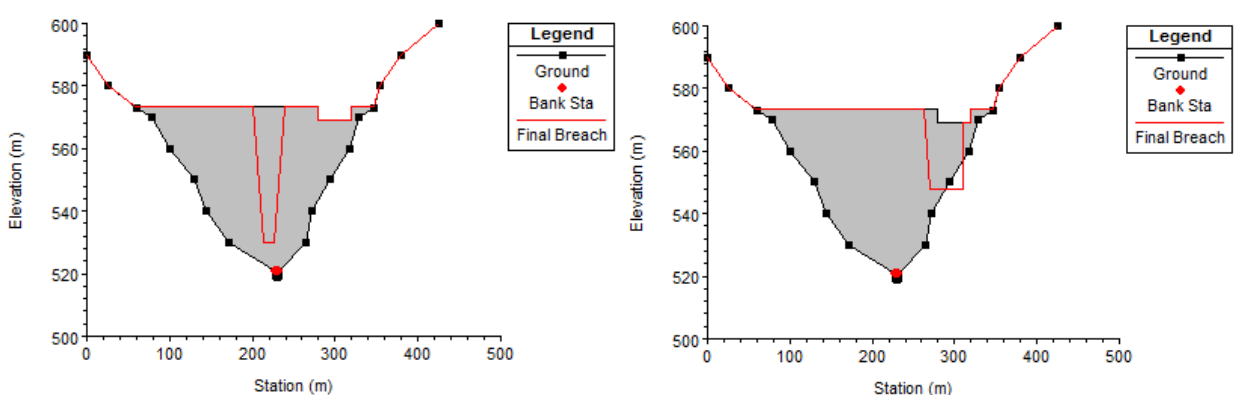

Fig. 5 – Dam breach geometry for both scenarios embankment failure (left), sconcrete block collapse (right)

Case 1		Case 2	
Breach progression [% of final size]	Time [min]	Breach progression [% of final size]	Time [min]
0 - 20	20	0 - 70	12
20 - 80	20	70 - 100	18
80 - 100	20		

Table 3 – Breach progression

2.4 Flood wave hydrograph

Both hydrographs were obtained using HEC-RAS. As for the breach geometry, uncertainties are common in the prediction of the maximum discharge and the shape of the hydrograph. The results are presented in Fig. 6 with the corresponding failure time. For completeness, the peak discharges are compared with common formulas in literature (Table 4). As for the breach parameters a certain scattering can be found in the estimation of the peak discharge.

AUTHOR	PEAK DISCHARGE [m^3/s]	
Froehlich (1995)	$0.60 \cdot V_w^{0.295} \cdot H_b^{1.24}$	10’582
Costa (1985)	$325 \cdot \left(\frac{H_w \cdot V_w}{10^6}\right)^{0.42}$	4625
MacDonald &Lanagridge (1984)	$1.153(V_w \cdot H_w)^{0.412}$	4625
Evans (1986)	$0.72(V_w)^{0.53}$	3643

Table 4 – State of the art of existing formulas to evaluate the peak discharge through the breach

