Preliminary determination of design flood

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At first consideration, methods to calculate design floods do not change according to the size of the project or the size of the catchment area, at least for catchment areas smaller than several hundred square kilometres.

But in practice there are significant differences when it comes to small and medium sized dams:

• the first concerns the recurrence interval of the flood event the designer is seeking protection from; in fact, fairly short recurrence intervals may be accepted for very small dams where dam failure would have practically unnoticeable consequences downstream; conversely for dams representing a risk for populations downstream, long recurrence intervals must be considered;

• the second difference stems from the fact that small and medium sized structures are generally located in small catchment areas which rarely have gauging stations. In this case the quality of the available hydroclimatological information is less good. Greater imprecision in the results of the hydrological study must be taken into account when the type of spillway and its dimensions are chosen.
PRELIMINARIES

Today it is clearly accepted that the flood study, which serves in dimensioning spillway and discharge structures, cannot rely simply on flow observations. The methods used are of the hydro-meteorological type and combine rainfall data with flow data. Those methods may rely simply on statistical concepts (GRADEX method or the AGREGEE model) or propose a deterministic approach to the transformation of rainfall into flows.

In many countries (in particular in Anglo-Saxon countries), the Probable Maximum Precipitation - Probable Maximum Flood (PMP - PMF) method, developed by North-American engineering, is commonly used. This method defines a probable maximum flow for the catchment area studied which is the highest flood that can reasonably be imagined. The risk of such flow occurring is in principle infinitely low, and in any case cannot be quantified.

In other countries, like France, a design flood is computed along with a corresponding risk of occurrence. Depending on the hazard this represents for the downstream area - human fatalities, economic considerations, whether or not the dam is overtoppable, etc. - the selected recurrence interval may be of the order of $10^2$ to $10^4$. In terms of frequency, this would mean for example that a dam designed to handle a flood with a frequency of occurrence of $10^3$ has a risk of $1 \cdot (0.999)^{100} = 9.5\%$ of suffering the design flood during 100 years in operation. The risk is of the order of $1\%$ if the dam is likely to have to discharge the 1000-year flood (in 100 years operation). The designer must therefore be aware that the dam runs a non negligible risk of being confronted with a design flood determined this way, while bearing in mind that it could well withstand a higher flood thanks to freeboard.

Although at first consideration all of these methods are only applicable for sites where gauging records are available, practical considerations sometimes result in applying them in downgraded mode. This is often the case for small catchment areas where flow records are rarely available. The reliability of the hydrological study is however always highly dependent on the quality of the available hydroclimatological information.

2. PMP - PMF : Probable Maximum Precipitation - Probable Maximum Flood.
DESIGN FLOOD AND SAFETY FLOOD

The design flood is the flood with the longest recurrence interval considered in the reservoir. It is taken into account to determine the Maximum Water Level (MWL) and dimension the spillway, incorporating possibilities of flood routing. Often, the considered design flood is the flood with the highest peak flow. It is not always certain that this flood is the worst case in calculation of the spillway. A flood with a lower peak flow but lasting a longer period could have worse consequences. The minimum recurrence interval recommended for such a flood is between 100 and 10,000 years (10^2 to 10^4). The choice of a recurrence interval depends on the risk involved in dam failure. The dam’s intrinsic risk can be quantified by means of the parameter $H^2 \sqrt{V}$. Table 1 sets out recommendations for the choice of a design flood relative to this criterion. However, global risk is also related to the vulnerability of the downstream area (population density in the zone likely to be flooded in the event of a failure). The recommendations in table 1 must be beefed up in case of serious vulnerability (for example by going from a 500-year flood to a 1000-year flood). When the dam is of public safety interest, the recurrence interval should never be less than 1000 years, whatever the value of $H^2 \sqrt{V}$.

Once MWL has been calculated, the dam crest is set at a higher level. The difference between these two levels is called freeboard. Freeboard is essentially intended to avoid overtopping due to wave action but also plays an essential role in safety from flooding. A method for calculating it is given in Chapter IV, p. 73.

Thanks to freeboard, a dam should be able to withstand a flood (which is known as safety flood) higher than the design flood. This is by definition the worst case flood that could occur at the dam without any risk of failure. In the case of an ungated spillway at an embankment dam, the safety flood would be any flood that would cause overspillage, provided that it did not cause overtopping at any point in the chute that would jeopardise the fill itself. For a gravity dam, the safety flood would also correspond to the crest of the non-overflow part. For a dam with an impermeable core, the safety flood would be reached when the reservoir water level reaches not the dam crest but the crest of the core.

