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The urban inundation model with bidirectional flow interaction between 2D overland surface and 1D sewer networks

Le modèle d'inondation urbain avec interaction des flux bidirectionels entre 2D surfaces émergées et 1D réseaux d'égout

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RÉSUMÉ

Cet article développe un modèle numérique intégrant surface et égouts pour l'étude des processus de ruissellement en milieu urbain. Les courbes précipitations ruissellement et les conditions d'écoulement dans les réseaux de drainage sont basées sur le modèle 1D. Les écoulements en surfaces sont détermines par le modèle 2D. Bien que les deux modèles soient résolus par des algorithmes différents et a des échelles différentes, la connection est possible, par l'intermédiaire des bouches d'égouts. Les effluents et influents au travers de ces dernières sont détermines par les équations correspondant aux déversoirs et orifices. Afin de garantir un interfaçage adéquats, un soin particulier est consacre a la synchronisation temporelle entre les modèles.

ABSTRACT

An integrated numerical model is developed in the study for simulating the runoff processes in urban areas. A 1D model is used for calculating the rainfall-runoff hydrographs and the flow conditions in drainage networks. A 2D model is employed for routing flow on overland surface. Both models are solved by different numerical schemes and using different time steps with the flow through manholes adopted as model connections. The effluents and influents via manholes are determined by the weir or the orifice equations. Timing synchronisation between both models is taken into account to guarantee suitable model linkages.

KEYWORDS

adaptive time step, coupled models, timing synchronisation, urban flood

1. INTRODUCTION

In urban areas, the storm sewer networks are commonly built for conveying runoffs caused by pluvial rainfall. Therefore, the condition of storm sewer system is highly related to its performance with respect to sewer flooding. Many models have been developed for assessing the hydraulic performance of drainage networks. The Storm Water Management Model (SWMM) developed by the U.S. Environmental Protection Agency (Rossman et al. 2005) is commonly used not only for hydraulic evaluation but also for water quality modelling in sewage systems (Ha et al. 2003). Because SWMM is a freeware, its hydraulic engine is widely used in commercial software, including PC SWMM (Computational Hydraulics Int. 2002), XP SWMM (XP Software Inc. 2002) and MIKE SWMM (DHI Software 2002). In addition, various other software packages that use other hydraulic solvers, such as MOUSE (DHI Software 2003) and Infoworks-CS (Wallingford Software Ltd. 2006), exist. Most of those models use the water stage-volume curve to determine the flood depth caused by manhole surcharge, but cannot properly simulate the water movement on the ground surface.

Hence, the dual drainage concept has been often introduced to describe the flow interactions between the above ground and below ground systems (Djordjević et al. 1999; Nasello and Tucciarelli 2005). Natural flow paths and retention basins are regarded as the major drainage system for routing the surface flow, while sewer pipes, manholes and inlets are considered as the minor drainage system for conveying the sub-surface flow. Both drainage systems are modelled by 1D hydraulic models and linked through manholes and inlets to reflect the bi-directional interactions between sub-surface and surface networks (Bolle et al. 2006; Mark et al. 2004; Schmitt et al. 2005).

When the major drainage system capacity is insufficient to handle the runoff caused by pluvial events, the overland flow occurs. The 1D modelling approaches are usually limited to circumstances where overland flows are confined to predetermined flow paths, while the 2D overland flow models should be employed to simulate the movement of surface runoff caused by large surcharges (Carr and Smith 2006; Chen et al. 2005; Dey and Kamioka 2006; Horritt and Bates 2002).

2. METHODOLOGY

In this study, flows in drainage pipes and over the ground surface were routed by the 1D sewer network model SIPSON (Djordjević et al. 2005) and the 2D overland flow model UIM (Chen et al. 2005), respectively. Both models use different numerical schemes and time steps with the discharge through manholes adopted as model linkages. The drainage runoff and surcharge effluents were calculated by SIPSON for every time step and treated as point sinks and sources, correspondingly, in the UIM model within the same time interval. Discharges are determined by weir or orifice equations by taking account of the hydraulic heads at manholes and ground surface.

