Evaluation of Inundation Models

LIMITS AND CAPABILITIES OF MODELS

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SUMMARY

No two flood events are the same, each being a unique combination of processes; for different events different processes will become important in flood propagation. Although in theory models of limitless complexity can be constructed that are capable of representing arbitrary flood events, model parameterisation becomes increasingly difficult with increasingly complexity, and for flood risk management computational efficiency is at a premium. From the class of all possible flood inundation models, the optimum set for flood risk management depends on which processes are important for the flood event and the task at hand (e.g. real time forecasting, planning etc).

In this report we aim to provide a comprehensive reference of flood inundation models for the flood manager. We begin by describing the processes important for flood propagation, the equations for modelling these processes, the simplifications and assumptions. Examples of each type of model are given, although this list will not be completely comprehensive it will span the range available. The final part of this section summarises the limits and capabilities of each model or class of models. Section 2 of the report gives further details of the specific models used in Task 8 of FLOODsite, whilst Section 3 discuss data requirements for inundation modelling along with the calibration and validation of hydraulic codes given data and model uncertainties. Classes of models together with example software and limits of applicability are given in a table at the end of the report.
1. **Introduction**

1.1 **Physical realism versus computational efficiency**

There are two main reasons to undertake numerical modelling of floodplain flow: first as an alternative to laboratory experiments or field data to improve understanding of the processes involved in floodplain flow; and second to obtain predictions of quantities useful for the management of floodplain systems, e.g. discharge, water surface elevation, inundation extent and flow velocity. In this context a model consists of a user’s best estimate of the processes that are perceived to be relevant to the application, and may be tested by comparison to analytical solutions, scale models or field data. Physical realism is of utmost importance in the first class of application, whereas for flood management the emphasis may be on computational efficiency. In this report our focus is on the latter of these two applications.

Compound channel flows are fully turbulent over a wide range of space scales and unsteady in time, but it is computationally prohibitive to simulate flows with this level of complexity. Fortunately, the processes perceived by modellers to be relevant to the accurate simulation of floodplain flow for a particular purpose are typically a small subset of the known physical mechanisms. The key step in selecting an appropriate numerical modelling framework for floodplain flows is therefore to identify those processes that are relevant to a particular modelling problem and decide how these can be discretized and parameterised in the most computationally efficient manner. In Section 1.2, flow processes in compound channels will be discussed along with the simplifying assumptions frequently made in floodplain flow modelling. Examples of numerical modelling tools that embody these simplifying assumptions and appropriate case studies will then be discussed in Section 1.3. Table 1 summarises the classes of model available and their limits of applicability.

1.2 **Flow processes in compound channels**

Flood basins normally consist of a main channel and one or two adjacent floodplain areas. When a flood wave exceeds bank full height, water will travel rapidly over the low lying floodplains. During a flood the floodplain may either act as storage or an additional means of conveyance. In the language of fluid dynamics a flood is a long, low amplitude wave passing through a compound channel with complex geometry. The size of the flood wave will be important when coming to select an appropriate flood management tool, in the very largest basins such waves may be \( \sim 10^3 \) km long and only \( \sim 10^1 \) m deep, and may take several months to traverse the whole system. Flood waves are translated downstream with speed or celerity, \( c \) \([LT^{-1}]\), and attenuated by frictional losses such that in downstream sections the hydrograph (the variation of discharge, \( Q \) \([L^3T^{-1}]\) with time, \( t \) \([T]\)) is flattened out.

Even before bank full height is exceeded there are already many in-channel processes to consider, each with characteristic length scales. At the scale of the channel planform shear layers form at the junction between the main flow and slower moving dead zones (Hankin *et al*., 2001). At the scale of the channel cross-section there are secondary circulations (Bridge and Gabel, 1992; Nezu *et al*., 1993). Finally, turbulent eddies range from heterogeneous structures at the scale of roughness elements and obstructions on the bed (Ashworth *et al*., 1996; McLelland *et al*., 1999; Shvidchenko and Pender, 2001), down through the turbulent energy cascade (Hervouet and Van Haren, 1996), to the Kolmogorov length scale, \( \eta \) \([L]\), where turbulent kinetic energy is dissipated. The smallest eddies may be only \( \sim 10^{-2} \) mm across, and the grid size required to include such processes in flood inundation models makes this infeasible for most real applications.

When bankfull height is exceeded and compound flow ensues, in addition to the above processes new physical mechanisms come into play. The principal mechanisms are momentum exchange between the fast moving channel and slower floodplain flow (Knight and Shiono, 1996) and interaction between meandering channel flows and flow on the floodplain (Sellin and Willetts, 1996). The channel-floodplain momentum exchange occurs across a shear layer which is manifest as a series of vortices with vertically aligned axes (Sellin, 1964; Fukuoka and Fujita, 1989; Shiono and Knight, 1991). Ervine and Baird (1982) conclude that failure to account for the momentum exchange can lead to errors of up to \( \pm 25\% \) in the...
discharge calculated using uniform flow formulae such as the Manning and Chezy equations. Further vigorous momentum exchange occurs during out-of-bank flow in meandering compound channels (see Sellin and Willetts, 1996 for a discussion). Here water spills from the downstream apex of channel bends and flows over meander loops before interacting with channel flow in the next meander. These three-dimensional interactions modify secondary circulations within the channel and represent an additional energy loss in the near channel area. Floodplain flows beyond the meander belt will not be subject to such energy losses and this region may provide a route for more rapid flow conveyance. The impact of these additional energy losses will be at a maximum at some shallow overbank stage, when the interaction between main channel and floodplain is at its greatest (Knight and Shiono, 1996), before slowly decreasing as depth increases and the whole floodplain and valley floor begins to behave as a single channel unit.

Away from the near channel zone water movement on the floodplain may be more accurately described as a typical shallow water flow (i.e. one where the width:depth ratio exceeds 10:1) as the horizontal extent may be large (up to several kilometres) compared to the depth (usually less than 10m). Such shallow water flows over low-lying topography are characterised by rapid extension and retreat of the inundation front over considerable distances, potentially with distinct processes occurring during the wetting and drying phases (see Nicholas and Mitchell, 2003). Correct treatment of this moving boundary problem is therefore important both to capture adequately the shallow water energy losses (which may be high due to large relative roughness) and because flood extent is a common prediction requirement from hydraulic codes.

Flow interactions with micro-topography (see Walling et al., 1986), vegetation (Lopez and Garcia, 2001) and structures (Meselhe et al., 2000) may all be important, thereby giving a complex modelling problem. In particular, where the floodplain acts as a route for flow conveyance rather than just as storage, energy losses are typically dominated by vegetative resistance. Yet despite a small number of pioneering studies (see for example Kouwen, 1988; Nepf, 1999; Ghisalberti and Nepf, 2002; Wilson and Horritt, 2002), the interaction between plant form, plant biomechanics, energy loss and turbulence generation is at present relatively poorly understood (see Wilson et al., 2005). Moreover, many numerical models of floodplain flow assume that the channel bed is fixed over the course of the event, and for very large floods this may not be the case as embankment failure or geomorphic change may considerably affect the flow field.

Whilst overbank flow in compound channels is clearly a two-dimensional process, many practical floodplain management questions only require the prediction of water levels at particular points of interest. In such cases, the modeller is primarily concerned with the downstream routing of flow through a compound cross-section, and may be less concerned to represent floodplain flow and storage accurately. Here, the flow processes of interest are one-dimensional in the down-valley direction and one-dimensional models may therefore be used to represent such flows. Whilst this is often considered a gross simplification of the flow field (see Knight and Shiono, 1996), one can justify the approach by assuming that the additional approximations involved in continuing to treat out-of-bank flow as if it were one-dimensional are small compared to other uncertainties (for a discussion see Ali and Goodwin, 2002). Alternatively, one can attempt to correct one-dimensional flow routing methods to account for the additional energy losses and/or mass transfers (see Knight and Shiono, 1996) or develop hybrid schemes that combine one dimensional modelling for channel flows with a two-dimensional treatment of the floodplain (see Bates and De Roo, 2000).

Lastly, whilst typical hydraulic models do not consider water exchanges with the surrounding catchment, for whole catchment modelling or flood inundation simulation over long river reaches such exchanges may, at particular times, become important (e.g. Stewart et al., 1999; Woessner, 2000). Such processes include direct precipitation or runoff to the floodplain surface (e.g. Mertes, 1997), evapo-transpiration losses, so called bank-storage effects (Pinder and Sauer, 1971; Squillace, 1996) resulting from interactions between the river water and alluvial groundwaters contained within the hyporheic zone (Stanford and Ward, 1988; Castro and Hornberger, 1991; Wroblicky et al., 1998), subsurface contributions to the floodplain groundwater from adjacent hill slopes (e.g Bates et al., 2000; Burt et al., 2002) and flows along preferential flow paths, such as relict channel gravels, within the floodplain alluvium (e.g. Haycock and Burt, 1993; Poole et al., 2002). Over particular reaches and in particular environments, integration of some or all of these processes with flood routing models may be required and necessitate complex modelling structures (e.g. Stewart et al., 1999, Kohane, and Welz, 1994) which may be difficult to fully parameterize.
1.3 Numerical modelling tools

Pender et al. (2006) divide hydraulic models for compound channels into classes according to the maximum dimensionality of the processes represented and the addition (or omission) of other flow processes, (see Table 1). It is clear that for particular problems different classes of models will be appropriate. In this section we discuss some details of numerical modelling and how the classifications given in Table 1 relate to the numerical scheme used.