<table>
<thead>
<tr>
<th>$H^2 \sqrt{V}$</th>
<th>&lt; 5</th>
<th>5 to 30</th>
<th>30 to 100</th>
<th>100 to 700</th>
<th>&gt; 700</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recurrence interval in years</td>
<td>100</td>
<td>500</td>
<td>1,000</td>
<td>5,000</td>
<td>10,000</td>
</tr>
</tbody>
</table>

Table 1 - Minimum recurrence interval of the design flood for an earthfill dam without consideration of vulnerability downstream ($H$: dam height in metres; $V$: reservoir volume in hm^3)
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THE GRADEX\(^1\) METHOD

This statistical method, developed by Electricité de France (EDF), is the standard used in France. Its success is in particular due to its (relative) simplicity of use, which results from an extreme simplification of the process of transforming rainfall into flow.

ASSUMPTIONS OF THE GRADEX METHOD

Rainfall is considered globally over a certain period of time, equal to the average duration of the hydrographs.

The probability of precipitation events lasting various durations is a simple exponential decay function. The main parameter is proportional to the standard deviation of maximum precipitation values. It is called the exponential gradient, GRADEX. GUMBEL’s law is often applied. Its distribution function is as follows:

\[
F(P) = \exp\left(-\exp\left(-\frac{P-P_0}{\alpha}\right)\right)
\]

The GRADEX (\(\alpha\)) may be obtained using the event method. In this case, it is equal to 0.78 times the standard deviation. \(\alpha\) is of course a function of the duration of the precipitation considered.

Remarks:

- When \(P \to \infty\), \(F(P) \to 1 - \exp\left(-\frac{P-P_0}{\alpha}\right)\) and the Napierian logarithm at recurrence interval \(T = \frac{1}{1-F(P)}\) is equal to \(\frac{P-P_0}{\alpha}\). The rainfall depth varies linearly with the logarithm of the recurrence interval, the slope (\(\alpha\)) of this straight line being equal to the GRADEX.

- If \(P_{1000}\) and \(P_{100}\) indicate respectively the rainfall depth at recurrence intervals of 1000 and 100 years, then:

\[
P_{1000} - P_{100} = \alpha (\ln 1000 - \ln 100) = 2.3 \alpha
\]

(\(\ln\) designating the Napierian (or natural) logarithm).

When the catchment area reaches a certain saturation level, any increase in rainfall generates an equal volume of runoff for the same lapse of time. As a first approximation, this state is reached for recurrence intervals of ten years (impermeable catchments, with low retention), to 50 years (permeable catchments, with high retention).

The runoff law is obtained quite simply by translating the rainfall depth law to the point of the 10 or 50 year recurrence interval.

A physical interpretation of this process can result from observation of the graph of runoff variations relative to the amount of rainfall (see fig. 1, p. 35). The retention capacity of the catchment area is schematically represented by the difference between the bisector (rainfall = runoff) and its parallel, plotted in the middle of a cloud of points.

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1. See Bibliography, references 5, 7 and 11, p. 36.
Figure 2 (p. 35) illustrates this principle: the adjustment of the rainfall values has the GRADEX as slope. In this application, the recurrence interval retained for the hypothesis concerning saturation of the catchment area is 20 years (this corresponds to a reduced GUMBEL variable equal to 2.97). For volumes exceeding the runoff corresponding to this recurrence interval, adjustment is made by plotting a straight line with a slope equal to the GRADEX.

In this example the catchment area is instrumented and it is therefore possible to make a statistical adjustment of runoff, up to the 20 year recurrence interval.

In the case of small catchment areas without flow records, this is not possible. A regional approach based on nearby, if possible similar, catchment areas is necessary. It is however possible to consult national analyses such as SOCOSE or CRUPEDIX. These methods essentially require rainfall data and give an order of magnitude for peak flow at recurrence intervals of 10 years (10 and 20 years for SOCOSE). It appears that even a sizeable error on a 10-year (or 20-year) flood has a relatively weak influence on the 1000-year flood or the 10 000-year flood calculated with the GRADEX method.

