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DESIGN FLOOD ESTIMATION: LESSONS LEARNT FROM SELLA ZERBINO DAM-BREAK

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ABSTRACT

In 1925 two dams were constructed across the Orba River to store water in Ortiglieto reservoir. The 13th of August 1935 the flood spillways of the main dam - Bric Zerbino - were unable to discharge the flood and both dams were overtopped. The main dam was not damaged while secondary dam - Sella Zerbino - collapsed. This paper describes in detail the rainfall event that caused the dam break and critically compares the historical peak discharge to the design discharges of the existing dams located in Northern Apennine region.

Key words: *dam safety, design flood, spillways design, Sella Zerbino dam break*

INTRODUCTION

Dams can be considered among the most reliable structures, as indicated by the centuries-old experience in their construction and operation. Nevertheless dam failures, including those with many casualties, occur all the same. Failures and accidents of many large dams in Europe and in USA have been recorded (ICOLD, 1974; USCOLD, 1988): 500 accidents of various severity occurred (200 of which were cases of failure) out of a population of about 9000 dams constructed up to 1965 (ICOLD, 1991).

The failure probability is in the order of 10^{-3} per dam year (CHENG, 1989; THANG & YEN, 1991) and the risk of victims due to the failure of dams of any type is estimated to be $5.1\ 10^{-8}$ persons/year, corresponding to five victims per 100 million bodies (KALUSTIAN, 1995): this statistics is clearly dominated by major catastrophes, such as South Fork and St. Francis in the USA, Bouzi and Malpasset in France, Gleno, Zerbino and Vajont in Italy, Banqiao in China, Tigra and Machhu II dam collapse in India.

Many cases of dam failure were analyzed and simulated (BOSA & PETTI, 2011; NATALE *et alii,* 2008; PILOTTI *et alii,* 2011; VALIANI *et alii*, 2002; ROGERS, 2006; BEGNUDELLI & SANDERS, 2007). Research projects, like CADAM (MORRIS, 2000), have been promoted and international conferences focused on dam safety (NATALE *et alii,* 1998; DE ALMEIDA & VISEU, 1997; DOUGLAS *et alii*, 1998; D.S.C., 1991) have been held.

The main causes of failure in concrete gravity dams are listed in table 1 (ICOLD, 1987; ICOLD, 2005).

Analysing a sample of 100 failures of concrete dam, JOHNSON & ILLES (1976) pointed out that the main causes of collapse are: overtopping (35%), problems concerning foundations (25%) and various causes due to design and/or construction errors, poor materials, earthquake and war actions (40%).

The number of masonry dam failures is high, even though the worst disaster in the history happened when a clay-core earth fill dam, the Banqiao Reservoir Dam

Main Cause of Failure in Concrete Gravity Dams	%
shear strength in the foundation	40
seepage in the foundation	40
Uplift	20
excess rates of flow	20

Tab. 1 - Main Causes of Failure in Concrete Gravity Dams

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in China, failed in August 1975 due to overtopping and caused more than 171000 casualties. The design flood discharge of this dam was 1-in-1,000-year while the estimated return period of the 1975 flood was 2000 years (Si, 1998; Graham, 1999, Xu *et alii*, 2008).

Also the masonry dam of Sella Zerbino broke due to an extreme flood that the spillways were not able to evacuate. The dam was overtopped and the falling jet eroded the foundations of the dam. The dam break drowned 111 folks (Natale *et alii*, 2008). After the event the designers of the dam were prosecuted. The criminal trial against the owner and the designers ended on the 4th July 1938 and the defendants were sentenced not guilty.

DESCRIPTION OF ORTIGLIETO RESERVOIR

The first plan to store water from Orba River, see Fig. 1, date back to years 1895-1899. The original project was soon modified to bring the reservoir capacity from 12 to 18 hm^3 by increasing the height of the main dam and obstructing a secondary saddle, named *Sella Zerbino*, with a smaller dam. Accordingly, two masonry gravity dams were built. The main dam, called *Bric Zerbino*, is 44.0 m high and 145.5 m long.

The secondary dam, that was 14.5 m high and 120.0 m long (Anonymous, 1925), was inappropriately founded on a poor bedrock (Novarese, 1938; Peretti, 1936; Accusani, 1936; De Marchi, 1940). The flood outlets of the main dam consisted in: 12 Heyn siphon spillways, a gated side spillway and a bell-mouth pressure outlet. The design discharge of these outlets amounted to 860 m^3/s in total. The secondary dam had no spillways. The reservoir went in operation in 1925.