2.1. SIPSON

SIPSON consists of two components, the hydrological and the hydraulic models. The hydrological model, BEMUS (Radojković and Maksimović 1984), was developed for computing rainfall-runoff hydrographs for sub-catchments to be used as inputs for manholes in the hydraulic model. The hydraulic model is used to analyse the dynamic flow behaviour in 1D drainage networks. It simultaneously solves continuity equations for network nodes, energy equations for nodes and pipe/channel ends, the complete Saint Venant equations for flow in conduits and streets, and equations for other link types (pumps, weirs, etc.).

2.2. UIM

UIM simulates flow movements on the ground surface. The 2D non-inertia flow equations, which are a simplified form of the Saint Venant equations, are adopted in the model. The conveyance reduction factor based on the building coverage ratio and the storage volume of underground structures are considered in UIM to reflect the influences caused by buildings on flow movements.

Like many other 2D models, the computing of overland flow is time-consuming when the proposed model is applied to cases with massive number of grids. Consequently, the adaptive time step is introduced to speed up the simulations. The time step in the 2D model is adjusted automatically based on the Courant criterion (Hunter et al. 2005; Yu and Lane 2006) such that larger time steps could be chosen whenever the numerical stability is satisfied.

The time step used in SIPSON is generally larger than the ones used for UIM. The default and upper bound of time steps in UIM are the same in SIPSON, i.e., $\Delta t_{2d_0} = \Delta t_{1d}$. By the end of computation for each time step, the Courant condition is checked based on the latest calculated water depth and velocities for setting the next time step:

$$\Delta t'_{2d_{m+1}} \le \frac{\Delta x}{\sqrt{g d_{2d_m}} + u_{2d_m}} \tag{1}$$

$$\Delta t'_{2d_{m+1}} \le \frac{\Delta y}{\sqrt{g d_{2d_m}} + v_{2d_m}} \tag{2}$$

where, m is the index of the time step; $\Delta t'_{2d_{m+1}}$ is the estimated time step length [s] used in UIM for $m+1$ th step; $d_{2d_m} = h_{2d_m} - z_{2d}$ is the water depth [m] of the computing grid at m th step; u_{2d_m} and v_{2d_m} are velocity components [m/s] along x and y directions, respectively. Theoretically, Eqs. (1) and (2) should be verified grid by grid for having the best value over the entire domain, nevertheless, the global maximum values $d_{2d\_\max_m}$, $u_{2d\_\max_m}$, and $v_{2d\_\max_m}$ are deemed to be satisfactory and are used for saving the computing effort.

2.3. Model linkage

Two distinct models are combined for simulating the flow dynamics in sewer networks and on overland surface. The models are executed individually and linked by exchanging information obtained at proper locations and times for appropriate linkages. In this study, the grids containing manholes are considered as the locations where interactions occur. The time step used in SIPSON is also regarded as the timing for model linkage.

2.3.1. Interacting discharge

The bidirectional interacting discharge is calculated according to the water level difference between sewer network and overland surface. The upstream and downstream levels for determining discharge are defined as $h_U = \max\{h_{mh}, h_{2d}\}$ and $h_D = \min\{h_{mh}, h_{2d}\}$, respectively, where h_{mh} is the hydraulic head [m] at manhole and h_{2d} is the water surface elevation [m] on the overland grid.

The ground levels acquired from sewer network dataset usually differ from the values extracted from the digital terrain model (DTM). Both values are correct because they represent different attributes of the system. The ground level $z_{mh_{top}}$ [m] used in sewer model reflects the point value on the top of a manhole, whereas, the grid elevation z_{2d} [m] in the overland flow model stand for the average level of the topography within a grid. Inconsistency between two datasets is often present when the manhole is located at local peak or depression inside the grid as shown in Figure 1.

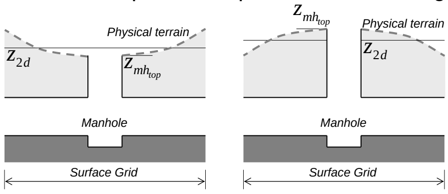


Figure 1 Inconsistency between top elevation of manhole and averaged grid elevation

The crest elevation $z_{crest} = \max\{z_{mh_{top}}, z_{2d}\}$ [m] is, therefore, defined for computing the discharge between both models. Three types of linkages are considered for applying different discharge equations.

