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HYDRAULIC COMPUTATIONS FOR FLOOD ROUTING OF A MODULAR URBAN STORMWATER DRAINAGE SYSTEM

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ABSTRACT

This paper discusses the hydraulic analyses carried out to evaluate flooding due to overbanking of stretches of a 4-meter-wide drainage canal through an existing subdivision in Metro Manila. The proposed land development will convert the upstream portion of the property into a mixed-use commercial-residential area. Computations are carried out to determine the spatial extent of the present flooding potential, and to quantify the upstream-migrated inundated areas under the present condition of a submerged exit. Hydraulic analyses are also undertaken to solve the recurring problem of a submerged culvert outfall. The study also provides the quantitative bases for the structural measures to mitigate the flooding hazards associated with the land use modifications. It is concluded that hydraulic computations are necessary in predicting local flooding conditions, as well as in analyzing the effects of proposed engineering interventions to mitigate this condition. The methodology is demonstrated via project application involving an urban stormwater drainage system.
Key words : flood routing, urban drainage, stormwater, hydraulic analysis

1. INTRODUCTION

The recent rapid trends in urban housing developments have placed renewed concern on the capability of engineered drainage systems to reliably discharge floods. In particular, for a modularized stormwater drainage development designed to carry a runoff discharge proportional only to the area being developed, it is important to review the existing system to ensure that new flows from additional catchments, as well as amplified flows due to modifications in land use of old catchments, can still be safely carried. The conditions of the outlet must also be re-assessed, since vertical siting conditions of the outfall may have to be adjusted to accommodate the additional loads.

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This paper focuses on the studies undertaken to investigate the hazards posed by the flooding of an existing drainage canal system on a proposed property development at the upstream area. The study specifically addresses the issue of whether the hazards posed by the observed downstream flooding will migrate or extend to the proposed development and whether engineering interventions are feasible and economical to address these potential problems.

In many cases where the interior drainage of a small urban catchment, such as a subdivision, is connected to a larger system such as a municipal drainage, problems of local flooding often arise. Since most surface flows of mildly sloping channels are subcritical even during periods of high flows, the conditions downstream influence the upstream flow regime. A typical situation is the inundation of the exit area of a connecting channel due to choke points in the receiving larger system (Guo, 2000). In such situations, deliberate correction of the downstream conditions is often not possible on account of legal issues.

This pre-engineering study was commissioned to focus on the flooding hazard aspects of the entire development project, which also includes water sourcing study. The specific objectives of this study are: (1) to identify the cause of the existing flooding; (2) to determine the spatial extent of the consequent inundation; (3) to investigate if the proposed upstream development will pose site drainage problems; and (4) to conceptualize and assess engineering measures to mitigate, if not eliminate, the persistent downstream flooding. Although this paper focuses on the last two project objectives, the resulting conclusions from the first two are also discussed to provide the substantive bases of the conduct of the flood routing studies.

2. MODULAR URBAN DRAINAGE STUDY – A PROJECT OVERVIEW

The area of interest is a block of undeveloped property having an area of 5.7 hectares located in southern Metro Manila. The existing development, called Phase I, includes residential lots with paved roads and sidewalks, stormwater drainage network, and other urban features. The proposed development, or Phase II, will have additional paved roads, drainage facilities, clubhouse, swimming pool, basketball courts, and commercial spaces with sizeable parking areas. As part of the overall objectives of the pre-engineering studies of the project, this component addresses the drainage characteristics of the project site.

After three extensive site inspections, it was observed that the head section of the existing drainage canal of the present development is a sag area of the adjacent villages and the surrounding public roads. The existing ground terrain and storm drainage network of a large subdivision upstream causes significant overland flow and surface runoff to collect at the sag area. The collected water flows into pipe culverts, then to the head section of the drainage canal under study. From interviews of residents, this sag area experiences knee-deep floods for about 20 minutes during heavy downpours. Although this area is legally outside the property limits, it was necessary to provide drainage structures such as road inlet, collecting basin, wall weirs, and reinforced-concrete pipe culvert to alleviate this local flooding problem. Moreover, at the upstream area of Phase II, the existing ground terrain is susceptible to depression storage as judged from the numerous ponded areas that appear even during moderate rainfall events.

