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INUNDATION FLOW ANALYSIS DUE TO HEAVY RAINFALL IN LOW-LYING RIVER BASIN

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SYNOPSIS

In Japan, inundation due to heavy rainfall occurs frequently in urban areas. A comprehensive inundation flow model, which treats runoff from mountainous areas, flood flows in the river networks, inundation flows in the drainage basin and drainage of inundation water through sewerage system, is developed in this study. The effects of buildings, bankings of railways and flood control facilities are also considered. This model is applied to the Neya River basin, Japan, which is a highly urbanized area. The validity of this model is considered by comparing with the actual records of inundated areas and water level of the river network. Finally, the computation using the design rainfall of this river basin is also conducted.

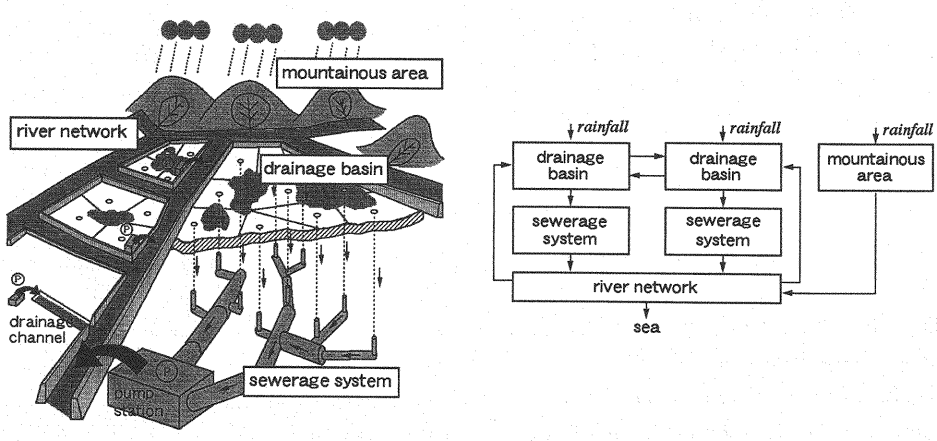
INTRODUCTION

As far as the management of a river basin is concerned, protection from flood disasters is one of the most important concerns in the field of civil engineering. More people suffer from flood disasters than the other kind of natural disasters in the world. In Japan, though very few people have been killed, flood disasters due to heavy rainfall occur frequently in urban areas. Japan is vulnerable to inundation because most its big cities are located along low-lying coastal areas and the elevation of the urban areas is usually lower than high water level of the urban rivers, which makes it very difficult to drain the inundation water. Since 1960s, inundation due to heavy rainfall has been a serious problem and continues to cause severe damage to urban areas. Therefore, many inundation flow models have been proposed so far, but many of them treat only a small catchment area (8)(9)(11) or use simple models for sewerage system (10). Furthermore, some of them are based on existing software packages such as SWMM and MOUSE (2)(5).

The objective of this study is to develop a numerical model for inundation flow analysis due to heavy rainfall. When inundation due to heavy rainfall is considered, inundation water is effected by runoff from the mountainous area, flood flow in the river network and drainage through the sewerage system and pump stations. Therefore, in this study, a comprehensive model is developed which can treat the inundation process in a whole river basin in detail. Then this model is applied to the Neya River basin, Japan, where there are frequent inundations. The possibility of flood hazards due to heavy rainfall is also discussed in this paper.

COMPUTATIONAL METHOD

As shown in Fig.1 (a), the studied river basin consists of four parts: (i) mountainous area, (ii) river network, (iii) drainage basin and (iv) sewerage system. In the mountainous area, runoff discharge is calculated by the kinematic wave model. In the river network, the characteristics method is applied to a one-dimensional unsteady flow analysis. In the drainage basin, a two-dimensional inundation flow analysis based on unstructured meshes is conducted. In the sewerage system, discharge from pump stations is calculated by the sewerage model for the one-dimensional unsteady flow with slot, which can treat both open-channel and pressurized flow conditions. The flow of rainwater is shown in Fig.1 (b). The runoff discharge from the mountainous area is assumed as lateral inflow of the river network. All the rainfall which falls into the drainage basin is drained through the sewerage system, and the water left on the drainage basin is treated as inundation water. The direct overflow from the river network to the drainage basin is also considered when the water level of the river network rises higher than the river bankings. In this study, the three sub-models except for the mountainous area are calculated simultaneously.



