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HONG KONG DRAINAGE SERVICES DEPARTMENT
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The Hydrological/hydraulic Investigation of the Hung Shui Kiu Drainage Channel

Final Report

HWR Report 145 (Vol. 1)

April 1995

Hydraulics and Water Research (Asia) Ltd

The Hydrological/Hydraulic Investigation
on
Hung Shui Kiu Drainage Channel

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Note to reader:

The Hydrological/hydraulic Investigation of the Hung Shui Kiu Drainage Channel Final Report comprises three (3) separate volumes:

1) Vol.1 is the main report of the above study. It describes the study background, the model setup as well as the details of the findings. It can be considered as a complete report for this study.
2) Vol.2 contains i) a 3.5" floppy disk with all the model input and results data in ASCII format;
   ii) a hard copy of the above data;
3) Vol.3 is the Executive Summary of the Final Report.

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1 INTRODUCTION

1.1 Background to the study

On 5 November, 1993 and 22 July, 1994, two severe flooding events took place at the upper reach of the Hung Shui Kiu Drainage Channel which drains stormwater from the upstream subcatchments of the Tin Shui Wai Drainage Basin. Figure 1 shows a location map of the catchment, and Figure 2 shows a plan of the catchment with the river network and major urban areas and roads. The influence of man upon the natural hydrological cycle of this catchment is considerable, in particular the impounding and canalisation of streams and rivers, and the general urbanisation.

The Hung Shui Kiu catchment is the most upstream subcatchment of the Tin Shui Wai Drainage basin. The area of the Hung Shui Kiu catchment is about 4.6 km^2. There is a cascade of dams and weirs along the mainstream of the catchment (shown in Figure 2), leading to the Hung Shui Kiu Drainage Channel, which starts just downstream of the recently constructed Tuen Mun-Yuen Long Eastern Corridor Highway (hereafter referred to as the TM-YL Highway).

Immediately upstream of the entrance to the Hung Shui Kiu Drainage Channel there is a 100 m long steep trapezoidal transition, with a general gradient of 1:30. A new drain from the TM-YL Highway joins the channel from the left at this point, through a twin 1200 mm diameter storm drain. Just after the entrance, another tributary joins the channel from the right. The channel starts with a 760 m long rectangular section 5.5 m wide and 3.6 m deep, which then expands into a trapezoidal section with a base width of 5.5 m. The longitudinal gradients of the rectangular and trapezoidal sections are generally 1:300 and 1:1,000 respectively. The rectangular section was designed to carry runoff from a 10-year design rainfall, and the trapezoidal section was designed to carry runoff from a 50-year design rainfall. For both parts of the channel, the design rainfalls were based on long-range rainfall statistics at RO headquarters and freeboard allowance. The project brief states that various hydraulic analyses have shown the rectangular section to be able to just cope with runoff from a 200-year design rainfall. During the two flood events of November, 1993 and July, 1994, the rectangular section of the channel was overtopped, and on each occasion water levels rose to approximately 2 m above the top of the channel, flooding the surrounding village of Tan Kwai Tsuen.

Preliminary assessments of the two floods have been carried out by DSD (Drainage Services Department). However, in these analyses the high runoff estimated from the observed flood marks could not be explained by the available rainfall data (DSD, 1994a). Furthermore, photographic evidence indicates that the channel might have overtopped on 18 July, 1992 during Typhoon Faye (DSD, 1994c). The frequent overtopping of the channel warrants a detailed investigation into the causes of the flooding and requires recommendations for appropriate remedial actions. DSD have contracted HWR (Asia) Ltd to carry out this study. The principal objectives of the study are:

- To determine the causes of flooding at the upper reach of the Hung Shui Kiu channel during the events of 5 November, 1993 and 22 July, 1994.

- To recommend short-term and long-term remedial measures.

The hydrological modellers visited Hong Kong from the UK between 28 November and 2 December, 1994 in order to collect data and visit the site. The hydraulic modelling was undertaken at HWR's office in Hong Kong.

1.2 Structure of the report

This report has been structured in accordance with Section 7.3 of the Brief. Its contents are a combination of the material already presented to the client in the First Report and Second Report. Some modification have been included to reflect DSD's comments and additional data received since the First Report and Second Reports and therefore these are now superseded. This Final Report should be considered as the sole complete account of the work undertaken during the study and the conclusions and recommendations made by the Consultants. Chapter 2 gives a detailed report of the work undertaken as part of the data review. Chapter 3 gives a summary of the initial tasks undertaken as part of the hydrological modelling, including the calibration of the hydrological model. Chapter 4 includes a discussion of the selection of the hydraulic model, its set up and the results of the modelling of the November, 1993 and July, 1994 calibration events. Chapter 5 examines the original design of the channel, the impact of the current remedial measures and leads on to discuss potential further remedial works. Finally, Chapter 6 summarises the conclusions of the Consultants and makes recommendations for future work.

2 DATA REVIEW

2.1 Introduction

Various maps, general reports and other sources of information relating to the study area were supplied by DSD, GEO (Geotechnical Engineering Office) and WSD (Water Supplies Department). Aerial photographs of the Hung Shui Kiu catchment taken in 1987 and 1993 were provided by DSD, and further aerial photographs of the catchment taken in 1993 and 1994 were obtained from GEO. Unfortunately, there were no aerial photographs showing the flooding. Various topographical maps and survey sheets, with scales ranging from 1:20,000 to 1:1,000, were provided by DSD.

In line with the project brief, this data review section has been divided into four parts:

- Hydrological data;

- Catchment characteristics;

- Drainage characteristics; and

- Meteorological data.

2.2 Hydrological data

In the following section, the hydrometric network most relevant to the Hung Shui Kiu catchment, as shown in Figure 2, is considered in detail. Other gauges which were used in the study are also described.

2.2.1 Rainfall

There are five recording raingauges in or near the Hung Shui Kiu catchment:

- R27 - a tipping bucket rain gauge located at Yuen Long RG Filters on the catchment watershed. The gauge is telemetered and transmits 5-min data to RO and DSD.
- No.17 - an autographic rain gauge located at Yuen Long RG Filters on the catchment watershed. The gauge provides 15-min rainfall data in chart form which is retrieved manually by WSD.
- No.174 - an autographic rain gauge located at Tin Shui Wai town. RO provided data for this gauge in tabular form, but did not provide any other information about the gauge.
- N07 - a tipping bucket rain gauge located at Tuen Mun town. The gauge is telemetered and transmits 5-min data to RO and DSD.
- N12 - a tipping bucket rain gauge located at Yuen Long town. The gauge is telemetered and transmits 5-min data to RO and DSD.

The gauge locations are shown in Figure 2. Tables 1 and 2 show the data collected from these rain gauges for the November, 1993 and July, 1994 events respectively. As can be seen, there is a considerable amount of 5-min and 15-min data for both these events.

In order to construct spatial and temporal rainfall patterns during the two events, it was necessary to refer to data from a larger area. The spatial rainfall patterns, presented in Section 3.1, were constructed at hourly data intervals. Gauges from which hourly data were abstracted for each event are shown in Figure 3 and listed in Table 3.

In order to recommend suitable design rainstorms of various return periods, a selection of rainfall statistics were collected:

- Synthetic design rainstorm profiles based on long-range (1884 - 1990) rainfall statistics at R01 RO headquarters (Lam & Leung, 1994).
- Synthetic design rainstorm profiles based on short-range (1981 - 90) rainfall statistics at No17 Yuen Long RG Filters (Lam & Leung, 1994).

There appear to be no great merit in analysing significant historical events from the autographic raingauge charts available at No17 Yuen Long RG Filters within the short time-scale of the current study since RO have recently published the results of such analysis for the period 1981-90 (Lam & Leung, 1994). Furthermore it is unclear how the results of such a study might be used in deriving flood estimates of required return periods. Experience shows that the most sensible design flood estimates are produced using generalised storm profiles, such as those derived by RO. Such a design storm profile is likely to be somewhat more peaky than most historically observed storms, but this yields conservative, or safe, peak flood estimates.

2.2.2 River stage and flow

There is only one river stage gauge in the catchment. It is gauge D07 located at Shek Po Tsuen downstream of the flood prone area, and installed under TELADFLOCOSS 2. The gauge is shown in Figure 2. The gauge is telemetered and transmits 5-min data to RO and DSD. There is no rating curve at this site, and so the stage values cannot be converted into equivalent flow figures. However, since this gauge will only be of interest for the hydraulic modelling component of the study, stage values may be adequate. However, it will be important to ascertain whether or not the gauge was bypassed in either of these events. The peak water level recorded at Shek Po Tsuen for the 5 November, 1993 event was 4.83 mPD at 08:45, whilst that for the 22 July, 1994 event was 5.20 mPD between 10:00 and 11:00.