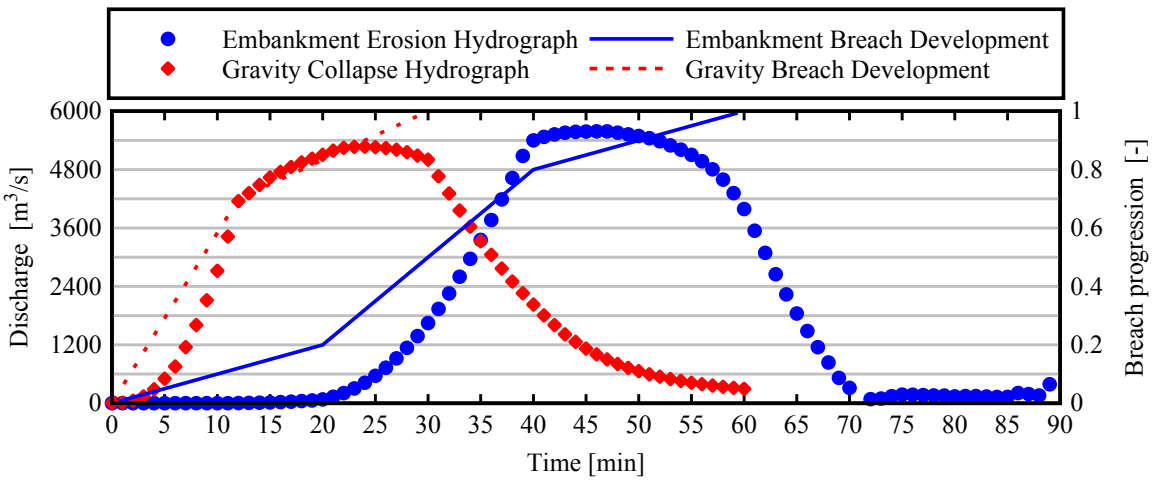

Fig. 6 – Hydrograph at the downstream end of the dam for both embankment erosion and concrete collapse

2.5 Downstream propagation

For the determination of the overflow areas, only the worst scenario was considered, i.e. the scenario which produces the highest peak discharge and the largest flood volume, that is the case of the erosion of the embankment dam.

The downstream propagation was simulated using the unsteady analysis in Hec-Ras. In accordance with the local regulation (i.e., *Circolare DSTN/2/22806*), the flood wave was investigated until the confluence with the Pesio River. The evolution of the hydrographs can be followed in some typical section along the valley as presented in Fig.7.

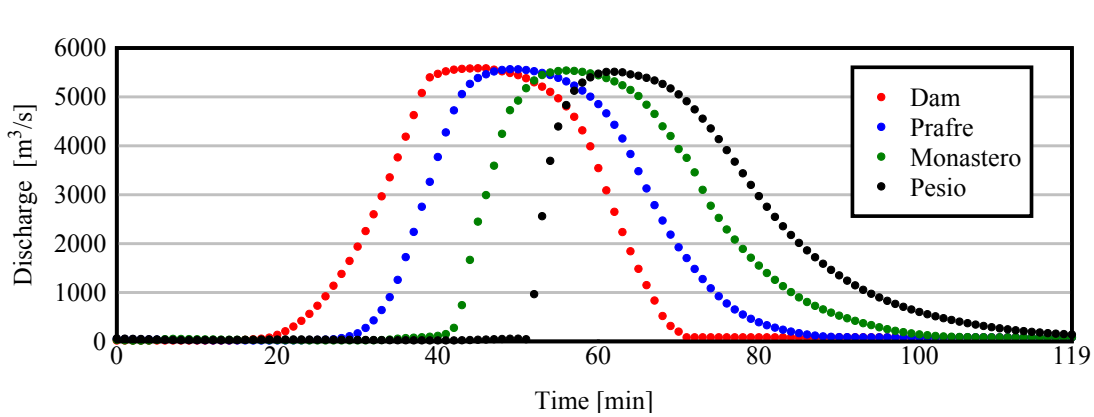

Fig. 7 – Propagation of the wave in the Pogliola Valley

As expected the results of the numerical simulation showed a negligible peak routing effect in the Pogliola valley because of the steepness of the riverbed and the absence of floodplains and expansion areas. The peak discharge at the dam is 5586.1 m3/s and drops to 5514.8 m3/s at the confluence of the Pogliola into the Pesio, reaching this

section after a travel time of 25 min, nearly 70 min after the inception of the breach . The Q500 = 5585 m3/s for the Pesio River is slightly greater than the routed peak which justifies the selection of the downstream boundary of the study area, stopped at the confluence. Downstream of the confluence, the Pesio River presents a wider section that allows a more effective river routing within its natural sections.

2.6 Inundation areas

The results of the HEC-RAS simulations were exported into AutoCAD and then combined with the official cartography (i.e., CTR-Regione Piemonte) in order to determine the major inundations areas. This information is fundamental to prepare hazard classification maps of affected areas. The graphical results obtained are presented in Figure 8. As expected the critical sections are represented by bridges and constrictions, provoking inundation upstream. In particular, the area of major risk is observed in correspondence of the small settlement of Monastero, where the abrupt restriction due to the presence of a narrow arch bridge (n.11, shown in Fig.8), is responsible of the upstream flooding of several houses, farms and a section of the Provincial Road n.564. A detail of the above mentioned inundated area is illustrated in Fig. 9 on both CTR map and aerial backgrounds.

