NUMERICAL MODELING OF TSUNAMI INUNDATION USING SUBGRID SCALE URBAN ROUGHNESS PARAMETERIZATION

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The importance of accurate numerical modeling of tsunami inundation in an urban area has clearly realized due to the devastating damage from 2011 Tohoku Earthquake Tsunami. Although, numerical inundation simulations using high resolution topography data (O(1m)), the medium resolution tsunami inundation model (O(10m)-O(100m)) needs and useful for tsunami hazard assessment. This study develops and validates a numerical model of tsunami inundation using upscaled urban roughness parameterization: Drag Force Model (DFM) which deals with the effect of structures as drag force acting on flow based on physical modeling. The validation of the DFM reveals that the DFM can express the effect of the flow direction and inundation ratio.

Keywords: tsunami; inundation; upscaling; urban roughness; drag force; drag coefficient

INTRODUCTION

Recently, survey technology such as light detection and ranging (LIDAR) or laser scanning has been developed and they are able to provide high resolution topography datasets (e.g. Marks et al., 2000). Such dataset can enable to deal with the effect of complex structures on a tsunami run-up and numerical tsunami inundation simulation started to use ultra-high-resolution topography data with O(1m) (e.g., Prasetyo, 2017; Park et al., 2013). However, the application of such models is not practical due to computational cost (Abu-raya and Imamura, 2002). The medium resolution model with O(10m) - O(100m) is necessary and useful for making hazard assessment or large number of simulations for local assessment or stochastic tsunami hazard assessment (e.g., Goda et al., 2015). Then numerical models of tsunami inundation in the medium resolution with higher accuracy than preceding models have been strongly required. The objective of this study is to propose new roughness parameterization by upscaling high-resolution topography data and to obtain more accurate bottom roughness coefficient with medium resolution than current parameterizations.

NUMERICAL METHOD

Governing Equation

The target of this study is improving the tsunami inundation over the artificial complex topography in the medium resolution (e.g. O(100m)), although the use of 3D Navier-Stokes equation is getting popular for understanding detail of tsunami inundation modeling in the scientific purpose. Therefore, a set of governing equations of tsunami inundation over the urban area is 2D nonlinear shallow water equation (NSWE).

$$\frac{\partial \eta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \tag{1}$$

$$\frac{\partial M}{\partial t} + \frac{\partial}{\partial x}\left(\frac{M^2}{D}\right) + \frac{\partial}{\partial y}\left(\frac{MN}{D}\right) + gD\frac{\partial \eta}{\partial x} + R_x = 0 \tag{2}$$

$$\frac{\partial N}{\partial t} + \frac{\partial}{\partial x}\left(\frac{MN}{D}\right) + \frac{\partial}{\partial y}\left(\frac{N^2}{D}\right) + gD\frac{\partial \eta}{\partial y} + R_y = 0 \tag{3}$$

where t is time, x and y are horizontal axes, η is sea surface elevation, M and N denote flow discharge fluxes for unit width in each direction, and R_x and R_x are resistance forces due to roughness or topography changes such as the Manning's coefficient, respectively.

Drag Force Model (DFM)

This study examines a roughness parameterization based on the subgrid topographical feature. Here, we introduce Drag Force Model (DFM) proposed by Fukui et al. (2018), which considers the effect of structures with drag force if we neglect inertial force. Figure 1 shows a conceptual view of the DFM. We

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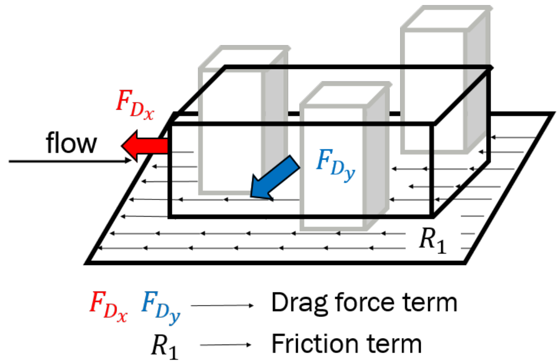


Figure 1: Conceptual view of Drag Force Model (DFM)