(a) Modeling of a river basin

(b) Flow of rainwater

Fig.1 Framework of total model

Mountainous area

The runoff analysis in the mountainous area includes both slope flow and mountainous river flow. The governing equations based on the kinematic wave model are as follows:
< slope flow >

$$\frac{\partial h}{\partial t} + \frac{\partial q'}{\partial x} = r_e \tag{1}$$

$$q' = \alpha h^m \tag{2}$$

where x : one-dimensional spatial coordinate, t : time, q' : discharge per unit width on the slope, r_e : effective rainfall ($= fr$, f : runoff percentage, r : rainfall intensity), h : water depth and m, α : coefficients ($m = 5/3$, $\alpha = \sqrt{\sin\theta_s}/N_e$, θ_s : slope gradient, N_e : equivalent roughness, according to the Manning's law).
< mountainous river flow >

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = \frac{q_s}{B} \tag{3}$$

$$q = \alpha h^m \tag{4}$$

where q : discharge per unit width of the mountainous river, q_s : lateral inflow per unit width from the slope, B : mountainous river width, $m = 5/3$ and $\alpha = \sqrt{\sin\theta}/n$, θ : river bed slope and n : roughness coefficient.

The lateral inflow from the slope to the river is calculated by the characteristics method, and the runoff discharge along the river is calculated by the finite difference method using the Leap-Frog method (7).

River network

In the river network, the following continuity and St. Venant equations are used (4),

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_m - q_0 \tag{5}$$

$$\frac{1}{g}\frac{\partial u}{\partial t} + \frac{u}{g}\frac{\partial u}{\partial x} + \frac{\partial h}{\partial x} = s_0 - s_f \tag{6}$$

where A : cross sectional area of flow, Q : discharge, $u = Q/A$: velocity averaged over cross-section, $s_0 = \sin\theta$: river bed slope, $s_f = n^2 u|u|/R^{4/3}$: friction slope, R : hydraulic radius, g : gravitational acceleration, q_m : lateral inflow per x-directional unit width from the mountainous area, pump stations and drainage channels. The overtopping discharge q_0 into the drainage basin per x-directional unit width is calculated as follows:

$$q_0 = 0.35 h_1 \sqrt{2gh_1} \tag{7}$$

where h_1 : overflow depth. The drainage basin is surrounded by sufficiently high bankings, so overtopping flow is assumed to be a perfect overflow, and neither the submerged overflow nor the overtopping flow from the drainage basin to river network are treated in this study.

Sewerage system

Rainwater on the drainage basin is drained into the river network through the sewerage system. This system consists of sub sewers, main sewers and pump stations. The drainage process through the sewerage system is modeled as follows (see Fig.2).

The rainwater in the sewer pipe is dynamically calculated based on the following continuity and momentum equations using Leap-Frog method.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{8}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (uQ)}{\partial x} = -gA\frac{\partial H}{\partial x} - \frac{gn^2|Q|Q}{R^{4/3}A} \tag{9}$$

where A : cross sectional area of flow, Q : discharge, q : lateral inflow, u : flow velocity, R : hydraulic radius, $H = z + h$: water level, z : elevation of sewer pipe bottom and h : water depth, which is calculated as follows,

$$h = \begin{cases} A/B & : \quad (A < A_p) \\ B' + (A - A_p)/B_s & : \quad (A > A_p) \end{cases} \tag{10}$$

where the case of $A < A_p$ is open-channel flow condition and that of $A > A_p$ is pressurized flow condition, B : pipe width, B' : height of pipe ceiling, A_p : cross sectional area of pipe and B_s : slot width. In order to treat both open-channel and pressurized flow conditions, the slot model (1), which considers the sewer pipe with a hypothetical narrow slot on its ceiling as shown in Fig.2, is introduced in this study. The value of the roughness coefficient is uniformly 0.015 (which is prescribed when the design velocity is determined in the Neya River basin) and the cross sectional shape of pipes is assumed to be rectangular.