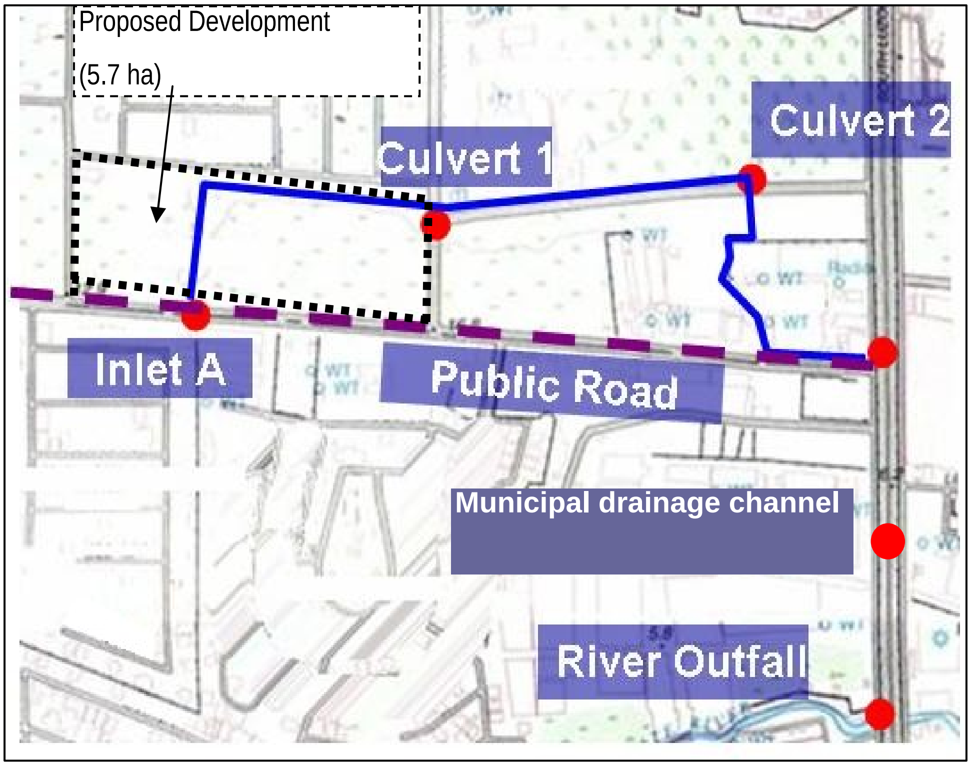

Fig. 1 Schematic of a modular drainage system

3. FLOODING ISSUES

Site drainage of Phase I is via a rectangular open canal that runs from Inlet A, which is a sag area of the existing municipal drainage system, to the local outlet downstream (see Figures 1, 2a). The canal is 6-meter wide at its northern reach, then narrows to 4 meters at upstream Culvert 1. The side walls have different heights depending on the elevation of the adjacent ground, but are generally 2.5- to 3-meter high along most length. The canal walls are of riprap type at the high portions and plastered concrete hollow block (CHB) type in the lower walls. Surface runoff from the undeveloped lots leads directly to the canal, while impervious roadway runoff from the road network is collected by a gutter-inlet-collector pipe system that eventually discharges into a 4-meter-wide drainage canal. The canal crosses two culverts along its entire stretch. The first is a 4-meter wide upstream RCBC, Culvert 1 (Figures 1, 2b and 2c), which is under a roadway rotunda of Phase I and is generally aligned with the canal. The second is another 4-meter wide RCBC, Culvert 2, crossing the main entrance road of the subdivision (Figures 1 and 2e). The drainage canal turns almost 90 degrees into Culvert 2 (Figure 1e).

The outlet of the canal connects to the municipal drainage system (Figure 2f) about 300 meters downstream from the exit of Culvert 2. Due to persistent inundation of the exit of Culvert 2, it was anticipated that the conversion of the idle downstream areas may exacerbate the problem and pose flooding hazards to the upstream Phase II. Part of the objectives of the study is to investigate if the existing canal system will be able to provide adequate drainage for the fully developed Phase II without further engineering interventions. Of paramount concern is the inundation of the large and relatively flat area around Culvert 2. As the area is perennially under water, it has become a grazing area for animals such as deer and goats.