Excepting 0D models, for which no physical laws are included, all hydraulic models are derived from the three-dimensional Navier-Stokes momentum equation which for an incompressible fluid of constant density may be expressed as:

$$\rho \frac{Du}{Dt} = -\nabla p + \mu \nabla^2 u + F$$  \[1\]

where $\rho$ is the fluid density [ML$^{-3}$]; $u$ is the velocity [LT$^{-1}$]; $t$ is the time [T], $p$ is the pressure [ML$^{-1}$T$^{-2}$]; $\mu$ is the viscosity [ML$^{-1}$T$^{-1}$] and $F$ is the set of source terms (e.g. friction, gravity and coriolis) to be included in the specification of a particular problem.

Combining the Navier-Stokes equation with the equation of continuity:

$$\nabla \cdot u = 0$$  \[2\]

gives a system of equations that can be solved to yield the three-dimensional velocity vector $u = (u, v, w)$, where $u$, $v$ and $w$ are the three components of $u$ in the $x$, $y$ and $z$ directions respectively, and pressure, $p$, for a given point in time and space. In free surface models the pressure is typically replaced with the flow depth, $h$ [L].

In theory these equations can be used to fully describe any open channel flow. However, to capture the details of turbulent flows (which can be as small as $10^{-2}$ mm) requires a very fine discretization both in space and time. In practice, we are often not interested in the velocity field at all scales but the mean flow properties. Reynolds (1895) averaging can be used to split each variable (say $u$) into a mean vector ($\overline{u}$) and a random variation about it ($u'\). Assuming the random variation averages to zero over some integration period we replace all components with their equivalent mean vector, this yields the Reynolds Averaged Navier-Stokes equations (RANS). As a consequence, new terms representing shear stress on the mean flow due to turbulence appear in equation [1]. Values for the Reynolds stresses must be provided (by introducing some turbulence model) to close the RANS equations.

1.3.1 Three-dimensional models (3D)

Representation of the three-dimensional processes, that dominate in the near channel region, requires a three-dimensional solution of the RANS equations that in turn necessitates an approximate numerical technique such as finite differences, finite elements or finite volumes. A number of codes are available for problems such as sediment transport and flow-vegetation interaction, where three-dimensional process representation is deemed important (e.g. CFX, FLUENT and PHEONIX).

The computational cost involved in three-dimensional codes means the modeller must trade off between cell or element size, the domain size and the complexity of the turbulence closure scheme employed. However, three-dimensional models for compound channels at scales of practical interest are feasible as evidenced by the work of Stoesser et al. (2003) who applied a steady state 3D RANS model with $k$-$\varepsilon$ turbulence closure to a 3.5 km reach of the upper river Rhine using 198 114 cells within a finite volume code. Higher order turbulence modelling approaches for three-dimensional flow in compound channels has also been attempted and methods include algebraic stress (Shao et al., 2003) and Large Eddy Simulation (Thomas and Williams, 1995) schemes. However these approaches are computational expensive and have thus far only been applied to channels of regular geometry. Even when it is possible for 3D models with higher order turbulence closure to be applied to complex natural geometry, they may be hard to parameterise due to lack of calibration data.

For dynamic simulations of floodplain flow with three-dimensional models additional approximations need to be introduced to deal with the changing domain extent in both horizontal and vertical dimensions. To date, however, none of the available methods provides a complete solution to this problem. For example,
models which use a deformable mesh (Feng and Perić, 2003) or the $\sigma$-transform (Stoesser et al., 2003) to discretize the grid in the vertical may suffer from stability problems during dynamic shallow water flows as with these methods the upper boundary of the computational domain moves with the water free-surface. If the number of vertical grid layers remains fixed, then cell height-length ratios may become highly distorted as water depth, $h$, approaches zero. For this reason most dynamic codes use the Volume of Fluid (VoF) method (Ma et al., 2002) to track the horizontal moving boundary in whilst retaining fixed vertical grid increments. The VoF method calculates the proportion of each 3D cell filled with water (1 for fully wet cells, 0 for cells that are fully dry and a value between 0 and 1 for partially wet cells) in order to track accurately the position of the surface and horizontal boundaries on a fixed grid in a computationally efficient manner. However, even with this method avoidance of distorted cells for flows with horizontal extent 100-10000 m and depths generally much less than 1-10 m still requires numerical grids that are horizontally very highly resolved and which are likely to incur a prohibitive computational cost. Thus, to date most three-dimensional numerical models of compound channel flow have not been applied to problems with significant changes in domain extent over low gradient floodplains. Neither may three-dimensional approaches be necessary, as for many scales of compound channel flows the two-dimensional shallow water approximation may be adequate. For these reasons, dynamically varying flows in compound channels have, to date, typically been treated with two-dimensional models.

1.3.2 Two-dimensional models (2D and 2D+)

Two-dimensional approaches typically use depth averaged velocity obtained by integrating the Reynolds Averaged Navier-Stokes equations over the flow depth. Examples are the St. Venant equations, which assume a hydrostatic pressure distribution, or the Boussinesq equations, which do not (see Hervouet and Van Haren, 1996). For example, the St. Venant equations are given in non-conservative form as:

Continuity equation

$$\frac{\partial h}{\partial t} + u_d \cdot \text{grad} (h) + h \text{div}(u_d) = 0$$  \[3\]

Momentum equations

$$\frac{\partial u_d}{\partial t} + u_d \cdot \text{grad} (u_d) + g \frac{\partial h}{\partial x} - \text{div}(v_t \cdot \text{grad} (u_d)) = S_x - g \frac{\partial Z_f}{\partial x}$$  \[4\]

$$\frac{\partial v_d}{\partial t} + u_d \cdot \text{grad} (v_d) + g \frac{\partial h}{\partial y} - \text{div}(v_t \cdot \text{grad} (v_d)) = S_y - g \frac{\partial Z_f}{\partial y}$$  \[5\]

Where $u_d,v_d$ are the depth-averaged velocity components [with dimensions LT$^{-1}$] in the x and y cartesian directions [L]; $Z_f$ is the bed elevation [L]; $v_t$ is the kinematic turbulent viscosity [L$^2$T$^{-1}$]; $S_x,S_y$ are the source terms (friction, coriolis force and wind stress) and $g$ is the gravitational acceleration [LT$^{-2}$].

Equations [3] to [5] can then be solved using some appropriate numerical procedure (see Wright, 2005) and turbulence closure (see Sotiropoulos, 2005) to obtain predictions of the water depth, $h$, and the two components of the depth-averaged velocity, $u_d$ and $v_d$. The Shallow Water equations are most often applied to flows that have a large areal extent compared to their depth and where there are large lateral variations in the velocity field. They are thus well suited to the computation of overbank flood flows in compound channels, tides, tsunamis or even dam breaks (see Hervouet and Van Haren, 1996). Whilst two-dimensional models cannot fully represent the complex flow processes in the near channel region of a compound channel, they will still capture certain aspects of these processes. Here, the modeller assumes that this is sufficient to reproduce the particular flow features that are of interest for the problem in hand at a given scale. Moreover, two-dimensional schemes can also more easily represent moving boundary effects and may therefore be of more use for simulating problems where inundation extent changes dynamically through time (see Bates and Horritt, 2005).

The 2D class of models includes full solutions of the two-dimensional St. Venant or shallow water equations (Gee et al., 1990; Feldhaus et al., 1992; Bates et al., 1998; Nicholas and Mitchell, 2003) and simplified shallow water models (see for example Molinaro et al., 1994) where certain terms, such as inertia, are omitted from the controlling equations. Each of these equations sets can be discretized using
structured or unstructured grids, and it may even be possible to allow the grid to deform to follow the moving inundation front through time (Benkhaldoun and Monthe, 1994). However the computational cost in re-meshing and problems with numerical stability mean that, to date, fixed grid approaches have been preferred. Also fixed grids, being cheaper computationally, may allow finer spatial resolution meshes to be employed which may be a considerable advantage given the complexity of floodplain topography.

Simpler methods for attempting to represent two-dimensional flow are storage cell codes, which are really a hybrid of one-dimensional and two-dimensional approaches. Storage cell codes may be classified as 1D+ or 2D- depending on the way floodplain flow is represented, these methods are discussed further in Section 1.3.4.

1.3.3 One-dimensional models (1D)

Most simply, for flow routing problems floodplain flow can be treated as one-dimensional in the down-valley direction. An equation for one-dimensional channel flow can be derived by considering mass and momentum conservation between two cross sections \( \Delta x \) apart. This yields the well-known one-dimensional St. Venant or shallow water equations:

**Conservation of momentum**

\[
\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial x} + gA \left( \frac{\partial h}{\partial x} + S_f - S_o \right) = 0 \quad [6]
\]

where \( Q \) is the flow discharge [L^3T^{-1}], \( A \) is the flow cross-section area [L^2], \( g \) is the gravitational acceleration [LT^{-2}], \( S_f \) is the friction slope [LL^{-1}] and \( S_o \) is the channel bed slope [LL^{-1}].