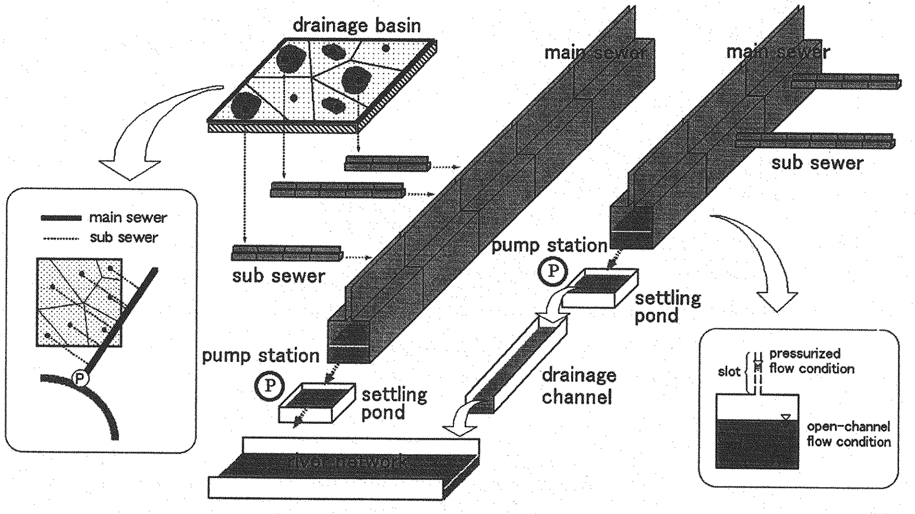


Fig.2 Sewerage model

There is only one sub sewer pipe in each drainage basin mesh. The route of the sub sewer is determined as the shortest path to the nearest main sewer from the mesh centroid (see Fig.2). The slope and the cross sectional shape of sub sewers are assumed to be uniformly 1/1,000 and 1m square, respectively. The inflow discharge from a drainage basin mesh into a sub sewer pipe will be described later. When the piezometric head of the upstream end of the sub sewer is higher than the inundation water level of the surface drainage basin, the drainage into the sub sewer does not occur.

The main sewers have a tree-type connection. The rainwater flows down from a few (or one) upstream main sewers into one main sewer. At some junctions or downstream ends of downstream sewer pipes, the pump stations are located. The lateral inflow q is defined as the value obtained by dividing the flow discharge at the downstream end of the sub sewer by the longitudinal spatial interval Δx of the main sewer.

Each pump station usually has a settling pond as shown in Fig.2. The water level of the settling pond is necessary for calculation of the flow discharge at the downstream end of the main sewer. In this study, the bottom area of the settling pond is defined in proportion to the drainage capacity of each pump station, and the bottom elevation of the settling pond is assumed to be 0.5m lower than that of the main sewer bottom.

The rainwater drained from some pump stations is conveyed to the river network through the drainage channel as shown in Fig.2. These drainage channels are assumed to be one-dimensional rectangular channels and the following equations are applied:

$$\frac{\partial h}{\partial t}+\frac{\partial M}{\partial x}=\frac{q_{in}}{B} \tag{11}$$

$$\frac{\partial M}{\partial t}+\frac{\partial (uM)}{\partial x}=-gh\frac{\partial H}{\partial x}-\frac{gn^2|M|M}{h^{7/3}} \tag{12}$$

where u, M : flow velocity and discharge per unit width in the x-direction, respectively, H : water level, q_{in} : lateral inflow per unit length from the pump station and B : channel width.

Drainage basin

The two-dimensional inundation flow analysis using Leap-Frog method based on unstructured meshes is applied (6). In order to determine the effects of buildings and railway bankings, the occupying ratio λ (the ratio of the buildings area to the mesh area) and invasion ratio β (the ratio of the side length, through which inundation water can go into or out, to the total side length) are introduced following Inoue et al.(3). The governing equations are the following continuity and momentum equations neglecting the non-linear terms, because such a large velocity will not appear in low-lying areas.

