

Keynote lecture:

Urban flood control in Hong Kong — challenges and solutions

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Abstract: Over the past four decades, changing land use patterns have led to the reduction in upland flood storage capacity and increased lowland runoffs in many parts of Hong Kong. A variety of measures have been developed to protect semi-rural areas, new towns, and metropolitan areas from flooding – including real time warning systems, village polder schemes, river training, stormwater diversion and storage schemes. Many of these flood control schemes share certain characteristics: intense storms and high inflows, tight space constraints and scarce land availability, proximity to densely populated areas and congested underground utilities, and enhanced backwater effect due to coastal reclamation. The design of these flood control schemes often involve complex three-dimensional flows: complicated subcritical-supercritical transitions, free surface and surcharged flows, and spatially varied flow at 3D junctions. Such designs can be effectively explored and studied in undistorted physical models supplemented by mathematical model calculations. In this paper, the characteristic hydraulic challenges and solutions of these schemes are illustrated by reference to the Tai Hang Tung Storage Scheme and the Kai Tak Transfer Scheme. These two projects, which are in an advanced stage of construction, constitute the essential components of the West Kowloon Drainage Improvement Scheme.

Keywords: urban flooding, stormwater management, flood storage, flow diversion, underground storage tank, side weir, hydraulic transitions, drainage tunnel, interception structure, supercritical spatially varied flow

1. INTRODUCTION

The long term average annual rainfall in Hong Kong is 2,200 mm; about 80 percent of the rain falls between May and September. The wettest month is August, with an average monthly rainfall of 391 mm. The topography of drainage basins consists typically of steep upland terrain and relatively flat valley floor in the downstream semi-rural or densely populated “new towns” or metropolitan/urban areas. Over the past forty years, changing land use patterns (in particular the conversion of farm land to fish ponds and increasing urban areas) have resulted in the loss of flood plain storage and increased runoff. For example, agriculture land (cultivated and fallow) have decreased from 2830 ha in 1963 to 925 ha around 1990; on the other hand, the fish pond area increased from 665 ha to 990 ha, and developed urban area from 460 to 1465 ha respectively (Mott Macdonald 1990). In addition, the 1990s witnessed several unusually warm years, which seems to be correlated with record high rainfalls. For example, 1997 was the fourth warmest year on record, with an average temperature of 23.4 °C; an extremely high annual rainfall of 3,343 mm was recorded, with extensive flooding in West Kowloon. The rainfall also exhibited significant spatial and temporal variability. In 2001 (the fourth wettest year), the annual

rainfall was 3,091 mm; 1,368 mm of rain (62 percent) fell on the Northern New Territories in June 2001 alone, causing wide spread flooding.

A variety of measures have been developed to protect semi-rural areas, new towns, and metropolitan areas from flooding – including real time warning systems, village polder schemes, river training, stormwater diversion and storage schemes. Many of these flood control schemes share certain characteristics: high intense storms and inflows, the need to design conveyance systems under tight space constraints, proximity to densely populated areas and congested underground utilities, and enhanced backwater effect due to coastal reclamation. The design of these flood control schemes often involve highly complex three-dimensional flows: complicated subcritical-supercritical transitions, free surface, surcharged and two-phase flows, and spatially varied flow at 3D junctions. Such designs can be most effectively explored and studied in undistorted physical models supplemented by mathematical model calculations. Over the past 15 years, we have undertaken a number of significant modeling studies that include: the River Indus Model Study (1987); the Tai Lam Chung Side Channel Spillway (1994); the Yuen Long Bypass Floodway (1998), the Tai Hang Tung Storage Scheme (THTSS) and the Kai Tak Transfer Scheme (KTTS; 2000-01). In this paper, the characteristic hydraulic challenges and solutions are illustrated by reference to the hydraulic design of the THTSS and the KTTS schemes (Fig.1). These two schemes constitute the West Kowloon Drainage Improvement -- Stage 2 Project that are being implemented to solve the flooding problem in MongKok - a major residential and commercial district of the Hong Kong Special Administration Region of China.

2. TAI HANG TUNG STORAGE SCHEME

Fig. 2a shows a schematic outline of the critical reach of the stormwater drainage system. Under existing conditions, the upstream stormwater runoff from Tai Hang Road and Tat Chee Road are conveyed in underground storm drains; the combined flow passes through a steep culvert downstream to join with the Boundary Street Nullah in an open conduit. The total flow ($Q_{\max}=110 \text{ m}^3/\text{s}$ for a 1 in 50 year design storm) is discharged into a steep 1.5 km long culvert under Nullah Road leading to the Hong Kong Harbour. Due to rapid urbanization, the existing drainage system (designed for 1 in 10 year storm) failed to cope with severe rainstorms in recent years; there are a number of flooding “black spots” in Mongkok (e.g. Location A).

Rather than digging up roads and widening/adding existing sewers, which would be highly disruptive to the busy traffic and interfere with numerous existing underground structures and utilities, an upstream storage scheme is devised. The economic drainage improvement plan calls for the construction of a 100,000 m^3 underground storage underneath the Tai Hang Tung Recreation Ground. Part of the flood flow will be intercepted by a system of side weirs. The stormwater that overflows into the underground tank will be temporarily stored; the flow diversion serves to attenuate the flood peaks and prevent downstream flooding. The success of the scheme depends on the proper functioning of the side weir system and the hydraulics at the channel/weir junctions. For example, too early a spill into the storage tank may use up needed storage capacity for a later storm peak.

Due to the complicated turbulent flows in the stormwater drainage system (including subcritical and supercritical open channel flows, pressurized flow in closed conduit, side weir overflows, and transitional flows at culvert junctions), a 1:22 undistorted Froude scale physical model of the THTSS system was built (Lee *et al* 2000; Henderson 1969; Novak 1981). Experiments were conducted to: a) study and optimize the design of the side weir system and the inlet and hydraulic transition; and b) provide the side weir

head-discharge characteristics for a one-dimensional numerical unsteady flow model developed to study the flow within the side weir system. The inflow hydrographs are obtained from catchment-wide drainage network models. River/drainage network models can be used to screen design alternatives and study catchment response provided the local flow behaviour at critical junctions or nodes is known. However, these 1D models by themselves are incapable of giving workable hydraulic designs. In addition, the side weir discharge characteristics for such drainage systems has previously not been investigated. The sensitivity of the system response to weir level, as well as the use of the storage tank to intercept part of the backflow (against steep gradient) from the Boundary Street Nullah also need to be addressed.