**Conservation of mass**

\[
\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad [7]
\]

Where \( q \) is the lateral inflow or outflow per unit length [L^2T^{-1}].

Equations [6] and [7] have no exact analytical solution, but with appropriate boundary and initial conditions they can be solved using numerical techniques (e.g. Preissmann, 1961; Abbott and Ionescu, 1967) to yield estimates of \( Q \) and \( h \) in both space and time. The river reach in one-dimensional St. Venant models is discretized as a series of irregularly spaced cross-sections. Boundary conditions typically consist of the inflow hydrograph at the upstream cross section and, for sub-critical flow, the stage hydrograph at the downstream boundary. For critical flow where the Froude number, \( F_r = u/\sqrt{gh} \), exceeds 1, information from the downstream boundary cannot propagate upstream and no boundary condition need therefore be prescribed. These equations form the basis of most standard commercial hydraulic modelling software such as HEC-RAS, MIKE11, ISIS and SOBEK.

Horritt and Bates (2002) demonstrate for the simulation of a large flood event over a 60km reach of the River Severn, UK that one-dimensional, simplified two-dimensional and full two-dimensional models perform equally well in simulating flow routing and inundation extent given uncertainties over inflow, topography and validation data. This suggests that although gross assumptions are made regarding the flow physics incorporated in a one-dimensional model applied to out-of-bank flows, the additional energy losses can be compensated for using a calibrated effective friction coefficient.

One-dimensional approaches that attempt to correct for some of these additional energy losses are reviewed by Knight and Shiono (1996, p155-159) and are based on subdividing the channel in the streamwise direction and then calculating the conveyance in each section using uniform flow formulae. The sub-area conveyances are then summed to give the total conveyance. Knight and Shiono (1996) identify three main variations on the channel division method which aim to simulate the channel-floodplain interaction more exactly. These are: (1) modification of the sub-area wetted perimeters (Wormleaton et al., 1982); (2) calculation of discharge adjustment factors for each sub-area based on a ‘coherence’ concept (Ackers, 1993) and (3) quantification of the apparent shear stresses on the sub-area division lines (Knight and Hamed, 1984). However, these methods have been developed to estimate the depth-discharge rating curve.
at particular cross-sections and are largely yet to be incorporated in standard flood routing models. One exception here is the LISFLOOD-FF model of De Roo et al. (2001). Here, a correction of the Manning roughness value is applied to simulate the momentum exchange, which occurs across the shear layer between main channel and floodplain flows.

These 1D codes are more appropriate when the width of the floodplain is no larger than 3 times the width of the main river channel and are not separated from the channel by embankments (Pender et al., 2006). One of the major disadvantages of this approach is that floodplain flow is assumed to be parallel to the channel, although as noted above cross-section conveyance can be computed by separating the cross-sections into a series of panels between which a separate conveyance is performed.

1.3.4 Coupled one-dimensional/two-dimensional models (1D+ and 2D-)

Whilst one-dimensional codes are computationally efficient, they do suffer from a number of drawbacks when applied to floodplain flows. These include the inability to simulate lateral spreading of the flood wave, the lack of a continuous treatment for topography and the subjectivity of cross-section location. Whilst all of these constraints can be overcome with higher order codes, the computational cost of running a two or three dimensional simulation may be high. Consequently, recent research has begun to examine hybrid one-dimensional/two-dimensional codes that seek to combine the best of each model class (see for example Bladé et al., 1994; Estrela and Quintas, 1994; Bechteler et al., 1994, Romanowicz et al., 1996; Bates and De Roo, 2000; Venere and Clausse, 2002; Dhondia and Stelling, 2002).

Such models typically treat in-channel flow with some form of the one-dimensional St. Venant equations, but treat floodplain flows as two-dimensional using a storage cell concept first described by Cunge et al. (1980). Here the floodplain is discretized as a series of regions, with flows between regions calculated using analytical uniform flow formulae such as the Manning equation. Initially, this approach was implemented in standard one-dimensional river routing packages such as ISIS (Wicks et al., in press) by defining the storage cells as large polygonal areas (surface areas of $10^3-10^4$ km$^2$) representing discrete flooding compartments (e.g. polders, storage basins etc) that are subjectively identified by the user. In Table 1 these are classified as 1D+ codes, examples being Infoworks RS, Mike 11 and SOBEK which can actually be used in either 1D or 1D+ modes.

Recent developments in topographic data capture have, however, allowed high (cell size $\sim 10^0$ m) resolution Digital Elevation Models of floodplain areas to be produced. This has allowed storage cells to be discretized as a high resolution grid (storage cells with surface area $10^{-5}-10^{-4}$ km$^2$), for example the LISFLOOD-FP raster flood routing model of Bates and De Roo (2000). We classify these codes as 2D- in Table 1. LISFLOOD-FP treats in-channel flow using either the simplified kinematic or diffusion wave forms of equations [6] and [7]. Floodplain flows are similarly described in terms of continuity and mass flux equations, discretized over a grid of square cells, which allows the model to represent 2-D dynamic flow fields on the floodplain. Flow between two cells is assumed simply to be a function of the free surface height difference between those cells (Estrela and Quintas, 1994):

$$Q^i,j = \frac{h_{flow}^{i,j}}{n} \left( \frac{h^{i-1,j} - h^{i,j}}{\Delta x} \right)^{1/2} \Delta y$$  \[8\]

Water depths in each cell are then updated at each time step based on the sum of the fluxes over the four faces of the cell.

$$\frac{dh^{i,j}}{dt} = \frac{Q^{i-1,j} - Q^{i,j} + Q^{i,j-1} - Q^{i,j}}{\Delta x \Delta y}$$  \[9\]

where $h^{i,j}$ is the water free surface height at the node $(i,j)$, $\Delta x$ and $\Delta y$ are the cell dimensions, $n$ is the effective grid scale Manning’s friction coefficient [L$^{1/3}$T$^{-1}$] for the floodplain, and $Q_x$ and $Q_y$ describe the volumetric flow rates between floodplain cells. $Q_x$ is defined analogously to equation [8]. The flow depth, $h_{flow}$, represents the depth through which water can flow between two cells, and is defined as the difference between the highest water free surface in the two cells and the highest bed elevation (this definition has been found to give sensible results for both wetting cells and for flows linking floodplain and channel cells.) This approach is similar to diffusive wave propagation, but differs marginally due to the de-
coupling of the $x$- and $y$- components of the flow. While this approach does not accurately represent diffusive wave propagation, it is computationally simple and has been shown to give very similar results to a more accurate finite difference discretization of the diffusive wave equation (Horritt and Bates, 2001a).

Equation [8] is also used to calculate flows between floodplain and channel cells, allowing floodplain cell depths to be updated using equation 12 in response to flow from the channel. These flows are also used as the source term, $q$, in the channel flow sub-model (see equation [7]), effecting the linkage of channel and floodplain flows. Thus only mass transfer between channel and floodplain is represented, and this is assumed to be dependent only on relative water surface elevations. While this neglects effects such as channel-floodplain momentum transfer and the effects of advection and secondary circulation on mass transfer, it is the simplest approach to the coupling problem and should reproduce the dominant behaviour of the real system.

Such cellular approaches are capable of high-resolution application to relatively long river reaches up to $10^3$ km in length and may provide an alternative to one-dimensional codes. For example, the LISFLOOD-FP storage cell code has been successfully used to model flow routing along a 60 km reach of the River Severn in the UK (see Horritt and Bates, 2002) and a 35 km reach of the River Meuse in The Netherlands (see De Roo et al., 2003).

### 1.3.5 Zero-dimensional or non-model approaches (0D)

In certain situations one may not even need a model at all to predict inundation extent. Given gauged water surface elevations along a reach, or water surface elevations predicted on the basis of flood frequency analysis, one can perform a similar interpolation to that used by Werner (2001). This approximates the flood wave as a plane (or series of planes) which are intersected with the DEM to give extent and depth predictions. Bates and De Roo (2000) compared this method to two-dimensional storage cell and full two-dimensional hydraulic models for a 35 km reach of the River Meuse, The Netherlands and found that in certain situations the planar approximation performed almost as well as hydraulic modelling. In this application, the methods were used to simulate the January 1995 event ($Q_{\text{peak}} = \approx 2700 \text{ m}^3\text{s}^{-1}$, 1 in 63 year recurrence interval) and validated by comparison to air photo imagery of flood extent taken at around the time of maximum flooding. For regions close to gauging stations where the recorded water level was used as a height control, the planar approximation achieved accuracy of 81% pixels correctly predicted as wet or dry, compared to 85% for a two-dimensional storage cell model. Away from the gauging station the planar approximation performed less well, and using a hydraulic model produced a better result in all circumstances examined, even if at times this difference was marginal. Clearly, the planar approximation will work well for reaches that are short compared to the wavelength of the flood and where there is good gauged data to constrain the position of the plane. Even in these circumstances, however, lack of mass conservation will mean that areas are predicted as flooded that are not hydraulically connected to the channel. Nevertheless, this may be a useful method under some circumstances, and provides a benchmark level of performance that all hydraulic models should exceed to be considered skilful.

