APPLICATION OF LASERSCAN TO FLOOD MAPPING OF AN URBAN STREAM

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ABSTRACT

The advent of air-borne laserscan systems capable of recording land elevation with a precision of 15 cm makes it possible to measure rivers cross sectional area with such accuracy as to calculate flood levels of given return periods by hydraulic formulas. The sequence of steps for the preparation of flood maps is as follows:

1) Estimation of a design flood discharge such as the 100 years at a given point on a river.

2) Calculation of the height of the 100 year flood at a given section of a river (point 1) from hydraulic formulas such as Manning s.

3) Application of step backwater techniques to calculate flood heights at upstream sections starting with (2) and including adjustments for constricted openings like bridges.

4) Preparation of flood inundation maps of river stretches by connecting points determined by (3)

The applicability of some of these techniques were evaluated for the Parma River within its course through the homonymous city taking an all purpose laserscan survey over the city as the data base. The 100 year flood discharge was extrapolated graphically from a Gumbel distribution plot of a 22 years record of average daily discharge available at the site and 6 corresponding values of maximum instantaneous floods. Flood heights were computed by trial and error application of the Manning formula using the river sectional data from the laserscan survey. The final simulated worst case condition for the 100 year flood indicates that the city can be inundated from overtopping of the river s banks at various points if even a small amount of clogging of bridge openings takes place.

The conclusion about the system is that it can be very useful in flood studies in that it can reduce the field work to a minimum where stream s slopes are well defined. Where the low flow is limited to a small portion of the within-banks cross section the laserscan data can be used as-is with little field work to assess roughness coefficients. In larger rivers with flat slopes, where the water depth is more than a few decimeters and it occupies most of its cross section there is the need to measure some wetted cross sections within the length of the reach under study. Where river constrictions occur (culverts, bridges etc.) the geometry of the structures must be assessed in the field.

INTRODUCTION

The problem of mapping maximum flood levels or flood levels with a given probability of occurrence, for example the 100 year flood , is of primary importance in regional planning especially in defining zoning areas, thus places subject to urban development. Of similar importance is the mapping of maximum water surface elevations downstream from dams in the event of a dam breach or, in general the application of hydrologic and hydraulic modelling to forecast the propagation of a flood wave through a river network. These applications are rare because of the great expense of surveying cross sections in the field by standard surveying methods. The recent advent on the market of systems that record directly the altimetry of the terrain with a decimetric precision makes it possible to straight forwardly apply hydrologic and hydraulic techniques to reconstruct and eventually map expected water levels. The paper gives an example applied to an urban stream.

THEORETICAL ASPECTS

The height of the water along a stretch of a river during the passage of a flood wave is a function of the discharge and the morphology of the cross sections of the river. If the flow is free, not restricted by the accumulation of debris (trees) on the river bed and if the river is not undergoing erosion the flow is considered uniform. In this condition the hydraulic gradient, the surface water gradient, and the stream bed gradient are parallel and the cross sectional area, hydraulic radius and water depth are constant; and Manning s equation, one of the many practical variations of Chezy s equation, is applicable (Dalrymple and Benson, 1967). The equation, written in terms of discharge is given by:

$$Q = 1/n\ A\ R^{2/3} S^{1/2} \qquad (1)$$

Where

Q = discharge, in m^3s^{-1}

A =area of the section, in m^2

R =hydraulic radius, or ratio of A to P, the wetter perimeter , in m

S =hydraulic slope, dimensionless

n =roughness coefficient of the section, in $m^{1/6}$

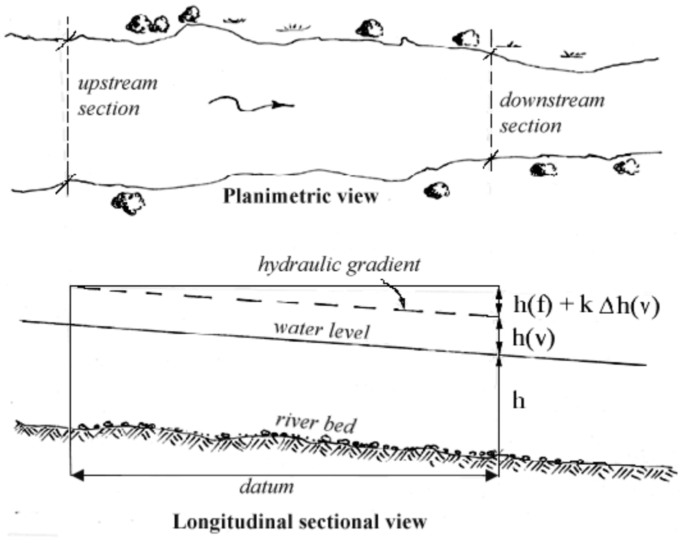

Figure 1 Definitions of hydraulic variables in fluvial sections

Because of its simplicity and lacking something better it is assumed that Manning s equation is also applicable for non uniform sections as are invariably found in nature, if the hydraulic gradient is modified to reflect only the losses due to the roughness of the section. Thus, starting with Bernouilli s equation to calculate the hydraulic profile between sections (see figure 1) one can write (Bailey and Ray, 1966):

$$h_0 + h_{v(0)} + h_f + k(\Delta h_v) = h_1 + h_{v(1)} \qquad (2)$$

where

h = water level at cross section, with subscritpt 0 for the downstream section and 1 for the upstream one

$h_v = \alpha V^2(2g)^{-1}$ = velocity head at section, in m, where V is the average velocity, α is a coefficient which is 1 for sections not subdivided and $\alpha = (\Sigma K_i^3/a_i^2)/(K_T^3/A_T^3)$ for subdivided sections where i refers to each subdivision and T to the total section, and g is the gravity acceleration constant, 9.8, m s^{-2}

$h_f = L_{(0-1)}\{(Q_0 + Q_1)/2\}^2(K_0 K_1)^{-1}$ = head loss between two sections because of friction, in m

$L_{(0-1)}$ = distance between the sections, in m

Q = discharge, in $m^3\ s^{-1}$

$K = 1/n\ (A\ R^{2/3})$ = total conveyance at a section (from Manning s formula)

$k(\Delta h_v)$ = head loss , in m, due to expansion or contraction between the sections, where Δh_v is the difference in velocity heads between the sections and k is a coefficient which is 0.5 for expanding sections and 0 for contracting ones

Where constrictions exist on rivers, such as at bridges and culverts, there is a change in the water surface profiles caused by these constrictions. The general equation that holds under these circumstances is, (Matthai, 1967),

$$Q = C\ A_d \sqrt{2g\ [\ \Delta h + \alpha_u (V_u^2/2g) \ldots h]} \qquad (3)$$

Where C is a coefficient related to the geometry and the roughness (material) of the constriction, $_d$ refers to the section downstream from the constriction and $_u$ to the upstream section. The other symbols are as previously defined.

The step backwater computational method solves the basic flow equation between sections by trial and error solution within specified tolerances. It is one of the many used to calculate water surface profiles under gradually varied flow conditions and it is considered one of the best when applied to natural streams. It can be applied to subcritical and supercritical flow conditions with the proviso that cross sectional calculations be made in an upstream direction for subcritical flow and in a downstream direction for supercritical flow. The theory assumes also that Manning s formula, derived for uniform flow, is also applicable to varied flow conditions. It is assumed that the following conditions are applicable:

1) Flow is permanent, 2) Slopes are small enough to accept the measured depths as equivalent to the vertical ones, 3) Water level is constant across the section, 4) Effects due to sediment load and air entrapment are insignificant, 5) All the energy losses are taken into account

Design flood derivation

Concerning the determination of the design flood discharge(s) to be applied, be that the maximum recorded or, better, that with a given probability of occurrence, it all depends on what hydrologic data are available. Under the best conditions there is a hydrologic station on the river stretch being studied with tens of years of record. In other cases, with less precision in determining the flood flow, there is a hydrologic station nearby within the same basin. With lesser precision yet, but probably mirroring reality more closely, there are no hydrologic stations in the basin and one must rely on extrapolating regional estimates of flow (Benson, 1962) to the segment of river being investigated. In the first case, when the hydrologic station is within the river stretch under study, the procedure to determine the flood flow, let s say the 100 year flood (probability of occurrence of 0.01), is straight forward. The annual maximum peak flood discharges for the period of record are ordered from the largest to the smallest. The probability of each flood is calculated from the formula m/(n+1) where m is the rank of the ordered series, 1,2,3 ..n and n is he total number of the data, that is the number of years of record. In practice the inverse of the probability is calculated, the so called return period, and thus the formula used is (n+1)/m. Next the data are plotted on Gaussian log probability paper or on a similar one, taking the flood values (or their logs if using normal probability paper) as the ordinates and the return period (or the probability) as the abscissa. If the data are aligned in such a manner that a straight line can be drawn safely through the points the line is extended graphically to intercept the value of the discharge at the return period of 100 or the value of 0.01 if using probability. Although it is always of help to obtain a graphic plot of the data one can calculate analytically the values from the appropriate Gaussian (or other) distribution equations as, by definition, a straight line drawn on a probability paper implies that the data that define the line come from that distribution or can be accepted as that. If the data do not follow a straight line but rather define a curve one can try to follow it to extrapolate the value wanted but only if the extrapolation is very short (the record is long). Alternatively one can search for a distribution that will linearize the data better; the choice is ample.

