Rain, runoff and rivers

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(Symons Memorial Lecture, delivered 21 March 1973)

To the civil engineer, rain (except as a slight impediment to construction) is barely worthy of remark until it gets into a reservoir (from where it can be sent for consumption); or into a river, from where it should be removed into the sea as soon as possible to avoid floods. This is not, of course, to say that a knowledge of rainfall physics and weather patterns is useless: many an engineering job has been more effectively and cheaply built or operated if this knowledge is exploited; but design of works is only rarely controlled directly by rainfall. The carefully garnered records of rain, gathered first by Symons through the services of the nobility, gentry, and clergy of these islands, must be converted to other forms for them to have the right significance. We have made slow progress in determining these conversions. Indeed, I could imagine Symons, coming back today, commenting on the present situation 'Yes! you have improved my techniques a good deal. Observations are now more professionally made; instruments and exposures are better understood; and retrieval of the data is quicker and more efficient; but your progress on using the data is still poor in fields outside agriculture, and your understanding of the quantitative aspects of the effects of rainfall on sanitary and engineering problems (to which I called attention in 1862) is positively rudimentary.' It is the purpose of my lecture to substantiate this view, and to give you an insight to the practical problems of interpretation that an engineer may have if he is responsible for works on a river which are, in the sonorous words of the Charter of the Institution of Civil Engineers, 'directing the Great Sources of Power in Nature for the use and convenience of Man.'

Consider then the situation of an engineer who for design purposes needs to know details of the hydrology of a river. These details may include, among others, the maximum flood likely in a given return period (usually needed for safety assessments); the distribution and magnitude of the response of the catchment to rain (for dilution, availability of water and drainage purposes), and the timing of floods (for warning purposes). For these he needs hydrographs, i.e. the time-variation of river flow, from which to judge and extrapolate the extreme and mean conditions with which he is concerned. It is very clear that the properties of a hydrograph depend strongly on many variables beside the rainfall and the catchment area. Pairs of hydrographs, taken with a number of variables chosen to be constant, but with one variable different, show strikingly different characteristics. The steepness of the river system affects the peak shape; the previous rain on the catchment affects peak heights; and the geological characteristics affect the width of the hydrograph (Fig. 1); and there are other, more subtle effects, usually non-linear, which interact.

Taken as a whole, averaging over many years, these differences show up as variations from one catchment to another of the ratio of the annual runoff to the annual rainfall. In Britain, this ratio varies from 86 per cent (14 years on River Wye at Cefn Berwyn) to 18 per cent (33 years on River Lee at Fielden Weir). Yet another way of showing the irregular response of catchments is to inspect the records for the peak daily discharge in each year on a catchment, and to compare it to the mean rain flow that caused it. The mean rain flow is the mean rate of rainfall over a period multiplied by the catchment area. Table 1 gives the Thames data for 1921-71, excluding the 9 years when the biggest floods were from a frozen catchment. The rainfall was taken for the preceding 4 days so that the ratio runoff/rainfall can exceed 1.0 if heavy rain on the 3rd and 4th day did not reach the gauging point in time to contribute to the flood peak.
### TABLE 1. Thames at Teddington 1921-71 (9 years excluded)

<table>
<thead>
<tr>
<th></th>
<th>Peak flood/mean rainfall</th>
<th>Rainfall (4 day total)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest daily discharge (1920)</td>
<td>1.62</td>
<td>12 mm</td>
</tr>
<tr>
<td>Lowest daily discharge (1953)</td>
<td>0.26</td>
<td>31 mm</td>
</tr>
<tr>
<td>Mean of 42 floods</td>
<td>0.33</td>
<td>25 mm</td>
</tr>
</tbody>
</table>

The variation of this ratio gives a good impression of the non-linearities and the probable omission of some important variables, possibly connected with the geology of the area. This is also obvious in the case of the 7 times larger catchment of the Vaal, in South Africa, given in Table 2. The data available are not quite the same as for the Thames, so it is necessary to take the monthly flow:monthly rainfall ratio; so direct comparison of the absolute value with the Thames is inappropriate. But the two sets of ratios have some consistency, with their lowest value about one-sixth that of their highest.