$$(1-\lambda)\frac{\partial h}{\partial t}+\frac{\partial M^*}{\partial x}+\frac{\partial N^*}{\partial y}=r_e-q_{out}+q_{over} \tag{13}$$

$$\frac{\partial M}{\partial t} = -gh\frac{\partial H}{\partial x} - \frac{gn^2 M\sqrt{u^2+v^2}}{h^{4/3}} \tag{14}$$

$$\frac{\partial N}{\partial t} = -gh\frac{\partial H}{\partial y} - \frac{gn^2 N\sqrt{u^2+v^2}}{h^{4/3}} \tag{15}$$

where u, v : x, y-components of flow velocity, respectively, $M^* = \beta M$, $N^* = \beta N$: x, y-components of corrected discharge per unit width, M, N : x, y-components of discharge per unit width, respectively, q_{out} : drainage discharge per unit area from computational mesh into sewerage system and q_{over} : overtopping flow discharge per unit area of computational mesh from the river network.

q_{out} is defined as follows. The maximum drainage capacity Q_m into the sub sewer is assigned to each drainage basin mesh m. Q_m is defined as distributed discharge of the design discharge of the main sewer q_I in proportion to the mesh area.

$$Q_m = \frac{A_m}{\sum_{k=1}^{K_I} A_k} q_I \tag{16}$$

where I : the main sewer which the rainwater of mesh m flows into, A_k, A_m : area of mesh k, m, respectively, K_I : the number of meshes from which rainwater flows into the main sewer I. q_{out} is defined as the smaller value of "the maximum drainage capacity per unit area, Q_m/A_m" and "the discharge per unit area when all the inundation water of mesh m is drained within $2\Delta t$, $h_m/(2\Delta t)$", where h_m : water depth of mesh m and Δt : computational time step.

APPLICATION TO THE NEYA RIVER BASIN

As shown in Fig.3, the Neya River basin is surrounded by hilly areas and river bankings. This is one of the most rapidly urbanized areas in Japan, including a part of Osaka City and its satellite cities such as Neyagawa City, Moriguchi City, Higashi-Osaka City and Yao City. The topographical conditions and rapid urbanization are factors that cause this river basin to be a severely damaged area due to heavy rainfall. The mountainous area which was studied is 49km^2, the drainage basin is 197km^2, and the total length of the river network amounts to 89km.

Mountainous area

There are 25 rivers flowing into the river network from the west slope of Ikoma mountainous area. Most of them have the similar topographical conditions, therefore, the runoff discharge from the Oto River is only calculated and it is assigned to the other rivers in proportion to their catchment area. However, the runoff discharge from the three rivers, the Sanra River, Kiyotaki River and Gongen River, which are located in the north part of the mountainous area is calculated individually because their topographical conditions may be different from those of

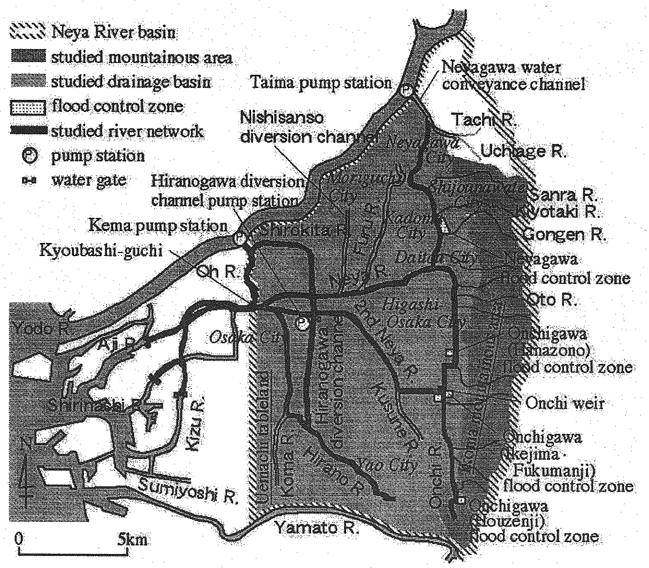


Fig.3 Studied area

Oto River. Some part of runoff discharge from Uchiage River, which exceeds the drainage capacity of Taima pump station (110m^3/s), is assumed to be inflow from the upstream end of the Neya River. The hydrological parameters used here are as follows: The runoff percentage is 80%, equivalent roughness of slopes, N_e, is 1.0 and the roughness coefficient of mountainous rivers, n, is 0.020 (7).