Inundation of Culvert 2 area is caused by channel flow stagnation due to choke points downstream of Culvert 2. The major choking area is a large tree blocking the canal reach fronting a factory close to the municipal drainage outlet. Although less consequentially, the skewed alignment of the canal reach downstream of Culvert 2, plant outgrowths, and possibly silting of the canal, also contribute to the stagnation of the flow.

(a)

(b)

(c)

(d)

(e)

(f)

Fig. 2 Site photos: (a) Head water is received at Inlet A from a culvert under the city road. (b) Entrance section of Culvert 1. (c) Exit section of Culvert 1. (d) Drainage canal between Culverts 1 and 2. (e) Entrance section of Culvert 2. (f) Downstream exit to city drainage system

4. CATCHMENT CHARACTERIZATION

The project watershed, shown in Figure 3, was manually delineated from NAMRIA topographic maps, with defined sub-catchment drainage channels additionally identified using information from the site inspections. The watershed has an area of roughly 117 hectares, with the highest point on the boundary at elevation 46.17 meters above datum. The outlet is at elevation 39 meters. The length of the main stream (canal) is 760 meters. The weighted slope of overland flow to the canal is 1.7 percent and that of the main stream is 0.7 percent. Land use is urban, primarily residential, partly commercial with pockets of undeveloped land, while land cover is mostly impervious and partly grassy field with trees. There are no natural streams within the watershed so that all surface runoff eventually lead to the open drainage channel.

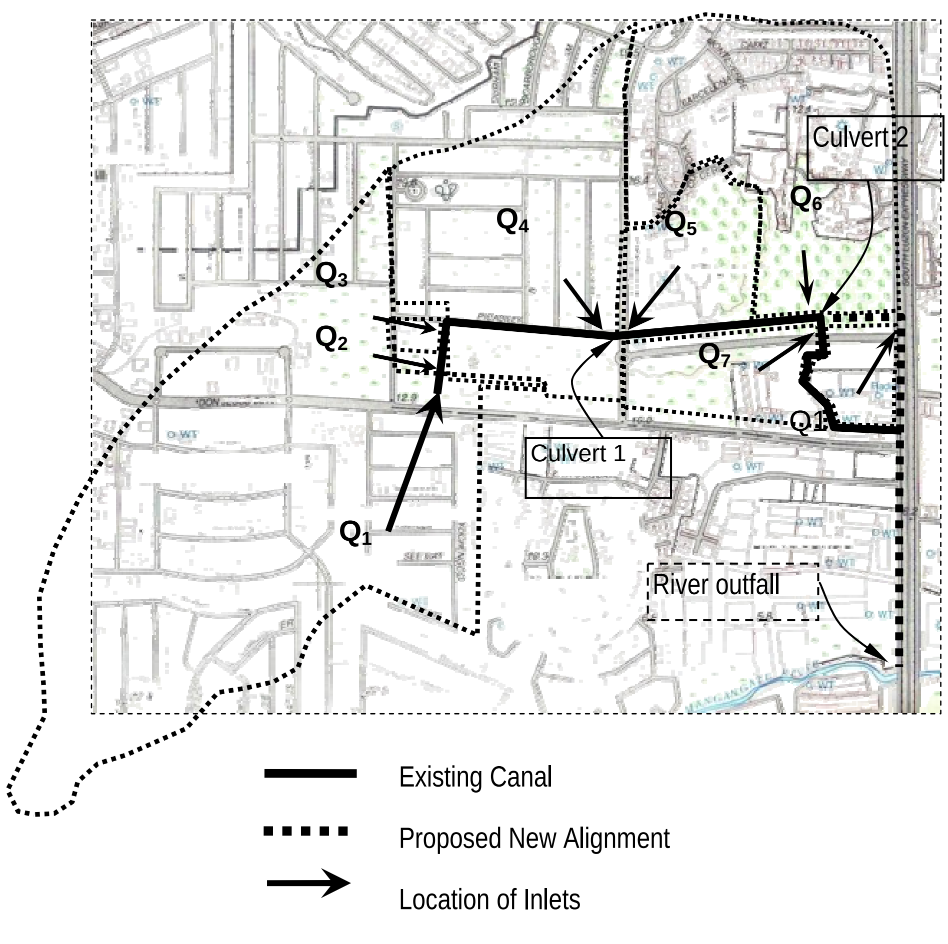

Fig. 3 Sub-catchment delineation, discharge outlets, and canal alignments

The time for rainfall to reach the watershed outlet, or time of concentration, was estimated from a number of prognostic formulas used in watershed models. Based on the watershed characteristics and rainfall intensities of the nearest PAGASA rain gage station, the results for the applicable formulas are summarized in Table 1.