- Free weir linkage

The free weir equation is adopted when the crest elevation z_{crest} is between the values of the upstream water level h_U and the downstream water level h_D, as shown in Figure 2. The discharge is calculated by using Eq. (3).

$$Q = sign\left[h_{mh} - h_{2d}\right] c_w w \sqrt{2g}\left(h_U - z_{crest}\right)^{\frac{3}{2}} \tag{3}$$

where, Q is the interacting discharge [m^3/s], whose positive value meant surcharge flow from sewer toward overland and negative value meant drainage flow from surface into sewer; c_w is the weir discharge coefficient; w is the weir crest width [m]; and g is the gravitational acceleration [m/s^2].

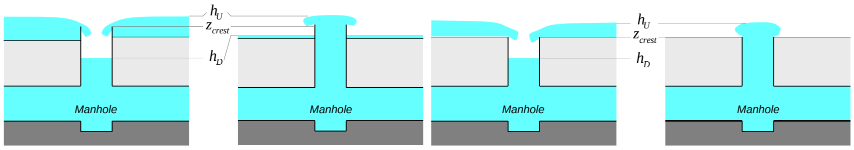


Figure 2 Free weir linkages

- Submerged weir linkage

The submerged weir equation is used (Figure 3) when both water levels at manhole and overland grid are greater than the crest elevation and the upstream water depth above the crest, $\left(h_U - z_{crest}\right)$, is less than A_{mh}/w, where A_{mh} is the manhole area [m^2]. Eq. (4) is employed for determining the interacting discharge.

$$Q = sign\left[h_{mh} - h_{2d}\right] c_w w \sqrt{2g}\left(h_U - z_{crest}\right)\left(h_U - h_D\right)^{\frac{1}{2}} \tag{4}$$

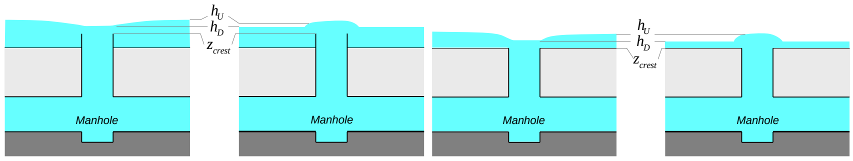


Figure 3 Submerged weir $h_U \le A_{mh}/w$ or orifice $h_U > A_{mh}/w$ linkages

- Orifice linkage

The manhole is considered fully submerged (Figure 3), when the upstream water depth above the crest $\left(h_U - z_{crest}\right)$ is greater than A_{mh}/w for the submerged weir linkages. The orifice equation is used for calculating the interacting discharge.

$$Q = sign\left[h_{mh} - h_{2d}\right] c_o A_{mh} \sqrt{2g}\left(h_U - h_D\right)^{\frac{1}{2}} \tag{5}$$

where, c_o is the orifice discharge coefficient.

2.3.2. Timing synchronisation

The flow dynamics in drainage networks and overland surface are solved by two separate models using different time steps. The timing synchronisation become an important issue for connecting both models appropriately, in particular because variable time steps are used. The exact values in both models are exchanged at the same time, as shown in Figure 4, to prevent unnecessary errors being introduced to model communication. The estimated time step $\Delta t'_{2d_{m+1}}$, by using Eqs. (1) and (2), is restricted to Eq. (6) for matching the next synchronisation timing.

$$\Delta t_{2d_{m+1}} = \max\left\{\left(T_{syn_l} + \Delta t_{1d} - \sum_{i=1}^{m}\Delta t_{2d_i}\right), \Delta t'_{2d_{m+1}}\right\} \tag{6}$$

where, $\Delta t_{2d_{m+1}}$ is the time step size [s] used for $m+1$ th step; T_{syn_l} is time of the previous (l th) synchronisation [s]; $\sum_{i=1}^{m}\Delta t_{2d_i}$ is the total length of time steps [s] after m steps of calculations in UIM; $(T_{syn_l} + \Delta t_{1d} - \sum_{i=1}^{m}\Delta t_{2d_i})$ is the time left [s] before next ($l+1$ th) synchronisation.