2.2.3 Flood marks

There are four flood marks for the November, 1993 event and nine for the July, 1994 event which will be useful in calibrating the hydrological and hydraulic models. These include flood marks at the irrigation weir at the outfall of the Hung Shui Kiu catchment for both events, and these stage values can be converted to flows using a standard broad-crested weir equation. This will provide data on which to calibrate the hydrological model. For the 5 November, 1993 event, the flood mark at the weir was approximately 19.56 mPD, whilst that for the 22 July, 1994 event was 19.06 mPD. The flood marks for both events are given in Table 36.

Flood mark D for the November, 1993 event is described as a "red mark at a tree" and is shown to be on the left bank adjacent to the upstream limit of the rectangular channel. Flood mark E for the July, 1994 event is also described as "red mark on a tree" and shown to be in same location as Flood mark D for the 1993 event. Both marks record the same level, 14.04 mPD. Visits to the site confirmed that there is only one tree at this location with a red mark on it and it is therefore believed that flood mark E for the 1994 event is in fact the old flood mark D from the 1993 event. Thus it has not been used in the calibration of the July, 1994 event.

2.2.4 Tide records

There is a tide gauge located near the outfall of the Tin Shui Wai catchment. It is gauge TBT located at Tsim Bei Tsui on Deep Bay. The gauge is shown in Figure 2. The gauge is telemetered and records data at 1-min intervals, but only transmits 5-min data to RO and DSD. Again this gauge will only be of interest for the hydraulic modelling component of the study. It was necessary to ascertain whether or not the tide is an important factor in the flooding. For the November, 1993 event, the peak tide level was 2.60 mPD, only 0.10 m higher than the 2.50 mPD channel bed level at Shek Po Tsuen. The observed peak level at the Shek Po Tsuen gauge was 4.83 mPD, significantly higher than the peak tide level and coincident with the peak discharge, negating any possible backwater influence from the tide. The July, 1994 peak tide level was 2.87 mPD against an observed peak water level at Shek Po Tsuen of 5.20 mPD again negating any possible backwater effect of the tide.

2.3 Catchment characteristics

The Tin Shui Wai catchment is situated in a complex Palaeozoic sedimentary basin surrounded by Mesozoic plutonic and volcanic rocks (HK Geological Survey, 1988; GEO, 1992). The Hung Shui Kiu catchment is underlain by fine-grained to medium-grained granite which is highly faulted and intruded with dacite and rhyolite dykes. The valley which forms the mainstream of the Hung Shui Kiu catchment falls on one of these geological faults. In the lower catchment down to Tin Shui Wai, Carboniferous metasediments and Jurassic volcanics predominate, but these are largely concealed by Pleistocene terraced alluvium and debris flow deposits and Holocene marine deposits. These superficial deposits comprise well-sorted clays, silts, sands and gravels, in contrast to the debris flow material which fills valleys in the hilly areas and spreads out downslope forming large fans.

The Hung Shui Kiu catchment has an area of approximately 4.6 km^2, with altitude varying from over 400 mPD on the watershed to only about 20 mPD at the outfall just upstream of the TM-YL Highway. The catchment is located on undulating hilly ground covered mainly with thin soil and sparse vegetation, but wooded at lower elevations. The underlying geology accounts for the poor soil cover and sparse vegetation that are found. The mean annual rainfall in this region is around 1800 mm, varying from 2200 mm on the high ground to 1600 mm near the coast (HK Geological Survey, 1988). The Hung Shui Hang reservoir is situated in the headwaters of the catchment, and a quarry is located just off the catchment boundary, west of the reservoir. There is another quarry to the north-east, towards Yuen Long. Nearer to the reservoir is an area of borrow land which is being variously quarried and reclaimed.

The Hung Shui Kiu Drainage Channel conveys the flows from the catchment outfall down to Deep Bay. This part of the Tin Shui Wai catchment is very flat with a large part of reclaimed area only a few meters above sea level. The channel starts with a 760 m long rectangular section which defines the flood-prone areas in Tan Kwai Tsuen; the rectangular section expands into a larger trapezoidal section. The flood-prone areas house a mixture of agricultural lands and light industry, with associated residential developments.

2.4 Drainage characteristics

The drainage characteristics of the channel were discussed at a meeting with P K Chan and C S Cheng of DSD on 23 November, 1994. Subsequently, relevant design drawings, calculations and maps were passed to the Consultants. The following description of the drainage characteristics has been based on this information and observations made during the three site visits undertaken by the Consultants.

There is a cascade of dams and weirs along the mainstream of the Hung Shui Kiu catchment, leading to the drainage channel. There are two other inflow points to the channel: a drain from the TM-YL Highway just before the channel entrance from the left, and a tributary just after the channel entrance from the right. Examination of the aerial photographs, visual inspection during a site visit, and supporting documentation (Binnies, 1987) suggest that the drain from the TM-YL Highway captures both road drainage and also runoff from two small catchments between the Hung Shui Kiu catchment and the neighbouring Lam Tei catchment, increasing the catchment area by around 6 %. This additional contributing area, believed to result from works associated with the recently constructed TM-YL Highway, has been estimated from aerial photographs and maps to be about 0.28 km^2 and was incorporated into the original design of the Hung Shui Kiu Channel (DSD, 1995a). Drainage from this area enters the Hung Shui Kiu Channel through twin 1200 mm diameter pipe culverts situated on the left bank just downstream of the highway bridge. This subcatchment has been modelled as a separate inflow. The estimated peak inflow for the 5 November, 1993 event was about 9 m^3s^{-1} which agrees well with the 9.2 m^3s^{-1} design flow of the pipe culverts (Highways Dept, 1994). The peak inflow for the 22 July, 1994 event was slightly lower, at about 7.5 m^3s^{-1}.

However, it may be that the contributing area of the tributary has also been increased, in part due to quarrying activities. From examination of aerial photographs and maps it appears that an area to the east of Tan Kwai Tsuen now drains into the new rectangular channel, known as the Tan Kwai Tsuen Channel, joining the Hung Shui Kiu Channel some 100m downstream from the upstream limit of the rectangular section. This area, about 0.38 km^2 has also been modelled explicitly. Through construction of the TM-YL Highway and flow diversion this catchment may have lost inflow into a very small subcatchment which inputs at the new road bridge, but through quarrying to the east of the Hung Shui Kiu catchment, it may have increased its contributing area by approximately 0.05 km^2.

Table 37 lists the areas of the significant subcatchments shown in Figure 6 contributing to the new rectangular channel and compares these area estimates derived at the time of the channel design with those derived by the Consultants for the current study.

The difference in catchment areas between the two studies is about 8 percent, which probably represents a slightly different interpretation of the poorly defined new contributing areas along the TM-YL Highway and old quarry.

The hydrology and hydraulics of the original channel design (DSD, 1985 onwards) has been briefly reviewed in order that the assumptions made can be kept in mind whilst the hydrological modelling component of the current study is carried out. The conclusions drawn concur with those drawn by DSD (1994a, 1994b), and are summarised below:

- The rectangular section of the channel was designed to pass flood from a 10-year design rainfall, which was calculated to cause peak discharges of 70 and 76 m^3s^{-1} at the upstream and downstream limits respectively. However, the as constructed drawings show that for the full length of the rectangular channel the width and depth are constant therefore the design capacity should be 76 m^3s^{-1}, plus a small freeboard (200-300 mm).

- A Manning's n of 0.015 was used in the design calculations of the rectangular channel.

- Energy dissipators have been introduced at the upstream entrance to the rectangular drainage to assist in the formation of a hydraulic jump.

- Outside the 1:10 year tidal limit the drainage channels have been designed assuming gravity flow only. No investigation of tide induced backwater effects above this limit have been made.

- Some calculations have been undertaken to establish the energy losses at hydraulic jumps along the channels but it is not clear how these head losses have been included in the overall design calculations.

- No calculation of energy loss at bends, effects of cross waves or potential energy loss caused by the access ramp into the rectangular channel have been identified.

- From examination of the drawings, it appears that the cross section of the upgraded channel is broadly similar to that of the original natural channel (DSD, 1994a).

- The time of concentration of 81 min for the channel entry point computed using the modified Bransby-Williams formula may be too long in view of the steep channel slopes and short stream lengths along the side of the valley (DSD, 1994b).

- The runoff coefficient of 0.6 adopted for use in the Rational Formula may be too low in view of the steep nature of the catchment and of the shallow soils (DSD, 1994b).