River network

At the upstream end of the river network, a certain small constant discharge is given. At the downstream end, where water gates are located, the boundary condition is given depending on each computational case. After some trials, Manning's roughness coefficient is determined from 0.020 to 0.055 considering piers of the expressways. First, under the above mentioned conditions, the computations are executed until the water level and flow discharge become unchanged. Then the obtained results are used as the initial conditions.

As the effect of flood control facilities attached to the river network, storage in the flood control zones (totally 3.59×10^6m^3), drainage through Kema pump station (maximum capacity is 200m^3/s) and the effects of the Onchi weir and the Hiranogawa diversion channel pump station are considered.

Sewerage system

The total pipe length of the main sewers considered here is 227km. The four drainage channels are also considered. The sewerage networks including the drainage channels are shown in Fig.4. Though the underground regulating ponds (totally 0.37×10^6m^3) are attached to the sewerage system as flood control facilities, their effects are not examined in this study because

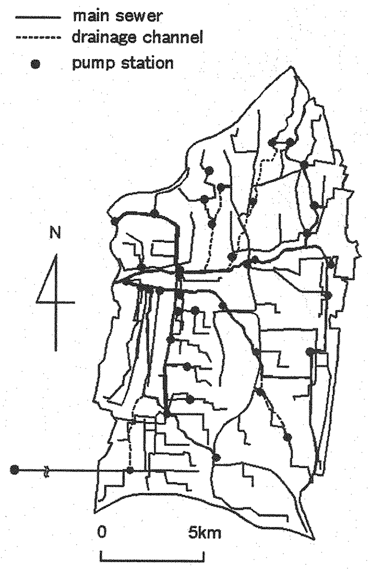


Fig.4 Sewerage network

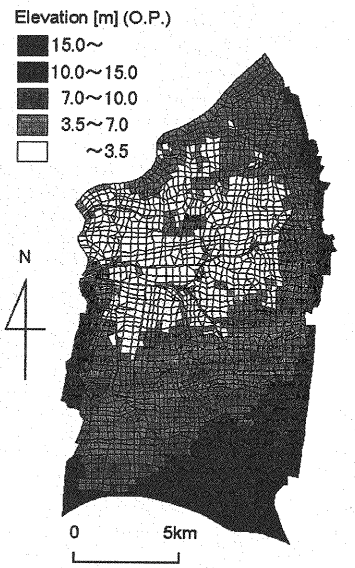


Fig.5 Computational meshes and elevation

the capacity of them is very small compared with the inundation water volume and the treatment of the starting time of flowing into the underground regulating ponds is very difficult.

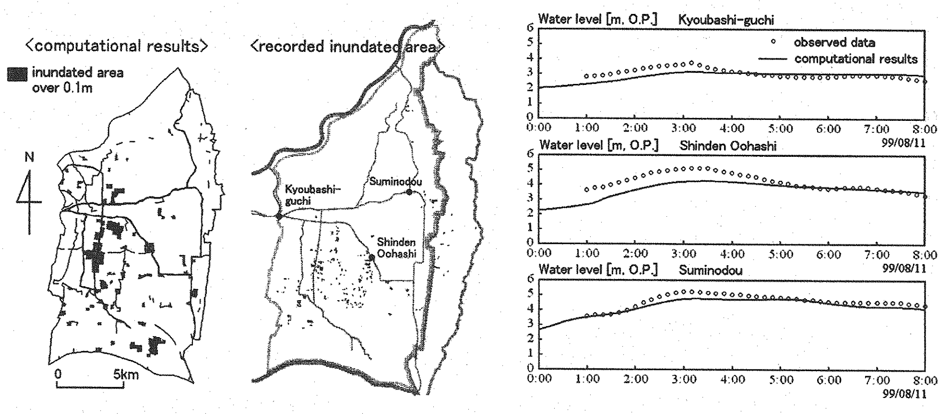
Drainage basin

The drainage basin is divided into 2,915 unstructured computational meshes based on 35 catchment areas of the sewerage system. The occupying ratio and the invasion ratio are defined at each computational meshes and the boundary of them, respectively. Fig.5 shows these computational meshes and elevation distribution. The runoff percentage is assumed 80% and roughness coefficient is set to be 0.067 (12).