Table 1 Times of concentration (sec) for project watershed

IZZARD	10.6*
KERBY	14.9
KINEMATIC WAVE	19.1
SCS LAG	39.6
TC AND R	16.8

Except for the SCS Lag, the formulas yield values in the range of 15 to 20 minutes. Izzard's formula yielded the lowest value, but was not further considered since the resulting value violates the assumption of the model (Veismann et al., 1989). In view of these and the rainfall intensity curve of the rain gage station, the time of concentration for the project watershed was taken as 14.9 minutes based on Kerby's formula.

5. HYDROLOGY AND SITE DRAINAGE CHARACTERISTICS

The rainfall event with return period of 50 years was chosen as the input to the flood routing analyses. Rainfall intensity-duration-frequency curves, synthesized from the data of PAGASA, were obtained from available publication (DPWH-JICA, 2003). Rainfall intensity triangulation was also carried out using two other proximate stations, applying weighting factors of ($1/d$) and ($1/d^2$), where d is the distance of the watershed centroid from the stations. The highest of the rainfall intensity at the nearest station and the two weighted intensities for the same duration is taken as the design rainfall. For a storm duration equal to the time of concentration, the flood routing analyses were carried out using a design rainfall intensity of 248 mm/hr.

The peak flows Q_p (m^3/s) resulting from runoff on overland surfaces were estimated from the Rational formula:

$$Q_p = 0.00278kiA_c \qquad (1)$$

where k is the overland runoff coefficient, i the design rainfall intensity (mm/hr), and A_c the catchment area (ha). The runoff coefficient depends primarily on the land use and secondarily on soil type and slope for pervious surfaces. The value of k for each sub-catchment was taken from published hydraulic tables (Bedient and Huber, 1995). The peak runoff from impervious roadways is computed using Izzard's formula (Linsley et al., 1992):

$$Q_p = iw\frac{\sqrt{r^2+1}}{r}L_p \qquad (2)$$

where w is the pavement width including the road gutter, r the ratio of cross (transverse) slope to longitudinal slope, and L_p the length of roadway. The pavement widths were based on the site inspections while the slopes were based on site topographic maps. The resulting peak flows are shown in Table 2.

Table 2 Peak flow rates from sub-catchments for existing channel alignment

Peak flow	Magnitude (cms)	Description
Q1	34.55	Adjacent subdivision + roads
Q2	0.49	Sub-catchment Phase II
Q3	0.65	Sub-catchment Phase II
Q4	13.49	Culv.1 entrance (Phase I + roads)
Q5	2.36	Culv.2 exit (overland+ roads)
Q6	4.56	Culv.2 entrance (overland)
Q7	12.40	Culv.2 exit (overland + roads)

6. FLOOD ROUTING ANALYSIS

The water surface elevations in open channels are computed based on the energy equation for free surface flows:

$$y_1 + z_1 + \alpha_1 \frac{V_1^2}{2g} = y_2 + z_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \tag{3}$$

where y_1, y_2 are the water depths at upstream and downstream sections 1 and 2 respectively, z_1, z_2 the invert elevations of the channel, V_1, V_2 the average velocities, α_1, α_2 the velocity weighting functions, and g the gravity acceleration. h_L is the head loss consisting of friction losses and contraction or expansion losses:

$$h_L = LS_f + C\left[\alpha_1 \frac{V_1^2}{2g} - \alpha_2 \frac{V_2^2}{2g}\right] \tag{4}$$

where L is the distance between the sections, S_f the weighted friction slope, and C the contraction or expansion loss coefficient. The friction slope is based on Manning's resistance equation for the channel (or conduit):

$$S_f = \left(\frac{nQ}{AR^{2/3}}\right)^2 \tag{5}$$

where Q is the discharge, n the roughness coefficient, A the flow cross-section area, and R the hydraulic radius. At sections where a free surface cannot exist, such as fully flowing culverts, the water surface elevations at the exit and entrance sections of the conduits are determined based on the adjacent free-surface sections, the geometric and hydraulic properties of the culvert, and the discharge through the conduit, which are related by pressure-conduit hydraulic theory.