## 2. Models used in Task 8

### 2.1 LISFLOOD-FP

Raster-based storage cell codes, such as the LISFLOOD-FP model of Bates and De Roo (2000), solve a continuity equation relating flow into a cell and its change in volume:

$$ \frac{\partial h_{i,j}}{\partial t} = \frac{Q_{i,j}^{+} - Q_{i,j}^{-} + Q_{i,j}^{x} - Q_{i,j}^{y}}{\Delta x \Delta y} $$

and a momentum equation for each direction where flow between cells is calculated according to Manning’s law (only the $x$ direction is given here):

$$ Q_{i,j}^{x} = \frac{h_{\text{flow}}^{5/3}}{n} \left( \frac{h_{i,j}^{+} - h_{i,j}^{-}}{\Delta x} \right)^{1/2} \Delta y $$

where $h_{i,j}$ is the water free surface height at the node $(i, j)$, $\Delta x$ and $\Delta y$ are the cell dimensions, $n$ is the Manning’s friction coefficient, and $Q_{x}$ and $Q_{y}$ describe the volumetric flow rates between floodplain cells.
Inundation Model Evaluation M8.1
Contract No: GOCE-CT-2004-505420

$Q_i$ is defined analogously to equation (11). The flow depth, $h_{\text{flow}}$, represents the depth through which water can flow between two cells, and is defined as the difference between the highest water free surface in the two cells and the highest bed elevation. These equations are solved explicitly using a finite difference discretization of the time derivative term:

$$
\frac{h_{ij}^{t+1} - h_{ij}^{t}}{\Delta t} = \frac{Q_{x,ij}^{t+1/2} - Q_{x,ij}^{t-1/2}}{\Delta x} + \frac{Q_{y,ij}^{t+1/2} - Q_{y,ij}^{t-1/2}}{\Delta y} \quad [12]
$$

where $'h$ and $'Q$ represent depth and volumetric flow rate at time $t$ respectively, and $\Delta t$ is the model time step.

The model time step is set by the user, however too large a time step was found to result in 'chequerboard' oscillations in the solution which rapidly spread and amplify, rendering the simulation useless. Ironically, these oscillations occur most readily in areas with low free surface gradients, where we might expect obtaining a solution to be easiest. For this reason, a flow limiter was found to be required in order to prevent instabilities in areas of very deep water, by setting a maximum flow between cells. This flow limit was fixed so as to prevent 'over' or 'undershoot' of the solution, and is a function of flow depth, grid cell size and time step:

$$
Q_{x,ij}^{t+1/2} = \min \left( Q_{x,ij}^{t+1/2}, \frac{\Delta x \Delta y \left( h_{ij}^{t+1/2} - h_{ij}^{t-1/2} \right)}{4 \Delta t} \right) \quad [13]
$$

This value is determined by considering the change in depth of a cell, and ensuring it is not large enough to reverse the flow in or out of the cell at the next time step. This limiter replaces fluxes calculated using Manning's equation with values dependent on model parameters, and hence when the flow limiter is in use floodplain flows are sensitive to grid cell size and time step, and insensitive to Manning's $n$.

In tests of the code conducted to date validation data has consisted, at best, of gauged hydrometric data and single 'snapshot' images of near-maximum flood extent derived from air photo data and satellite-borne and airborne Synthetic Aperture Radars. The LISFLOOD-FP model has been tested within a simple calibration framework against inundation and hydrometric data for the Rivers Thames (Horritt and Bates, 2001a) and Severn (Horritt and Bates, 2001b; 2002) in the UK and the Meuse (Hunter et al., 2005a; Bates and De Roo, 2000) in The Netherlands and compared to two alternative hydraulic models; a planar approximation to the water surface (in effect not a model at all) and a standard two-dimensional finite element scheme. In all cases LISFLOOD-FP equalled or outperformed the alternatives when simulations of inundation extent were compared, even when the grid resolution was comparable.

Despite their ability to successfully replicate this maximum flood extent data, the simplifying assumptions made in the development of storage cell codes do lead to a number of theoretical and practical constraints to their usage. Firstly, when the flow limiter (equation [13]) was invoked for a significant proportion of the domain the model was found to lack any meaningful sensitivity to floodplain friction and seemed to be over-sensitive to channel friction (Horritt and Bates, 2002). Use of the flow limiter also means that the results are not independent of the space and time step selected by the modeller. While this result is common in both explicit and implicit (Romanowicz and Beven, 2003) formulations of storage cell schemes and is in agreement with the hypothesis of Cunge et al. (1980) - that the floodplains act primarily as additional storage, while the main body of the wave is transported by the main channel - it should not be overlooked as a potentially erroneous artefact of the model structure. A consequence of this sensitivity is that predictive performance of the model may be adversely affected (Horritt and Bates, 2002). Hence even though the optimum calibration for two events may be drawn from a similar region of the parameter space, steep gradients within this space may mean that when calibrated parameters from one event are used to predict another, the absolute model performance may vary markedly. In other words optimum parameter sets need only differ by a small amount to be considered non-stationary between events. Secondly, when applied without calibration (i.e. single-realisation deterministic mode), LISFLOOD-FP can be shown to under predict markedly both the spatially-distributed (i.e. inundation extent, flood depths) and bulk (i.e. wave volume, travel time) flood characteristics within the domain when compared with other modelling approaches (Werner and Lambert, in press).
As a solution to the above problems Hunter et al. (2005b) have recently proposed a modified version of the LISFLOOD-FP based on adaptive time stepping. This approach seeks to remove the need to invoke the flow limiter (equation (13)) by finding the optimum time step (large enough for computational efficiency, small enough for stability) at each iteration. Stability depends on water depth, free surface gradients, Manning’s $n$ and grid cell size and thus varies in time and space during a simulation.

This method uses an analysis of the governing equations and their analogy to a diffusion system to calculate the largest stable time step. Equations (10) and (11) are essentially discretizations of the continuity and momentum equations:

$$\frac{\partial h^{i,j}}{\partial t} = \frac{Q^{i-1,j} - Q^{i,j} + Q^{i,j-1} - Q^{i,j}}{\Delta x \Delta y}$$

$$Q^{i,j}_x = \left(h_{flow}^3 \left(\frac{h^{-3/2}}{\Delta x} - \frac{h^{3/2}}{\Delta x}\right)\right)^{1/2} \Delta y$$

where $q_x$ and $q_y$ are components of the flow per unit width. Equation (15) differs from the usual definition of Manning’s equation in 2D shallow water models in that the two components are decoupled, but this has been found to have negligible effect on model predictions (Horritt and Bates, 2001a). The sense of the flow is determined by whether the free surface gradient is positive or negative. Combining equations (14) and (15) we obtain:

$$\frac{\partial h}{\partial t} + \nabla \cdot \mathbf{q} = 0$$

$$q_x = \pm \frac{h_{flow}^{3/2}}{n} \left|\frac{\partial h}{\partial x}\right|^{1/2}, \quad q_y = \pm \frac{h_{flow}^{3/2}}{n} \left|\frac{\partial h}{\partial y}\right|^{1/2}$$

Diffusion Terms

$$\frac{5}{3n} \frac{h_{flow}^{3/2}}{\partial x} \left|\frac{\partial h}{\partial x}\right|^{1/2} \frac{\partial h_{flow}}{\partial x} \pm \frac{5}{3n} \frac{h_{flow}^{3/2}}{\partial y} \left|\frac{\partial h}{\partial y}\right|^{1/2} \frac{\partial h_{flow}}{\partial y} = 0$$

The terms with the second spatial derivatives make up the diffusion part of the equation, and will dominate when free surface gradients are small and stability problems are likely to arise. The solution is unlikely to mirror the behaviour of classical diffusion problems since the diffusion coefficient varies in space and time, and is anisotropic, but we can use the analogy to estimate the most efficient time step. For the diffusion equation:

$$\frac{\partial h}{\partial t} - \alpha \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2}\right) = 0$$

and its explicit discrete counterpart on a square grid (subscripts are spatial grid locations, superscripts time):
\[ \frac{h^{i+1}_{ij} - h^i_{ij}}{\Delta t} - \frac{\alpha}{\Delta x^2} \left( h^i_{i+1,j} + h^i_{i-1,j} + h^i_{i,j+1} + h^i_{i,j-1} - 4 h^i_{ij} \right) = 0 \]  

[18]

A von Neumann stability analysis produces the following time step condition:

\[ \Delta t \leq \frac{\Delta x^2}{4 \alpha} \]  

[19]

At equality, the \( h^i_{ij} \) terms in equation (18) cancel, and it becomes the well known Jacobi relaxation approach to the solution of Laplace’s equation, where the value at a node is iteratively replaced by the mean of neighbouring values. This would imply that an optimal time step for the hydraulic model at a specific location is given by:

\[ \Delta t = \frac{\Delta x^2}{4} \min \left( \frac{2 n}{h^{5/3}_{\text{flow}}} \left| \frac{\partial h}{\partial x} \right|^{1/2}, \frac{2 n}{h^{5/3}_{\text{flow}}} \left| \frac{\partial h}{\partial y} \right|^{1/2} \right) \]  

[20]

We thus arrive at an expression for the time step similar in form to that used by Werner & Lambert (in press) but larger by a factor of 2. In Werner & Lambert (in press) the time step was set to allow small checkerboard oscillations to decay down to a flat free surface, whereas in this analysis we counter the build up of these oscillations directly, and hence can use a larger time step. A scheme that uses this criterion can be implemented by searching the domain for the minimum time step value and using this to update \( h \). The time step will thus be adaptive and change during the course of a simulation, but is fixed in space at each time step.