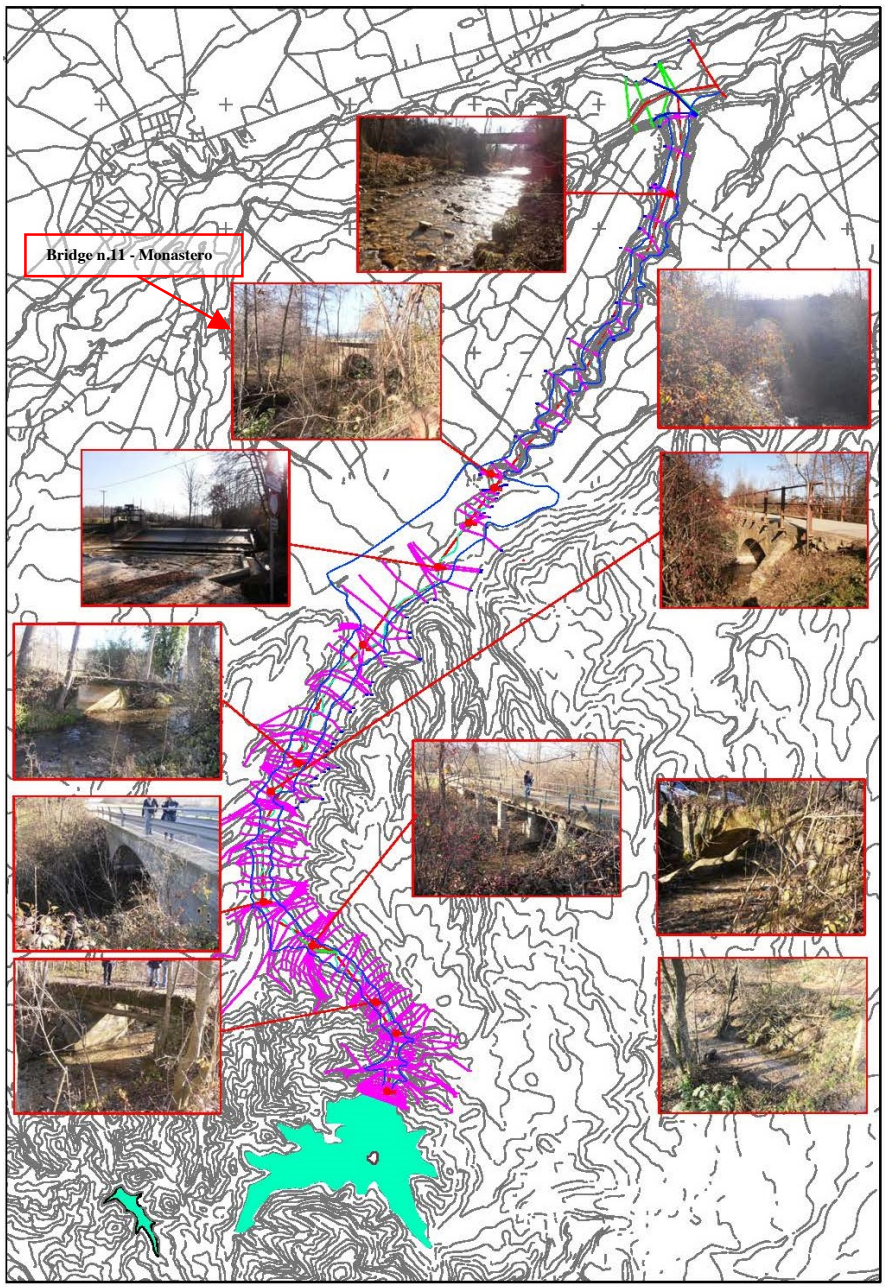

Fig. 8 – CTR Map from the dam to the Pesio confluence with Hec-Ras sections and overflow areas.