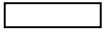
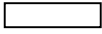

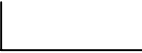
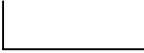
2.1 Hydraulic Scale Model Design And Experiments

2.1.1 Design Parameters of the THTSS System

Fig.2 and Table 1 show the basic layout and parameters of the THTSS model: i) the Tai Hang Tung road culvert (8 m x 2.5 m), with a maximum inflow of $Q_{\max} = 46.5 \text{ m}^3/\text{s}$; ii) the Tat Chee Road culvert with $Q_{\max} = 11 \text{ m}^3/\text{s}$, and iii) the Boundary Street Nullah (7 m x 2.6 m) with $Q_{\max} = 48.7 \text{ m}^3/\text{s}$ (BBV 1999). Table 1 shows the typical normal and critical depths in the system

In the storage scheme, overflow weirs must be designed to allow the safe spill of the large stormwater flows into the underground tank when the water level in the THT culvert exceeds the weir level. The heart of the THTSS scheme lies in the weir design, and the hydraulic transition linking: i) the THT upstream to the weir system; and ii) thereon to the Tat Chee Road inlet and the downstream Tai Hang Tung system. A number of weir layouts and transitions designs have been studied and discarded; various considerations led to a scheme with three side weirs each of 8 m width and 30 m length (Fig.2): a flow through weir (W1) with invert level at 4.5 m PD; and two dead end weirs (W2 and W3) with a slight adverse bottom slope; the weir level is set at 5.5 m PD. The concept is to intercept most if not all of the THT overflows in the weirs W1 and W2, and to intercept the Tat Chee Road inflows as well as part of the Boundary Street nullah backflows into weir W3. i.e. the design minimizes the interaction between the stormwater flows from either side of the weir system; any undesirable hydraulic behaviour can hence be minimized.

Table 1 Design parameters of the THTSS stormwater culverts

Parameter Channel	Design flow Q_{\max} (m^3/s)	Bed slope S_0	Normal depth y_n (m)	Critical depth y_c (m)	Froude number V/\sqrt{gy}	Section 
Tai Hang Tung Culvert	46.48	0.0028	1.60	1.51	0.91	 8m(W)×2.5m(H)
Tat Chee Culvert	10.99	0.0050	surcharge	1.14	Surcharge	 2.9m(W)×1.45m(H)
Boundary Street Culvert (upstream)	48.74	0.0027	1.88	1.70	0.87	 7m(W)×2.6m(H)
Boundary Street Nullah (downstream)	38.80	0.0027	1.50	1.39	0.89	 7.6m(W)×3.85m(H)

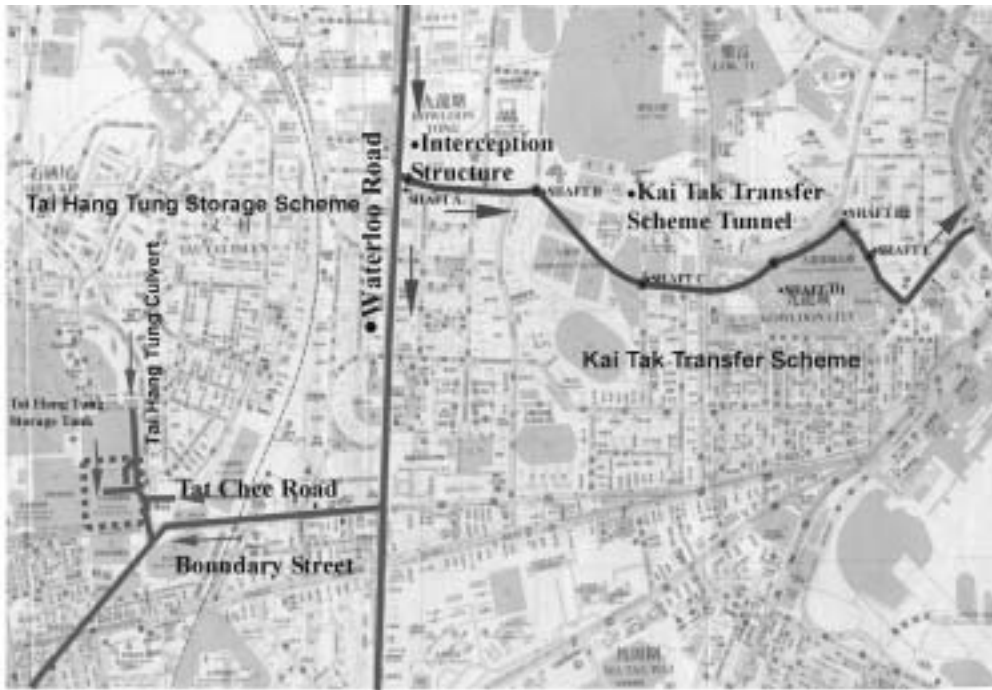


Fig. 1 Tai Hang Tung Storage Scheme and Kai Tak Transfer Scheme for urban flood control