### TABLE 2. Vaal river at Vaal Dam 1936-60 (no years excluded)

<table>
<thead>
<tr>
<th></th>
<th>Flood/monthly rain</th>
<th>Rainfall for the month</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest monthly flow (1944)</td>
<td>0.36</td>
<td>200 mm</td>
</tr>
<tr>
<td>Lowest monthly flow (1942 and 1947)</td>
<td>0.15</td>
<td>109 and 110 mm</td>
</tr>
<tr>
<td>Mean of 27 years</td>
<td>0.17</td>
<td>140 mm</td>
</tr>
</tbody>
</table>

![Graphs of river flow hydrographs](image)

Figure 1. Three pairs of river flow hydrographs demonstrating the effects of changing conditions of catchments.
(a) The effect of river slope; a steep river, such as the Dart, gives not only a high peak flow, but a narrow hydrograph. Notice the difference in the stream-flow scale for the Wensum hydrograph.
Figure 1. (b) The effect of geology: the permeable catchments give a much lower hydrograph than the impermeable ones.

Figure 1. (c) The effect of antecedent rain: two hydrograph from the same catchment, one for a rainstorm at the end of a wet previous month, the other with previously dry conditions.
The effect of the antecedent rain on the condition of a catchment might be standardized if the statistics are taken only of the first flood after the sudden ending of a long, totally dry period (i.e. nil antecedent rain). One would expect that the range of the runoff rainfall ratio for such floods would be distinctly less than similar ratios for floods on catchments with varying degrees of soil moisture. Reliable data of this sort are rare, but fragmentary evidence from the Groote River, South Africa, where there may be a total lack of runoff for 5 months or more in the dry season, rather disappointingly shows the highest-lowest values of the ratio much greater than that of the Vaal and the Thames. It might prove rewarding to examine floods from catchments which have some degree of natural standardization in them.

I. RUNOFF PREDICTIONS FOR THE DESIGNER

With these complications in mind, and with the pressure of successful completion of the job always on him, what is our hypothetical engineer to do? If he is fortunate, his site is close to a place where a long-term record has been kept and an extensive time-series of his required variable is available. To a meteorological audience, accustomed to the problems of analysis and extrapolation of long rainfall records, it is obvious that river records should not be taken immediately at their face value. Inconsistencies and non-homogeneities, often occurring many years ago, must first be carefully assessed before any reliability can be placed on the records: but once so adjusted, they can become the happy hunting ground of statisticians. With really long records, the changes of statistical parameters from one section to another introduce an uncertainty of extrapolation. This is important when the question arises as to the economical size of a reservoir to even-out the flow of a river to give a high reliable yield. For this purpose, the total annual volume of the river flow is needed: Fig. 2, again with the Thames data, shows how a 90-year record, if cut up into three 30-year series can be analysed for the tendency towards zig-zag reversals of the annual flow (the auto-correlation coefficient $\rho$ of one year's value to the following year); and for the tendency for persistence of a graph to climb and go on climbing, or vice versa (the Hurst coefficient $h$). The two coefficients $\rho$ and $h$ show, roughly, the short-term and long-term variations in a record. It is very clear from inspection of the three short runs, both of rain and of river

![Figure 2](image_url)  

Figure 2. Ninety years of data from the River Thames. The records of both rainfall and runoff have been sub-divided into three 30-year series, and statistical coefficients $\rho$ and $h$ for each series obtained.
volume, that very different decisions might have been made at the ends of any 30-year record from a decision made at the end of another. The non-linear action of the catchment is also shown by comparison of the rain and runoff records, the latter showing much larger relative fluctuations than the former. The reasons behind these changes of statistical parameters may be partly due at least to changes of climate of which we are only slowly becoming aware, and partly to changes of agricultural and other practices on the catchment. There is obviously much scope for further research in both respects.