- The design rainfalls were based on data from RO headquarters in Kowloon and from Kings Park, as these were the only intensity information available at the time of channel design. RO have now published additional depth-duration-frequency data (Lam & Leung, 1994), which include a rain gauge at Yuen Long RG Filters on the catchment watershed, and these were compared with the long term rainfall statistics at RO when generating the design rainfalls and flows.

The Hung Shui Hang reservoir is situated in the headwaters of the catchment. The reservoir was built in 1957 for the storage and supply of irrigation water to local villages. The following reservoir details come from Taylor-Fox (1988). The catchment area to the dam is 3.23 km^2. The dam is approximately 19.8 m high, with the crest at 86.87 mPD. The reservoir has a surface area of 0.02 km^2 and a capacity of 91 Ml. A further surface area of approx 0.008 km^2 is formed by water impounded at the intake weir for the Yuen Long Treatment Works, this weir being located just upstream of the main reservoir. This intake is no longer in use due to heavy siltation in the supply pipeline. The reservoir spillway is a 50.29 m overflow weir with an ogee profile with the crest at 85.34 mPD.

The reservoir capacity is relatively small compared with the runoff volume expected in typical floods, and hence is unlikely to have a significant impact upon the natural flood response of the catchment. Part of the immediate remedial measures following the recent floods has been to install twin siphon pipes to keep the reservoir drawndown by around 5 m. This will provide some additional flood storage and is discussed further in Section 5.2.

There are no reliable records at the reservoir of levels over the spillway during either of the two events which could be used to estimate flow over the spillway. During an inspection of the dam following the November, 1993 event, engineers found debris on the walkway, damage of handrails above the stilling basin, and signs of extensive scouring of vegetation both at the sides of the dam and within the watercourse downstream of the dam (WSD, 1993). If the footbridge was overtopped the discharge over the dam could be in excess of 200 m^3s^{-1} (WSD, 1993). However, the reservoir safety engineers do not believe that the footbridge was overtopped (pers. comm. P. Wood, 1994; pers. comm. S. Wong, 1994). Trash was noticed only on the handrails and stanchions at either end of the footbridge and not in the middle; the greatest amount of trash and flattening of vegetation occurred near the left bank abutment of the dam, and there was no evidence of high trash levels around the perimeter of the reservoir after the event. The most likely explanation is that the debris was washed down the valley sides from above the dam. There is a steep access path with concrete steps leading down to the dam at the left bank abutment where the greatest volume of trash was noted, supporting this theory.

2.5 Meteorological data

As well as the various hydrological data available, meteorological information, such as weather radar imagery, were considered. RO was the source of all meteorological data.

It was anticipated that the weather radar operated by RO could be used to supplement data from the rain gauge network for the two flood events, particularly that of 22 July, 1994 where many of the gauges around the catchment failed to operate. However, RO advised that it was unlikely that the radar data could help to either infill the period of missing data in July, 1994, or examine the storm movements (pers. comm. S. Wong, 1994). The current radar operated by RO can operate at four different ranges: 512 km, 256 km, 128 km or 64 km range. For both events, data archives were made at the 256 km radius, and the resolution was too coarse to enable accurate identification of the study area and small scale convective cells (RO, 1993b). Monthly weather summaries published by RO (1993a, 1994) provide daily mean wind speeds and directions at Waglan Island.

3 HYDROLOGICAL MODELLING

3.1 Spatial and temporal rainfall patterns

3.1.1 The 5 November, 1993 event

Severe flooding took place on the upstream section of the Hung Shui Kiu Channel on 5 November, 1993 (Typhoon Ira). According to flood marks, the channel was generally overtopped by 1.5 m, and the flooding was localised along the 0.8 km long rectangular section of the drainage channel. The flooding ceased after entering the trapezoidal section. According to some villagers the channel was running full around 08:00, and within 10 min the flood level rose to approximately 1.5 m above ground level. The flood quickly subsided within half an hour. The autographic rain gauge No17 recorded 119.5 mm of rain between 07:45 and 08:45, but unfortunately the tipping bucket rain gauge R27 had been out of order since 07:40. During the period of flooding, the tide level at Tsim Bei Tsui TBT was quite low i.e. 0.6 m to 0.9 mPD between 07:00 and 09:00. The maximum river stage recorded at Shek Po Tsuen D07 was 4.83 m at 08.45 (within river bank).

The RO Monthly Weather Summary for November, 1993 (RO, 1993a) reports that relatively cool weather brought by a surge of the winter monsoon towards the end of October continued into November. The weather was brilliantly fine to start with but turned cloudier over the next couple of days. As Typhoon Ira moved into the South China Seas, coming in a north-westerly direction from the Philippines and finally hitting the coast of China just north of Vietnam, northerly winds freshened on the night of 2 November (mean daily wind speed of around 30 km hr^{-1} at Waglan Island) and became strong offshore late in the evening of 3 November (around 50 km hr^{-1}). Winds turned easterly on 4 November and reached gale force strength offshore and on high ground (around 80 km hr^{-1}). Rain was persistent throughout the day on 4 November and turned heavy during the night. From midnight to daybreak, a north-south oriented rainband remained almost stationary over the western part of the territory. Along this corridor of active convection, intense rain cells developed over the coastal waters and moved northwards, hitting Lantau Island and Tuen Mun repeatedly. Intense convection embedded within one of the typhoons trailing rainbands brought concentrated heavy rainfall to the western part of the territory early on 5 November.
Figure 4 shows the hourly isohyets between 00:00 and 10:00 on 5 November, 1993, and clearly shows bands of heavy rainfall passing over the catchment. This rainfall would wet-up the catchment, saturating the thin soils and filling any storage capability in the reservoir. Between 08:00 and 09:00 a rain cell is centred directly over the catchment, indicating over 100 mm rain in that hour. Winds and rain eased off during the day on 5 November (around 30 km hr^{-1}). Following some light rain that night and early next day, the weather turned brighter.

3.1.2 The 22 July, 1994 event

Again severe flooding took place at the same location on 22 July, 1994. According to flood marks the channel was generally overtopped by around 2 m. The flooding was again restricted to the 0.8 km long rectangular section of the drainage channel, and again ceased after entering the trapezoidal section. According to some villagers, the channel was overtopped at around 03:00, but the most serious flooding occurred at around 07:00 to 08:00. Heavy rainfall concentrated over the New Territories on the morning of 22 July, starting around 01.00. However the tipping bucket rain gauge R27 was out of order since 01:40, and only a limited record (15 min rainfall data from 06:15 to 09:00) was retrieved from the autographic rain gauge No17. During the period of flooding, the tide level at Tsim Bei Tsui TBT was rather high. The maximum recorded tide level was 2.87 mPD at 08:10. The maximum river stage recorded at Shek Po Tsuen D07 was 5.20 m at 10:00.

The RO Monthly Weather Summary for July, 1994 (RO, 1994) reports that the month was extremely wet (1147.2 mm), with the highest July rainfall total on record at three and a half times the July normal, and the second highest monthly rainfall total on record. Following the passage of Typhoon Tim on 11-12 July, and under the influence of an unstable south-west monsoon, the weather in mid-July was generally fine but cloudy, with some sunny spells and isolated showers. An active trough of low pressure developed over the south China coast on 21 July, bringing very unsettled weather, thunderstorms and torrential rain to the territory from 22-24 July. Winds were predominantly from the south, with the mean wind speed remaining relatively steady throughout the period (20 to 40 km hr^{-1} at Waglan Island).

Figure 5 shows the hourly isohyets between 00:00 and 12:00 on 22 July, 1994. The heavy rain on 22 July fell in two episodes, one in the small hours (maps from 00:00 to 04:00) and the other in late morning (maps from 06:00 to 10:00), and the maps show the two rain cells passing over the catchment. Again the rainfall would wet-up the catchment, satisfying soil moisture deficits and reducing storage. Altogether over 300 mm were recorded in the north-western part of the territory on 22 July. Torrential rain fell again between 4 am and 6 am on 23 July, and heavy rain continued throughout 24 July. Although there were still periods of rain on 25 July, the intensity and total amount lessened. As the trough of low pressure moved south gradually into the south China Sea, rain began to ease off on 26 July.

For the current study, the raingauge at Tsuen Wan was not used further in the analysis, but the extremely high hourly point rainfall between 10:00 and 11:00 is suspicious and worth considering briefly in a wider context. This figure was originally supplied as 172.0 mm with some missing data but RO (1995a) report the exact value to have been 185.5 mm and confirm this to be the highest hourly total in the whole country. Bell & Chin (1968) suggest that the maximum hourly rainfall in Hong Kong can be in excess of 200 mm hr^{-1}. However, the available data for various durations, including this value, would put the recorded rainfall right beyond the upper limits of the Bell & Chin (1968) maximum rainfalls.