It is noteworthy that for small catchment areas with no gauging stations, the evaluation of rare flood flows is almost exclusively based on rainfall information. Luckily this information is generally available for most of France.

A simple ratio of affinity is used to go from runoff in the considered time-frame to peak flow. This ratio is estimated from hydrographs; its average value is used (laws governing the probabilities of ratios and average runoff may also be combined, resulting in ratios that increase with the recurrence interval). For catchment areas with no water level gauging stations, we may use a ratio determined for similar catchment areas.

An example of this application is given farther along in this chapter (p. 31).

**DIFFICULTIES IN APPLYING THE GRADEX METHOD**

Strictly exponential decrease in precipitation with recurrence intervals leads to assigning extremely high recurrence intervals to certain observed events at a random point in France. It is true that total rainfall over 500 mm in 24 hours is not really exceptional in certain areas of France, but it is limited to certain regions: around 1000 mm in the Canigou region of the eastern Pyrenees, in October 1940; 800 mm in the Solenzara region of Corsica, in October 1993, etc. The standard design flood used in dimensioning a dam is therefore not the maximum flood that could occur.

There is no well-defined rule to calculate the time during which the hypothesis of equal increase in rainfall and in runoff is applied. Only a detailed coupled analysis of rainfall and floods will result in an estimate that is not too risky. In the absence of

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1. See Bibliography, reference 1, p. 36. The SOCOSE method is derived from the work of the American Soil Conservation Service.
precise data, the formulation of the characteristic time of the catchment area, developed in the SOCOSE\(^1\) method, may be used. In this method, the characteristic time is defined as the period during which the runoff is more than half of peak runoff. If no data is available on runoff on the site, the following regionalised formula may be used:

\[
\log D = -0.69 + 0.32 \log S + 2.2 \left( \frac{P_a}{P_t} \right)^{0.5}
\]

- \(D\): characteristic time (hours)
- \(S\): surface area of the catchment area (sq. km)
- \(P_a\): average annual rainfall (mm)
- \(P\): daily rainfall with a 10-year recurrence interval (mm)
- \(t_a\): average yearly temperature (°C)

**Note:**
This method is frequently used at a daily time-step when the catchment area is of a certain size, by virtue of a greater availability of daily information on rainfall and runoff.

The sudden rupture that affects the runoff equation at the pivot point (start of the rainfall equation) leads to an over-estimation of flow with intermediate recurrence intervals (50 to 500 years).

The affinity ratio to obtain the peak runoff is extremely variable. The method recommends keeping its average value. If we have properly chosen the duration in which the increase in runoff is equal to the increase in rainfall, it should be of the order of 1.5 to 2.0.

This method does not give a design hydrograph in a form suitable for simulation of flood attenuation. A bi-triangular shape which respects the duration, the peak runoff, and the volume of runoff can be used. In general, these design hydrographs result in over-estimations of attenuation capacity, as they represent only a part of the flood. One must often take into account the base flow in the river before the flood, when it represents a non-negligible proportion of the flood flow.

### The Agregee Model\(^2\)

A recent development by Cemagref\(^3\), this model is an extension to the GRADEX method. Re-using its statistical concepts and the hypothesis that when the catchment area is saturated, any increase in rainfall generates an equal increase in runoff. The modifications are based on:
- combining the laws of probability of rainfall and runoff to progressively go from the runoff equation to the rainfall equation;

\[1. \text{ See Bibliography, references 1 and 12, p. 36.} \]
\[2. \text{ See Bibliography, reference 9, p. 36.} \]
taking into account the statistical distribution of the affinity ratio (passage from average runoff to peak discharge),

the probability approach to instantaneous flow in order to obtain an overall design hydrograph.

This model makes no hypothesis on the equation of rainfall probability. The simple exponential decay function for rainfall relative to the recurrence interval is not imposed. Thanks to the progressive passage from the flow equation to the rainfall equation, the model avoids the over-estimation of the discharges of intermediate recurrence intervals (50 to 500 years). Although not very realistic, the single-frequency hydrographs obtained are easy to use in calculation of flood attenuation.

The AGREGEE model and the GRADEX method give similar results in estimation of extreme floods (1000 years to 10 000 years).

**The PMP - PMF method**

This method is very rarely used in France. It is based on knowledge of the Probable Maximum Precipitation (PMP) in the catchment area and a rainfall-runoff model to calculate Probable Maximum Flood (PMF). It results in a design hydrograph.