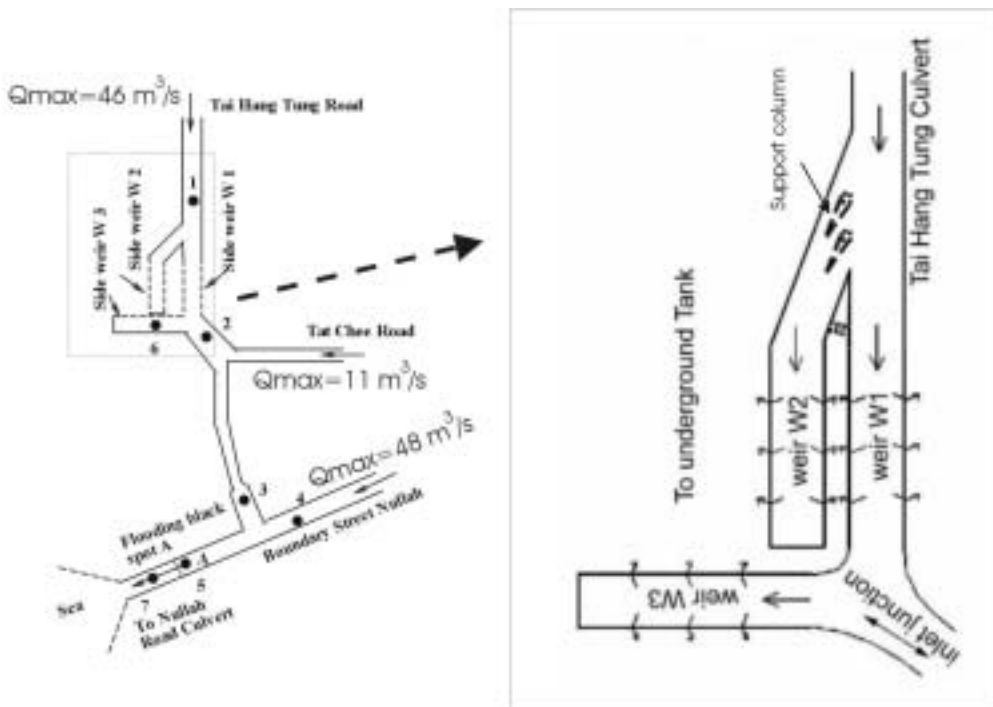


Fig. 2 Schematic diagram of Tai Hang Tung Storage Scheme and triple weir system

Extensive experiments have been performed to study the detailed flow in each side weir channel as well as alternative inlet junction designs. In a typical experiment, a prescribed steady inflow is fed into the inlet ends of Tai Hang Tung culvert, Tat Chee Road culvert, and Boundary Street Road Nullah, and the downstream water level at Boundary Street Nullah adjusted according to preset values. The piezometric head along the side weirs, velocities at key points, and side weir flows are accurately measured

2.1.2 Head-Discharge Characteristics of Side Weirs

The flow in each side weir is a spatially-varied flow. The lateral discharge per unit weir length, q_w , from the side weir (for double side) is related to the local head on the weir $H_w = y - y_w$ via a side-weir discharge coefficient C_M :

$$q_w = 2 \times \frac{2}{3} C_M \sqrt{2g} H_w^{3/2} \quad (1)$$

where y = water depth, and y_w is the level of the weir crest from the bottom of the weir channel. The experimental results (Fig.3) show that the discharge coefficients C_M of weir W1, W2 and W3 are not a constant. C_M decreases with the Froude number $F = V / \sqrt{gy}$ where V is the average velocity in the cross-section. As the flow velocity increases (increasing F), the “spilling efficiency” decreases. The data appear to follow a generalized form of the relation previously established for irrigation lateral weirs (Subramanya 1972):

$$C_M = C_{M0} \sqrt{1 - \frac{3F^2}{F^2 + 2}} \quad (2)$$

where $C_{M0} = 0.611 + 0.075 H_w/y_w$.

2.1.3 Performance of Side-weir System for Different Hydraulic Transitions

Experiments were carried out at low, medium and high flow conditions to study the side weir discharge capacity and the flow pattern of the different junction designs; these are conditions taken at representative instants of the hydrograph, obtained from the HydroWorks model. A variety of options of the inlet design were considered; initial tests quickly narrowed down to three options that involve the use of airfoil shaped piers, steps, and bends placed in the transition to aid the flow. Experiments, however, show that best performance is obtained with no obstruction and the transition shape shown in Fig. 4. With this design, the flow pattern is generally smooth. At low flow, the stormwater from Tat Chee Road flows towards Boundary Street with a stagnation region. At medium flow, the flow from Tat Chee Road impinges on the opposite wall of the junction and is split into two streams, one directed to weir W3 and the other to Boundary Street. At high flow, back flow from Boundary Street (against bottom gradient) joins the flow from Tat Chee Road towards W3. The combined flow is discharged satisfactorily in the downstream Boundary Street Nullah without overflowing the banks (overtop level = 6.7 m PD). The flow visualization confirms the design concept; i.e. the flows from either side of the weir system do not interact strongly.

2.1.4 Effect of Bottom Slope and Support Column

The experimental results reveal marked three-dimensional effects at the inlet sections to W2 and W3 and near the dead ends. The physical model has been invaluable in providing insights to the problem. As an example, for an inflow from the Tai Hang Tung upstream, a distinct hydraulic jump occurs at the entrance to Weir W1 (Fig.5a, left view). Such an unexpected flow behaviour is not desirable, as the system can spill into the side weir prematurely at a lower flow than anticipated. This phenomenon can be traced to the mismatch of specific energy between the Tai Hang Tung Nullah and at the side Weir W1,

and the wavy conditions can be greatly reduced by increasing the bed slope of the Tai Hang Tung culvert from 0.0004 to 0.0028 (Fig.5a, right view). A second important feature is the presence of support columns in the weir channels. In Fig.5b, we show the side weir system with the installation of two major square shaped (850 mm) support columns at the entrance to weir W2. The dramatic flow separation around the square pier (right view) leads to a noticeable depression of the water surface downstream in Weir W1; consequently the flow in W1 will not spill along a significant length of the weir (left view) - a clearly undesirable situation. This led to the use of a streamlined pier with a sharp-edged cutwater as support column, with much improved results (Fig.5c).