It is, however, a fortunate engineer with so much data available. A more common situation occurs when only a short-term, perhaps very short-term, record of streamflow can be obtained, but there is ample and long-term evidence of the rainfall characteristics. How can the latter be used to extend and predict the extreme values of the former? One well- tried technique is the Unit-Hydrograph method (sometimes misguidedly elevated to a Theory). This merely consists in the linear addition of successive unit-hydrographs in time to extend the effect of long-period rainfall, and to regard the stream flows as being directly proportional to the rainfall intensity. In this way, one record of streamflow for a unit time and unit rainfall (suitably adjusted for evaporation and other losses) can be converted to the hydrograph for a much heavier rainfall, which may never have been experienced at all, and which has been obtained using purely meteorological techniques. It would be idle to suggest that a simple linear process can be totally successful in predicting such a complicated process. Fig. 3 shows hydrographs from a small agricultural catchment. They have been non-dimensionalised in time by the very best authority—the rain itself—for each storm lasted the same period of time. The streamflow is non-dimensionalised, dividing in each case by the total volume of water discharged. There is only moderate success in correlating the several storms: peaks are often well correlated—but there are notable exceptions; the width and shape of the curves have some similarities—but again both slopes and details show large discrepancies. On larger catchments there is usually even less similarity between unit-hydrographs derived from different storms. Nevertheless, on small and medium catchments this simple idea can be used, with a fair degree of satisfaction to the engineer, if good judgement is employed in selecting the storms for the analysis. For example, Swinnerton, Hall and O'Donnell (1972) have recorded storms on 8 motorway catchments of size up to 2-2 ha; having derived hydrographs in non-dimensional terms from half the storms recorded, they tested them on the other half—and found the peak calculated rate of runoff differed by an average of less than 20 per cent from the observed runoff.

The possible sources of errors in unit hydrograph analysis are many. For a start, the engineer is always hopeful that the few rain gauge readings will represent the distribution of rain over the whole catchment—a process rather like finding the volume of a hill from a few spot levels on its surface. The conditioning of the catchment, with its underlying geological aquifers, by any antecedent rain obviously varies from one storm to the next. The effect of storage, in aquifers, puddles, ponds and lakes may be very variable depending on man-made interference with the catchment surface. All these, and others, are of course under extensive research effort, and many catchments, with their hydrographs, are being analysed for this purpose. A useful start has been made by Nash (1960) whose method enables some properties of a unit-hydrograph to be estimated in Britain. Whether sufficiently general rules will emerge in order to synthesise accurate hydrographs from non-hydrological data on well-mapped catchments remains to be seen, but it is unlikely in the very near future.

Although the unit-hydrograph idea, with its simplifications and limitations, is often accepted as one method, other ‘models’ of catchment behaviour have been proposed. For instance, the effective part of the catchment can be considered as a series of reservoirs which delay the passage of water from the instant it falls as rain to the time when it passes the desired gauging point in the river. Fig. 4 demonstrates the effect. A 1 km² catchment is assumed, consisting entirely of one reservoir into which rain falls at 25 mm per hour for 1 hour. The outlet of the reservoir is a weir which connects directly to the gauging point.
Figure 3. Comparison of Unit-Hydrographs from an agricultural catchment. (Above) Four storms all of a natural duration 12 minutes ± 2 minutes of varying total rainfall R. (Below) Three storms, duration 24 minutes ± 2 minutes. The runoff rate h has been non-dimensionalized by dividing by the total volume of runoff W.

The hydrograph then takes the form shown, with a long weir causing much less delay to the water than a short weir. It is clear that observed hydrographs have a strong similarity to these rather simplified calculated ones. Considerable efforts have been made to represent real catchments in this way either by fitting a reservoir pattern to a few observed hydrographs, then to use the pattern for other rainfalls; or, with more difficulty, to take geographical and geological information and to synthesize the equivalent reservoirs. The results are only slightly more encouraging than for unit-hydrographs, since the conditioning
of the catchment by previous rain also affects the reservoirs that are already completely full and so give no storage.

But our engineer may now be in a desperate strait. He may find no hydrological data at all for his catchment (and this is unfortunately only too common, particularly in under-developed countries). Then there is only recourse to that oldest of artifices — the empirical equation of other men’s experiences. There have been many attempts to produce such
equations, largely for the maximum flood (which is of course of such importance for safety reasons). As an exercise, 37 of these equations have been taken from the literature and assembled into 2 groups—one of 20 where the simplest variable (catchment area) only is involved, the other of 17 where other variables (rate of rainfall, mean slope of catchment, etc.) are needed. In each case, the predicted flood was found for the same (imaginary) circular catchment in Britain, of 50 m² area (129 km²). Where other variables are needed they are consistently applied to the various formulae. Treating, rather unfairly, all predictions as of equal weight, they give

'Simple' formulae

mean of 20
410 m³s⁻¹ mean dev ±240

ignoring three outrageously high or low values
mean of 17
412 m³s⁻¹ m.d. ±193

'Complicated' formulae

mean of 17
387 m³s⁻¹ m.d. ±258

ignoring one very high value
mean of 16
311 m³s⁻¹ m.d. ±177

A well known enveloping curve of real floods experienced in Britain would have indicated 34 m³s⁻¹ per 1,000 ha, i.e. a total flow of 272 m³s⁻¹.