A problem with this approach is that there is no lower bound on the time step. As free surface gradients tend to zero (standing water), \( \alpha \) tends to infinity and hence the time step also tends to zero. Furthermore, as flow reverses during the transition between the wetting and drying phases, the time step is driven to zero, causing the model to ‘stall’. For a fully dynamic model, some way of dealing with this pathological behaviour as surface gradients tend to zero is required. This is avoided by introducing a linear scheme that is applied to cells where free surface elevations in neighbouring cells differ by less than a specified threshold, \( h_{\text{lin}} \) (Cunge et al., 1980) the flow equation then becomes:

\[ q_x = \frac{h^{5/3}_{\text{flow}}}{n} \left( \frac{\Delta x}{h_{\text{lin}}} \right)^{1/2} \left| \frac{\partial h}{\partial x} \right|^{1/2} \]  

[21]

with a similar expression for \( q_y \). For cells where this linearised flow equation is applied, an equation similar to equation (20) above is used to determine the time step.

Hunter et al. (2005b) tested this new adaptive time step (ATS) formulation against analytical solutions for wave propagation over flat and planar slopes and showed a considerable improvement over the original fixed time-step version of the model. Moreover, the ATS scheme was shown to yield results that were independent of grid size or choice of initial time step and which showed an intuitively correct sensitivity to floodplain friction over spatially-complex topography.

2.2 UCL

A first-order accurate finite-volume model is used to solve the 2D shallow-water equations on unstructured meshes (usually triangular cells). The fluxes are computed using Roe’s scheme. The bed slope source terms are treated in a lateralised way (Soares Frazão, 2002), which is somewhat similar to an upwind treatment. This technique appears to be very robust and yields good results on irregular topographies such as the present one. The friction term is computed using Manning’s formula.
In two-dimensional conservative vector form, the Saint-Venant shallow-water equations stating mass and momentum conservation read:

\[
\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} + \frac{\partial G(U)}{\partial y} = S \tag{22}
\]

with the variables defined as

\[
U = \begin{pmatrix} h \\ uh \\ vh \end{pmatrix}, \quad F = \begin{pmatrix} uh \\ u^2h + gh^2/2 \\ uvh \end{pmatrix} \tag{23}
\]

\[
G = \begin{pmatrix} vh \\ uvh \\ v^2h + gh^2/2 \end{pmatrix}, \quad S = \begin{pmatrix} 0 \\ gh(S_{0x} - S_{fx}) \\ gh(S_{0y} - S_{fy}) \end{pmatrix} \tag{24}
\]

\[
\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} + \frac{\partial G(U)}{\partial y} = S
\]

In equations \([22, 23 \text{ and } 24]\), \(h\) is the water depth, \(uh\) and \(vh\) the unit discharge in the \(x\)- and \(y\)-direction respectively, \(S_{0x}\) and \(S_{0y}\) the bed slope in the \(x\)- and \(y\)-direction respectively and \(S_{fx}\) and \(S_{fy}\) the components of the friction slope, computed using Manning’s formula.

The finite-volume scheme is built upon an integral form of equations \([23 \text{ and } 24]\), yielding the generalised expression for non-Cartesian grids (Toro, 1997):

\[
\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} + \frac{\partial G(U)}{\partial y} = S \tag{25}
\]

where \(\Omega_j\) is the cell-base area, \(L_j\) the \(j\)-interface length and \(nb\) the number of cell interfaces (3 for triangular cells). The vectors \(\mathbf{U}\) and \(F(U)\) express \(U\) and \(F\) in terms of normal and tangential velocity components, \(u_n\) and \(v_n\), attached to the considered interface, using a transformation matrix \(T\), where \(n_x\) and \(n_y\) are the components of the outwards unit vector normal to the interface:

\[
\mathbf{U} = T U = \begin{bmatrix} 1 & 0 & 0 \\ 0 & n_x & n_y \\ 0 & -n_y & n_x \end{bmatrix} \begin{pmatrix} h \\ uh \\ vh \end{pmatrix} = \begin{pmatrix} h \\ u_n h \\ v_n h \end{pmatrix} \tag{26}
\]

The flux normal to the interface becomes:

\[
F(U) = \begin{pmatrix} u_n h \\ u_n^2h + gh^2/2 \\ u_n v_n h \end{pmatrix} \tag{27}
\]

The problem is thus solved by computing the mass and momentum fluxes in the direction normal to each interface, and then combining those fluxes to obtain the mass and momentum balance over each cell.
The normal fluxes are computed using Roe’s scheme adapted to the shallow water equations (Alcrudo and Garcia-Navarro, 1993), taking into account the wave propagation celerity of the system.

2.3 **SOBEK**

2.3.1 **Concept and numerical approach**

SOBEK is a hydraulic modelling software package developed by WL | Delft Hydraulics that consists of several modules. The modules used for inundation modelling are the Overland Flow module (SOBEK-OF) and the Channel Flow module (SOBEK-CF).

SOBEK Overland Flow consists of a 2-dimensional modelling system based on the Navier-Stokes equations for depth-integrated free surface flow. All equations are solved through a fully implicit finite difference formulation for all terms in the Navier-Stokes equations, based upon a staggered grid. The special way in which the convective momentum terms have been formulated allows for the computation of mixed sub- and supercritical flows. Based upon this formulation it is also possible to compute the behaviour of standing and moving hydraulic jumps. For these computations to be robust and accurate, there is no need to introduce artificial viscosity.

In combination with the 2D modelling system, SOBEK is able to handle 1D elements such as (small) water courses and hydraulic structures. In this 1D-2D combination, the 2D overland flow, including the obstructing effects of embankments and natural levees, is simulated through the 2D equations of SOBEK Overland Flow, while the sub-2D grid gullies and the hydraulic structures are modelled with SOBEK Channel Flow. Both modelling systems produce implicit finite difference equations, which are also linked through an implicit formulation for joint continuity equations at locations where both modelling systems have common water level points, as shown in Figure 2.1.

![Figure 2.1 Schematisation of the Hydraulic Model: a) Combined 1D/2D Staggered Grid; b) Combined Continuity Equation for 1D2D Computations](image)

The main advantages of combination of flow in the 1D and 2D domain are:

- 2D grid steps can usually be significantly larger, as no refinement of the 2D grid is required for the correct representation of hydraulic structures and gullies;
- as a result, the simulations will run much faster for a comparable level of accuracy;
- a wide variety of hydraulic structure descriptions can be used.
- robustness and accuracy

The Overland Flow and the Channel Flow modules of SOBEK are based upon the same numerical principles and both allow for extremely stable and robust computations. In the first place this is based upon the properties of the numerical schemes applied. In the second place, a number of checks are made at every step in the computation to prevent physically unrealistic results, such as negative water depths. If such a constraint is not satisfied, the time step will be reduced. Such a procedure is also applied in the flooding and drying of cells in the Overland Flow module. Every time only one neighbouring computational cell can be wetted or dried, otherwise the time step will be reduced to satisfy this criterion.
The robustness and accuracy of the numerical schemes follow to a large extent from the particular way in which the convective momentum terms have been discretized. The formulation implemented also suppresses the development of oscillating velocity directions at irregular model boundaries. Here the finite difference scheme offers the same robust behaviour as models which follow irregular boundaries with their grid contours, such as curvilinear grids and finite elements. Of particular interest is the strict volume conservation. This feature is of particular importance in the simulation of transport of pollutants.

2.3.2 Additional features

SOBEK offers several additional features that can be used for inundation modelling. The main features are:

- Simulating breach growth
- Rainfall and evaporation
- Wind set up

Breach growth can be simulated using three approaches:

1. location, timing and breach size prescribed by the user. In this case SOBEK only computes the breach growth in time until the maximum width is reached.
2. location and timing prescribed by the user, breach width computed by SOBEK. SOBEK computes breach width (B in meters) in time (t), as a function of the difference in water level on both sides of the breach (H in meters) and the critical flow velocity for erosion (u_c in m/s):

\[
B = 1.3 \frac{g^{0.5} H^{1.5}}{u_c} \log \left( 1 + \frac{0.04 g H}{u_c t} \right) \quad [28]
\]

3. location and timing are determined by SOBEK, breach width equals cell size. This method allows for the application of a criterion for failure. The Real Time Control module of SOBEK (RTC-module) checks for each time step whether this criteria is met. Different criteria can be applied such as the exceedence of water levels or flow velocities, either instantaneous or during a certain time period. The width of the breach does not vary in time, but equals the width of a grid cell.