The PMP is defined as the highest theoretical precipitation that is physically probable for a specific geographic location, over a defined period of time. Its estimation is based on observed rainfall data and on maximising the meteorological parameters linked to precipitation: humidity, temperature, pressure of saturating vapour in the air, wind speed, convection, etc. Such calculation requires the skills of a meteorologist. In order to facilitate the calculation, some countries have published regional PMP estimates.

**The SHYPRE model: simulation of flood scenarios**

Cemagref has developed a model for simulating flood scenarios called the SHYPRE model.

This approach is based on gaining maximum value from temporal information about rainfall episodes in order to generate flood hydrographs with realistic shapes. By coupling a stochastic model for simulation of hourly rainfall and a simple model for transforming rainfall into discharge, the method generates a collection of flood hydrographs over a very long period.

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1. See Bibliography, references 4 and 8, p. 36.
2. See Bibliography, references 2, 3 and 10, p. 36.
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In this way, with no prior assumptions concerning the law of probability, the frequency distribution, from routine to exceptional, can be constructed empirically. This goes for both peak flows and for average flows and threshold discharges of various durations. Such floods are available to calculate hydraulic transients, including attenuation in a reservoir and modelling a flood plain.

In this method, the stochastic model of rainfall not only complies with temporal information, it also gives an original approach to infinite rainfall behaviour. Processing some 50 rainfall gauging stations along the French Mediterranean coastline has given an idea of regional trends simply through the characteristics of daily rainfall and has opened up the possibility of building a spatial regional model. The method has proven to be more stable than simple statistics, which depend heavily on sampling, and the model helps to evaluate the impact of anthropic effects.

Empirical formulas and regional formulas

These methods of estimating flow are extremely succinct and under no circumstance may be a substitute for a complete hydrological study.

The Francoù-Rodier envelope curve

From observations of maximum floods over the last 2 centuries in 1400 catchment areas throughout the world with surface areas in the $10 \cdot 2 \cdot 10^6$ sq.km range, FRANCOÙ and RODIER established an envelope curve formulated as follows:

$$Q/Q_o = (S/S_o)^{-k/10}$$

- $Q$ is the peak discharge of the flood (m$^3$/s) for an area of the catchment $S$ (km$^2$).
- $Q_o = 10^6$ and $S_o = 10^8$.
- $k$ is a regional parameter. It varies in France in the 5.5 bracket (Mediterranean zone) to 3.5 (oceanic zone in the north of France). At a global scale, the highest $k$ values (and thus in proportion the highest flow) are close to 6 like in Texas, New-Mexico and in some of the Pacific areas affected by typhoons (Korea, Japan, Philippines, …). On the opposite, the large African tropical streams are quite well characterised by an exponent $k$ value close to 2 (it is the case for the Niger and Senegal rivers).

Concerning an envelope of observed maximum floods, these estimates of discharge are not affected by the frequency of appearance. The authors, however, consider that a good part of the floods correspond to a recurrence period of around 100 years.

1. See Bibliography, reference 6, p. 36.
SYNTHESIS OF FLOW WITH A RECURRENCE INTERVAL OF 1000 YEARS CALCULATED WITH THE GRADEX\textsuperscript{1} METHOD

The GRADEX method has been applied by EDF for numerous French catchments of surface areas varying between a few square kilometres to a few thousand square kilometres. The regression established on 170 catchment areas of the peak discharge with a 1000-year recurrence interval relative to the surface area of the catchment is:

\[ Q = \lambda S^{0.72} \]

\( S \) is the area of the catchment in square kilometres and \( \lambda \) a parameter given in the table hereafter for the following three zones:

- **zone I**: the catchment areas of tributaries to the lower Loire river (Vienne, Creuse, etc.) located in the north of Central France, those of the Saône and Moselle rivers, and Brittany;
- **zone II**: the catchment areas of the eastern and central Pyrenees, of Aude and Ariège, of the Dordogne and the Lot rivers, the catchment areas of the Durance, the Fier and the Arve rivers, the Dranses, and the Isère rivers;
- **zone III**: the catchment areas of Haute Loire, Cévennes, the Tarn river, the right-bank tributaries of the Rhône river downstream of Lyons (Eyrieux and Ardèche rivers, etc.), Alpes-Maritimes, and Corsica.