3.2 Selection of hydrological model

The selection of an appropriate hydrological model for rainfall-runoff modelling purposes will depend ultimately on the data available with which to calibrate and validate it. No reliable flow data exist within the Tin Shui Wai catchment against which a rainfall-runoff model may be calibrated. Although there is a telemetering water level recorder at Shek Po Tsuen, no rating curve exists for this site, and it is unlikely that a unique relationship between stage and flow exists because the site is affected by periodic tidal backwaters. Some peak flow estimates may be derived from flood peak levels observed during the 5 November, 1993 and 22 July, 1994 events. The estimated flows at the irrigation weir just upstream of the new TM-YL Highway will provide the best data against which the hydrological modelling may be checked.

There is no single method for estimating flood runoff which can be recommended for the catchment in view of the lack of reliable downstream flow data for calibration and validation purposes. Consequently, a range of methods will be considered.

The WSD unit hydrograph method based on catchment characteristics will be used, although it is believed that this may produce a unit hydrograph which has too long a time-to-peak and an underestimated peak discharge. Empirical formulae for estimating unit hydrograph parameters from elsewhere in the world will also be applied in order to check the WSD method.

WSD (1968) considered loss rates for a range of storms, and this work was checked by both Mott-Macdonald (1990) and BMC (1992a). The range of loss rates quoted by WSD varied from 3 to 85 mm hr^{-1} and recommended adopting 3 mm hr^{-1} for design. Mott-Macdonald found a smaller range of 3 to 27 mm hr^{-1} and recommended 8 mm hr^{-1} for design. BMC analysed only four events, but quote a range of 4 to 12 mm hr^{-1}, although for one event the final rate was 24 mm hr^{-1}. During both the November, 1993 and July, 1994 floods, catchment average hourly rainfall exceeded 65 mm hr^{-1}. Using the WSD recommended loss rate, the equivalent runoff coefficient is about 0.95, and using the Mott-Macdonald figure, it would be 0.88. Thus the figure of 0.6 used for the original design implies a loss rate of 26 mm hr^{-1}, which is at the upper end of most of the studied storms.

Runoff rates will therefore be determined using a range of methods, ranging from the 3 mm hr^{-1} loss rate method proposed by the current design procedures, through the 8 mm hr^{-1} loss rate recommended in TELADFLOCOSS 1 and the modified SCS method used in TELADFLOCOSS 2, to engineering judgement. Whatever method is finally selected must take account of the very limited soil and vegetation storage available in the catchment. Any losses method must recognise that the majority of any large rainfall input will produce flood runoff. Therefore, estimation of losses must consequently be a relatively unimportant task as runoff may be in excess of 90%.

Once a range of flood estimation models have been applied to the data, the most appropriate design method will be selected using a combination of engineering judgement and comparison of the model results with the single peak flow estimate available from the irrigation weir. DSD have undertaken some flood estimates using various different models (DSD, 1993), and their figures will also be of value.

3.3 Calibration of the hydrological model

3.3.1 Derivation of hydrological model parameters

The objective of the hydrological component of the study is to provide inflows to the hydraulic model at specific input points. This section describes derivation of the inflows for the observed events of November, 1993 and July, 1994, used for hydraulic model calibration.

The choice of the gauging station D07 Shek Po Tsuen, downstream of Castle Peak Road bridge, as the lower limit of the hydraulic model, and the positioning of the recently constructed TM-YL Highway, divides the Hung Shui Kiu catchment naturally into seven subcatchments, as shown on Figure 6. The largest of these is the catchment to the irrigation weir upstream of the TM-YL Highway, denoted catchment 1 in Figure 6. This is the only catchment on which calibration of the hydrological model is possible. Catchment 1a refers to the catchment of Hung Shui Hang reservoir, and has been included for completeness. Catchment 2 is the small catchment joining the main channel between the irrigation weir and the TM-YL Highway. Catchment 4 is the drain from the TM-YL Highway, and joins the main channel from the left, just upstream of the start of the rectangular section, whilst catchment 3 is the tributary inflow from the right which enters the rectangular section some 100 m below its upstream end. Catchment 5 represents the flat, urbanised area along the right bank of the rectangular and trapezoidal sections of the drainage channel. Catchment 6 represents the flat, urbanised area along the left bank of the drainage channel, whilst catchment 7 represents the area between the Castle Peak Road bridge and Shek Po Tsuen. The hydrographs from catchments 6 and 7 will be scaled from that for catchment 5 on the basis of area. Table 4 lists the catchment characteristics abstracted from maps for each of the seven subcatchments.

The hydrological model selected for use in the current study is the unit hydrograph and losses model, which has also been used in TELADFLOCOSS 2, and which is commonly applied for flood studies such as the present problem. The catchment characteristics listed, therefore, are those required for estimation of the hydrological model parameters.

Derivation of unit hydrograph

Because no suitable flow data are available for unit hydrograph derivation, the parameters of the unit hydrograph must be estimated from catchment characteristics. The most important parameter is the time-to-peak Tp of the unit hydrograph, which is near equivalent to catchment lag LAG, which is the time delay between the centroids of the storm rainfall and the peaks of the resulting hydrograph. A range of methods exist for estimating either Tp or LAG, or for estimating the very closely related time of concentration Tc of a catchment. A range of these were applied to the five main subcatchments.

One method commonly used in Hong Kong is the Bransby-Williams formula (Williams, 1922) for estimating Tc in minutes. This formula, in metric units is:

$$Tc = 144.65 * MSL / \{Sl^{0.2} * (Area*10^{6})^{0.1}\} \qquad \text{Eqn (1)}$$

where MSL, Sl and Area are as given in Table 4.

However, experience in Hong Kong suggests that this formula gives Tc estimates which are too long for small, steep headwater catchments such as the Hung Shui Kiu. DSD have suggested that the estimated Tc of 77 min for catchment number 1 to the irrigation weir is too long, and the consultants concur with this view. Given a stream length of 3900 m, a Tc of 77 min represents an average water velocity in the channel of only 0.84 m s^{-1}, or allowing for some overland flow, little over 1 m s^{-1}. Considering the steep valley sides and channel slopes of the catchment, this velocity seems too low, and the Tc estimate is believed to be rather too long. However, estimated Tc values for the five main subcatchments are given in Table 5.

The WSD manual for design flood estimation in Hong Kong (WSD, 1968) proposes two methods of estimating LAG, and hence Tp, both derived from analysis of all local rainfall and flow data available at the time. These methods have subsequently been checked by the consultants for both Phase 1 and Phase 2 of the TELADFLOCOSS studies (Mott-Macdonald, 1990; BMC, 1992a), and shown to be reasonable for steep, upland catchments such as the Hung Shui Kiu. The first method is based on the Snyder formula, as used by the U.S. Department of the Interior Bureau of Reclamation (USDoI, BR, 1960), which estimates LAG in hours as:

$$LAG = 5.52 * 10^{-4} \{L*Lc/S^{0.5}\}^{0.4} \qquad \text{Eqn (2)}$$

where LAG is basin lag time (hr), L is mainstream length, MSL (ft), Lc is length from basin outlet to point on the stream nearest to catchment centroid (ft), and S is the channel slope as given in Table 4.

A second equation based on catchment area alone was also proposed by WSD. This was found to have a slightly higher correlation coefficient for the catchments in their study, and has the form:

$$LAG = 0.0398 * A^{0.5373} \qquad \text{Eqn (3)}$$

where A is catchment area (hectares), i.e AREA * 10^2.

Although this has a superior correlation coefficient to the earlier equation, it seems to be intrinsically inferior in the sense that intuitively catchments of any given area having different shapes, and particularly slopes, would be expected to have different lag times.

However, it should be noted that either equation is based on analysis of data from only 22 events from seven catchments. For simplicity, the second equation based on area alone was used to estimate LAG for the five main subcatchments, and results are shown in Table 5.

Another commonly applied formula is that proposed by Kirpich (1940), who published one of the first means of estimating Tc for small, ungauged agricultural catchments. This relationship was based on data from only six small agricultural catchments in Tennesee, and was derived from work by Ramser (1927). However, this work was extended by Watt & Chow (1985) using data from a number of published sources, and their derived general relationship for LAG is believed to be one of the best methods of estimating catchment lag in hours, for ungauged catchments, currently available. Their relationship has the form:

$$LAG = 0.000326 * \{(MSL*1000)/S^{0.5}\}^{0.79} \qquad \text{Eqn (4)}$$

where MSL and S are as given in Table 4.

Results from applying this equation to the five main subcatchments are given in Table 5.