RESULTS AND DISCUSSIONS

Validity of the model

The computational results are compared with the actual records. On August 11, 1999, there was comparatively heavy rainfall in the Neya River basin, which exceeded sewerage capacity and caused inundation in urban area. In this study, four sub-models are effected complicatedly with each other, so by using the observed inundated area and water levels in the river network, these four sub-models can be expected to be validated. The rainfall, observed on August 11, 1999 at 8 rainfall gauging stations (maximum hourly and total precipitation are 56mm/hr and 244mm, respectively), is given to each catchment area considering spatial distribution of rainfall based on the Thiessen method. The temporal change of tidal stage ob-



(a) inundated area (b) water level of the river network

Fig.6 Comparison between computational results and observed data

served at Ajigawa water gate is used as the boundary condition at the downstream end of the river network. The computational time step Δt is 0.1s for the mountainous area and 0.5s for the river network, drainage basin and sewerage system. The total computational time is 8hr.

A comparison of inundated area and water level of the river network between the computational results by this model and the observed data is shown in Fig.6 (a) and (b), respectively. By means of this model it can be well expressed how wide the inundated area spreads and that inundated area is concentrated on the left side of the 2nd Neya River, though detailed inundated spots cannot be reproduced. The water level of the river network in the latter part of the computation can be well expressed, but there are some differences of the peak level and the water level at Kyoubashi-guchi and Shinden Oohashi in the initial part of the computation. The reason for this discrepancy between the inundated area and water level of the river network may be that rainfall spatial distribution is not appropriate or that the pump stations are empirically operated based on the operators' experiences. As a result, the behavior of inundation water and drainage process from the drainage basin can be well expressed by the model developed here, but the connection between sewerage model and river network cannot be discussed in detail, and further investigation using more detailed rainfall spatial distribution is necessary.

Inundation by the design rainfall

The actual rainfall observed at Yao in 1957 shown in Fig.7, which is the design rainfall of the flood control plan in the Neya River basin, is used here. The maximum hourly precipitation is 63mm/hr and the total precipitation is 311mm. At the downstream end of the river network, the mean high tidal level of the Osaka Bay is constantly given as the boundary condition. The computational time step Δt is the same as the previous case and the total computational time is 40hr.

The discharge hydrographs of the Oto River, Gongen River, Kiyotaki River and Sanra River obtained from the runoff analyses are shown in Fig.7.

The calculated discharge hydrograph at Kyoubashi-guchi, the reference point of the Neya River basin, and the calculated flow discharge and direction of the river network when the maxi-