<table>
<thead>
<tr>
<th>Zone</th>
<th>( \lambda )</th>
<th>Bracket for 90 %</th>
<th>Bracket for 70 %</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>4.05</td>
<td>3.07 - 5.36</td>
<td>3.4 - 4.8</td>
</tr>
<tr>
<td>II</td>
<td>7.4</td>
<td>5.2 - 10.4</td>
<td>5.9 - 9.2</td>
</tr>
<tr>
<td>III</td>
<td>16.4</td>
<td>9.1 - 29.7</td>
<td>11.3 - 23.9</td>
</tr>
</tbody>
</table>

This formula does not apply to catchment areas smaller than a few square kilometres. It only gives an order of magnitude which must always be rendered more precise with a local study. This order of magnitude is only to give a first opinion on the spillway capacity. It should never replace a more complete study.

RAINFALL RUNOFF ANALYSIS IN A WELL DOCUMENTED CATCHMENT

It is very rare to find a small catchment where we have a good understanding of the hydrology. The subject of this study is therefore not determination of a dam’s design flood. It presents data from an experimental catchment which support the recommended methods and justify the conclusion that the peak discharge of the design flood depends very little on how long the considered rainfall lasts.

\[ 1. \text{ See Bibliography, reference 5, p. 36.} \]
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PRESENTATION OF THE CATCHMENT AREA

Located in the Massif des Maures, a Mediterranean mountain zone, this catchment has been monitored by Cemagref since 1967, in the frame of the Réal Collobrier Research Project. The lands adjacent are mostly compact gneiss. There is little soil cover. The vegetation consists of scrubby briar and evergreen arbutus. The catchment area measures 1.47 square kilometres. Mean annual rainfall (1967-1990) is 1164 mm and the corresponding runoff is 626 mm.

The largest river floods are caused by intense storms occurring mostly in September and October. The following two events, with contrasting time histories, are an excellent illustration of the different types of response of the catchment (table 2).

<table>
<thead>
<tr>
<th>Date</th>
<th>Maximum rainfall (in mm)</th>
<th>Maximum runoff (in mm)</th>
<th>Peak discharge (m³/s)</th>
<th>Ratio Peak discharge/Runoff in 24 h</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.09.68</td>
<td>159</td>
<td>88</td>
<td>57</td>
<td>21</td>
</tr>
<tr>
<td>29.10.83</td>
<td>165</td>
<td>25</td>
<td>25</td>
<td>3.4</td>
</tr>
</tbody>
</table>

Table 2 - Floods of 13/09/68 and 29/10/83

APPLICATION OF THE GRADEX METHOD

The GRADEX method is applied with time steps of 1, 2, 4, 8, 12 and 24 hours, although these last times are much longer than the catchment’s characteristic time. Figures 1 and 2 (p. 35) are examples of observed 4-hour rainfall and runoff.

The plots show all events during which rainfall in excess of a given value was recorded.

CATCHMENT RETENTION

Figure 1 (p. 35) shows that the retention loss was between 70 and 100 mm in the three heaviest rainfall events, and between 55 mm and 100 mm in the three largest flood events. Statistical analysis as described later indicates that the "pivot point" for the GRADEX method should be taken as the 0.95 frequency (20-year recurrence interval, reduced GUMBEL variable, u = 2.97).

Table 3 shows retention versus duration of rainfall.

<table>
<thead>
<tr>
<th>Duration (hours)</th>
<th>1</th>
<th>2</th>
<th>4</th>
<th>8</th>
<th>12</th>
<th>24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retention (mm)</td>
<td>30</td>
<td>43</td>
<td>57</td>
<td>76</td>
<td>86</td>
<td>98</td>
</tr>
</tbody>
</table>

Table 3 - Retention for a 20-year recurrence interval for different durations of rainfall.
STATISTICAL RAINFALL AND RUNOFF DISTRIBUTIONS

Figure 2 (p. 35) shows rainfall and runoff on the vertical scale versus the standardised GUMBEL $u$.

The concave shape of the two curves is explained by the fact that events of very short return periods are plotted. The asymptotic rainfall distribution appears if we limit ourselves to the 27 most severe events (one event per year). It is not unusual for the most severe event (September 13th, 1968) to lie some distance off the curve.