The estimates derived using Watt & Chow's equation appear physically most realistic of those computed and have been adopted as the best estimates of unit hydrograph Tp for the current study. This Tp estimate was able to reproduce the two calibration events successfully (see Section 3.3.4).

In view of the fact that the Watt & Chow (1985) equation is based on significantly more data than those of either Kirpich (1940) or WSD (1968), it is suggested that this method be used to estimate LAG for the various subcatchments within the Hung Shui Kiu basin, even though it has been derived primarily with data from North America. Since LAG or Tp is a function primarily of land surface and channel slope and roughness, there is no reason to believe that the results of Watt & Chow's work should not be applicable in Hong Kong. The computed LAG values given in the final row of Table 5 have been adopted as unit hydrograph time-to-peak estimates for hydrological model calibration, and these LAG values have been assumed to be reasonable estimates of time-to-peak for a 5-minute unit hydrograph.

The unit hydrograph was assumed to be adequately represented by a simple triangle, as proposed in the UK Flood Studies Report (NERC, 1975). Thus the time base of the unit hydrograph TB has been taken as:

$$TB = 2.52 * Tp \quad (hr) \qquad \text{Eqn (5)}$$

and the peak flow of the unit hydrograph Qp has been taken as:

$$Qp = 2.2 / Tp \ (m^3s^{-1}km^{-2}) \qquad \text{Eqn (6)}$$

for a storm of 10 mm of rainfall in 5 min.

WSD (1968) presented an average dimensionless curvilinear unit hydrograph, but this was derived from a very small data set. In the current study, the WSD unit hydrograph was scaled from Tp which has been taken as equivalent to LAG, and Figure 7 compares the WSD and FSR unit hydrographs for subcatchment 1. The WSD unit hydrograph has a slightly earlier and lower peak, than the FSR unit hydrograph, and also a longer recession. The effect of using the WSD unit hydrograph rather than the FSR unit hydrograph was tested on subcatchment 1 for the July, 1994 event, and the derived hydrographs are shown in Figure 8. The hydrographs have the same total volume, as identical percentage runoffs were used, but the peak flows from the WSD unit hydrograph are about 3 to 4 % lower than those using the FSR unit hydrograph, with the flows being sustained after the peaks due to the longer recession of the WSD unit hydrograph. Because the FSR unit hydrograph gives conservative peak flow estimates, and because of its computational simplicity, it has been adopted in the current study.

The U.S. Department of the Interior Bureau of Reclamation (USDoI, BR, 1960) also provide a means of estimating unit hydrograph ordinates for ungauged sites using the soil conservation service (SCS) method. This approach has been successfully used in both phases of the TELADFLOCOSS studies (Mott-Macdonald, 1990; BMC, 1992a), partly at least because this method allows the effects of changing land use on the flood response of a basin to be modelled. However, the SCS method is very similar to that proposed here, with TB being 2.67 * Tp rather than the 2.52 * Tp adopted here.

However, for the present study the aim is to identify an overall hydrological model which is best able to reproduce the observed flood peaks estimated from trash marks during the floods of 5 November, 1993 and 22 July, 1994, and this justifies adoption of the simple UK Flood Studies Report triangular unit hydrograph.

The relationship between time of concentration, unit hydrograph time-to-peak and catchment lag is complex and imprecise. Various authors have presented empirical formulae relating LAG to Tp for example, but there is no firm physical relationship between these three time characteristics of a basin. In view of the general uncertainty over this issue, the LAG estimate computed using the Watt & Chow formula given in equation (4) has been adopted for simplicity, and taken as equivalent to Tp, partly since recent research into response times of small catchments has shown that $Tp = LAG^{0.94}$ (Marshall & Bayliss, 1994). The assertion that the Bransby-Williams formula appears to overestimate Tc is based on the evidence of Table 5, and also on consideration of likely overland and channel flow rates. Beran (1979) concluded that the Bransby-Williams formula in fact underestimates Tc of catchments, but these were in Britain, and were generally much larger catchments than those found in Hong Kong.

Derivation of losses

Three rainfall loss rate mechanisms have been applied in Hong Kong over the years and these are:

- proportional losses (i.e. a runoff coefficient or percentage runoff approach),
- constant loss rate, or Φ index method,
- time varying losses (SCS loss mechanism).

The proportional loss rate is used in the Rational Method, and is also used in the UK Flood Studies Report rainfall-runoff method (NERC, 1975).

The constant loss rate, or Φ index method, was recommended by WSD (1968), and loss rates of from 3 to 85 mm hr^{-1} were observed. WSD recommended that to be conservative, the lower rate of 3 mm hr^{-1} should be adopted for safe design. However, for both phases of the TELADFLOCOSS studies (Mott-Macdonald, 1990; BMC, 1992a), a loss rate of 8 mm hr^{-1} was used. In view of the very high storm rainfalls experienced in Hong Kong, the difference between these figures is insignificant, but to be conservative, the 3 mm hr^{-1} figure recommended by WSD has been used in the current study. Of course, there must be a distinction between recommended loss rates for design events, which will never in fact occur exactly as modelled, and loss rates fitted as part of a calibration process. However, given that the estimated flood peaks observed for both the November, 1993 and July, 1994 events can be reproduced using the more conservative loss rates suggested, then the same parameters should be used for estimation of design floods.

The time varying loss rate method, of which the SCS method is perhaps the best known, has a number of advantages over the two approaches described above. The SCS method was used for the Basin Management Plans of the TELADFLOCOSS 2 study (BMC, 1992a), but suffers from one or two drawbacks as far as the current study is concerned. The first is that there is a considerable degree of subjectivity in the method, where the curve numbers, or CN, that control runoff rates must be estimated by determining soil type and land use, and the basic CN varies according to the antecedent state of the catchment. Thus it is difficult to select a simple, consistent set of CN which will both reproduce past events, yet still be applicable for estimation of design events. The variability in runoff response allowed by the method is potentially attractive in that it allows the hydrologist to adjust the parameters of the method to fit observed flood events, but this same flexibility becomes something of a liability at the design stage, where antecedent moisture condition AMC II is generally adopted in consequence.

For the current study, the proportional and constant loss rates methods were tried in the calibration stage, and results showed that there was little to choose between them. From past experience, it would be anticipated that the SCS method would yield broadly similar flood responses to the two calibration storms, and in light of this, just the first two of the methods described above have been employed. The loss rates assumed are given in Table 6.

Derivation of baseflow

In their study of the Hung Shui Hang reservoir, Taylor-Fox (1988) stated that their adopted baseflow value of 0.12 $m^3s^{-1}km^{-2}$ would occur during moderately wet conditions. In the current study the more conservative adopted value of twice this amount i.e. 0.24 $m^3s^{-1}km^{-2}$, has been chosen to represent a wetter catchment. No guidelines appear to exist for selection of a design baseflow for Hong Kong. However, in practice, for this catchment, the chosen value is fairly arbitrary as the baseflow is insignificant compared to the volume of the total flow hydrograph, for both the calibration events and the design events. This constant baseflow has been applied to all of the subcatchments.

Summary

Table 6 lists the definitive set of hydrological model parameters used for catchments 1 to 5. Time-to-peak values vary from 0.55 hr (i.e. 33 min) for catchment 1 to only 0.08 hr (i.e. 5 min) for catchment 2, necessitating a 5 minute data interval in the modelling. Percentage runoff values vary from 95 % for catchment 1 to 80 % for catchment 5, whilst for the alternative loss rate approach values vary from 0.25 mm $5min^{-1}$ for catchments 1 to 4 to 0.5 mm $5min^{-1}$ for catchment 5. As stated previously baseflows are a constant 0.24 $m^3s^{-1}km^{-2}$ for all catchments.

3.3.2 Derivation of catchment average rainfall profiles

The rainfall-runoff models for each of the sub-catchments of the Hung Shui Kiu catchment, the parameters of which were estimated in the previous section, require areal rainfalls as input. The isohyetal method is considered to be one of the most accurate methods for determining catchment areal rainfall, but is subjective and dependent on skilled, experienced analysts having a good knowledge of the rainfall characteristics of the region containing the catchment. In this study, the method selected to derive the areal rainfalls is called the iso-percentile method. The iso-percentile method is a modified version of the isohyetal method and is easier to apply, and the development and use of the iso-percentile method has been encouraged by the UK Met. Office. The iso-percentile method was previously employed in Hong Kong in the real-time flood forecasting system for the Indus Basin as part of TELADFLOCOSS 2 (BMC, 1992b). The method is discussed briefly in Jones (1984).