The scatter of these different pieces of opinion is great, so that much skill and experience is necessary to select the formula whose original data fall nearest the conditions being predicted. There is clearly no substitute in a particular catchment for a long-term series of measurements, and their analysis can form as exciting a 'relevant' type of scientific endeavour as high-speed weather forecasting by computer.

2. RIVERS AND HYDRAULICS

However, when an estimate of the flow variable has been made, by one artifice or another, the river engineer's problem is only just begun. He must now use the flow statistics to determine river depths, widths, velocities, forces and discontinuities. There has been much study of the hydrodynamics of open channel flows and some fascinating phenomena can be predicted. Regarding only gravity and inertia effects (i.e. not concerning changes of depth caused solely by the shear, 'friction', stress on the river bed), a fundamental property is a critical depth and velocity for each flow. This is not to be confused with the critical value of the Reynolds number. Taking incompressible, steady, two-dimensional flows, having no vertical accelerations and with a uniform velocity at every point in any one cross-section, two useful relations can be set up ultimately from the equations of motion

\[ E = \frac{u^2}{2g} + d; \quad F = \rho u^2 d + \frac{1}{2} \rho g d^2 \]

E and F are respectively called the specific energy and the flow-force. Because E can be changed either by the conversion of fluid energy to heat, or by changes of elevation of the bed of the channel, so the effects of friction and of bed levels on u and d can be forecast. Similarly, since F can be changed by forces imposed on the stream, so the effects of gravity forces or of drag forces can be forecast. Since the flow q in such a simplified system is \( q = ud \), both equations are converted to

\[ E = \frac{q^2}{g d} + d; \quad F = \frac{\rho q^2}{d} + \frac{1}{2} \rho g d^2. \]

Both equations are cubics in d with a minimum value at a critical depth \( d_e \). This value is of considerable importance, as a decrease in \( E \) or \( F \) when \( d > d_e \) gives the opposite effect on d from a decrease when \( d < d_e \). A drag force applied to a slow stream gives a fall in the water surface; in a fast stream the same force causes a rise. With the idea of \( d_e \), the pheno-
Figure 5. The functions $E$ and $F$ for flow in an open channel, plotted for one discharge $q$ only. Observe the critical depth at which $E$ and $F$ are minima.

memon of the sudden rise in water level—the hydraulic jump—becomes explicable, together with the curious zig-zag line waves seen on fast flows. The properties of $d_c$ fix the connection between the flow of water past an obstacle like a weir and the depth of water upstream of it—a device frequently used to produce the hydrologic data necessary for its own design! A considerable variety of hydraulic structures can be effectively designed by use of these ideas—if the flow over them is sufficiently near the simplified form assumed for this 'theory.'

The surface slopes and depth relationships which result from purely frictional influences are, however, not covered by the above simplified scheme. There are analogies with the assessment of pressure gradient in a pipe, for so far as the roughness of the bed is concerned, a river is but an unusually shaped pipe with a free surface as part of its boundary. Rivers though are rarely straight, and so secondary or spiral currents are caused at each bend. These act as slow stirring actions which increase the momentum exchange across a section, and so increase the apparent frictional forces. The much wider variety of bed surfaces in rivers than in pipes, and their variability along the river also gives much more uncertainty in assessment of the gradients caused by friction.

But the idea of the critical depth (and with it a critical velocity) carries with it the basis of a useful tool for the hydraulic engineer. This is the Scale Model (now sometimes called a Physical Model) of a set of boundaries so complicated that the simple assumptions can no longer be invoked. If a section of a river is modelled to a small scale then the boundary conditions of the flow are met; but the question arises as to how the flow $q$ should be modelled. If similarity of performance of the water surface is to be achieved, model to prototype river, then at any one place in the model the depth $d$ must bear the same relation to $d_e$ as the prototype depth is to its $d_e$. Using suffix $m$ for model and $p$ for prototype, thus

$$\frac{d_m}{d_e} = \frac{d_p}{d_e} \quad \text{(the 'Froude' scale law).}$$

When this is so, and also the simplifications of the theory are adhered to, then confidence can be put into predictions from very complicated boundaries of the resultant water profiles. It is this fact that permits accurate calibrations of water levels against flows to be made for many structures.