If the existing hydrologic record is from stations outside the area of interest one may resort to regional techniques of flood flow analysis to obtain an estimate of the 100 year flood where needed (Dalrymple, 1960).

Perhaps the most complex and documented study of mapping design floods in urban areas is that of Anderson (1970) relative to the tributaries of the Potomac river in Virginia within the Washington, D.C.,. metropolitan area This study made use of aerial photography combined with field surveys to assess the river geometry and included also the evaluation of the increased flood levels brought about by the urbanization process; one should consult the cited paper for details.

When the appropriate design flood has been determined, one proceeds with the hydraulic equations 1-3 to calculate the relative flood levels along the river banks and subsequently draw the corresponding flood inundation maps. Traditionally the river cross sections used for the calculations are surveyed in the field, a time consuming work that has not facilitated flood mapping (Giusti, 1984). The recent appearance on the market of laser instruments that can be mounted on aircraft to sense altimetry with a precision of less than 20 cm and with an equivalent planimetric positioning by differential GPS promises to improve the future of flood mapping by indirect methods.

LASERSCAN APPLICATION TO THE PARMA RIVER

A general laserscan survey test was made by the Compagnia Generale Ripreseaeree of Parma in collaboration with the German TopoSys Company, a leading producer of laserscan operating systems in Europe (Casella et al, 1998). Various surveys were made over the Province of Parma and one was conducted over the city itself covering the segment of the Parma River that crosses it. The characteristics of the survey were as follows: Flight speed 350km/hr; Flight height 810m above ground; Aircraft twin engine Piper Navajo Chief PA31; Camera RC 30 with 150mm lens;

Figure 2 Aerial photography of the Parma River where it crosses the city. The thin yellow lines indicate the sections used in the study. Flow is from bottom to top of figure.

Positioning and navigation with differential GPS with 2 Trimble stations on the ground; Laserscan viewing angle ± 7° implying a recording strip of 210m at the standard operating mode of 70m/s and 850m height resulting in a 5points/m^2 density and a ground imprint diameter of 30cm. Final data consisted in a network of points one meter apart in x,y and with an altimetric precision of ± 15cm. As indicated this was a test survey for a general evaluation of the system with no particular intent to maximize its capability for hydrologic modelling which is one of its common applications (Lohr and Schaller, 1999).

A photographic portrayal of the stretch of the river that was covered in the laserscan survey over the city is shown in figure 2. Also shown on the aerial photo as thin yellow lines and probably not readily visible in the paper are the cross sections selected for this study which are listed in the table sequentially in the y direction or longitudinally for the stretch between the first 2 bridges (bottom of the photo) and for the most downstream stretch covering the last three bridges (top of the photo).

For the purpose of this study it was important to convert the uncalibrated laserscan elevations to the absolute values of the terrain elevations above mean sea level so as to be able to map the computed water level elevations over the existing cartography. The availability of a large scale cartographic coverage of the city (scale 1:2000) made possible a comparison of the variability of the recorded DTM (uncalibrated with respect to the terrain altimetry) with the absolute values of the terrain elevations referenced to mean sea level. About 30 elevations of cartographic points chosen at recognizable positions, i.e. street intersections and plazas center points, were paired with the equivalent laserscan values. The results indicated a difference in values of 31.65m with a standard deviation of .19m which is somewhat in line with the indicated altimetric precision of the laserscan data of ± 15 cm.

Table 1 Longitudinal profile of Parma River within the city as defined by laserscan. Elevations given are in m, uncalibrated. Subtract 31.65m for actual ground elevations a.m.s.l.