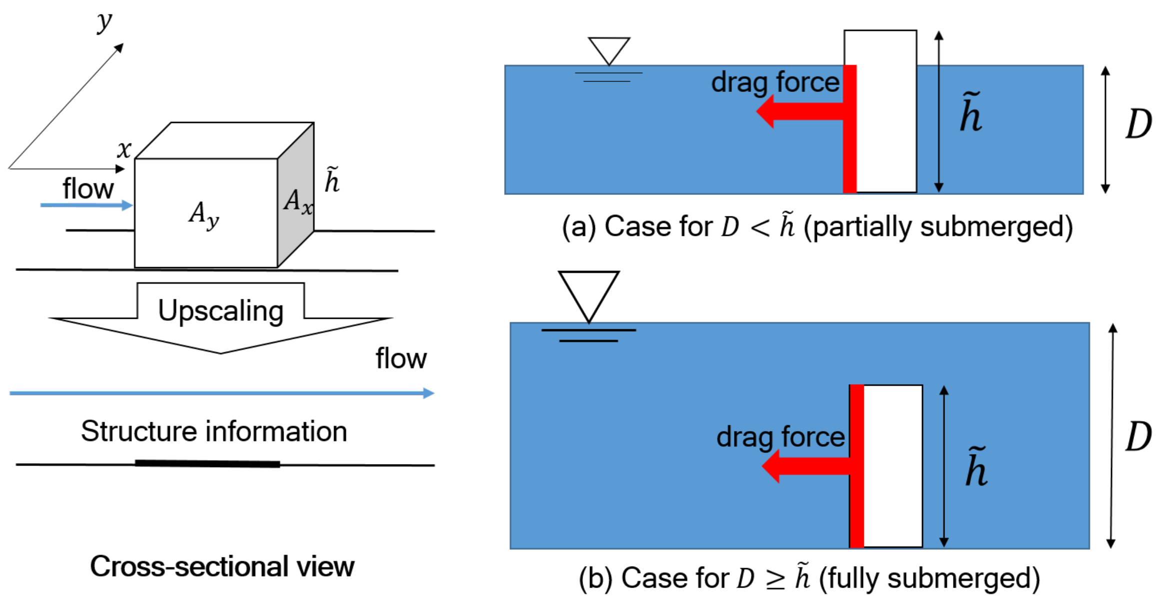


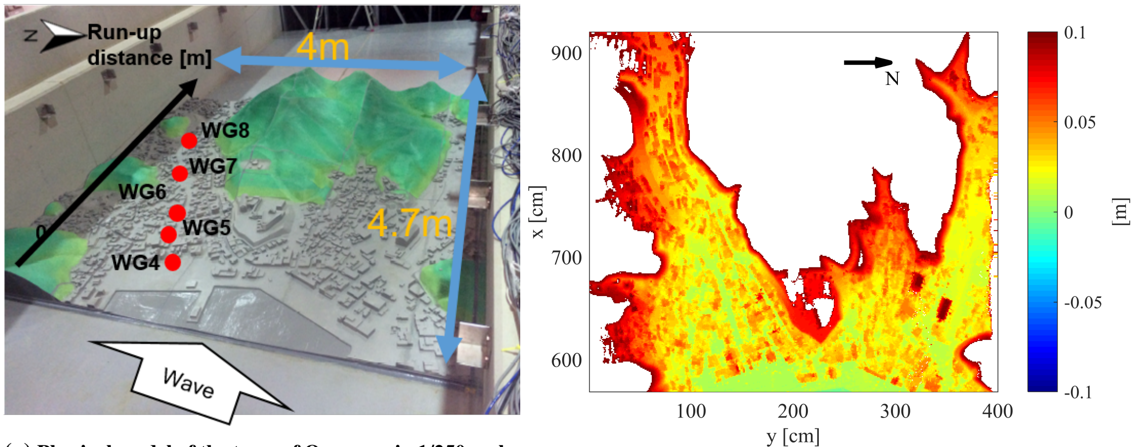
Figure 2: Conceptual view of roughness parameters (left) and case division for submergence of structures (right); (a) case of partial inundation and (b) full inundation

define the drag forces by the structures as in Eq.(4) and (5).