DESCRIPTION OF THE CRITICAL STORM

After a long dry period, at 6:15 a.m. of 13th August 1935 an exceptionally heavy rain hit the *Orba* basin.

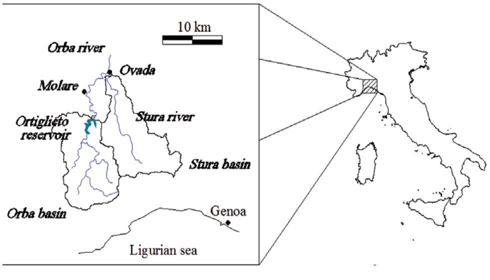

Fig. 1 - Orba and Stura Basins

At 7:00 a.m. the rain intensity increased and kept on without interruptions until 3:00 p.m. The most intense rain fell between 7:00 and 8:00 a.m. and between 2:00 p.m. and 3:00 p.m.

The synoptic representation of that very day can be summarised as follows: the warm and humid wind coming from south east over the Tyrrhenian Sea came across the cold front descending from north, as confirmed by the wind direction recorded during the event (Alfieri, 1936; Coyne, 1937; Vicentini, 1936). The humid front streamed towards the three gaps in the water divide of the Apennine indicated by dashed lines in Figure 2 and very intense showers hit Orba and Stura watersheds.

The records of daily rainfall at the gauging stations shown in Figure 2, indicate that the rainfall of August 13th was: (1) the maximum ever recorded by the rain gauges denoted by the symbol (○); the maximum in year 1935 for that indicated by the symbol (■). The rainfall recorded elsewhere was not remarkable (+). The area hit by the thunderstorm is limited to an extension of 350 km^2 as indicated by the ellipse in Figure 2 which is 39 km long and 14 km wide.

Due to the exceptional intensity of the rainfall, the rain-gauge pans in the area were not emptied at 9:00 a.m. as prescribed by the Italian Hydrographic Service operational protocol; as a consequence, the measure of the cumulative rainfall height is assigned to very day of the storm. Instead, the records of 6 rain-gauges located along the shoreline, southward of the critical area, assign the rainfall of the first part of the storm to the previous day, the 12th of August.

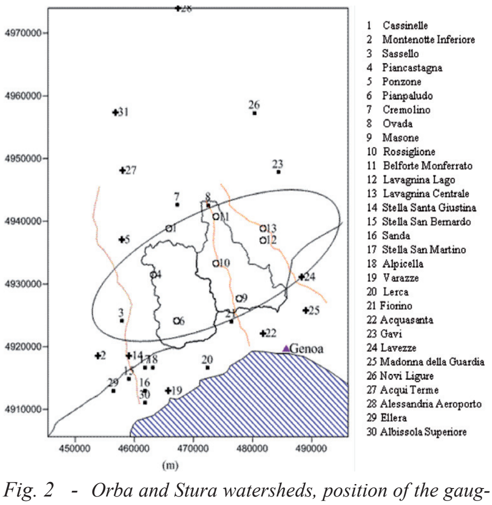

Fig. 2 - Orba and Stura watersheds, position of the gauging stations and area hit by the storm

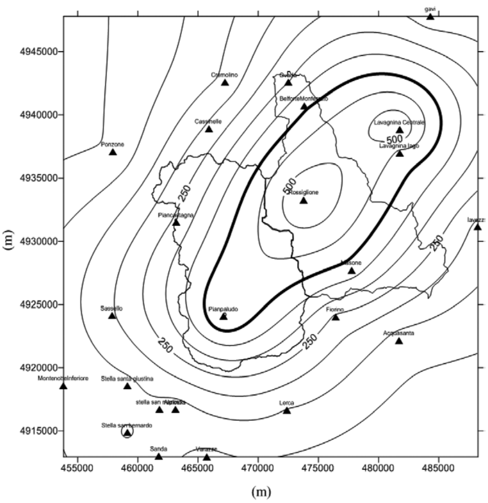

Fig. 3 - Isohyetal map of the 13/8/1935 event

The isohyetal map of Figure 3 shows that the total rainfall, averaged on the Orba basin area from 9:00 of August 13th to 9:00 o'clock of the day after, was 366 mm during the 8 hours event; De Marchi (1937) and Mangiagalli (1937) estimated 389 mm.