When there is friction acting, and the water surface is partially controlled by the shear stress of the bottom, difficulties and uncertainties soon arise. A model of the same relative roughness as the prototype, with smaller stresses on it because the velocities are lower in order to obey the Froudeian relationship, will have the roughness elements more deeply immersed in a laminar sub-layer: such a model would therefore act more like a perfectly smooth river. However, it is acting at a lower Reynolds number, so that its frictional properties are, relative to a square law for friction, greater than that of a smooth prototype. With luck and skill, an engineer may find a smooth-walled model acts like a much rougher prototype—and so get good correspondence of observed phenomena. Without this, some very misleading observations can be made on models. To avoid this, some models are made
with increased relative roughness, but there becomes a risk that secondary currents are then so distorted that other errors are too great. It has indeed been rightly said that the interpretation of hydraulic models is an art rather than a science.

When changes of water level are gradual and detailed knowledge of every small local level is not required, it is possible to set up a Mathematical Model of a river. By use of some simplifying assumptions, much observational data from the existing river, and of the equations of motion expressed in finite difference form, it is perfectly possible to predict levels for other more severe conditions. This is particularly useful when the flow is non-steady, either because of tidal influences at its mouth, or because of the impositions of a flood wave near its source. Such calculations depend, rather like the physical hydraulic model, on the uncertainties of the frictional terms. Extrapolation of observed friction gradients to extreme conditions is as hazardous as the problems of deciding on the size of roughness elements in a 'wet' model. For a design engineer, wishing to explore quickly the effect of many variables or of several courses of action, these calculations are quicker than a physical model. They have far less appeal and credibility to the lay client, who when faced with a wet model forgets the crucial importance of friction in the delight of the 'birds-eye' view of the whole of a river scene.

3. RIVERS AND SOLIDS

If the definition of 'rivers' is (as the O.E.D.), 'A copious stream of water flowing in a channel towards the sea, a lake or another stream,' this review of the effects of rain as runoff would be complete. However, a more general definition is to include the solids moved inexorably downstream with the water. Most engineering works involve some consideration of the sand and silt flow, and in many of them this consideration is probably more important than that of controlling the water.

The flow of heavy grains has in it elements both of the mechanics of fluids and of the mechanics of solids. At low water speeds (i.e. low shear stresses) the solids will not move at all. At a certain threshold stress, grains start to move, rolling at first over other grains, later, at higher stresses, starting to bounce or 'saltate' in short, low trajectories clear of the bed. At high shears, grains are carried so high into the water that they fall under the influence of turbulent motions, which appear to suspend them for very long trajectories and to distribute them all through the whole depth of the river. In some ranges of stress, and with some grain sizes, elegant and mysterious ripples spontaneously appear on the bed; at others, much larger and longer dunes appear, which are linked with small, but noticeable, standing waves on the water surface. With a mixture of grain sizes present, all these effects may occur at the same time, presenting a confused picture.

The complexity of the flow does not lend itself to analysis by the usual equations of motion either for fluids, or for solids. Convincing fundamental theories, not needing empirical coefficients, are yet to be made. However, the economic importance of predictions of grain movement have led to a vast bibliography relating to many experimental approaches. These can be grouped into 3 distinct families.

(i) Experiments concerning the threshold of movement—obviously of importance in works where no movement of the bed can be tolerated.

(ii) Experiments concerning bed feature shapes and their effects on the friction in the stream—of importance to studies where water surface gradients are needed.

(iii) Experiments to find the rate of solids flow, as a function of fluid parameters—of importance when equations or erosion or deposition are involved.