Flooding due to heavy rainfall can be accounted for. The same applies to the impact of evaporation. Evaporation can play an important role at locations where flooding last for months and evaporation losses are high. An example of an area where this option was prerequisite is the Doñana wetland in southern Spain. The rainfall distribution is assumed homogeneous over the area of interest.

The impact of strong winds on water levels can be simulated as well. This is of importance when vast areas are flooded.

2.3.3 Calibration and validation

For proof of accuracy, comparison of results has been made with experimental studies, both with published data and obtained through own laboratory experiments. A dyke break flood onto a flat, horizontal basin has been undertaken in the Fluid Mechanics Laboratory of Delft University of Technology, the Netherlands (Liang et al., 2004). Figure 2.2 illustrates the experimental layout, which consists of a closed reservoir containing water of initial depth equal to 0.6 m separated by a solid dyke wall from a flat basin. The basin was initially dry or contained still water to a low depth (0.05m for the case considered here). A gate opening of 40cm represented the breach, and was located in the middle of the dyke. The dyke-break was modelled by lifting the gate at a constant speed of about 16 cm/s. According to Duinmeijer (2002), the basin has a smooth concrete bed corresponding to a Manning roughness coefficient of 0.012 m s^{-1/3}. Reflective boundary conditions are imposed at all solid walls.
Figure 2.2 Delft University of Technology dyke break: top view and side view of the experiment layout (Liang et al., 2004)

Figure 2.3 shows the progress of the flood wave as observed from above. The upper part of the figure shows the image taken with a camera above the flume. The lower part shows the computed water levels at the same time step. The computed front of the flood is at the same position as the measured front. The same applies to the location of the hydraulic jump. Even the reflection from the walls is simulated correctly.

Figure 2.3 Comparison of measured and simulated water levels using the experiment carried out by Delft University of Technology (Duinmeijer, 2002).
3. Model parameterization, validation and uncertainty analysis

Section 1.3 has made clear the variety of models available for floodplain flow modelling. Choice of model has been shown to depend on the scale of the problem, the computational resources available and the needs of the user. However, any model is only as good as the data used to parameterize, calibrate and validate it, and in this section the data sources available for floodplain flow models are discussed along with the methods available to assimilate these data into hydraulic models. The data required by any hydraulic model are principally: (1) boundary condition data; (2) initial condition data; (3) topography data; (4) friction data and (5) hydraulic data for use in model validation.

3.1 Boundary condition data

Boundary condition data consist of values of each model independent variable at each boundary node and at each time step for unsteady simulations. For one and two-dimensional codes these can typically be assigned either from stage or discharge measured at river gauging stations or must be measured in the field by the user. The precise data required depends on the model and the reach hydraulics, but for sub-critical flow will, as a minimum, consist of the flux rate into the model across each inflow boundary and the water surface elevation at each outflow boundary. These requirements reduce to merely the inflow flux rates for super-critical flow problems as when the Froude number $F_r > 1$ information cannot propagate in an upstream direction. Rating curves may provide an alternative means of parameterizing the outflow water surface elevation in certain models. In addition, 3D codes require the specification of the velocity distribution at the inlet boundary and values for the turbulent kinetic energy. In most cases, hydrological fluxes outside the channel network, e.g. surface and subsurface flows from hill slopes adjacent to the floodplain and infiltration of flood waters into alluvial sediments, are ignored (for a more detailed discussion see Stewart et al., 1999).

3.2 Initial condition data

Initial conditions for a hydraulic model consist of values for each model dependent variable at each computational node at time $t=0$. In practice, these will be incompletely known, if at all, and some additional assumptions will therefore be necessary. For steady state (i.e. non-transient) simulations any reasonable guess at the initial conditions is usually sufficient, as the simulation can be run until the solution is in equilibrium with the boundary conditions and the initial conditions have ceased to have an influence. However, for dynamic simulations this will not be the case and whilst care can be taken to make the initial conditions as realistic as possible, a ‘spin up’ period during which model performance is impaired will always exist. For example, initial conditions for a flood simulation in a compound channel are often taken as the water depths and flow velocities predicted by a steady state simulation with inflow and outflow boundary conditions at the same value as those used to commence the dynamic run. Whilst most natural system are rarely in steady state, careful selection of simulation periods to coincide their start with near steady state conditions can minimise the impact of this assumption.

3.3 Topography data

Topography is frequently considered the key data set for flow routing and inundation modelling. High resolution, high accuracy topographic data are essential to shallow water flooding simulations over low slope floodplains with complex micro-topography, and such data sets are increasingly available from a variety of remotely mounted sensors. Traditionally, hydraulic models have been parameterised using ground survey of cross sections perpendicular to the channel at spacings of between 100 and 1000 m. Such data are accurate to within a few millimetres in both the horizontal and vertical and integrate well with one-dimensional hydraulic models. However, ground survey data are expensive and time consuming to collect and of relatively low spatial resolution. They hence require significant interpolation to enable their use in typical two- and three-dimensional models, whilst in one-dimensional models the topography between cross-sections is effectively ignored and results are sensitive to cross-section spacing (e.g. Samuels, 1990). Moreover, topographic data available on national survey maps tends to be of low accuracy with poor spatial resolution in floodplain areas. For example in the UK, Nationally available contour data are only recorded at 5m spacing to height accuracy of ±1.25m and for a hydraulic modelling study of typical river reach
Marks and Bates (2000) report finding only 3 contours and 40 unmaintained spot heights within a ~6 km² floodplain area. When converted into a DEM such data lead to relatively low levels of inundation prediction accuracy in hydraulic models (see Wilson and Atkinson, 2005).

Considerable potential therefore exists for more automated, broad-area mapping of topography from satellite and, more importantly, airborne platforms. Three techniques which currently show reasonable potential for flood modelling are aerial stereo-photogrammetry (Baltsavias, 1999; Lane, 2000; Westaway et al., 2003), airborne laser altimetry or LiDAR (Krabill et al., 1984; Gomes Pereira and Wicherson, 1999) and airborne Synthetic Aperture Radar interferometry (Hodgson et al., 2003). Radar interferometry from sensors mounted on space-borne platforms, and in particular the Shuttle Radar Topography Mission (SRTM) data (Rabus et al., 2003), may in the future provide a viable topographic data source for hydraulic modelling in large, remote river basin where the flood amplitude is large compared to the topographic data error.

LiDAR in particular has attracted much recent attention in the hydraulic modelling literature (Marks and Bates, 2000; Bates et al., 2003; French, 2003; Charlton et al., 2003). Major LiDAR data collection programmes are underway in a number of countries, including The Netherlands and the UK, where so far approximately 20% of the land surface area in England and Wales has been surveyed. In the UK, helicopter-based LiDAR survey is also beginning to be used to monitor in detail (~0.2 m spatial resolution) along critical topographic features such as flood defences, levees and embankments. LiDAR systems operate by emitting pulses of laser energy at very high frequency (~5-100KHz) and measuring the time taken for these to be returned from the surface to the sensor. Global Positioning System data and an onboard Inertial Navigation System are used to determine the location of the plane in space and hence the surface elevation. As the laser pulse travels to the surface it spreads out to give a footprint of ~0.1m² for typical operating altitude of ~800m. On striking a vegetated surface, part of the laser energy will be returned from the top of the canopy and part will penetrate to the ground. Hence, an energy source emitted as a pulse will be returned as a waveform, with the first point on the waveform representing the top of the canopy and the last point (hopefully) representing the ground surface. The last returns can then be used to generate a high resolution ‘bare earth’ DEM.

3.4 Friction data

Friction is usually the only unconstrained parameter in a hydraulic model. Two and three-dimensional codes which use a zero equation turbulence closure may additionally require specification of an ‘eddy viscosity’ parameter which describes the transport of momentum within the flow by turbulent dispersion, however this prerequisite disappears for most higher order turbulence models of practical interest. Hydraulic resistance is a lumped term that represents the sum of a number of effects: skin friction, form drag and the impact of acceleration and deceleration of the flow. These combine to give an overall drag force $C_d$ that in hydraulics is usually expressed in terms of resistance coefficients such as Manning’s $n$ and Chezy’s $C$ and which are derived from uniform flow theory. This assumes that the rate of energy dissipation for non-uniform flows is the same as it would be for uniform flow at the same water surface (friction) slope. The precise effects represented by the friction coefficient for a particular model depend on the model’s dimensionality, as the parameterization compensates for energy losses due to unrepresented processes, and the grid resolution. Thus, the extent to which form drag is represented depends on how the channel cross-sectional shape, meanders and long profile are incorporated into the model discretization. Similarly, a high-resolution discretization will explicitly represent a greater proportion of the form drag component than a low-resolution discretization using the same model. Complex questions of scaling and dimensionality hence arise which may be somewhat difficult to disentangle. Certain components of the hydraulic resistance are, however, more tractable. In particular, skin friction for in-channel flows is a strong function of bed material grain size, and a number of relationships exist which express the resistance coefficient in terms of the bed material median grain size, $D_{50}$ (e.g. Hey, 1979). Equally, on floodplain sections where conveyance rather than storage processes dominate, the drag due to vegetation is likely to form the bulk of the resistance term (Kouwen, 2000). Determining the drag coefficient of vegetation is, however, rather complex, as the frictional losses result from an interaction between plant biophysical properties and the flow. For example, at high flows the vegetation momentum absorbing area will reduce due to plant bending and flattening. To account for such effects Kouwen and Li (1980) and Kouwen (1988) calculated the Darcy-Weisbach friction factor $f$ for short vegetation, such as floodplain grasses and crops, by treating such vegetation as flexible, and assuming that it may be submerged or non-submerged. $f$
is dependent on water depth and velocity, vegetation height and a product \( MEI \), where \( M \) is the number of stems per unit area, \( E \) is the stem modulus of elasticity and \( I \) is the stem area’s second moment of inertia. Whilst \( MEI \) often cannot be measured directly, it has been shown to correlate well with vegetation height (Temple, 1987).