The storm hyetograph used in this study and given in Figure 4 was reconstructed from the one measured at the Lavagnina Centrale raingage (Alfieri, 1935).

ESTIMATION OF THE EXCEPTIONALITY OF THE EVENT

The Authors analysed the measures of annual maximum daily rainfall in 39 gauging stations in the area, recorded in the period 1930-2011. The position of the rain gauges is shown in Figure 2. The number of elements in each one of the samples ranges from 9 to 75

First of all, Studentized Statistics non-parametric test (Kottegoda & Rosso, 2008, page 307) was applied to detect outliers in the samples: only for Lavagnina Centrale and Lavagnina Lago stations, the 1935 rainfall could be considered an outlier at 5% significance level.

The Authors used the regional quantile estimator to evaluate the return period of the rainfall event under consideration (Cunnane, 1989) assuming that all the samples included in the same statistically homogeneous region, have the same Generalised Extreme Value parent distribution, when data are normalised by dividing the measures by the average of their sample. Rossiglione, whose rain-gauge was seriously damaged during the event, and Lerca samples were rejected according to the chosen homogeneity criterion

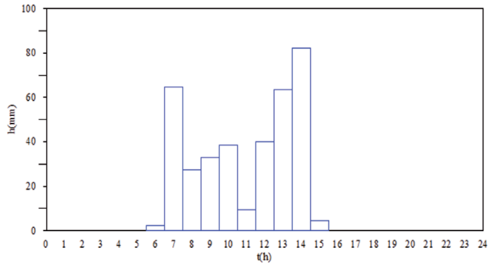

Fig. 4 - Reconstruction of the 1935 hyetograph on Orba basin

based on statistical distribution of standard deviation and skewness coefficient.

To satisfy the requirement that only independent data should be included in the regional sample, only one among multiple data recorded in different rain gauges in the same day was retained. The final sample includes 572 values.

The parameters of the parent distribution are estimated by the Probability Weighted Moment method to obtain:

$$P(h) = \exp\{-[1-k(h-u)/\alpha]^{1/k}\}$$

being $\alpha = 0.286058$; $u = 0.792868$; $k = -0.130374$.

The return period of the 13/8/1935 rainfall event was then estimated for the gauging station in the area

In Tab. 2 the return periods for the gauging stations having the 13/8/1935 rainfall depth as the maximum ever registered, are highlighted in yellow.

Gauging Station	h 13/8/1935 (mm)	Return Period (years)
Cassinelle	212	26
Piancastagna	261	84
Ponzone	57	2
Montenotte Inferiore	103	2
Sassello	195	14
Piampaludo	453	297
Lavezze	185	6
Masone	377	112
Cremolino	161.2	10
Lavagnina Lago	433	721
Lavagnina Centrale	554	2980
Belforte Monferrato	390	298
Gavi	220	57
Ovada	300	224
Rossiglione	550	4682
Santa Giustina	135.4	3
San Bernardo	156.2	5
Sanda	112.8	3
San Martino	138	3
Alpicella	124	3
Varazze	90.5	2
Lerca	147.4	7
Fiorino	241.5	11
Acquasanta	128	4
Acqui terme	29.4	1
Alessandria Aeroporto	11.8	1
Ellera	102.0	2
Albisola Superiore	90.5	2

Tab. 2 - Return periods of the 13/8/1935 events estimated from the GEV distribution

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Among these 8 stations only for Lavagnina Centrale and Rossiglione the return period exceeds 1000 years. We can conclude that the storm that caused the failure of Sella Zerbino dam can be considered as exceptional only in a very confined area.

The VAPI project (De Michele & Rosso, 2001) had the target to determine a uniform procedure to evaluate flood discharges on the Italian territory. Some gauging stations that recorded the 13/8/1935 rainfall are in the database of this project as well. For some of these stations the return period of the 13/8/1935 event was calculated, and the estimates are higher than our ones. The estimated return period of Piampaludo rainfall exceeds 10,000 years, 588 years for Masone, 343 years for Piancastagna. This can be explained since the region that was considered by VAPI to estimate the parameters of the probability distribution is very wide and has characteristics that are almost different from the limited zone interested by the thunderstorm. As a matter of fact the VAPI procedure overestimates the exceptionality of the rainfall.