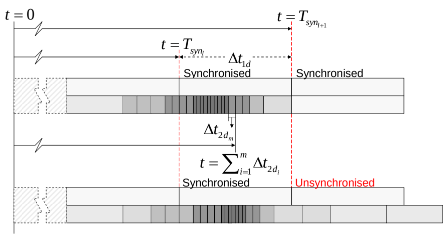


Figure 4 Synchronized and unsynchronized timing between 1D and 2D models

3. APPLICATION

Two examples are presented: the artificial study area that was employed to analyse and discuss the detailed flow interactions and model application to the real world case study. The intention was to test whether the proposed model can effectively and simultaneously describe the bidirectional flow interactions and the surface runoff movements.

3.1. Idealised case study

The idealised artificial case was used to demonstrate the ability of the proposed model to simulate bidirectional interactions between the sewer network and overland surface. The 1500 x 1000 m^2 study area shown in Figure 5 consisted of three inclined planes. Surface slopes were 0.002, -0.002 and 0.002 in x-direction and 0.001 in y-direction. Ten manholes and ten pipes were installed for draining the surface runoff and the sub-catchments (shown in Figure 5).

Manhole 1 had the lowest elevations of the ground surface and the drainage system, and was connected to the outlet. Manhole 3 was located in a local depression 0.4m below the elevation of the exit point.

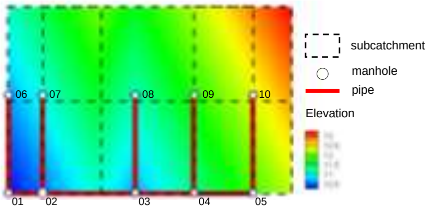
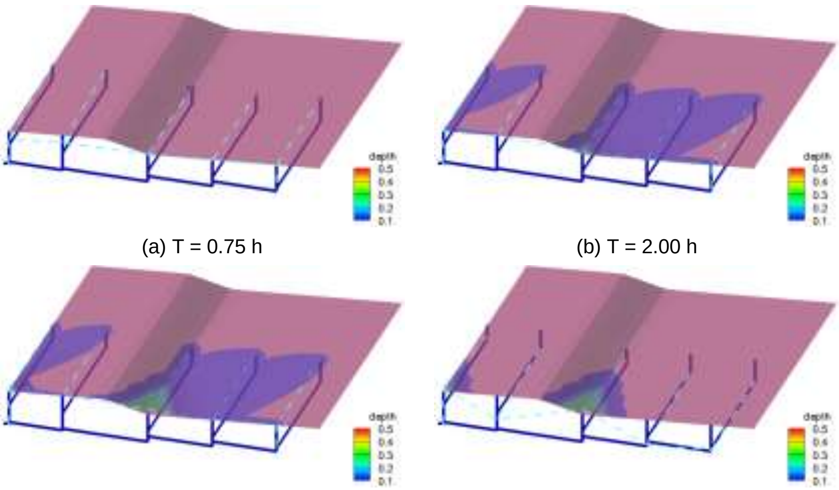


Figure 5 Elevation, drainage network and sub-catchments of idealised case

The design rainfall intensity for the drainage system was 10 mm/h. A six-hour rainfall event was applied herein for simulation. The rainfall intensity started from zero and increased to 20mm/h in the first hour, kept constant for four hours, and decreased to zero in the sixth hour. The duration of numerical experiment was 12 hours to simulate the receding process of surface inundation. The rainfall-runoff hydrographs were calculated by BEMUS and imposed as inflows at manholes. Figure 6-(a) reveals that manholes became surcharged when the inflow discharge had exceeded the design capacities of pipes and the hydraulic head had reached the surface elevation. The surcharged flow spread along the ground surface toward lower elevations and caused surface inundation as displayed in Figure 6-(b). The overflow between sub-catchments occurred when the flood extent reached the lowest exit point of the sub-catchment boundary as shown in Figure 6-(c). The surface runoff kept moving on overland until it reached a downstream manhole where the hydraulic head was below the surface elevation and the drainage capacity was available. After the rainfall stopped, the flows in sewer pipes began decreasing such that the flood extent was reduced as a consequence of the surface runoff had returned to the sewer system.