Similar to topography, ground survey of grain size and vegetation parameters is extremely time consuming and, recently, research has begun to consider the use of remotely sensed techniques to determine certain of the above data. For example, photogrammetric techniques to extract grain size information from ground-based or airborne photography are currently under development (Butler et al., 2001). Similarly, recent research developments allow extraction of specific plant biophysical parameters from LiDAR data. For vegetation > 4m in height, this latter technique uses the timing difference between the first and last points on the returned LiDAR waveform to determine the height of the canopy. For vegetation < 4m the method uses the local standard deviation of the last return heights. Cobby et al. (2000 and 2001) demonstrate that the height of short vegetation up to 1.4m high can be estimated with such a technique to 0.14m rmse. Taller vegetation (>10m) is subject to greater height estimation error (~2-3m) as canopies are typically denser and it is less likely that the laser pulse will penetrate the full depth of the canopy, however for the purpose of determining hydraulic resistance this is less of a problem. Given that other plant biophysical properties, e.g. MEI, correlate with plant height, Mason et al. (2003) have presented a methodology to calculate time and space distributed friction coefficients for flood inundation models directly from LiDAR data. Much further work is required in this area, however such studies are beginning to provide methods to explicitly calculate important elements of frictional resistance for particular flow routing problems. This leads to the prospect of a much reduced need for calibration of hydraulic models and therefore a reduction in predictive uncertainty.

3.5 Model data assimilation

Increasing use of the above remote sensing techniques has caused a rapid shift in hydraulic modelling from a data-poor to a data-rich and spatially complex modelling environment with attendant possibilities for model testing and development. Despite an increase in computational power, the relative resolution of model and topography data has now reversed for most codes typically used to simulate flood inundation at the reach scale (see Bates and De Roo, 2000 for a review). A newly emergent research area is therefore how to integrate such massive data sets with lower resolution numerical inundation models in an optimum manner that makes maximum use of the information content available. This is the direct opposite to the problem that most environmental modellers have traditionally faced (see Grayson and Blöschl, 2001 for a general discussion).

For example, Marks and Bates (2000) describe the integration of a LiDAR data set with a two-dimensional hydraulic model where the unstructured mesh discretization was derived independent of the topography. The topography was then assimilated into the model in an \textit{a posteriori} step using weighted nearest neighbour interpolation to assign an elevation value to each computational node. This is typical of finite element mesh construction in many fields, but may not produce a mesh that captures those attributes of the original surface that are critical to the modelling problem in hand and may also lead to high data redundancy. To overcome these problems, Bates \textit{et al.} (2003) describe a processing chain for high resolution data assimilation into a lower resolution unstructured model grid. This consists of: (1) variogram analysis to determine significant topographic length scales in the model; (2) identification of topographically significant points in this data set; (3) incorporation of these points into an unstructured model grid that provides a quality solution for the relevant numerical solver and (4) use of the data left over from the mesh generation process to parameterize the Bates and Hervouet (1999) sub-grid scale algorithm for dynamic wetting and drying. This method is demonstrated for the case of LiDAR topographic data but is general to any data type or model discretization. Cobby \textit{et al} (2003) take this process further and develop an automatic mesh generator that produces an unstructured grid refined according to vegetation features (hedges, stands of trees etc) on the floodplain identified automatically from LiDAR. These methods show some promise, however much further work is required in this area to more fully analyse the numerical quality of meshes generated by competing techniques.

Similarly for friction parameters, Mason \textit{et al}. (2003) use an area-weighting method to calculate area-effective frictional resistance from the high resolution height information contained in LiDAR data. This
method aims to yield model parameters that are appropriate to the particular discretization used, rather than being scale and discretization independent, although this has yet to be fully tested.

3.6 Calibration, validation and uncertainty analysis

In all but the simplest cases (for example, the planar free-surface ‘lid’ approach of Puech and Raclot, 2002), some form of calibration is required to successfully apply a floodplain flow model to a particular reach for a given flood event. Calibration is undertaken in order to identify appropriate values for parameters such that the model is able to reproduce observed data and, as previously mentioned, typically considers roughness coefficients assigned to the main channel and floodplain and values for turbulent eddy viscosity, $\nu_{t}$, if a zero-equation turbulence closure is used. Though these values may sometimes be estimated in the field with a high degree of precision (Cunge, 2003), it has proven very difficult to demonstrate that such ‘physically-based’ models are capable of providing accurate predictions from single realisations for reasons discussed in the critiques of Beven (1989, 1996, 2001) and Grayson et al. (1992). As such, values of parameters calculated by the calibration of models should be recognised as being effective values that may not have a physical interpretation outside of the model structure within which they were calibrated. In addition, the process of estimating effective parameter values through calibration is further convoluted by a number of error sources inherent in the inundation modelling process which cast some doubt on the certainty of calibrated parameters (Aronica et al., 1998; Horritt, 2000; Bates et al., 2004). Principally, these errors relate to the inadequacies of data used to represent heterogeneous river reaches (i.e. geometric integrity of floodplain topography and flow fluxes along the domain boundaries) but also extends to the observations with which the model is compared during calibration and the numerical approximations associated with the discrete solution of the controlling flow equations. The model will therefore require the estimation of effective parameter values that will, in part, compensate for these sources of error (Romanowicz and Beven, 2003).

Given that the number of degrees of freedom in even the simplest of numerical models is relatively large, it is no surprise that many different combinations of effective parameter values may fit sparse validation data equally well. Such equifinality in floodplain flow modelling has been well documented (see, for example, Romanowicz et al., 1994, 1996; Aronica et al., 1998, 2002; Hankin et al., 2001; Romanowicz & Beven, 2003; Bates et al., 2004) and uncertainty analysis techniques, based on the Generalised Likelihood Uncertainty Estimation (GLUE) of Beven & Binley (1992), have been developed and applied in response. Based on Monte Carlo simulation, GLUE explicitly recognises this equifinality and seeks to make an assessment of the likelihood of a set of effective parameter values being an acceptable simulator of a system when model predictions are compared to observed field data. In studies reported thus far, this level of ‘acceptability’ has typically been calculated by considering the deviation of simulated variables from one of the following classes of observed data (according to the terminology of Bates & Anderson (2000)):

External and internal bulk flow measures – for example, time series of stages and/or discharges recorded continuously at river gauging stations. Routinely available in national flow archives, these data have proven utility in testing the wave routing behaviour of flood models (for example, Cunge et al., 1980; Klemeš, 1986) and have shown to be replicable by even the simplest of numerical schemes (Horritt and Bates, 2002). As Dietrich (2000) notes “if the goal of a modelling exercise is to predict solely the time evolution of some bulk property of a complex system then spatial lumping (or spatial integration) can be invoked to filter out spatial variability”. As such, they are unlikely to provide a sufficiently rigorous test for competing model parameterisations (i.e. sets of effective parameter values) within spatially distributed schemes capable of directly simulating hydraulic patterns. Nevertheless, automatic collection of these data, often at hourly intervals or less, may provide records of sufficient temporal resolution to reasonably evaluate dynamic model performance and/or identify compensating errors operating within the model structure (for example, Bates et al., 1998). However, given the considerable intermediate distances between gauging stations, internal observations are typically much less common than external ones. Clearly, bulk flow data, while being an important source of observational data, have only limited strength as a stand-alone piece of evidence for the evaluation of hydraulic models and are sometimes of questionable accuracy, particularly for large out-of-bank flows (Bates & Anderson, 2000). Recourse must therefore be made additional sources of data, such as spatially distributed fields of model state variables (i.e. vector and point scale data) to further discriminate between competing models.
Vector data, for example, instantaneous observations of flood extent, either processed from remotely-sensed data (Horritt, 1999) or ground-survey methods (Simm, 1993), which may be considered an approximation to the zero water depth contour. Observational data of this type has the advantage that for relatively flat floodplain topography small changes in water surface elevation can result in large changes in shoreline position. Hence, in order to replicate a flood shoreline adequately a model must generate, either directly or through secondary interpolation, an accurate distributed field of the flow depth. In studies thus far, inundation extent has proven a useful test of model performance as it is both a relatively sensitive measure and its simulation requires a model capable of dealing with dynamic wetting and drying over complex topography (for example, Bates et al., 1997; Horritt, 2000; Bates & De Roo, 2000). However, during events where the valley is entirely inundated, and the flood shoreline is constrained mostly by the slopes bounding the floodplain, changes in water levels may produce only small changes in flood extent (Horritt and Bates, 2001a and b). In these instances, internal validation based on hydrometric data may prove a more exacting test of model performance than extent data. Furthermore, inundation data have typically been limited to one synoptic view per event due to the limited temporal resolution of satellite-borne imaging radars and thus provide only single ‘snapshots’ of inundation extent which do not allow adequate checking of the inundation dynamics simulated by the model.