The iso-percentile method is a simple means of deriving a weighted average rainfall for any catchment using whichever raingauges are available, and is best explained by reference to Figure 9 which shows the Hung Shui Kiu catchment and the five nearest recording raingauges, listed in Section 3.2. All of these raingauges are assumed to be representative of the rainfall falling on the Hung Shui Kiu catchment. The relative influence of each of the raingauges on the catchment is likely to be a function of its distance from the catchment centroid, and the inverses of the distances are normally used as weighting factors when computing the average. The definition of the catchment centroid is not critical, and the distances may be measured in any units, in this case km. However, the main feature of the iso-percentile method is that it uses the long-term average annual rainfall at each raingauge as an indicator of how the raingauge might influence the catchment rainfall. These point average annual rainfall values, together with the average annual rainfall for the catchment, were estimated from the RO map for the standard period 1953-83 (Kwan & Lee, 1984). Table 7 lists the five raingauges, their distances from the catchment centroid and their average annual rainfalls.

For any time period t, the catchment average rainfall $catR_t$ is estimated from the recorded rainfall at each of the five raingauges using the following formula:

$$catR_t = saarCAT \frac{\left(\frac{1}{dR27} \cdot \frac{rR27_t}{saarR27}\right) + \left(\frac{1}{dNo17} \cdot \frac{rNo17_t}{saarNo17}\right) + \left(\frac{1}{dNo174} \cdot \frac{rNo174_t}{saarNo174}\right) + \left(\frac{1}{dN07} \cdot \frac{rN07_t}{saarN07}\right) + \left(\frac{1}{dN12} \cdot \frac{rN12_t}{saarN12}\right)}{\left(\frac{1}{dR27} + \frac{1}{dNo17} + \frac{1}{dNo174} + \frac{1}{dN07} + \frac{1}{dN12}\right)}$$

where saarCAT is the average annual rainfall (in mm) for the catchment, and the prefixes d, r and saar for each raingauge refer to the distance (in km) of the raingauge from the catchment centroid, the rainfall (in mm) falling at the raingauge in the time period t, and the average annual rainfall (in mm) at the raingauge.

This algorithm therefore uses the relative average annual rainfall for each raingauge compared with the catchment average annual rainfall, and combined with a weighting factor which is the inverse of the distance of that raingauge from the catchment centroid.

The method is robust and will always produce an areal rainfall even when several of the raingauges do not have data for a particular time period. Clearly, the best estimate of the catchment rainfall would be derived from use of all five raingauges, but if any one or more of the raingauges did not have data for the time period, the catchment rainfall would simply be computed using the remaining raingauges. Because the individual gauge weightings, average annual rainfalls and distances are fixed for each raingauge, and are not influenced by the number of gauges being used in the averaging algorithm, raingauges may drop in and out of the procedure without causing problems.

The tight timetable for the work, imposed by the client, has severely restricted the flexibility of the consultants to compare the results from different methods. However, comparisons of the 1-hr catchment areal rainfalls by the isopercentile methods and isohyetal methods for the calibration events show that, in both cases, whilst the two methods compare well at lower rainfall intensities, at the higher intensities it is the isohyetal method which gives the lower figures. This is in part due to the poor data available for the events, which although sufficient for constructing isohyetal maps to illustrate the general movement of the storms across the catchment, are not adequate for detailed analysis of the storms by the isohyetal method.

One major problem in estimating areal rainfall for the catchment is that the available raingauges are all located at low elevations and hence may not represent rainfall over the higher portions of the catchment. It is clear that there is a relationship between rainfall and elevation, and this orographic effect is one that should ideally be used explicitly to adjust the observed low level rainfall records in order to derive better estimates of areal rainfall over the upland parts of the Hung Shui Kiu catchment. However, little reliable research has been undertaken on this topic in Hong Kong, and no readily available relationship between short duration storm rainfall and elevation seems to exist. BMC (1992c) plotted 30-year long-term annual average rainfalls against elevation and derived a relationship, but the scatter was very high and the report commented that altitude is not the only factor affecting rainfall, and that aspect must also be significant.

However, any relationship between annual average rainfall and gauge elevation is unlikely to be of much use, as to an extent higher rainfall at high altitudes is just as likely to be a result of the generally higher number of rain days at these sites rather than truly reflecting solely the orographic effect. Such a relationship would have little relevance to the short duration storm rainfalls of interest for drainage studies. Consequently a number of hourly and daily rainfall records from throughout Hong Kong were plotted against gauge elevation in order to search for an underlying relationship. The data used were hourly rainfalls for the 22 July, 1994 between 02:00 and 05:00 and between 10:00 and 13:00 hours, daily rainfalls for the 22nd, 23rd and 24th July, and the 3-day total for 22 to 24 July, 1994. The results of these trials are shown in Figures 10 and 11, where the marked scatter of points about the fitted regression lines is apparent.

One additional piece of work which DSD could usefully commission would be a thorough analysis of all available short duration rainfall from a large number of gauges throughout the Territory. It is possible that such a study might provide a means of estimating high level rainfalls more reliably than possible at present.

Considering the six separate hourly rainfalls studied, the fitted lines do generally show an increase of rainfall with elevation in three cases (03:00 to 04:00, 04:00 to 05:00 and 10:00 to 11:00 hrs), but the other three showed no such trend, and the relationships for 11:00 to 12:00 and 12:00 to 13:00 hours have a downward gradient. In some cases the correlation coefficient was just acceptable, with two having values exceeding 0.6, however three other cases had coefficients of less than 0.05, which implies no relationship between rainfall and elevation at all.

The daily study was a little more successful, although the fitted trend lines only have a significant correlation coefficient of 0.82 in the case of the 3-day total. The daily relationships were generally weak, and whatever gradients there were to the trend lines are created by the generally very high rainfalls on Tai Mo Shan.

Overall, the study of hourly, daily and annual rainfall against gauge elevation has shown that whilst there is generally an overall trend for rainfall to increase with altitude, the relationships are generally weak and other factors, such as the spatial and temporal distribution of storm rainfall have a marked effect on observed rainfalls at any point in time. The brief study was unable to demonstrate a sufficiently strong linkage between storm rainfall and altitude to provide a sufficiently robust model for estimation of rainfall over the upper catchment. To a large extent however, the iso-percentile method used to of estimate areal rainfall does take into account altitude effects. By using mapped average annual rainfall over the catchment as a scaling factor, the method recognises the fact that rainfalls are generally higher over the upper catchment than at lower elevations, and the model implicitly allows for the altitude effect.

The following sections describe in detail the derivation of the 5-minute catchment average rainfalls for the two calibration events.

The 5 November, 1993 event

Figure 12 shows the hourly stage data at D07 Shek Po Tsuen and the hourly rainfall data at each of the five raingauges for 5 November, 1993. The event is a simple, single-peaked event, and there is much similarity between the rainfall profiles recorded at each of the raingauges. The match between the rainfalls and the recorded stage is good.

The 5-minute catchment average rainfall profile for 5 November, 1993 was derived as follows:

- The hourly rainfall data for 5 November at the five raingauges were used to derive an hourly catchment average rainfall profile using the iso-percentile method. Figure 13 shows the hourly rainfall data at each of the five raingauges together with the hourly catchment average rainfall.

- The hourly catchment average data were then desegregated to 15-minute totals by scaling the 15-minute totals at raingauge No174 by the ratio of the catchment average rainfall to the rainfall at raingauge No174 for each hour in turn i.e. the four 15-minute totals at raingauge No174 for 00:00, 00:15, 00:30 and 00:45 would be scaled by the ratio of the hourly catchment average rainfall at 00:00 to the rainfall at raingauge No174 at 00:00, and so on. Figure 14 shows the 15-minute catchment average rainfall plotted with the hourly stage data.

- Finally, the 15-minute catchment average data were desegregated to 5-minute totals by simply dividing the 15-minute totals into three equal depths.

The choice of raingauge on which to base the sub-hourly profile was not critical for this event, since the raingauges had very similar profiles, and No174 was chosen simply because 15-minute data were available for the whole day.

The catchment average rainfall profile shows a total rainfall depth of 288.64 mm to have fallen in 9.5 hr, and using Lam & Leung (1994) (Table 10a) this corresponds to a return period of 13 years. The most intense 4 hr contains a depth of 219.20 mm, and using Lam & Leung (1994) (Table 6) this also corresponds to a return period of 13 years. However, the most intense 1 hr rainfall is 114.35 mm which corresponds to a return period of approximately 136 years.

The 22 July, 1994 event

Figure 15 shows the hourly stage data at D07 Shek Po Tsuen and the hourly rainfall data at each of the five raingauges for 22 July, 1994. The event is a complex, multi-peaked event with three defined peaks. Furthermore, the rainfall profiles recorded at each of the raingauges show small, but significant differences. No174 has similar rainfall values for each of the three peaks, whilst N07 has a maximum on the first peak, and N12 a maximum on the last peak and best matches the recorded stage.