Flood routing computations were implemented via HEC-RAS (USACE, 1998). Computationally, the water surface elevations at all channel stations are determined using the energy equation with the following input data (Table 3):

Table 3 Description of input data to HEC-RAS

Cross-section	coordinates and elevations of the invert points, reach lengths along the channel edges
Flow data	flood flows into the channel stations
Channel hydraulic properties	roughness parameters, contraction and expansion loss coefficients
Culverts	geometry and size of barrel, inlet and exit hydraulic characteristics, roughness coefficient, road deck elevations, no-flow areas if applicable
Boundary conditions	specified water surface elevations and/or friction slopes

Figure 4 shows the resulting flood profiles for the existing canal with various exit conditions of Culvert 2. Cases A to C correspond to submerged exit conditions, where the exit section is submerged to various depths. Case A corresponds to the observed maximum ponded depth of 1.55 meters (above the culvert's top surface) and Case B to 1.0 meter. Case C corresponds to a submerged but non-ponded exit. Case D represents a free exit with half-full Culvert 2 exit. The results in cases A and B show that the area between Inlet A and Culvert 1 entrance will not be overtopped even when the exit of Culvert 2 is ponded to its potential depth of 1.55 meters. However, the area immediately upstream of Culvert 2 will be almost bankful even if the exit is not ponded (case C). The results for case D reveal that the local flooding is caused by the submergence of Culvert 2 exit.

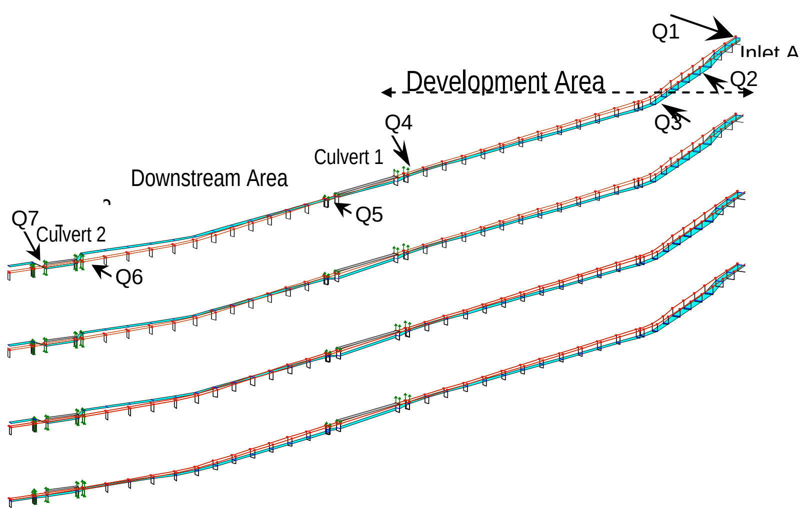

Fig. 4 Flood water surface profiles for existing channel alignment

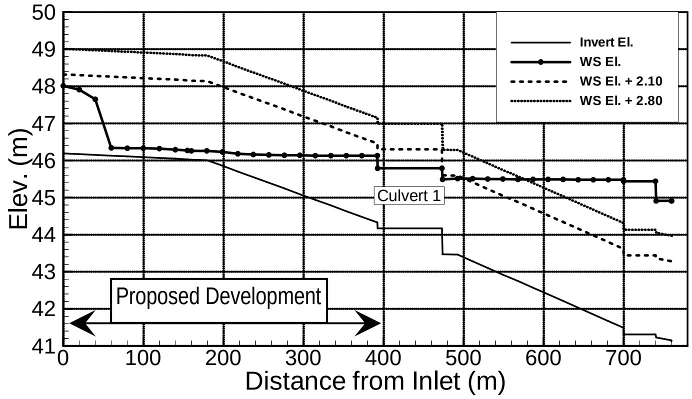

Fig. 5 site grading implications

7. IMPLICATIONS ON PROPOSED PROPERTY DEVELOPMENT

Based on the flood routing results, a number of measures were proposed to eliminate the potential flooding. The first is the provision of a high-enough ground level for all new constructions in the area upstream of Culvert 1. The minimum height of canal with sufficient freeboard is 2.80 m above the highest elevation of the water surface (Figure 5). This elevation can be used to determine the depth of backfill to needed to raise the final ground elevation.