2.2 Numerical Model Results

The one-dimensional, unsteady, free-surface flow in open channels is governed by the mass and momentum conservation equations. The numerical model solves the full Saint Venant equations (with the side weir overflows accounted for) using the four-point Preissmann scheme, and is capable of handling both subcritical and supercritical, free surface and surcharged flows. Super-critical and transitional flows are handled through a technique described by Havno and Brorsen (1986), whereby the influence of the convective terms in the momentum equation is proportionally reduced to zero as the flow approaches critical flow ($F = 1$). It is then possible to maintain the double-sweep algorithmic structure for subcritical flow characteristic over the whole domain of solution, including subdomains of supercritical flows. The method is adopted because the magnitude of convective acceleration term in supercritical drainage flows is usually an order of magnitude less than the friction and gravity terms. With this technique, the detailed shape of the hydraulic jumps will not be simulated; however, the jump's general location can be determined by an examination of Froude numbers. The pressurised flow in closed conduits is simulated by the governing equations on the basis of the "Preissmann slot" idea (Preissmann 1961; Cunge and Wegner 1964). The model can also deal with a branched channel system including bifurcations and confluences. Extensive computations against analytical test cases have confirmed the validity of the numerical model. More details can be found elsewhere (Lee *et al* 2001).

Using the experimentally derived side weir discharge coefficients, the model can be used to study side weir behaviour and system performance. Fig.6 shows example comparisons of predicted levels and velocities in side weir W3 (steady flow). Overall, the flow patterns along the side weirs can be simulated, but due to the existence of a hydraulic jump the difference between the computed and measured values along weir W1 is relative large. The numerical model predictions of total side weir flow agree reasonably well with observations (Table 2); for the high flow condition, the predicted flow is in excellent agreement with measurements. However, the predicted total side weir flow is somewhat greater than the measured value for medium inflow; the discrepancy of about 20 percent is acceptable as many three-dimensional details cannot be considered by the 1D model. Note that up to 50-60 percent of the total inflow is intercepted by the side weir system for the medium and high flow conditions. Fig.7a) shows the design inflow hydrographs at the Tai Hang Tung road, Tat Chee Road and Boundary Street for a 1 in 50 year storm. For this unsteady flow computation, it can be seen that without the THTSS, the peak flood level exceeds the road level of 6.7 m PD; with THTSS the flood is contained; the peak level is reduced by about 0.8-0.9 m (Fig.7b). The unsteady model calculations thus confirm and complement the physical model experiments. The model can be used to derive the required capacity of the storage tank as well as optimal weir level settings.

3. KAI TAK TRANSFER SCHEME (KTTS)

The success of the THTSS depends on an upstream diversion scheme – the Kai Tak Transfer Scheme; Fig.1 shows a schematic diagram of the layout of the stormwater transfer route. Under existing conditions, part of the stormwater runoff from upstream is conveyed in a steep box culvert (1:64 bed slope) underneath Waterloo Road, a trunk road in Kowloon. For a one in 50 year flood, a maximum flow of 65 m³/s runs along the Waterloo Road culvert to join with the downstream Boundary Street nullah which flows into the Mongkok area. The Kai Tak Transfer Scheme aims to divert about two-thirds of the design flow from the Waterloo Road culvert (3.1 m wide by 2.9 m deep) into a 4 m diameter underground tunnel (the Kai Tak Transfer Tunnel) by means of an interception structure, thus relieving downstream flood prone areas in MongKok of the stormwater load. The 2 km long tunnel will be constructed in soft ground and runs underneath parks and roads to discharge in the Kai Tak Nullah in East Kowloon. The success of the scheme depends on the ability of the interception structure to divert the desired flow in a hydraulically satisfactory manner for all flows. The basic channel parameters are given in Table 3.

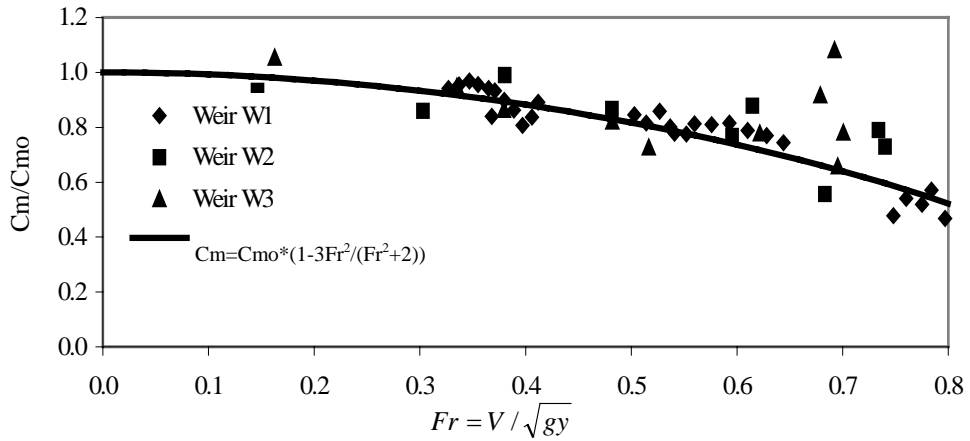


Fig. 3 Measured side weir discharge coefficient for Weir W1, W2, W3 as a function of Froude number

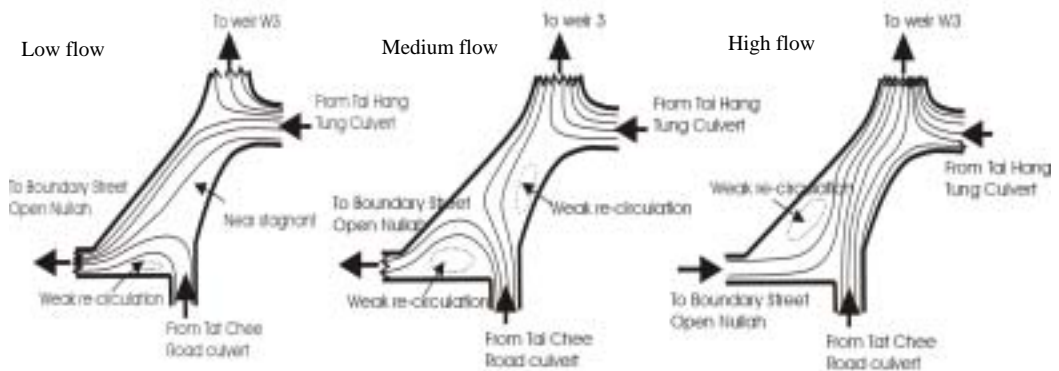


Fig. 4 Observed flow patterns in inlet junction (final design)