		Most upstream reach						
*station	*y value	D y	Cum.D y	z min.	D z min.	z average	D z aver.	
35	958	-82	-82	88,3	0	station above bridge		
27	1040	0	0	86,98	1,32	station below bridge		
31	1095	55	55	86,55	0,43			
30	1143	48	103	86,59	-0,04			
28	1201	58	161	86,44	0,15			
29	1274	73	234	86,39	0,05			
34	1404	130	364	86,41	-0,02			
	Most downstream reach							
8	2228	0	0	82,64	0	82,94	0	
5	2279	51	51	82,62	0,02	82,98	-0,04	
14	2318	39	90	82,57	0,05	82,91	0,07	
10	2389	71	161	82,53	0,04	82,86	0,05	
13	2435	46	207	82,27	0,26	82,7	0,16	
11	2475	40	247	82,11	0,16	82,41	0,29	
12	2545	70	317	82	0,11	82,45	-0,04	
7	2595	50	367	82,04	-0,04	82,23	0,22	above bridge
6	2670	75	442	81,48	0,56	81,75	0,48	below bridge
4	2707	37	479	81,52	-0,04	81,78	-0,03	
5	2754	47	563	81,21	0,31	81,7	0,08	
2	2786	32	679	81,26	-0,05	81,72	-0,02	

* station numbers and y (position) values are shown as listed and have no special meaning

A few plots of the cross sectional data, x and z values, indicated that computations of hydraulic parameters from points one meter apart are readily made for this type of river, about one hundred meters wide, for which water occupies but a small portion of the river bed during the low flow conditions that prevailed when the survey was made (late September). Of more concern is the capability of the system to record the small longitudinal variations of the streambed elevations that are needed to calculate the river slopes between the sections. An examination of the longitudinal elevations shown in the table indicate that occasionally there is no fall in elevation between successive sections, indeed the differences in elevations between lowest points of successive sections are negative as are negative the elevations computed by averaging the minimum streambed elevations between the bank slopes (last two columns of downstream reach). Hydraulic computations are not possible here; one would have to conduct a field survey to assess the true slope of the stream or discard these sections chosing others that would meet requirements thus introducing gaps in the computations which may not be acceptable in some cases. According to the TopoSys Company (oral communication, 2000) the average accuracy in the vertical of plus and minus 15 cm can be improved to less than 10 cm under optimal survey conditions, i.e. best ground positioning by differential GPS with 4 instruments, low solar activity (night survey), and availability of signals from 8 satellites. A laserscan survey to meet these conditions could be scheduled in advance as the optimal satellite configuration over an area is known . This would entail, however, higher costs which may condition the economic feasibility of the survey. Admittedly the stretch of river chosen here is poor for studying the applicability of laserscan survey for hydraulic modelling of floods. It is an extreme case where man has changed the natural conditions of the riverbed not only because of the presence of the bridges but because most bridges have an apron at their downstream end which conditions the natural bed slope of the river. The presence of the bridges also requires information on the bridge geometry and field surveying of their hydraulic characteristics thus diminishing the usefulness of the laserscan method which is that of replacing or minimizing the collection of field data. On the other hand, most urban streams have been surveyed at least in the stretch where they cross the city and bridges information is available. Indeed the bridge design must have entailed computations of design floods to assess bridge openings and thus the information may already exist. Where the usefulness of the laserscan system is apparent for preparing flood inundation maps is in the countryside at large where there are no close spaced altimetric data. It is also clear that these surveys should be conducted along the river channel so as to span the channel itself and the flood plain and that the surveys should be carried out during minimum flow and vegetational cover conditions.

Flood levels on the Parma river

Although it is not the purpose of this paper to show hydraulic computational methodology it is instructive to complete the discussion by providing a practical example of computing flood levels along the Parma river.

The first problem is the assessment of the design flood. Fortunately, for the river stretch under study there existed a hydrologic measuring station at the downstream bridge with discharge data from 1956 to 1977, thus with 22 years of record. Hydrographic services in their data compilations publish only the annual average daily floods as opposed to the instantaneous values. These data are shown in figure 3 as a frequency plot on Gumbel s extreme value logarithmic paper (Tonini, 1959), a distribution widely used which often linearizes series of extreme hydrologic data. The line drawn ignores the 2 values of low frequency. This line, however, has little interest for us as the maximum daily flood levels originate from peak flows. These are shown by the upper points fitted by a line drawn through all the points and by one drawn through the upper 4 points only and parallel to the bottom line. The resulting 100 year peak floods are extrapolated , respectively, as 1400 m^3/s with the steeper line and as 1120 m^3/s with the flatter line, almost double the