(a) T = 0.75 h (b) T = 2.00 h

(c) T = 5.50 h (d) T = 12.00 h

Figure 6 Simulation results of idealised case

3.2. Keighley case study

The Stockbridge region in Keighley (Bradford, UK) shown in Figure 7-(a) was chosen for practical application. The region between the River Aire in the south and the Leeds and Liverpool Canal in the north is a flood prone area. The northeast part next to the Canal is residential area with dense buildings. The rest of the region is green lands with a pond situated next to the River Aire. The major storm drainage system passes along the roads in residential area, as highlighted in Figures 7-(b-e),. The fluvial flooding from the River Aire was the major factor that led to inundation in the region. Nevertheless, the analysis was focused on the consequence of a pluvial event in this study. An extreme rainfall was assumed in the upstream catchments and within the region. The resulting runoff was too large for the drainage system capacity. The DTM with buildings was obtained from the LiDAR data, and resampled as 2m x 2m grid resolution for overland flow simulation. The time steps ranged from 0.5s to 5s with an average 0.732s which meant that the adaptive time steps saved around 31% of computing effort compared to using the uniform minimum step size in UIM.

The flow surcharged from Manholes A, B and G moved along the surface toward lower lying areas. Not only the pre-identified pond was filled but also two other minor depressions were inundated. The water was trapped in the ponds and flood extent kept increasing due to the lack of drainage inlets inside ponds. The flow surcharged from manholes C, D, E and F was confined by the buildings and moved along the streets until it reached downstream manholes where the drainage capacity was available. Part of the water was detained in local depressions next to the buildings near Manhole C. The simulation results clearly indicated the potential inundation area caused by pluvial rainfall although there was no overbank flooding from the channels.

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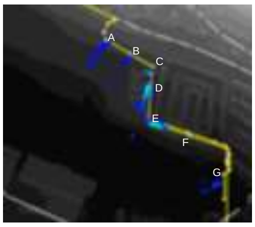
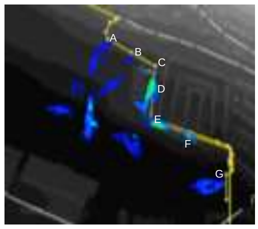
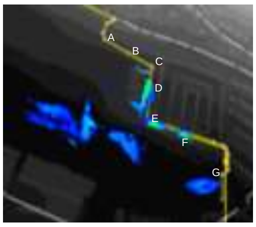
(a) Stockbridge region in Keighley



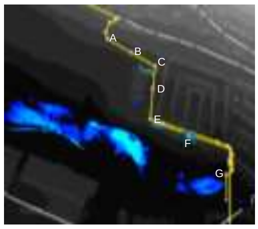
(b) Simulation result at T = 30 min



(c) Simulation result at T = 40 min



(d) Simulation result at T = 50 min



(e) Simulation result at T = 120 min

Figure 7 Study area and simulation results of an extreme rainfall event in the Stockbridge region (sewer pipes shown in yellow, various shades of blue indicate different water depths)

4. DISCUSSION AND CONCLUSIONS

The initial rainfall-runoff processes, including from rainfall to surface runoff, from surface runoff to drainage inlets, from inlets to gullies, and from gullies to major manholes, were simplified and calculated by using hydrological model in the study. In other words, it was assumed that the surface runoff was collected and conveyed directly to major manholes. Consequently, the overflows occurring at the most upstream manholes in both study cases were due to the concentrated runoff and the insufficient capacities of sewer pipes such that the surface runoff could not enter the drainage system. Those overflows were actually excess runoff rather than surcharges. The surcharged runoff was not allowed to return to the sewer system until it reached a major manhole. The assumption was made due to lack of detailed inlet information and simplifications of the initial rainfall-runoff processes. In reality, the surface runoff in urban area is collected by distributed inlets and conveyed to major manhole via gullies. Therefore, if the inlets were taken into account in the Stockbridge case, the flood water in local depressions next to buildings could be drained and returned to the sewer system.

By taking the sewerage flow component and the overland runoff component into account, the proposed model simulates the complex flow processes on overland surface of pluvial flooding cases better than the approach that uses 1D model only.

Presented results illustrate several issues inherent to coupled 1D-2D modelling of urban flooding. Linkages between the two models should be formulated carefully by taking care of the physical situation and by ensuring appropriate time synchronisation. Adequate spatial resolution is important, both in terms of the number of inlets taken into account (1D model) and the surface grid size (2D model). Coarse resolution may cause unrealistic results such as ponds not being drained or the influence of buildings not represented accurately, whereas too detailed resolution may make the simulation unacceptably slow. Based on a range of case studies, further research will focus on addressing these issues and on the objective comparison between results obtained by the presented 1D-2D approach and different 1D-1D models.