Since there is considerable sampling uncertainty in a 27 value sample, the computations were run on the 150 most severe rainfall events observed for each of the durations considered in determining the asymptote slopes, proportional to the standard deviations. For this example, the fitting method would appear to overestimate the frequency of the highest values. This is attributable to the sampling alone, and application of this fitting method to samples of other rainfall durations yields estimates that are either entirely consistent, or underestimated (points concave downwards). Results are listed in table 4.

<table>
<thead>
<tr>
<th>Duration (h)</th>
<th>1</th>
<th>2</th>
<th>4</th>
<th>8</th>
<th>12</th>
<th>24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Runoff (L)</td>
<td>a 2.24</td>
<td>4.43</td>
<td>7.72</td>
<td>11.9</td>
<td>14.9</td>
<td>22.3</td>
</tr>
<tr>
<td>(mm)</td>
<td>b 3.90</td>
<td>7.60</td>
<td>13.5</td>
<td>22.4</td>
<td>29.3</td>
<td>44.2</td>
</tr>
<tr>
<td>Rainfall (P)</td>
<td>a 6.82</td>
<td>10.9</td>
<td>15.9</td>
<td>22.5</td>
<td>26.8</td>
<td>34.3</td>
</tr>
<tr>
<td>(mm)</td>
<td>b 20.1</td>
<td>31.5</td>
<td>46.5</td>
<td>66.2</td>
<td>79.7</td>
<td>106</td>
</tr>
</tbody>
</table>

Table 4 - GUMBEL parameters $a$ and $b$ for rainfall or runoff: $P$ (or $L$) = $au + b$

Application of these equations to a recurrence interval of 0.95 ($u = 2.97$) was the basis for preparing table 3.

The rise in the GRADEX and therefore the trend in the $a$ values (slope of the rainfall distribution line) matches duration perfectly and can be expressed as:

$$\text{GRADEX (t)} = 7.4 t^{0.51}$$

Rare runoff is derived directly from these equations; by setting the pivot point at 0.95 ($u = 2.97$), runoffs of different duration at recurrence intervals of 0.999 are estimated at the values shown in table 5.

<table>
<thead>
<tr>
<th>Duration (hours)</th>
<th>1</th>
<th>2</th>
<th>4</th>
<th>8</th>
<th>12</th>
<th>24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Runoff (mm)</td>
<td>37.2</td>
<td>63.3</td>
<td>98.6</td>
<td>146</td>
<td>179</td>
<td>245</td>
</tr>
</tbody>
</table>

Table 5 - Estimation of runoff at recurrence interval 0.999
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PEAK/MEAN FLOW COEFFICIENTS

Ratios between peak flow and mean flow have been calculated for all the events considered. Their distribution makes it possible to estimate the ratios to be used in determining peak flood flows for very long recurrence intervals.

The ratios are very variable, especially for very small flood volumes. Obviously they increase with the duration of the event: between 1 and 2 for 1 to 2 hour events, they may be as high as 25 for some 24 hour events.

With the ratios recorded during the three largest floods for the 1, 2, and 4 hour durations (i.e. 1.6 - 1.74 and 2.67), the estimated 1000-year flood peaks are very similar, i.e. 24.3 - 22.5 and 26.9 m³/s.

CONCLUSION

This application highlights the insignificant impact of the choice of duration on the GRADEX estimates of flood peaks for very rare events.

This is similar to what was found for much larger catchments where floods were estimated by studying one and two-day rainfalls.

DESIGN FLOOD HYDROGRAPH

The design flood hydrograph is calculated within the following constraints:

- A flood peak of 24.5 m³/s (average of the 3 estimates);
- Flood volume of the 1000-year 24-hour value (245 mm, runoff yielding 0.36 hm³).

The following formulation is used:

\[ q(t) = \frac{q_p \cdot 2 \cdot (t/D)^\alpha}{1 + (t/D)^{2\alpha}} \]

where:
- \( q_p \): peak flow
- \( q(t) \): flow at time \( t \)
- \( D \): characteristic catchment time, as defined above page 28.

A value \( \alpha \) of 2.7 meets these two requirements and gives the design hydrograph shown in figure 3.
Fig. 1 - 4-hour runoff and rainfall

Fig. 2 - Statistical distribution of 4-hour runoff and rainfall

Fig. 3 - Design flood hydrograph
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