Point scale data – for example, water levels recorded discretely during post-event surveys or velocities measured in the field. Maximum water levels can often be observed after the recession of a flood event, either as high-water marks on surviving structures or as trash or wracklines deposited at the limit of maximum inundation, and have been used in isolation as calibration data in a number of studies (for example, Aronica et al., 1998; Beven, 1989). However, Lane et al. (1999) have drawn attention to the potential dangers of using these data for calibration purposes, observing that variables simulated by the model are integrated over both space and time and, as such, are rarely reconcilable with point scale observations collected in the field.

While it is clear from the preceding discussion that each class of observed data has some value in the calibration and uncertainty estimation processes, it also apparent that the limitations of each data source are such that, when used in isolation, they may preclude any definitive judgements to be made about model performance. These limitations may be considered in direct relation to the dimensionality of the respective data sources and their failure to provide explicit definition in higher dimensions – for example, vector data which may be considered two-dimensional in space but zero-dimensional in time. However, by making use of more than one type of observational data in the calibration process we should (hopefully) be able to overcome the inherent spatial or temporal limitations of any single source.

To date, only very limited attempts to calibrate distributed models against more than one particular data type have been made. Horritt and Bates (2002) tested the predictive performance of three industry-standard hydraulic codes on a 60 km reach of the River Severn, UK using independent calibration data from hydrometric and satellite sources. They found that all models were capable of simulating inundation extent and floodwave travel times to similar levels of accuracy at optimum calibration, but that differences emerged according to the calibration data used when the models were used in predictive mode due to the different model responses to friction parameterisations. However, Horritt and Bates (2002) did not consider either the potential for combining both data sources in the calibration process or the uncertainties associated with the model predictions.

Multiple observation data sets for historical flood events are still exceedingly rare and, as such, the potential value of additional observations in the calibration process has yet to be explored in the case of floodplain flow models. Furthermore, the use of uncertainty estimation techniques during this conditioning process allows the relative value of individual (sets of) observations to be precisely quantified in terms of the reduction in uncertainty over effective parameter specification (cf. Beven & Binley, 1992; Hankin et al., 2001). Interpretation of these uncertainty measures may also provide guidance over how much and of what type of observed data would be required to achieve given levels of uncertainty reduction in simulated variables.
4. The Future of Flood Inundation Modelling

Numerical modelling of floodplain flows has made major advances over the last decade as a result of increased computational power and the availability of rich and detailed remotely sensed data sources capable of accurate spatial parameterization of key input data. As a result, resources are now available for relatively inexperienced modellers to begin to run complex simulations on a desktop PC. Over the same period there has been relatively less progress on designing new models or solution algorithms, although laboratory experimental programmes continue to yield new insights into the physics of compound channel flow which serve to refine our perceptual model. These trends are likely to persist as the numerical solution of shallow water flow equations is now relatively well understood, whilst further advances in the field of floodplain flow modelling will continue to come from the development of new data sources. This in turn will lead to a better understanding of the uncertainties inherent to such codes when used in a calibration framework.

Examples of possible future research areas might include the use of increased computer power to simulate multi-dimensional flows over large domains with sophisticated turbulence closure. Application of such codes to flows in natural channels with relatively small grid sizes will also force researchers to consider how flow-vegetation interactions (see Wilson et al. 2005) and flows in the near-wall region can be better represented. Increased computer power may also be used to extend the size of model domains, as well as enhance process representation. Here one could envisage basin scale applications of simple two-dimensional codes or the representation of significant portions of globally significant rivers to understand better the dynamics of the flood pulse in such environments (see for example Coe et al. 2002).

An improved understanding of the generation of friction and turbulence by vegetation combined with enhanced retrieval of plant biomechanical properties from remotely sensed data also has great potential. Mason et al. (2003) have already published an algorithm to determine first order floodplain roughness coefficients from vegetation heights derived from airborne laser altimeter data, whilst Cobby et al. (2003) have used the same data to automatically generate unstructured grids for two-dimensional floodplain models. With further research it is possible to imagine that we can move towards the automatic generation of uncalibrated hydraulic models that have reduced predictive uncertainty. Other types of remotely sensed data capture, for example surface flow velocity from along-track airborne radar interferometry (Srokosz, 1997) and synoptic sequences of inundation images through a flood, may also aid the model validation process and allow us to discriminate better between competing models, algorithms and data assimilation methods as suggested by Mason et al. (2003).

Lastly, whilst flow equations for floodplain flow modelling and their numerical solution are in general well understood, one obvious exception is in the specification of appropriate schemes that are capable of handling dynamic wetting and drying effects over complex, low lying topography (see Bates and Horritt, 2005). Much research has already been undertaken in this area for moving and fixed grids (Lynch and Gray, 1980; Kawahara and Umetsu, 1986; Leclerc et al., 1990; Benkhaldoun and Monthe, 1994; Defina et al., 1994, Ip et al., 1998; Tchamen and Kawahita, 1998; Bates and Hervouet, 1999; Defina 2000), however a definitive solution has yet to be found. Such research is one area where further numerical analysis and new forms of the controlling equations are required, particularly in the case of three-dimensional codes.

In conclusion, research into floodplain flow modelling has begun to enter a particularly dynamic phase as a result of new data and computing opportunities which will enable fundamental questions about distributed data assimilation, distributed calibration, the predictability of natural systems and the limits of physically-based modelling to be critically assessed. Surface hydraulic models are a powerful tool with which to begin to answer these general environmental modelling questions as they are parametrically simple and we can actually make direct measurements of the process of interest. Floodplain flow modelling is also a key applied problem where distributed predictions are required and used to inform planning and insurance decisions that may have serious consequences for individuals but where, until recently, our ability to undertake such research was limited by the data available. It is likely that that the answer to the question “which model is appropriate for a particular task?” will continually change as computer power increases and data availability improves.
Floodplain flow modelling is therefore an area where there is considerable potential for exciting and innovative science aimed at solving a major societal problem.
<table>
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<th>Method</th>
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<td>0D</td>
<td>No physical laws included in simulations.</td>
<td>Broad scale assessment of flood extents and flood depths.</td>
<td>ArcGIS Delta mapper</td>
<td>DEM Upstream water level Downstream water level</td>
<td>Inundation extent and water depth by intersecting planar water surface with DEM</td>
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<td>1D</td>
<td>Solution of the one-dimensional St Venant equations.</td>
<td>Design scale modelling which can be of the order of 10s to 100s of km depending on catchment size.</td>
<td>Mike 11 HEC-RAS SOBEK-CF Infoworks RS (ISIS)</td>
<td>Surveyed cross sections of channel and floodplain Upstream discharge hydrographs Downstream stage hydrographs</td>
<td>Water depth and average velocity at each cross section Inundation extent by intersecting predicted water depths with DEM Downstream outflow hydrograph</td>
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<tr>
<td>1D+</td>
<td>1D plus a storage cell approach to the simulation of floodplain flow.</td>
<td>Design scale modelling which can be of the order of 10s to 100s of km depending on catchment size, also has the potential for broad scale application if used with sparse cross-section data.</td>
<td>Mike 11 HEC-RAS Infoworks RS (ISIS)</td>
<td>As for 1D models</td>
<td>As for 1D models</td>
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<tr>
<td>2D-</td>
<td>2D minus the law of conservation of momentum for the floodplain flow.</td>
<td>Broad scale modelling or urban inundation depending on cell dimensions.</td>
<td>LISFLOOD-FP</td>
<td>DEM Upstream discharge hydrographs Downstream stage hydrographs</td>
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<tr>
<td>2D</td>
<td>Solution of the two-dimensional shallow wave equations.</td>
<td>Design scale modelling of the order of 10s km. May have the potential for use in broad scale modelling if applied with very course grids.</td>
<td>TUFLOW Mike 21 TELEMAC SOBEK-OF Delft-FLS</td>
<td>DEM Upstream discharge hydrographs Downstream stage hydrographs</td>
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<tr>
<td>2D+</td>
<td>2D plus a solution for vertical velocities using continuity only.</td>
<td>Predominantly coastal modelling applications where 3D velocity profiles are important. Has also been applied to reach scale river modelling problems in research projects.</td>
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5. References


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