Table 2 Summary of measured and predicted weir overflows for triple weir system

Flow rate	Inflow (m ³ /s)			Side weir overflow (m ³ /s)				Predicted (total)
	Tai Hang Tung	Tat Chee Road	Boundary Street	Measured	W1	W2	W3	
Low	7.84	3.08	14.63	1.83	1.09	2.32	5.2	0.98
Medium	26.03	7.98	37.14	10.22	1.79	17.03	28.4	36.95
High	46.48	10.99	48.74	15.44	24.97	23.61	64.1	64.34

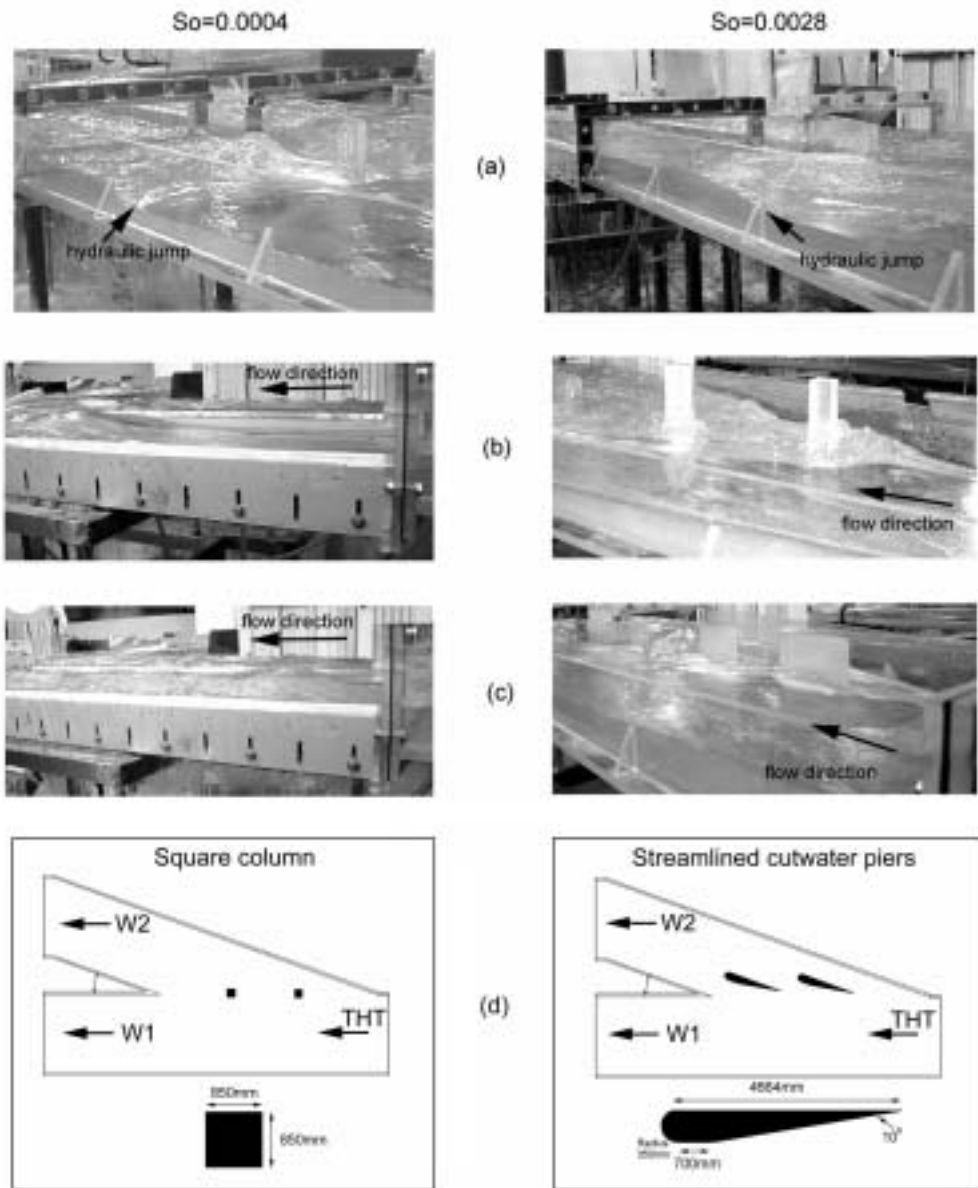


Fig. 5 Significant flow features observed in THTSS model: a) hydraulic jump due to mismatch of specific energy; b) malfunctioning weir flow due to square support column; c) weir flow with streamlined cutwater piers