$$\frac{F_{Dx}d}{\rho} = \frac{1}{2}C_D\frac{A_x d}{\Delta x \Delta y D}M\frac{\sqrt{M^2+N^2}}{D^2} \tag{4}$$

$$\frac{F_{Dy}d}{\rho} = \frac{1}{2}C_D\frac{A_y d}{\Delta x \Delta y D}N\frac{\sqrt{M^2+N^2}}{D^2} \tag{5}$$

where F_{Dx} and F_{Dy} are drag force by flow water, C_D is drag coefficient, A_x and A_y are cross-sectional area of structure in x and y direction, $\tilde{h}$ is characteristic height of structure in the grid, ρ is density of fluid, d is depth of the area where drag force acts, and Δx and Δy are computational mesh size. A_x, A_y, and $\tilde{h}$ represent the characteristics of subgrid scale structures in the computational mesh as shown in Figure 2 (left). The DFM uses A_x, A_y, and $\tilde{h}$ as roughness parameters representing sub-grid scale structure information. Also, d represents the water depth of the area that the drag force acts on, which is equal to the characteristic building height $\tilde{h}$ or the water depth D according to the submergence of the building as shown in Figure 2



(a) Physical model of the town of Onagawa in 1/250 scale (red mark: wave gauge points)

(b) Computational domain of the town of Onagawa

Figure 3: Study area of this study ((a) physical model of the town of Onagawa, (b) topography of the town of Onagawa)

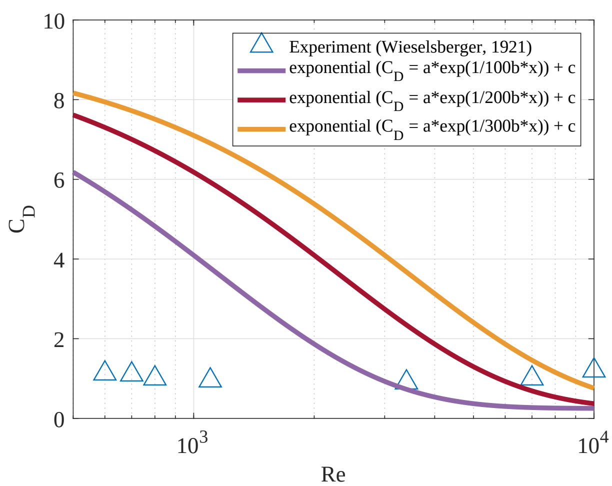


Figure 4: Assumed C_D - Re relation

(right). Therefore, the local water depth d is given by

$$d = \begin{cases} \tilde{h} & (D \geq \tilde{h}) \\ D & (D < \tilde{h}) \end{cases} \tag{6}$$

OUTLINE OF NUMERICAL SIMULATION OF TSUNAMI INUNDATION EXPERIMENT

The physical modeling of tsunami inundation was conducted in the flume named the Hybrid Tsunami Open Flume in Ujigawa Labolatory (HyTOFU) installed in Ujigawa Open Laboratory in 2015 (Prasetyo, 2017). The study area is a coastal area; the town of Onagawa in Miyagi Prefecture. The town of Onagawa is located on the northern part of Tohoku and was severely damaged by the 2011 Tohoku Earthquake Tsunami. Figure 3a shows an overview of the physical model of the town of Onagawa in 1/250 scale. The width, length, and depth of the model are 4.0 m, 4.7 m, and 2.0 m, respectively. Time series of the water level at the locations of 13 wave gauges mounted on the physical model were measured. The experimental results were compared to the numerical results by NSWE. A series of comparison proves the experimental results can be used as a benchmark for validation of other numerical models of tsunami inundation (Prasetyo, 2017).

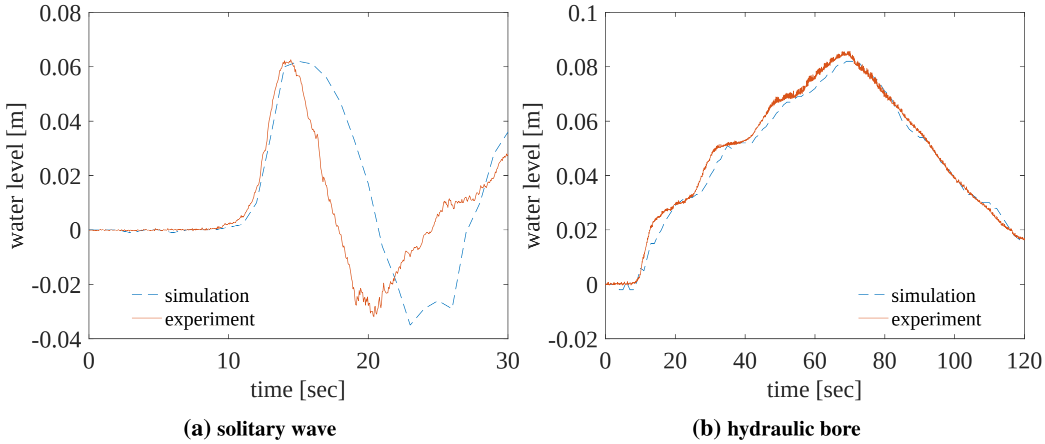


Figure 5: Time series of water level at WG3 as incident waves for (a): solitary wave case, (b): hydraulic bore case (blue dashed line: numerical result, red line: experimental results).