FLOOD WAVE RECONSTRUCTION

The *Orba* river basin, shown in Figure 1, has an extension of 141 km^2 (Visentini, 1936). The basin is shaped like a fan, as shown in Figure 5, since its four sub-basins join in Ortiglieto reservoir. Table 3 shows

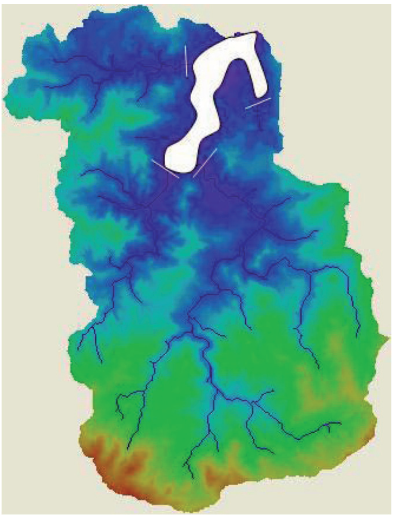

Fig. 5 - Orba Basin, closure sections of the 4 sub basins and the reservoir

River	A (km^2)	Mean Slope	T_c (h)	L(km)
Orba	141	0.29	5.50	11.12

Tab. 3 - Orba Watershed characteristics

the main characteristics of the basin, taken from a digital elevation model (DEM) of 20m square grid.

The effective rain was evaluated using SCS method (S.C.S., 1985). The basin Runoff Curve Number was determined from the land use and geologic maps (www.clc2000.sinanet.apat.it) edited by Piedmont and Liguria Regions, and considering the soil initially dry (CN I) since the rainstorm occurred after a long droughty period. An initial abstraction loss of 80% was considered in a time period of 1.5 hours.

The rainfall runoff process was simulated using the Instantaneous Unit Hydrograph (IUH) method. A semi distributed model based on the Geomorphological Unit Hydrograph (Rodriguez-Itube & Valdez, 1979; Rosso, 1984; Jain *et alii*, 2000) was determined. The basin was divided into 4 sub basins closed at the reservoir. To evaluate the flood discharge also the rainfall fallen directly over the reservoir water surface was considered, whose extension was in 1935 of 14 km^2. All the contributions were then added. The parameters of the GIUH were estimated from the available DEM. The estimated base time of the IUH, shown in figure 6, is t_c=2 hours.

The IUH's shape reflects the contribution of the four different sub-basins.

The rain falling on the water surface and on the shores of the reservoir is directly added to the inflow excluding losses and surface routing.

Figure 7 shows inflow and outflow hydrograph of 1935 event. Reservoir outflow was evaluated taking into account all the outlets that worked during the event (the siphon battery and the side spillway) and the flow running over both the dams but the dam break wave (Natale & Petaccia, 2012).

The reservoir level hydrograph is reproduced in figure 8 that shows the good agreement between the calculated values and the data collected by the dam keeper during the event, before the collapse of the secondary dam.

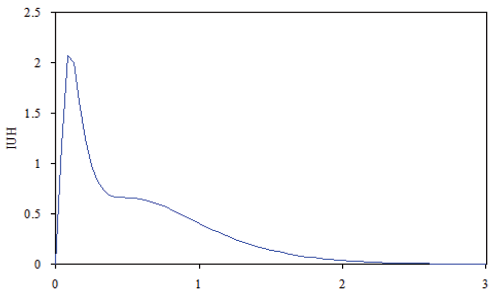

Fig. 6 - Orba basin IUH

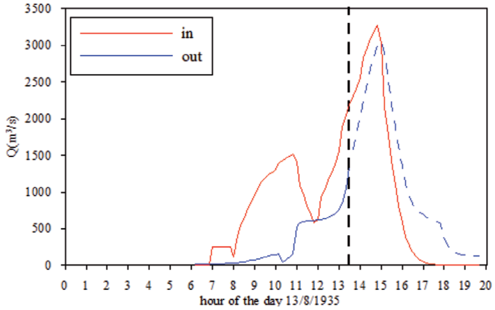

Fig. 7 - Discharge entering and leaving the reservoir during the 13/8/1935 event

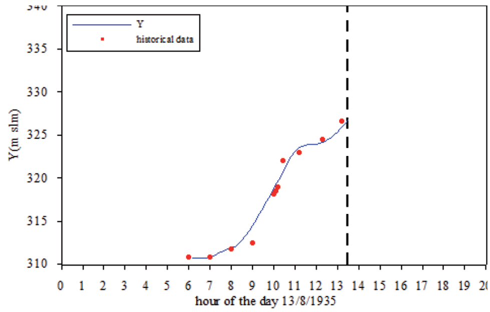

Fig. 8 - Ortiglieto reservoir levels: comparison between historical data and simulation