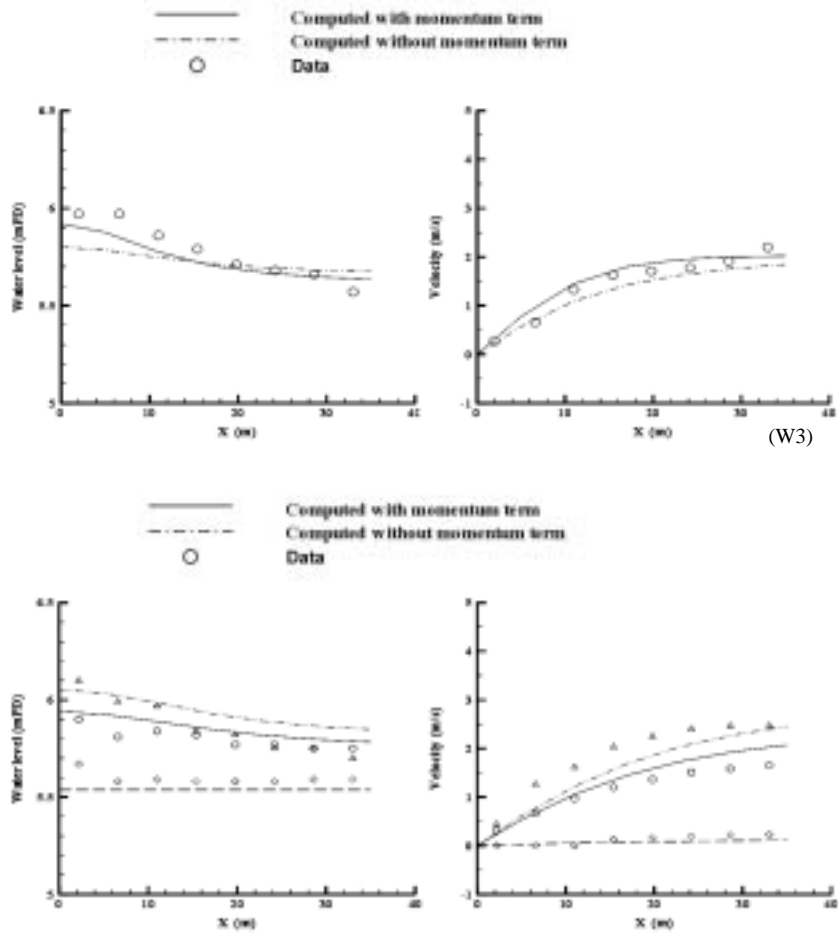


Fig. 6 Example comparison of computed versus measured water levels and velocities in weir system

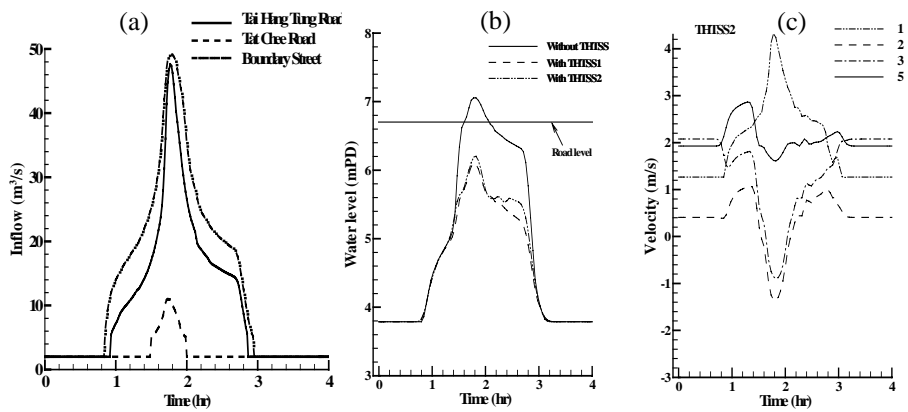





Fig. 7 Computed water levels and velocities in THTSS system: a) inflow hydrographs; b) water level at flooding black spot (A) with and without THTSS; c) velocity at key locations

Table 3 Representative parameters of the Kai Tak Transfer Scheme (free flow condition)

Parameters	Waterloo Road upstream of interception structure	Waterloo Road at interception structure	Waterloo Road downstream of interception structure	Interception structure	Kai Tak Transfer Tunnel
Design flow Q(m ³ /s)	30	From 30 to 10	10	20	20
Length (m)	50	37.5	60	40	420.5
Bed slope	1 in 64	1 in 64	1 in 180	Vertical drop 6.37m	1 in 1000
Normal depth (m)	1.49	From 1.49 to 0.98	0.98		2.72
Critical depth (m)	2.12	From 2.12 to 1.02	1.02		1.80
Fr. Number	1.70	From 1.70 to 1.07	1.07		0.45
Section	 3.1m (W) x 2.9m (H)	Shown in Fig.8	 3.1m (W) x 2.9m (H)	Shown in Fig.8	 D = 4m

Practical constraints dictate that the interception structure must be designed to intercept and convey the required flow from the Waterloo Road into the tunnel underneath Hereford Road (Fig. 1); it needs to achieve a drop of 6.4 m within a distance of about 40 m and keep a 1.5 m clearance from an existing sewer main and a freshwater supply main running parallel to (at different elevation) the Waterloo Road culvert (Fig.8a). For a typical flow of 30 m³/s, the normal depth along Waterloo Road is 1.5 m (F=1.7). Thus the interception structure is essentially a supercritical takeoff from a supercritical flow. In view of the complicated three-dimensional diversion flow, a 1:25 undistorted Froude scale hydraulic model study is carried out. The physical model covers a 60 m reach of the Waterloo Road culvert flow, the interception structure, and 500 m reach of the tunnel, extending to beyond Shaft B (Fig.1). Fig.8b) shows the perspective view of the initial design of the transition structure (the model is a mirror image of the prototype): consisting of a relatively straight spillway chute accounting for most of the drop in elevation, and a transitional bend leading into a 4m square tunnel which is connected to the circular 4 m tunnel at Shaft A. The provision of a sufficient length of model tunnel beyond Shaft A also permits useful qualitative observations of the air entrainment and release behaviour in the complicated two-phase flow.

Extensive model tests were performed for representative upstream flows of Q = 10, 20, 30, 40, 50, 65 m³/s covering the entire range of free flow and surcharged conditions. The downstream boundary conditions at the model limits are obtained from an independent Hydroworks calculation for the entire catchment. For low flows (Fig.9a), it is observed that the free surface flow near the diversion is analogous to an energy conserving spatially-varied flow, discharging non-uniformly into the interception structure. The flow in the interception structure is highly three-dimensional and consists of a spiral flow down the spillway chute, with clear zones of recirculation and separation. A concentrated jet stream is observed towards the outer bend - the jet impinging on the outer part of the structure is deflected 180 degrees; at higher flows this leads to intense air entrainment and a highly complicated two-phase flow. Fig.9b (top) shows the view of the observed flow down the interception structure leading to a hydraulic jump past Shaft A. For Q ≥ 40 m³/s both the upstream Waterloo Road flow and the downstream tunnel flow are surcharged, and the hydraulic jump in the tunnel is forced to occur on the spillway chute; the flow

near the diversion remains a highly turbulent two-phase flow. A second hydraulic jump is also observed on the Waterloo Road flow downstream of the diversion; this jump appears to arrest all the hydraulic control from downstream. In addition, a tornado vortex emanating from the outer corner of the interception structure (point S) is clearly observed (Fig 9b, bottom). The measured hydraulic grade line along the transfer tunnel route for different flows are shown in Fig.10. Detailed flow studies show that the diversion works very similarly to a side weir of zero height. Fig.11 shows the measured free surface and specific energy contours in Waterloo culvert near the diversion outlet; Fig.12 shows example spatial varied flow predictions of discharge and level along the diversion structure.