This study performed numerical simulation of tsunami inundation by the DFM based on the topography with high resolution (O(1m)) obtained by scanning the physical model of the town of Onagawa. Figure 3b shows the computational domain of this study. Topographical data of the structure was removed and upscaled roughness parameters were installed instead for the DFM. The numerical simulation using the original toporaphy with a fine grid (Δx = 1cm) wil be denoted as the Structure Resolving Model (SRM). Roughness parameters are calculated in each upscaled mesh and 10, 20, and 40 cm are used for the upscaled mesh size. The different assumptions of the drag coefficient C_D (constant: 0.5, 1.0 and 3.0 and empirical formula depending on the Reynolds number) were applied to the DFM. Regarding the empirical formula of C_D, we have started several classical formulae of C_D for simple structures. Next, assuming C_D and Re relations considering asymptotic behavior for Re to 0 and infinity. An experimental result for a cylinder by Wieselsberger (1921) is given by an exponential approximation as follows

$$C_D = a \exp(bRe) \tag{7}$$

The values of a and b are 9.1481 and -0.0867 as estimated by the least squares method, respectively. This study tuned the empirical variable C_D formula for a group of structures based on Eq.(7) and adding some parameters. The developed empirical formula is as follows

$$C_D = a \exp(\lambda bRe) + c \tag{8}$$

where λ is a tuning parameter and the values 1/100, 1/200 and 1/300 are examined and optimized to adjust maximum inundation depth with experimental results. The value of c is also a tuning parameter that represents the minimum limit of C_D, and this study adopts a value of 0.25. Figure 4 is the resultant curve of the empirical formula for C_D.

This study adopts a hydraulic bore and solitary wave as an incident wave. Figure 5 shows the shapes of incident wave for (a) the solitary wave and (b) hydraulic bore.

NUMERICAL RESULTS AND DISCUSSIONS

Sensitivity Analysis of C_D

We consider two attributes of the drag force coefficient for the DFM: 1) a constant value based on previous experimental studies by Aburaya and Imamura (2002), and Shimamora et al.(2007), and 2) a variable value based on the developed empirical formula as a function of Re. The use of an Re dependent drag force lacks sufficient data to support a theoretical and empirical formula or to estimate the effect of the energy dissipation due to a group of structures in different geometrical configurations, Re values, and water depths. This study conducts calibration and Re dependent relations to compare with the Onagawa experiment results of maximum inundation depth, arrival time (figure is not shown). We focus on two cross-sections along the wave gauges (southern urban area and northern urban area). In this paper, the results at WG4 to 8 in Figure 3a are shown since the tendency of the results is common for each cross-section. Figures 6a and 6b show the results of the numerical simulations of the DFM while changing the drag coefficients with the mesurement results. The fixed drag coefficient values 0.5, 1.0 and 3.0 were used following the previous study (e.g. Aburaya and Imamura, 2002; Shimamora et al., 2007). The empirical coefficient values of a and b were set to 9.1481 and −0.0867 following the results in section 3, and the empirical coefficient λ was changed from 1/100 to 1/300, respectively.

The numerical results for the constant C_D cases of 0.5, 1.0 and 3.0 show a change in water depth that is sensitive to the incident wave condition. Overall, the choice of a C_D value is more significant for solitary wave run-up condition rather than the bore condition. On the other hand, the value of maximum inundation depth for the cases with variable C_D values, Eq.(8) changes much less than the constant C_D case and the difference is at most about 15 %.

The wave run-up changes water depth and velocity from the onshore to land side, and the dependence of C_D on Re is important. Therefore, the values of the maximum inundation depth and maximum momentum flux change considerably according to the fixed value of C_D. On the other hand, the DFM with the variable C_D values enables the model to consider the change of vortexes around structures to some extent. Then, the differences in the numerical results are within about 15 %. These results are seemingly good, but an error of around 20 % still exists between the experimental results. It implies that we need to develop a formulation for C_D and calibrate the tuning parameters more carefully.