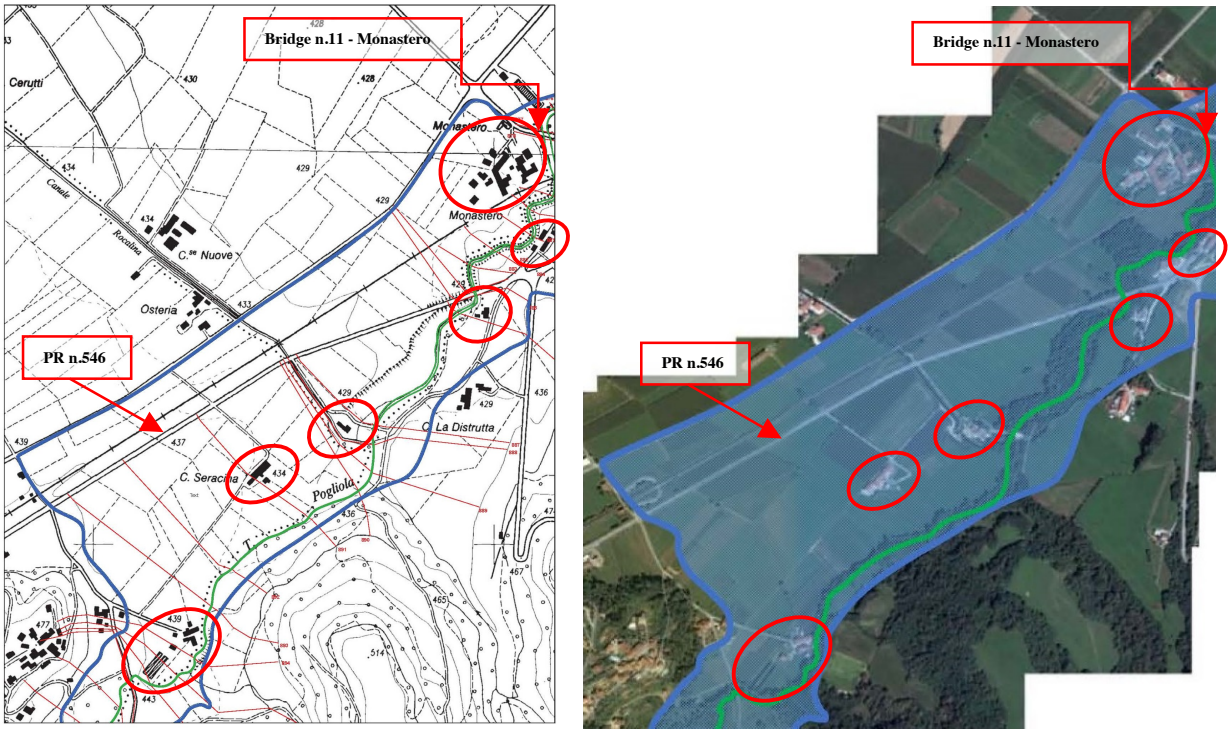

Fig. 9 – Detail of the most affected areas - Map (source: CTR – Regione Piemonte) vs Aerial photo (source: GoogleMaps)

3. Conclusions

The purpose of a Dam Break Hazard Analysis is to determine the time required by the hydrodynamic wave to reach inhabited centres and identify the main inundation areas. This information is crucial for the design of alarm systems, evacuation plans and hazard maps. In this context, the present study analysed the Dam Break failure of Serra degli Ulivi Dam. Being the latter an asphalt core dam, no previous case history was found in literature and the failure mechanism was based on conceptual assumptions. The hypothesis of a narrow breach with steep side slope was tested in parallel with the sudden collapse of the concrete overflow block. Both scenarios showed similar peak outflow discharges and the worst case was then used to determine the inundation areas. The propagation of the wave into the downstream valley was simulated by means of Hec-Ras. Results showed a negligible dampening of the flood peak; nevertheless, the peak discharge reaching the Pesio River is smaller than the magnitude of the natural floods of the Pesio River and the wave propagation analysis was stopped at the confluence. The results also allowed to determine the inundated areas along the Pogliola. The most affected areas are observed immediately upstream of bridges and river crossing where the hydraulic section contracts.

The authors would like to point out that the study was conducted base on the preliminary design of the dam. The basic assumptions may change after design changes to the dam in next stages of the project. Furthermore the hypothesis made concerning the breach formation are based on a conceptual thinking and engineering judgement. Due to the large diffusion during recent years of the asphalt-core embankments, the authors also like to emphasize the necessity of developing breach models for asphalt-core dams based on physical and numerical modelling for a better understanding of the real or most probable failure mechanism.

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Notation

A	Wet surface [m^2]	L	Breach length [m]
C_b	Constant	Q	Discharge [m^3/s]
H_b	Breach height [m]	Q_{500}	Discharge with return period of 500 years [m^3/s]
H_d	Dam Height [m]	S	Surface catchment area [km^2]
H_w	Water height above the breach [m]	V_w	Reservoir volume [m^3]
K_0	Constant	Y_m	Average elevation of the free water surface [m a.s.l.]

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