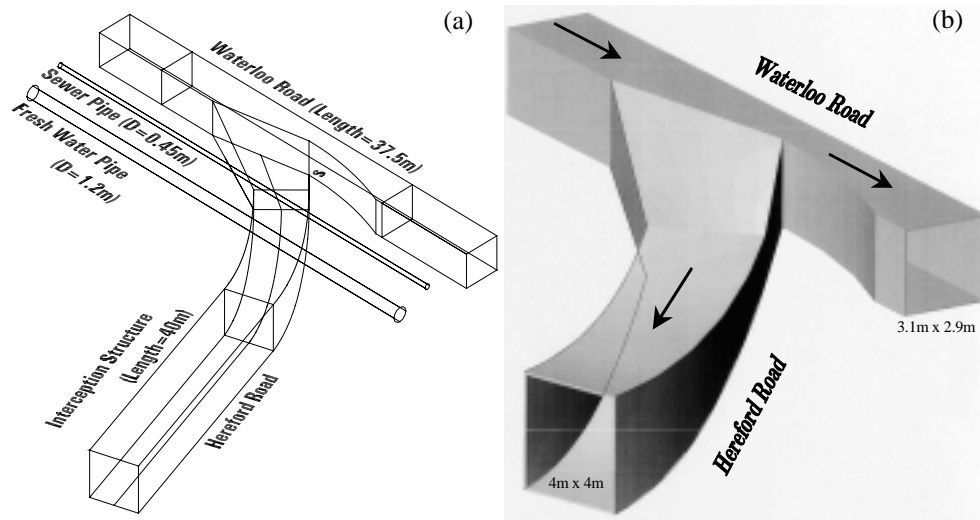


Fig. 8 Schematic diagram and perspective view of the KTTS interception structure

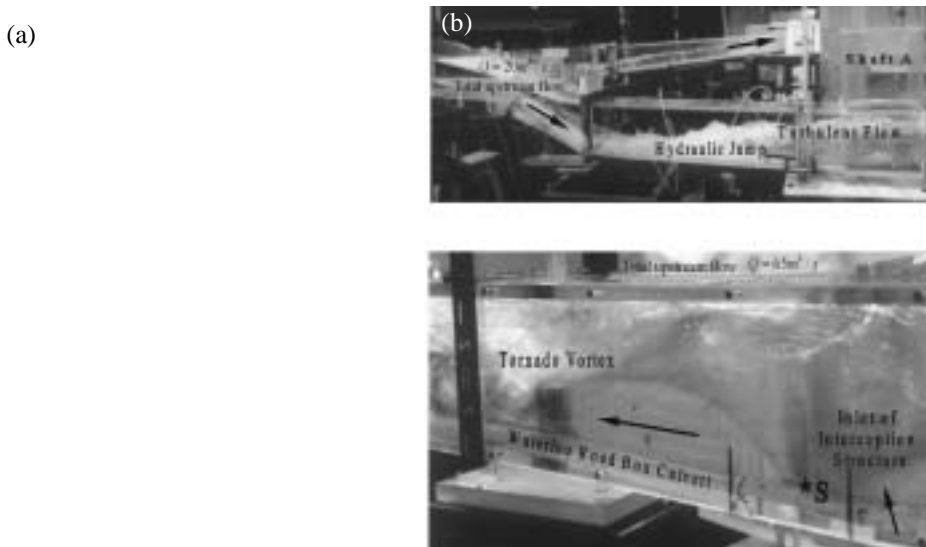


Fig. 9 Observed flow features in KTTS interception structure

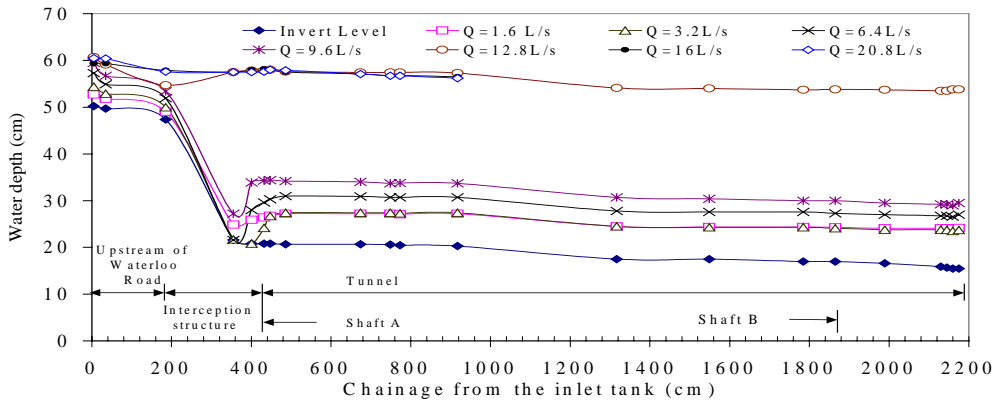


Fig. 10 Measured hydraulic grade line along the KTTS tunnel

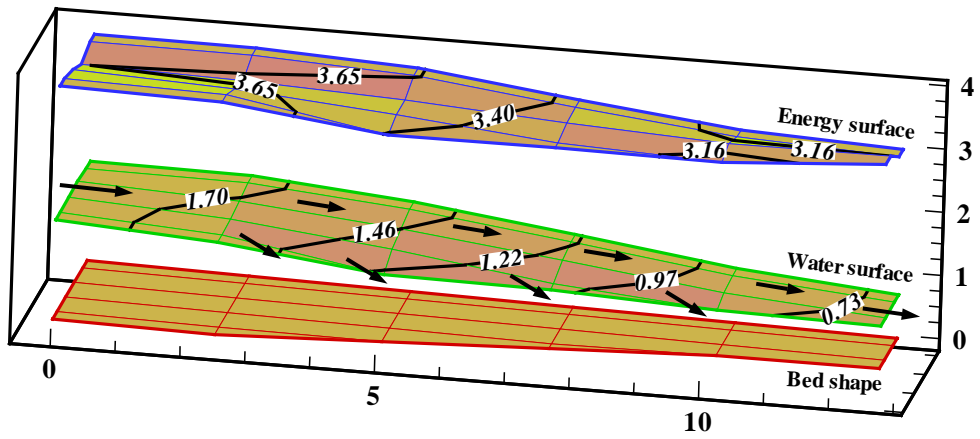


Fig. 11 Measured free surface and specific energy contours in diversion spatially varied flow