Comparison between DFM and SRM

This section discusses the comparison of each bottom roughness boundary condition and parameterization (DFM and SRM). Similar to the previous section, the maximum inundation depths and arrival times are compared with the wave gauge data from the experimental results. The drag force coefficient is determined by the variable Re formula of Eq.(8) with $\lambda = 1/200$ based on the discussion in the previous section. The maximum inundation depths and arrival times are compared with in-situ measured data at each location from the shoreline inland. Figures 7a and 7b show the comparison of the maximum inundation depth and arrival time between the experimental results and each roughness parameterization. The solid lines in the figures show the numerical results for the hydraulic bore and solitary wave as the incident wave (red solid line: DFM, and blue solid line: SRM), the dashd line indicates bottom elevation along the street, and the circles and triangles indicate the bore and solitary condition by experiments, respectively.

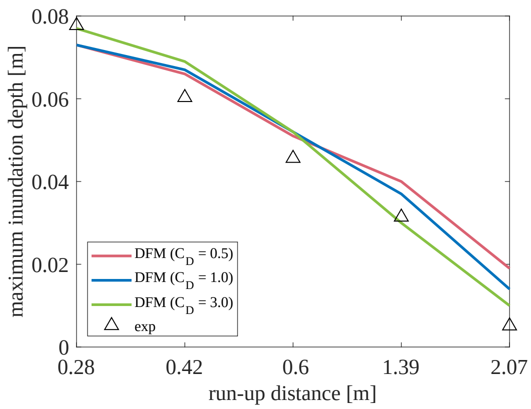
The incident wave conditions yield significant differences between the different parameterizations. The solitary wave condition shows large differences in the numerical results across each parameterization. First, we focus on the solitary wave case. The DFM result shows about 10 % error, though it underestimates values in the onshore area ($x_{\text{run-up}}$ = 0.6 m). As we described in the previous section, wave run-up simulated by the DFM is lower when the characteristic structure height $\tilde{h}$ is large. The point of inland extent ($x_{\text{run-up}}$ = 2.07 m) is located where the combination and numbers of structures is complex. The DFM basically shows equal or better results than the SRM and others for solitary wave run-up. The DFM can reproduce the effect of flow damping and change of direction based on the parameterization.

The SRM overestimates the maximum inundation depths, except for the first group of structures ($x_{\text{run-up}}$ = 0.28 m) for the solitary wave case. The error of inundation depth for the SRM is the largest, about 35 %, at the inland area ($x_{\text{run-up}}$ = 1.39 m), and then the maximum inundation depth decreases more than 50 % from this point ($x_{\text{run-up}}$ = 1.39 m) to the middle of the domain ($x_{\text{run-up}}$ = 2.07 m). Despite the decreasing water depth, the SRM still overestimates the inundation depth for complex urban areas. The SRM can consider the topographical characteristics, but the NSWE itself does not account for short wave diffraction and turbulence around the structures. Therefore, the wave energy does not dissipate and this results in higher maximum inundation depths that depend on the layout of structures.

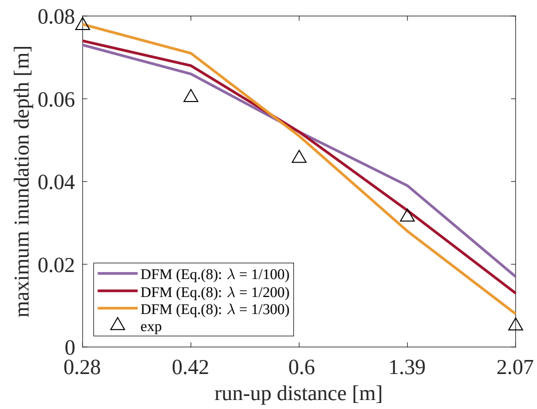
On the other hand, the hydraulic bore case shows less differences between each parameterization than the solitary wave does. The DFM gives better results than CERM for the bore case, but the maximum differences are only about 10 % from the onshore to inland areas ($x_{\text{run-up}}$ = 0.42 m to 1.39 m).

The errors at the points from onshore to inland ($x_{\text{run-up=}}$ 0.6 m and 1.39 m) are mainly due to $\tilde{h}$, which is quite large onshore ($x_{\text{run-up}}$ = 0.42 m). On the other hand, the DFM overestimates the maximum inundation depth in the middle of the domain ($x_{\text{run-up}}$ = 2.07 m) and the SRM estimates the best result for the bore case. The errors by the SRM are within 10 % and the difference between the SRM and DFM is within 10 % at all locations.