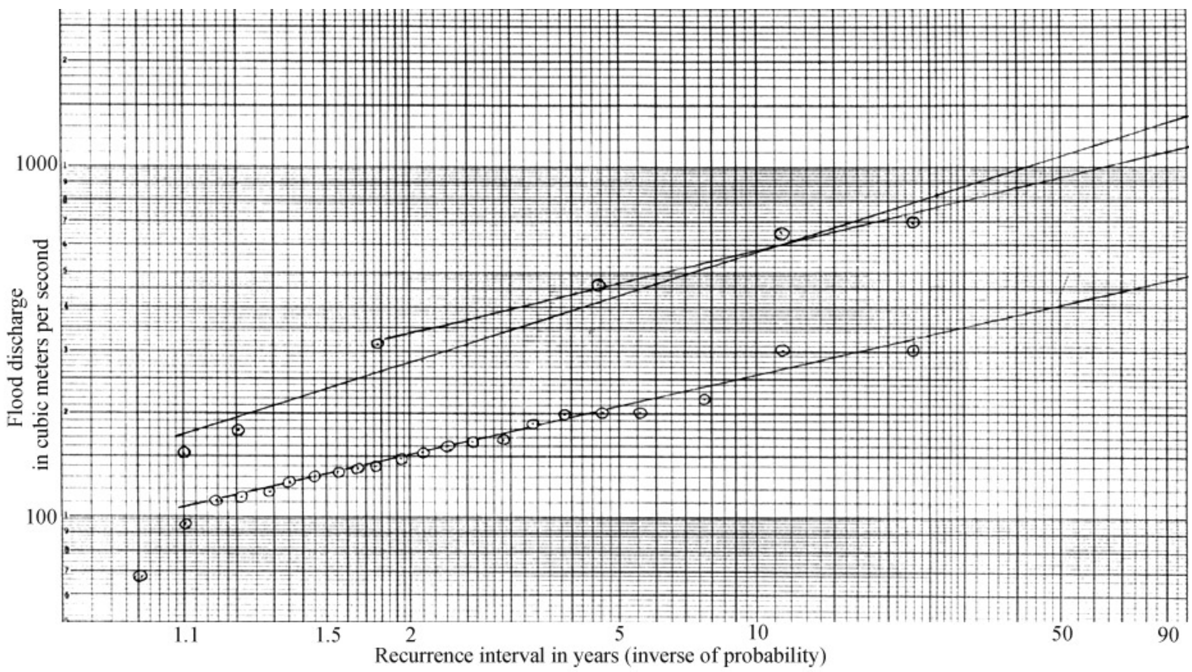

Figure 3 Extreme log data plot of the magnitude and frequency of Parma River floods at Parma City. Lower data are average daily values; upper data are instantaneous values.

maximum recorded of 680 m^3/s.

The estimated flood values are used with part of the data from the most downstream reach, stations 10 to 7 of table 1, to calculate flood levels. In applying Manning s equation the slope was computed by fitting an average line through the points rather than by taking actual differences in elevation between successive sections and the value of the roughness coefficient, n, was taken as 0.025. Computations were carried out using slopes defined by the minimum z values at the sections and also using slopes defined by the average z values at the sections. Under worst conditions, i.e. taking the highest flood of 1400 m^3/s and the lowest slope of 0.0011 one computes for station 7 a height of 56.33m which is short of the road elevation on the nearby bridge of 58.21m. Under the more reasonable conditions of 1120 m^3/s one computes a height of 55.28 m which would wet the bottom part of the houses flanking the river on the left bank but would not overtake the containment wall flanking the river on the right. Nonetheless it should be recalled that these computations refer to free flowing conditions without considering backflow effects from the reduced openings of the bridges and further potential decrease of the passageways openings caused by suspended debris (trees) clogging them. For these reasons it would be advisable that a field survey be taken of the Parma River channel and flood plain as it crosses the city and that the computations here indicated be carried out fully with more precise data.

CONCLUSION

It is possible to use laserscan surveys of river channels to derive most cross sectional properties of river reaches for hydraulic computations of flood heights and ensuing mapping of flood contours. All purpose routine surveys would not be useful and specific surveys with adequate ground GPS instrumentation for differential measurements should be planned to meet best satellite configuration for the area under study so that the vertical resolution can be brought to the 10cm level. The flight should be planned for periods of minimal vegetational cover and minimal river flow thus, during the winter low flow period. River sections that would meet the resolution of the laserscan data are most intermediate and large size rivers with a bank to bank cross sectional width of at least 20m, a fall in elevation (slope) of about 1% and where low flow depths are small and occupy but a small part of the river channel. In rivers with flat slopes exceeding the laserscan vertical resolution some field measurements of slopes need to be made. Extensive field work will be required where river constrictions, bridges and culverts, abound.

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