Another measure is the improvement in hydraulic design of Inlet A so that the peak flow Q_1 is substantially reduced. The existing Inlet A is found to be inadequate in capacity, causing a concentrated inflow to be funneled from the surface runoff from the upstream sub-catchments at the head section (see Figure 3) of the existing drainage canal. A wide and comparatively shallow inlet section would help to distribute the flow and reduce the headwater depth.

The results of the study also suggest the use of scour-resistant channel lining in the structural design of the upstream canal to mitigate flow-induced scour in the canal bed and banks. The simulations indicate that supercritical flow regime would obtain in the upper reaches of the canal, which would need scour-resistant lining material for channel stability.

Recommended non-structural measures include the clearing of the entrance section of Culvert 1 (Figure 2b) to avoid clogging, say, by floating logs or debris. Another is the clearing of flow-impeding outgrowths in the canal to optimize its conveyance characteristics and minimize flow resistance. The resulting lower water surfaces from such improvements are revealed by separate flood routing simulations with improved roughness parameter values.

8. ENGINEERING INTERVENTIONS

Since the flood flow simulations show that a ponded condition of Culvert 2 is the principal cause of the upstream migration of the flooded reaches of the existing channel, an improvement of the conveyance efficiency of the channel's downstream segments could eliminate or reduce the flooding in Phase II. However, actual site conditions (Figure 6), e.g. obstructing fence walls of adjoining properties, significant outgrowths along the channel banks and existence of fully-grown trees in the main canal section outside the property boundaries of Phase II, implied that there was limit to how this measure could mitigate the ponding at Culvert 2.

(a)

(b)

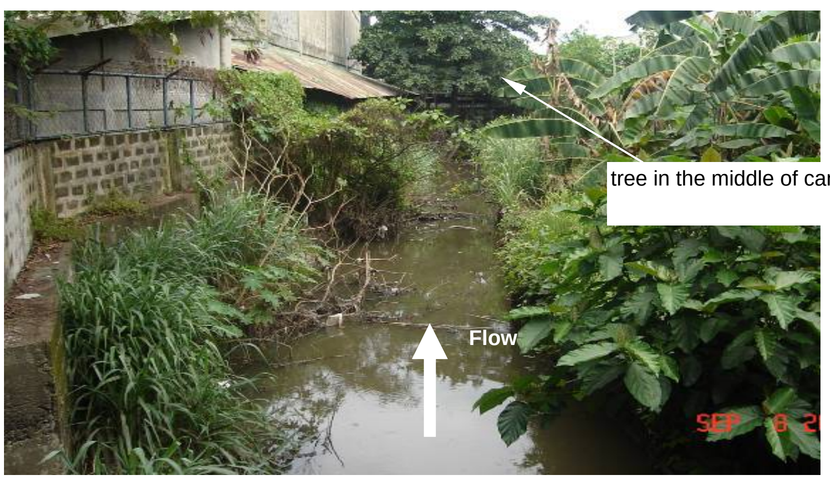

Fig. 6 Exit section of downstream Culvert 2; (b) channel downstream of Culvert 2 leading to municipal drainage outfall (see Fig. 2f)

To alleviate the potential flooding of the entrance area of Culvert 2, the realignment of the existing canal was examined. This calls for extending the 4-meter wide canal linearly to the national highway, maintaining its one-percent invert slope, and building from there a new subsurface culvert (conduit) to carry the flow onto a river about 650 meters downstream (Figures 1 and 3).

Three schemes of channel re-alignment were considered to mitigate the predicted downstream flooding for the fully developed Phase I, in which the peak flow rates Q_6 and Q_7 are now increased due to the land use modifications. In addition, a new flow Q_8 (see Figure 3) is now imposed on the realigned channel. Scheme A involves constructing a new 2-meter wide box culvert with the river outfall located at the existing drainage outfall (Figure 7) of the national road.