The 5-minute catchment average rainfall profile for 22 July, 1994 was derived as follows:

- The hourly rainfall data for 22 July at the five raingauges were used to derive an hourly catchment average rainfall profile using the iso-percentile method. Figure 16 shows the hourly rainfall data at each of the five raingauges together with the hourly catchment average rainfall.

- The hourly catchment average data were then desegregated to 5-minute totals by scaling the 5-minute totals at raingauge N12 by the ratio of the catchment average rainfall to the rainfall at raingauge N12 for each hour in turn i.e. the twelve 5-minute totals at raingauge N12 for 00:00, 00:05, 00:10,.....00:55 would be scaled by the ratio of the hourly catchment average rainfall at 00:00 to the rainfall at raingauge N12 at 00:00, and so on. Figure 17 shows the catchment average rainfall summed to 15-minute totals plotted with the hourly stage data.

The choice of raingauge on which to base the sub-hourly profile was more important for this event, because of the differences between the individual raingauge profiles. N12 was chosen because it best matched the recorded stage and fortunately 5-minute data were available for the whole day.

The catchment average rainfall profile shows a total rainfall depth of 425.49 mm to have fallen in 14.75 hr, and using Lam & Leung (1994) (Table 10a) this corresponds to a return period of 38 years, confirming the estimate of DSD (1994c). The most intense 4 hr contains a depth of 220.17 mm, and using Lam & Leung (1994) (Table 6) this corresponds to a return period of only 13 years The most intense 1 hr rainfall was 80.00 mm which corresponds to a return period of 14 years, indicating that the unique aspect of this event was the duration, rather than a particularly intense rainfall and highlighting the difficulties of assigning return periods to multi-peaked events.

Comments

In the derivation of the catchment average rainfall profiles for the calibration events, the reason why two slightly different approaches were adopted was explained previously in our responses to the comments on the second report. However, to summarise, from the information supplied by DSD and RO during the data collection phase of the study, the Consultants understood that no 5-min data were available for the 1994 event. No174 was therefore used as the raingauge on which to base the sub-hourly profile for both events because: (i) it was the only gauge for which sub-hourly data were available for the 1994 event, (ii) it was the only gauge for which sub-hourly data were available for the entire day for both events, and (iii) the Consultants wished to be consistent about which raingauge they used for this purpose and how they derived the catchment average rainfall profiles. Therefore, 5-min profiles were derived for both calibration events based on gauge No174 and disaggregation of 15-min catchment rainfalls into three equal 5-min rainfalls. For the 1993 event the choice of No174 was not critical since all the raingauges had very similar profiles. However, it was acknowledged that for the 1994 event the individual raingauge profiles were quite different with N12 best matching the recorded stage. However, after all the work had been done to derive the 5-min profiles, DSD supplied some previously undiscovered 5-min data for the 1994 event. These data were supplied late, and the Consultants could have ignored them. However, since N12 provided a better fit to the recorded stage for the 1994 event, and data were available for the whole day, the Consultants reworked the catchment rainfall for this event. It was not felt necessary to subsequently rework the catchment rainfall for the 1993 event because: (i) No174 provided an adequate fit to the recorded stage and compared well with other raingauges, and (ii) the disaggregation of the 15-min catchment rainfalls into three equal 5-min rainfalls provided an adequate fit to the recorded stage.

3.3.3 Estimation of flows at irrigation weir upstream of TM-YL Highway

Some peak level data (i.e. flood marks) are available for each of the two key flood events, and from these it was possible to estimate peak flood flows at the irrigation weir located just upstream of the TM-YL Highway. This weir is a round-nosed broad-crested weir, and weir dimensions were taken from DSD drawing NDD 5157D and site inspections.

The weir has a crest elevation of about 17.92 mPD (the crest level varies from 17.90 mPD to 17.93 mPD), and is 39.7 m long. The upstream nose has a radius of about 50 mm, and the downstream edge a larger radius of about 300 mm. The crest width, from upstream to downstream, is 0.84 m. It was difficult to establish the depth of the upstream face of the weir due to standing water upstream at the time of the site visit, but it is certainly greater than 2 m, and hence there would be no effect of upstream bed levels on flows.

Flow over the weir may be computed using the standard round nosed broad crested weir formula (ISO, 1977; ISO, 1982) as:

$$Q = 0.544 * Cd * b * g^{0.5} * h^{1.5} \qquad \text{Eqn (7)}$$

where Q is weir discharge (m^3s^{-1}), Cd is a weir coefficient, b is crest length of the weir (m), g is the gravitational acceleration constant (= 9.81 m s^{-2}), and h is head over the weir (m).

ISO (1982) gives a Cd value of 0.864 for a range of conditions where:

$$0.1 \leq h/L \leq 0.4$$

and $0.15 \leq h/P \leq 0.6$

where L is width of weir crest from upstream to downstream face (m), and P is depth of upstream face of the weir (assumed to be 2 m).

The assumption is that the crest width will be large relative to the head in order to ensure horizontal flow over the crest, but this is not the case for either of the two observed floods. For the 22 July, 1994 flood, a paint mark on the right hand wing wall of the weir indicates peak stage, and this was 19.06 mPD, or a head of 1.14 m over the weir. Thus the ratio h/L was 1.36 for this event. For the 5 November, 1993 event, there does not appear to be an observed peak level at the irrigation weir itself, and the only useful recorded peak is one of 20.11 mPD on a pig house some 60 m or so upstream. For the July, 1994 flood, a level of 19.61 mPD was recorded on a house wall at approximately the same point. Thus given that the water level at the weir for this event was 19.06 mPD, the water level fall was 0.55 m to the weir. Under the assumption that the water level fall was the same during the November, 1993 event, and that the pig house and house walls referred to are close together, the water level at the weir in November, 1993 would have been approximately 19.56 mPD, or 1.64 m head over the weir. The ratio of h/L in this case would be 1.95, which is well outside of the ISO range for the standard weir coefficient of 0.864.

It should be noted that the discharges may have been estimated using a peak stage estimate that is not a true head over the weir because of the lack of good observed stage data for the flood events. However, best use has been made of the available data, given that it is not believed that the upstream heads measured on the wall of either the house or pig house are valid estimates of upstream stage. The channel bed slope upstream of the irrigation weir is very steep, and hence these flood marks are unlikely to provide accurate representations of upstream head. Whilst the figures utilised may not be entirely accurate, they are considered more realistic that those based upon these upstream flood marks.

ISO standards give a simple formula for estimating Cd for values of h/L beyond the normal range, which is:

$$Cd = 0.191 * (h/L) + 0.782 \qquad \text{Eqn (8)}$$

This formula yields values of the weir coefficient Cd of 1.15 for 5 November, 1993 and 1.04 for 22 July, 1994.

Applying these weir coefficients and heads of 1.64 m and 1.14 m respectively yields the following peak flow estimates for this weir:

5 November, 1993	h = 1.64 m	Peak = 166 m^3s^{-1}
22 July, 1994	h = 1.14 m	Peak = 86 m^3s^{-1}

The peak estimate for the November, 1993 event lies at the top end of the various flood estimates undertaken by DSD (undated), which ranged between 84 m^3s^{-1} and 175 m^3s^{-1}. However, the estimated head over the weir may have been significantly overestimated, because of the need to very roughly extrapolate where the flood mark at the irrigation weir would have been. This overestimation is thought to potentially be as much as 10 %

to 20%, and so the discharge of this event may well have been only $120m^3s^{-1}$ to $140m^3s^{-1}$. Hence the calibration of the model is based on the July, 1994 event for which more data are available, and validation of the model is based on the November, 1993 event.

There would be considerable merit in DSD, or a contractor, undertaking a number of current meter measurements during high flows in order to assess flows over the irrigation weir more reliably. Ideally these measurements should be taken in the rectangular channel because it has a stable, known cross-section and the flow will be reasonable uniform. There are also a number of bridges which will provide good platforms from which to suspend the current meters.

3.3.4 Derivation of flow hydrographs

Inflow hydrographs for each of the five main subcatchments were derived for the November, 1993 and July, 1994 calibration events. In each case, the derived catchment average rainfall profiles were convoluted with the unit hydrograph and losses model based on the estimated model parameters to produce the calibration event inflows. The inflows for subcatchments 6 and 7 were then scaled from that for subcatchment 5 on the basis of area.

Figure 18 and 19 show the calibration inflow hydrographs from catchments 1 to 5 for the event of 5 November, 1993 based on the proportional loss method, whilst Figures 20 and 21 show the same based on the loss rate method.