Of these, the third family presents probably the most difficult problem of observation; there is an enormously wide spread to estimates of solids flow found for any one set of fluid variables, by the many empirical equations proposed. There is an absence of a generally accepted theoretical approach, and this is largely due to lack of knowledge of the details of movements of individual grains. Views through the glass sides of laboratory channels at a combined sand-water flow are misleading, since the visible grains are close to
the glass and so may move unusually of the rest; and instruments to measure any properties of this sand disturb the flow so that it becomes unusually at the point of measurement. Recently, however, some progress has been made in understanding the motion of individual grains by using the artifice of fixing the bed grains by adhesive, and then observing the motion of a single similar grain over them, with no other moving grains near. Multi-exposure photographs show up the trajectories as the grain is propelled along the bed by the shear stress of the water stream. A long-standing difficulty is the understanding of how a grain motion rather suddenly changes from a low, 'bouncing' trajectory to a much higher wavering one, with the grain apparently suspended for long periods by the action of turbulence on it. It seems that at a critical stress, about 16 times the threshold stress necessary just to start movement, the grains bounce high enough to be captured and supported by vertical turbulent impulses of sufficiently long period and high velocity: the grains then follow a long, wavering path perhaps to a considerable height in the stream, before they suddenly find a fast downward eddy stream which brings them back to the bed.

Figure 6. Four photographs of a solitary grain propelled by a water stream over a rough bed. Flow from right to left. Photographs made by multi-exposure technique using a light flashing at 40 s⁻¹.

Top: At shear stress just above the critical threshold, grain rolls, always in contact with the bed.
Second: At higher stress, grain moves in a series of regular ballistic leaps of low elevation, with no interference by fluid turbulence.
Third and Fourth: At highest stresses, grain moves in very large and long leaps, of which the horizontal portion is wavy, showing the effect of turbulence.
The nature of the difference between the low height 'bed load' of grains and the far better mixed suspended flows of solids is only likely to be fully understood by an intimate knowledge of the mechanics of the processes. The time has gone when a crude observation of a mass sand flow can give an adequate background of knowledge to provide accurate estimates for this complicated set of processes.

With such uncertainty, and with the pressure to provide economical and safe solutions to particular problems, it is no surprise to find the engineer readily turns again to model studies. In local erosion situations, such as the immediate neighbourhood of a solid obstacle (a bridge pier or a river narrowing), the sediment flow is principally controlled by the fixed part of the boundaries; the consequent erosion or deposition is therefore rather well modelled at a smaller scale, and considerable reliance can be placed on the final 'equilibrium' heights or depth of sediment.

![Figure 7](image)

Figure 7. The dried out bed of a laboratory river where water had flowed through a narrowed portion (as if at a bridge crossing). Notice the deep erosion at the upstream end of the narrowing, which critically affects the design of the foundations of the bridge. Also the ripple formation on the bed, which increases the hydrodynamic resistance on the stream.

In situations where there is no permanent control of the water conditions, the sediment performance at one place is highly dependent on the past history of both sediment and water; so to model the bed accurately, all details of the motions must be accurately modelled at all points. This proves to be quite impractical at model scales less than 1:1, so that compromises must be made, with consequent uncertainties of the result. This aspect is highly noticeable in cases where rivers meander or braid on an alluvial plain, a situation where vast sums of money may be wasted if a river changes course. The incentive to provide predictions is so great, however, that many model studies of this sort are started in the hope that some clues will emerge about the full size river. The success rate is mediocre with small models, but better for larger ones: but that latter may then be so expensive that they represent no economy on anything less than a very large and extensive project.

4. The Future

And so back to rain: at the end of a tale of approximations, simplifications, and of just plain guesswork, the meteorologist may well feel self-congratulatory, both on account of
the apparent high accuracy of his rainfall measurements, and partly because he is so clearly ahead of others in the chain of responsibility. But looking at the basic data from a position somewhat removed from rainfall meteorology, the observer may well ask if point measurement of rain is the only variable which might be monitored. In so enquiring, one appreciates the skill, devotion and ingenuity put into successively more complex instrument designs, but somehow the purpose of the measurement may have become lost. The point measurement must be, of course, the ultimate reference point for more sophisticated variables, but little attention seems to have been given to the possibility, say, of direct integrating methods over a large spatial extent, or of improved methods of determining infiltration on the timescale of a single rainstorm. How long is to be before the forecasting technique for rainfall will include parameters which are more helpful to your professional brothers battling with Nature? Symons brought rainfall measurement from the jam-pot to the standard gauge: where are we now taking it?

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REFERENCES