5. ACKNOWLEDGEMENTS

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LIST OF REFERENCES

Bolle, A., Demuynck, A., Willems, P., Bouteligier, R., Bosch, M. S., Verwey, A., and Berlamont, J. (2006) Hydraulic modelling of the two-directional interaction between sewer and river systems. In: *7UDM&4WSUD Intl. Conf.*, Melbourne.

Carr, R. S., and Smith, G. P. (2006) Linking of 2D and Pipe hydraulic models at fine spatial scales. In: *7UDM&4WSUD Intl. Conf.*, Melbourne.

Chen, A. S., Hsu, M. H., Chen, T. S., and Chang, T. J. (2005) An integrated inundation model for highly developed urban areas *Wat. Sci. & Tech.*, 51(2), 221-229.

Computational Hydraulics Int. (2002). Introduction for PCSWMM 2002. http://www.computationalhydraulics.com/pcswmm.html, Guelph.

Dey, A. K., and Kamioka, S. (2006) An integrated modeling approach to predict flooding on urban basin. In: *7UDM&4WSUD Intl. Conf.*, Melbourne.

DHI Software. (2002). MIKE SWMM - the new generation SWMM. Hørsholm.

DHI Software. (2003). *MOUSE Pipe Flow, Reference Manual*, Hørsholm.

Djordjević, S., Prodanović, D., and Maksimović, C. (1999) An approach to stimulation of dual drainage *Wat. Sci. & Tech.*, 39(9), 95-103.

Djordjević, S., Prodanović, D., Maksimović, Č., Ivetić, M., and Savić, D. (2005) SIPSON - Simulation of interaction between pipe flow and surface overland flow in networks *Wat. Sci. & Tech.*, 52(5), 275-283.

Ha, S. R., Park, S. Y., and Park, D. H. (2003) Estimation of urban runoff and water quality using remote sensing and artificial intelligence *Wat. Sci. & Tech.*, 47(7-8), 319-325.

Horritt, M. S., and Bates, P. D. (2002) Evaluation of 1D and 2D numerical models for predicting river flood inundation *J. Hydrol.*, 268(1-4), 87-99.

Hunter, N. M., Horritt, M. S., Bates, P. D., Wilson, M. D., and Werner, M. G. F. (2005) An adaptive time step solution for raster-based storage cell modelling of floodplain inundation *Adv. Wat. Res.*, 28(9), 975-991.

Mark, O., Weesakul, S., Apirumanekul, C., Aroonnet, S. B., and Djordjević, S. (2004) Potential and limitations of 1D modelling of urban flooding *J. Hydrol.*, 299(3-4), 284-299.

Nasello, C., and Tucciarelli, T. (2005) Dual multilevel urban drainage model *J. Hydraul. Eng.*, 131(9), 748-754.

Radojković, M., and Maksimović, Č. (1984) Development, testing and application of Belgrade urban drainage model. In: *3rd ICUD*, Göteborg, 1431-1443.

Rossman, L. A., Dickinson, R. E., Schade, T., Chan, C., Burgess, E. H., and Huber, W. C. (2005) SWMM5: The USEPA's newest tool for urban drainage analysis. In: *10th ICUD*, Copenhagen.

Schmitt, T. G., Thomas, M., and Ettrich, N. (2005) Assessment of urban flooding by dual drainage simulation model RisUrSim *Wat. Sci. & Tech.*, 52(5), 257-264.

Wallingford Software Ltd. (2006). InfoWorks CS - Simulation Engine. http://www.wallingfordsoftware.com/products/infoworks_cs/simulation_engine.asp.

XP Software Inc. (2002). XP-SWMM Technical Description, Portland. http://www.xpsoftware.com.au/products/pdfs/SWMMTech%20Description.pdf.

Yu, D., and Lane, S. N. (2006) Urban fluvial flood modelling using a two-dimensional diffusion-wave treatment, part 1: mesh resolution effects *Hydrol Process*, 20(7), 1541-1565.