(a)

(b)

Fig. 7 Conditions at the river outfall of municipal drainage system: (a) looking downstream; (b) view upstream

The resulting flood profile, shown in Figure 8(a), indicates that a culvert height of 1.3 meters is sufficient to carry the design flood to the river outfall with enough freeboard. Since the existing low river outfall (see Figure 7b) will likely be submerged during a flood event, its invert was raised by 1 meter in Scheme B. Figure 8(b) shows that a culvert at nominally higher invert elevation, resulting in milder invert slope of the re-aligned channel, will be needed to realize this without choking the channel. In these two realignment schemes, the canal reach between Culvert 1 and the head section of the realigned reach will experience almost identical flood stages.

Scheme C is where the realigned channel is narrower (one-meter wide) to satisfy potentially restrictive right-of-way requirements. Flood routing results shown in Figure 8(c) indicate that a significantly deeper culvert is necessary to carry the flood to the outfall. In addition, the narrow culvert will aggravate the flooding of the previous Culvert 2 area. Finally, if excavation costs would be prohibitive, this scheme would require a culvert profile that may extend above the existing ground.

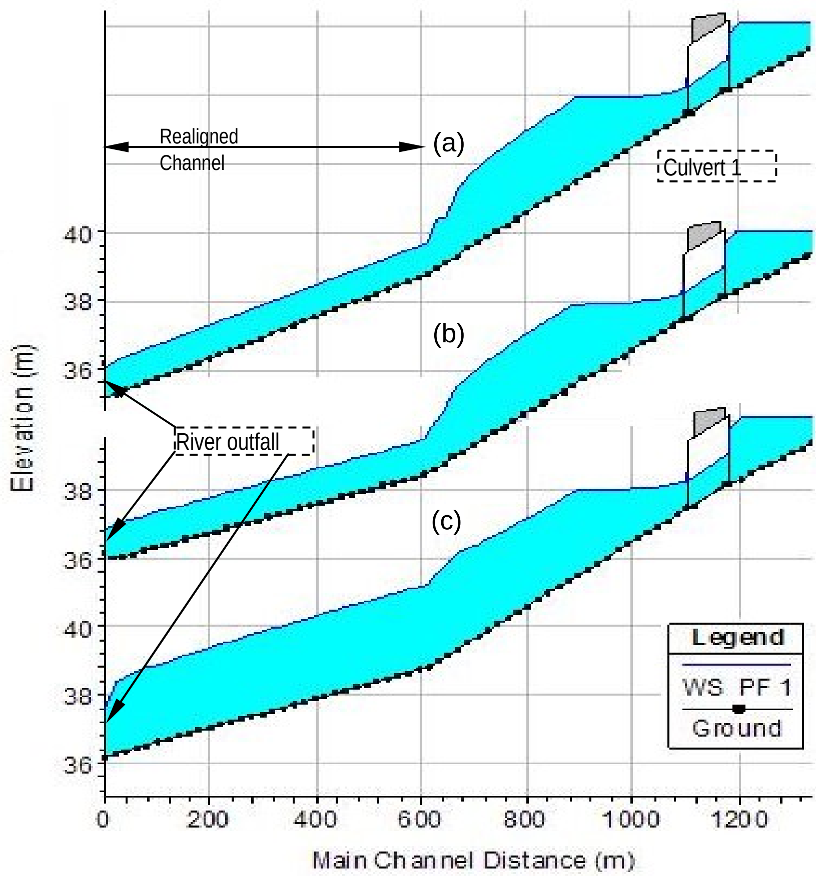

Fig. 8 Flood flow profiles for 3 cases of realigned canal: (a) 2-meter wide culvert to river outfall; (b) same as in (a) except outfall is higher by a meter; and (c) same as (b) except with narrower culver.

9. CONCLUSIONS

The flood routing studies undertaken for the proposed land development in an urban catchment provided quantitative information on the cause of local flooding in the existing drainage canal. Flood routing computations indicate that correcting the conditions of the exit section of the downstream culvert can reduce the potential flood stages. Where deliberate engineering interventions are considered, realignment of the channel in the flood-causative reach may prove effective. In this case, flood routing computations are essential not only in evaluating the effectiveness of various alignment schemes but in quantifying the resulting structure sizes as well. The flood profile computations also indicate the effectiveness of various geometric alternatives as required by, say, right-of-way constraints. These alternatives include adopting narrow but deep channels and and/or changing the elevations of outfalls. Using a wider and relatively shallower section may improve the hydraulic efficiency of the realigned channel and lower the total cost of the engineering works, but may present constructability limitations on account of right-of-way considerations.

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