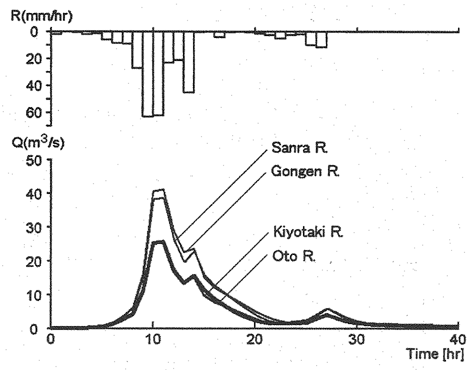


Fig.7 Rainfall and runoff discharge from the mountainous area

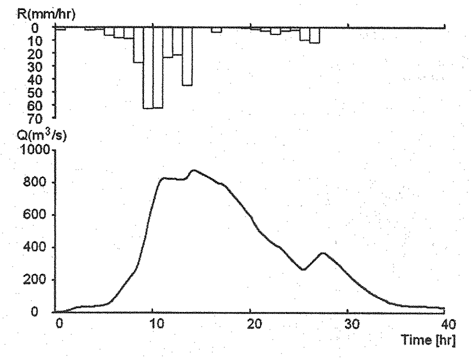


Fig.8 Discharge hydrograph at Kyoubashi-guchi

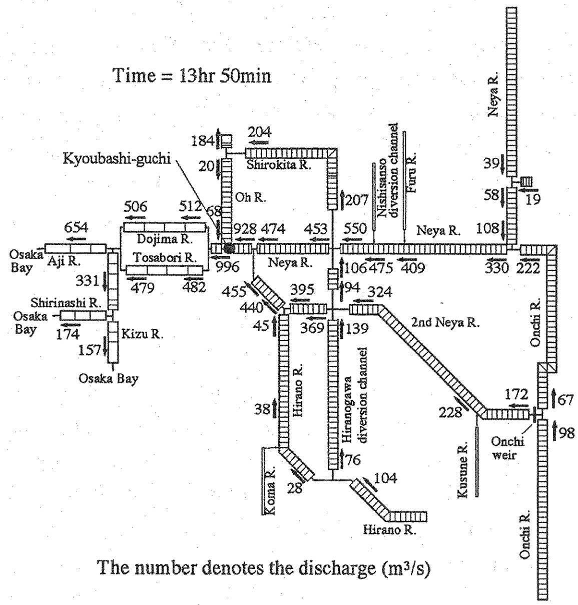


Fig.9 Flow discharge and direction in the river network

mum discharge at Kyoubashi-guchi appears, are shown in Fig.8 and Fig.9, respectively. The maximum discharge at Kyoubashi-guchi is $928 m^3/s$, which exceeds design flood discharge $850 m^3/s$.

Fig.10 and Fig.11 show the temporal change of inundation water depth and maximum water depth, respectively. From these figures, it is seen that the inundation water left on the drainage basin is flowing toward lower areas. Especially along the rivers, such as 2nd Neya River, Onchi River and Hiranogawa diversion channel, the inundation water depth becomes larger due to blocking by river bankings. In these figures, the effects of railway bankings and overtopping from Shirokita River are illustrated.

Fig.12 shows temporal change of water volume drained through sewerage system, stored