DAM	A(km²)	Q(m³/s)	q(m³/s/km²)	YEAR OF CONSTRUCTION
MENEZZO	0.89	3.60	4.04	1975
TENARDA	2.00	57.00	28.50	1959
LOMELLINA	2.63	34	12.83	1894-1896 ; 1908-1910
LOMELLINA updated	2.63	69.70	26.50	1938
GIACOPIANE	2.60	85.02	32.70	1920-1926
PIANSAPEIO original	5.70	53	9.30	1921 - 1926
PIANSAPEIO updated	5.70	150.77	26.45	1998
LAGO BADANA original	4.80	92.00	19.17	1907-1914
LAGO BADANA updated	4.80	160.00	33.33	2008
VALNOCI	7.52	158.00	21.01	1923-1931
BUSALLETTA	8.86	280.68	31.68	1971-1975
LAGO LUNGO	9.05	371.50	41.05	1887-1901
LAGO BRUNO (Lavezze)	6.01	308.00	51.25	1880-1883 e 1925-1927
ZOLEZZI	18.89	190.60	10.09	ND
OSIGLIETTA	20.50	680.60	33.20	1937-1939
BRUGNETO	26.91	769.63	28.60	1956-1959
LAVAGNINA	30.00	768.90	25.63	1911-1917
VALLA	68.00	276.08	4.06	1923-1925
ORTIGLIETO	141.00	860.00	6.10	1899-1927
CASTELLO	67.50	409.73	6.07	1936-1942

Tab. 4 - Characteristics of the 20 dams analysed in this study

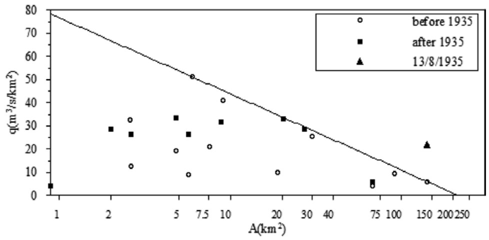

Fig. 9 - Design discharge of the dams in the region of the study and envelope curve

At 13:20 when the secondary dam collapsed, as indicated by the dotted line in Figs. 7 and 8, the flood discharge into the reservoir was about 1900 m³/s - between 2200 and 2500 m³/s for some others authors - and was still increasing. Some reports discussed in the trial (Lelli, 1937; De Marchi, 1937; Mangiagalli, 1937) state that after the dam break the incoming discharge kept on increasing to the value of 2500 m³/s and De Marchi calculated the peak discharge to be around 3000 m³/s. From our calculation the peak discharge is about 3300 m³/s.

CONCLUSIONS

From the statistical analysis carried out on the maximum daily rainfall depths we can assume that the 1935 event was really severe only for some stations close to the Ortiglieto reservoir. The peak discharge of the flood wave reaching the lake exceeded 3000 m³/s; for such a discharge the spillways were under designed.

Now consider the flood discharge capacity of 20 dams, listed in table 4, located in the region between the Ligurian Apennine and the sea (Anidel, 1952).

Looking at the envelope curve shown in Figure 9, the unit discharge of 21.46 m³/s /km² corresponding to the event that brought to Sella Zerbino failure (see the symbol ▲ in Fig. 9), is almost three times higher than the value given by envelope curve. Referring to an event of these characteristics all the dams built before (see the symbol ○ in Fig. 10) as well as after (see the symbol ■ in Fig. 9) the 1935 event would have suffered since the spillways are designed to release discharges significantly lower.

More updated techniques, like the one proposed in VAPI project, would have determined for the 1935 event a discharge lower than the one occurred. For the station Erro at Sassello the index discharge method gives a unit discharge, for a return period of 1000 years, of 9.59 m³/s / km², which is under the envelope curve.

Concluding, the event of 1935 would still be critic for the any existing dam in the region: the hydraulic design on the dam spillways should be reconsidered in order to avoid future risk.

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