Secondly, we focus on the arrival time as shown in Large differences between each parameterization



(a) **Case for C_D is constant (magenta: $C_D = 0.5$, blue: $C_D = 1.0$, $C_D = 3.0$)**



(b) **Case for C_D is variable (purple: Eq.(8), λ = 1/100, red: Eq.(8), λ = 1/200, yellow: Eq.(8), λ = 1/300)**

Figure 6: Comparison of maximum inundation depths from onshore to inland by DFM for the solitary wave case (solid line: numerical results by DFM, triangle mark: experimental results).

are observed for the bore condition while little difference is observed across the solitary wave condition.

The bore propagation simulated by the DFM shows a slower run-up than for the experiment in the onshore area ($x_{\text{run-up}}$ = 0.42 m and 0.6 m). However, the arrival times by the DFM is underestimated in the inland area ($x_{\text{run-up}}$ = 1.39 m and 2.07 m). This result indicates that the DFM includes stronger structural effects in the onshore area that weaken in the inland area. This is because there were higher structures near the coast ratherthan inland in Onagawa town. The commercial and local governmental buildings were located onshore but te lower residential houses were located following them. This kind of local characteristics are included in the DFM but the other roughness parameterization are not.

Additionally, the arrival time by the SRM is not accurate apart from the maximum inundation depth. The SRM overestimates arrival times except near the coast ($x_{\text{run-up}}$ = 0.28 m and 0.42 m). This is because flow velocity between each structure is less than for the experiment.

CONCLUSION

This study proposed and examined a subgrid scale model for urban roughness parameterization so-called DFM. The DFM considering the effect of the structures was validated by the experimental results in wave flume HyTOFU targeting the maximum inundated extend and maximum inundation depth. The numerical results were compared with Structure Resolving Model (SRM). The major conclusion of this study can be summarized as follows:

1. The effect of turbulent flow passing a group of structures should be considered, and the target range of Reynolds numbers should be optimized to correspond to each computational domain for a real scale.

2. The DFM enables to express the effect of the flow direction and the inundation ratio on the numerical results to some extent.

3. The DFM improves the estimation of inundated extend and maximum inundation depth comparing the SRM.

4. Differences between each roughness parameterization largely affect the estimated inundation characteristics when a wave with short wavelength is used since it has small mass flux from offshore.

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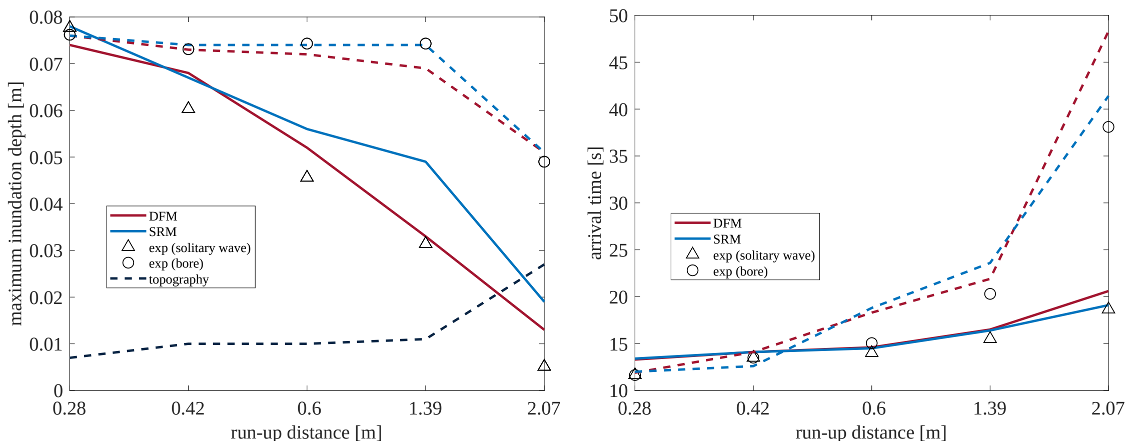

(a) Maximum inundation depth **(b) Arrival time**

Figure 7: Comparison of inundation characteristics from onshore to inland by DFM, SRM, and experimental results for the solitary wave and hydraulic bore case case (solid line: solitary wave case, dashed line: hydaulic bore case; red: DFM and blue: SRM, mark: experimental results for solitary wave case (triangle) and hydraulic bore (circle)).

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