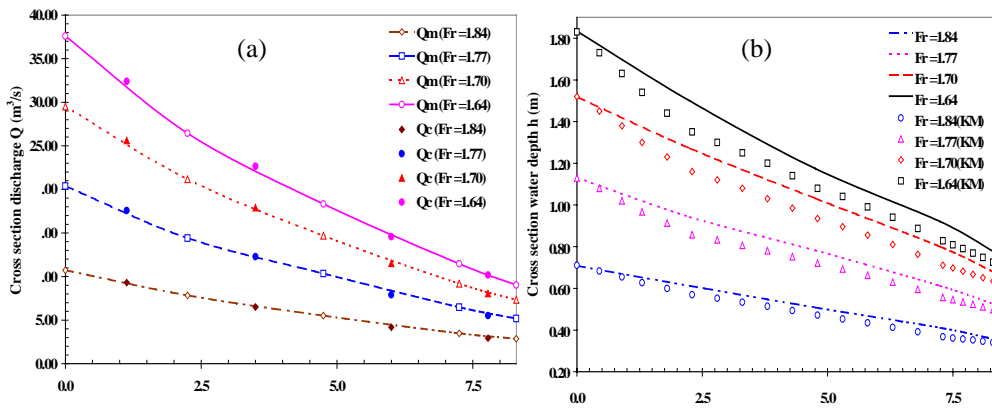


Fig. 12 Example predicted and measured discharges and depths in diversion spatially varied flow

In general, the results show that the required flow diversion of about 60 percent can be achieved. However, the flow in the interception and transition structure is highly turbulent and unstable; leading possibly to undesirable flow-induced vibrations. Model tests show that the design can be improved by the following measures (Fig.13): i) a splitter wall down the middle of the interception structure, dividing it into two compartments, thus effectively reducing the chute width and strength of the cross-wave disturbance; ii) a sharp cutwater of incident angle 10 degrees to the oncoming flow at the upstream end of the splitter wall; iii) a precisely rounded outer corner of the diversion structure, at the location S; iv) the installation of stilling blocks in the square tunnel to promote and fix the hydraulic jump before Shaft A; and v) the provision of air vents on the interception structure and outlets near Shaft A.

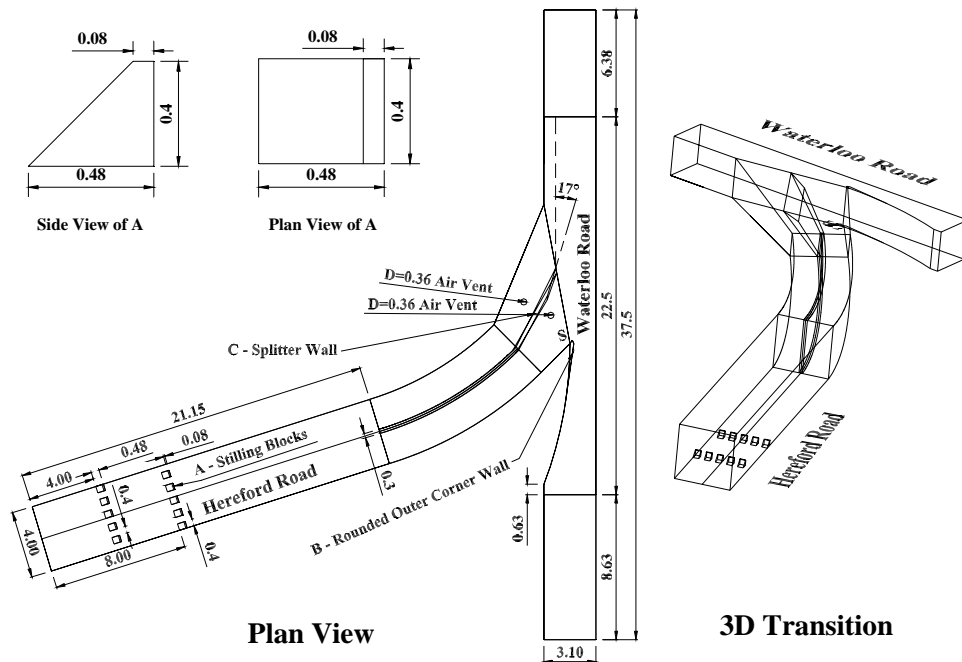


Fig. 13 Final design for the KTTS interception structure

4. CONCLUSIONS

Some characteristics of the hydraulic design of large urban flood control schemes in Hong Kong have been summarized. The successful design of these schemes depends on an understanding of the highly complex 3D junction flows. Both the physical model tests and the numerical model results indicate that THTSS and KTTS can enhance the flood control capacity to cope with a one in 50-year rainstorm. The proposed designs with minor modifications are being adopted, with the aim to achieve flood protection by 2004. The schemes are environmentally acceptable and can be built safely with minimal public disruption. The scheme will reduce the length of storm drains required from 94 km to 48 km, with savings of nearly one billion dollars. Additional examples of urban flood control schemes can be found in the references. For example, the Yuen Long Bypass Floodway was designed under difficult land availability constraints - its success depends crucially on the use of the jet principle to lower water levels at a critical river junction, thus enabling the required flow diversion into the floodway (Lee *et al* 1998).

ACKNOWLEDGEMENTS

The senior author is indebted to his former students and colleagues who have assisted in the various model studies that have formed an important part of their postgraduate training at the University of Hong Kong. In particular, the contributions of Dr HC Chan, Dr CP Kuang, Dr HW Tang, and Ms Chai Hua are gratefully acknowledged. The collaboration of BBV (Hong Kong) Ltd and the Drainage Services Department, HKSAR Government, in the studies are also much appreciated.

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