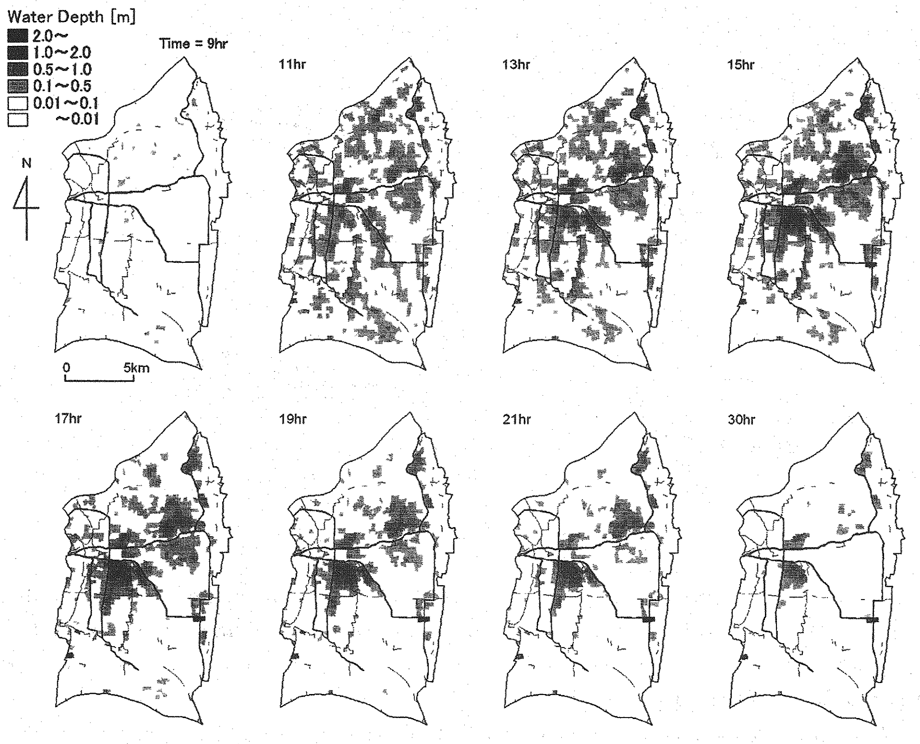


Fig.10 Temporal change of inundation water depth distribution

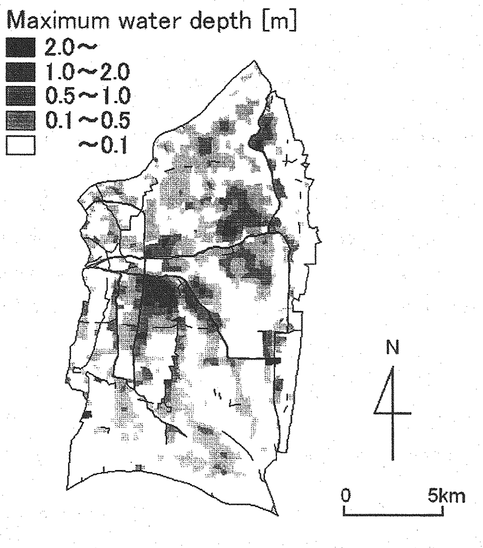


Fig.11 Maximum water depth

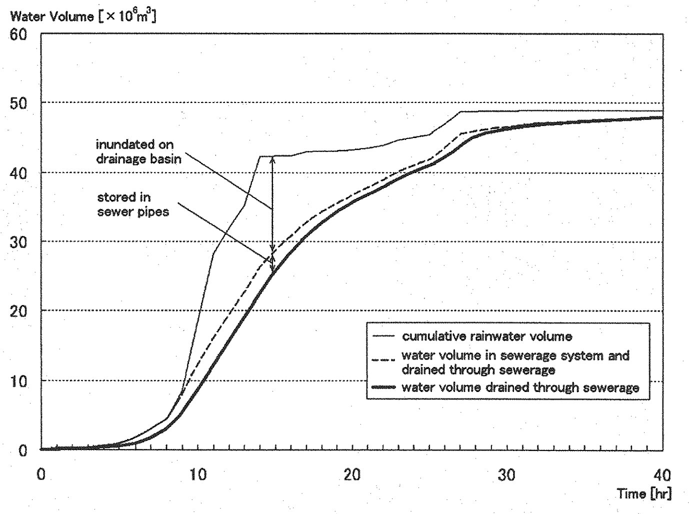


Fig.12 Temporal change of water volume

in sewer pipes and inundated on the drainage basin. From this figure, after Time=10hr, rainfall intensity exceeds drainage capacity of the sewerage system and inundation water volume is increasing rapidly. Thus the water volume drained through the sewerage system gradually approaches the cumulative rainwater volume.

Therefore, from the computational results, the dangerous area for heavy rainfall or where drainage capacity should be strengthened can be designated.

CONCLUSIONS

The inundation flow model, which can treat inundation by heavy rainfall in urban area, was developed and applied to the Neya River basin, Japan. This model can express the inundation process in the low-lying river basin appropriately. The point to be improved is how to determine the parameters used for the sewerage system, for example, but this model will enable us to obtain some important information to estimate areas vulnerable to the heavy rainfall, and to establish countermeasures against the flood disasters due to heavy rainfall.

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APPENDIX - NOTATION

The following symbols are used in this paper:

A	=	cross sectional area;
A_p	=	cross sectional area of sewer pipe;
B	=	river width;
B'	=	height of pipe ceiling;
B_s	=	slot width;
g	=	gravitational acceleration;
H	=	water level;
h	=	flow depth;
h_1	=	overflow depth;
m	=	numerical constant of the kinematic wave model;
$M,\ N$	=	respective discharge per unit width in x and y directions;
$M^*,\ N^*$	=	respective corrected discharge per unit width in x and y directions;
N_e	=	equivalent roughness;
n	=	Manning's roughness coefficient;
Q	=	flow discharge;
Q_m	=	maximum discharge capacity from drainage basin mesh m;
q	=	water discharge per unit width in the longitudinal direction of slopes or tributaries;
q'	=	discharge per unit width on the slope;
q_0	=	overtopping discharge into the drainage basin per unit width of $x-$direction;
q_m	=	lateral inflow per unit width of $x-$direction;
q_{out}	=	drainage discharge per unit area from computational mesh into sewerage system;
q_{over}	=	overtopping flow discharge from the river network;
q_s	=	lateral inflow discharge per unit length from side slopes;
r_e	=	effective rainfall intensity;
t	=	time;
$u,\ v$	=	respective flow velocity in the x and y directions;
$x,\ y$	=	coordinates of the flow;
z	=	elevation of sewer pipe bottom;
α	=	numerical constant of the kinematic wave model;
Δt	=	computational time step;
θ	=	river bed slope;
θ_s	=	slope gradient; and
λ	=	occupying ratio.

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