Figures 22 and 23 show the calibration inflow hydrographs from catchments 1 to 5 for the event of 22 July, 1994 based on the percentage runoff method, whilst Figures 24 and 25 show the same based on the loss rate method.

For the November, 1993 event, the peak flows for catchment 1 are 117 m^3s^{-1} from the percentage runoff method and 120 m^3s^{-1} from the loss rate method. These compare favourably with the lower estimate of the peak flow at the irrigation weir. For the July, 1994 event, the peak flows for catchment 1 are 85 m^3s^{-1} from the percentage runoff method and 86 m^3s^{-1} from the loss rate method, and these are exactly the same as the peak flow estimated at the irrigation weir. In view of the good reproduction of the estimated observed flows, no further adjustment of the hydrological model was carried out.

3.4 Generation of design event flows

The design event inflows are generated in the same way as the calibration event inflows, except that whereas the calibration flows were derived from observed rainfall events, the design flows are derived from synthetic, design rainfall events.

3.4.1 Design rainfalls

Lam & Leung (1994) provide extreme depths of rainfall corresponding to various return periods for RO Headquarters in Kowloon, Yuen Long RG Filters, Tai Po Tau Treatment Works and Tai Lung Farm. It is the first two of these raingauges which are of most interest to the current study. For RO Headquarters figures are given for durations between 15 seconds and 31 days, whilst for Yuen Long RG Filters figures are given for durations between 15 minutes and 31 days. The results are derived from annual maxima data, 100 years from RO Headquarters, but only 11 years from Yuen Long RG Filters. The lowest return period for which results are given is two years, and the highest is 1000 years.

There appears to be no great merit in analysing significant historical events from the autographic raingauge charts available at No17 Yuen Long RG Filters within the short time-scale of the current study since RO have recently published the results of such analysis for the period 1981-90 (Lam & Leung, 1994). Furthermore it is unclear how the results of such a study might be used in deriving flood estimates of required return periods. Experience shows that the most sensible design flood estimates are produced using generalised storm profiles, such as those derived by RO. Such a design storm profile is likely to be somewhat more peaky than most historically observed storms, but this yields conservative, or safe, peak flood estimates.

The objective of the current study was to generate design flows from rainfalls of return periods 2, 10, 50 and 200 years. In order to determine the critical design storm duration, a selection of durations i.e. 1, 2, 4 and 6 hours, were chosen. In line with the calibration rainfall data, the design rainfall data again used a time interval of 5 minutes.

Lam & Leung's results for Yuen Long RG Filters were plotted up on log-log paper as depth-duration curves for particular return periods, as shown in Figure 26. The 5-minute rainfall statistics for Yuen Long RG Filters were derived by extrapolating the lines back to a duration of 5-minutes. The equations fitted to the lines for the specified return periods were:

2-yr $\quad \log \text{Depth} = 0.5233 \log \text{Duration} + 0.7863 \quad R^2 = 0.9976$

10-yr $\quad \log \text{Depth} = 0.6669 \log \text{Duration} + 0.7229 \quad R^2 = 0.9997$

50-yr $\quad \log \text{Depth} = 0.7255 \log \text{Duration} + 0.7341 \quad R^2 = 0.9998$

200-yr $\quad \log \text{Depth} = 0.7568 \log \text{Duration} + 0.7554 \quad R^2 = 0.9998$

The results were replotted as depth-frequency curves for particular durations, including the 5-minute duration, as shown in Figure 27.

The equations were then used to derive the rainfall profiles for the four return periods for storm durations of 1, 2, 4 and 6 hours. These are tabulated in Table 8, together with the corresponding figures from RO Headquarters, and it can be seen that the results derived by the current study tend to be on the conservative side. For instance, for the design storms of duration 4-hours, the derived results are 7 % lower than RO for the 2-year event, but 10 %, 13 % and 14 % higher for the 10, 50 and 200 year events respectively. Adopting the derived results, rather than those for RO, introduces a small safety factor into the design flood estimates.

3.4.2 Design flows

In order to determine the critical duration, it was necessary to compare the design hydrographs derived from storms of various durations. Figure 28 shows the 2-year design inflow hydrographs for catchment 1 derived from storms of 1, 2, 4 and 6 hours duration, and it is clear that beyond the 2-hr duration there is very little difference in catchment response, apart from the length of the "shoulders" on the hydrograph, during which time the catchment will wet-up. Therefore, a design duration of 4-hr was chosen in order to incorporate the central peak, but also allow a reasonable wetting-up period.

Inflow hydrographs for each of the five main subcatchments were derived for the four design events. In each case, the 4-hr design rainfall profiles were convoluted with the unit hydrograph and losses model based on the calibrated model parameters to produce the design event inflows. The flows for subcatchments 6 and 7 were then scaled from that for subcatchment 5 on the basis of area. Figures 29 to 33 show the design inflow hydrographs from catchments 1 to 5 for return periods of 2, 10, 50 and 200 years based on the percentage runoff method.

4 HYDRAULIC MODELLING

4.1 Selection of hydraulic model

The requirements for the computational hydraulic model are as follows:

- simulation of steady state and unsteady open channel sub and super critical flow using fixed boundary hydraulics;
- simulation of in-channel structures including weirs, bridges and culverts;
- simulation of energy losses at channel constrictions and expansions;
- simulation of super-elevation and energy loss at bends and formation of cross waves;
- explicit representation of out-of-bank flow including spilling to and from the channel, storage on the floodplain and longitudinal flow along the floodplain; and
- simulation of looped and dendritic networks.

Selection of a single computational hydraulic model which has the full functionality identified above is not possible as no such model is currently commercially available. There are a number of commercially available river modelling packages which can satisfy many of the above requirements and of these MIKE11 could be considered the most prominent due to its application in other similar studies in Hong Kong. Unfortunately MIKE11 is not suitable for this study because of it will automatically add Preissmann's slots to prevent the flow becoming super-critical. The user has no control over this feature which can lead to significant increases in the model flow area at a given section and thus an over estimate of the channel conveyance. Instead it is recommended that ISIS, the 1-dimensional computation river model developed by the HR Wallingford / HALCROW joint venture, is adopted for the study. ISIS will represent all the above hydraulic features except super-elevation at bends and the formation of cross waves. Calculation of the magnitude of these phenomena will be undertaken outside the program but included in the simulations by using the software's user defined headloss feature or by increasing the channel roughness coefficients.

4.1.1 Description of ISIS

ISIS is a system for simulating flow and water quality in canals, rivers and estuaries which has been developed as a joint venture between H R Wallingford and Sir William Halcrow & Partners. ISIS combines the skills and experience of both these organisations to offer proven hydraulic and water quality modelling capabilities with a state of the art user environment. ISIS is derived from the SALMON and ONDA hydraulic and water quality engines and benefits from three decades of development and application on both simple and complex open channel systems throughout the world.

The hydrodynamic module, used in this study, employs the Saint-Venant Equations for shallow water open channel flow reduced to 1-Dimension which are as follows:

the conservation of mass or continuity equation;

$$\frac{\partial Q}{\partial x} \cdot \frac{\partial A}{\partial t} \cdot q \qquad \text{Eqn (9)}$$

where Q is flow in m^3s^{-1}, A is cross-sectional area in m^2, q is the lateral inflow in m^2s^{-1}, x is the longitudinal channel distance in m and t is the time in seconds

and the momentum conservation or dynamic equation;

$$\frac{\partial Q}{\partial t} \cdot \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right) \cdot gA\frac{\partial H}{\partial x} \cdot g\frac{AQ|Q|}{K^2} \cdot q\frac{Q}{A}\cos\alpha \cdot 0 \qquad \text{Eqn (10)}$$

where H is the water surface elevation above datum in m, β is the momentum correction coefficient, α is the angle of inflow and K is the channel conveyance calculated from:

$$K^2 - \frac{A^2R^{\frac{4}{3}}}{n^2} \qquad \text{Eqn (11)}$$

where n is the Manning's roughness coefficient, R is the hydraulic radius (A/P) and P is the wetted perimeter.

Thus the stage is assumed to be constant across a given section and the velocity is assumed to be constant across a particular section and through its depth. The system uses the Preissmann four-point implicit finite difference scheme to solve the channel equations and the matrix is inverted using a powerful sparse matrix solver.

The software includes a wide range of standard hydraulic units to model differing channel features. The selection of units to represent the key features of the Hung Shui Kiu channel is discussed later.

4.1.2 Data sources

The basic topographic data for the channel and the dimensions of the hydraulically significant structures has been taken from the as built drawings supplied by DSD and HyD. The hydrological inputs to the model were derived from observed rainfall profiles. converted to discharge hydrographs using the rainfall runoff model described in Chapter 3. The channel roughness coefficients, structure discharge coefficients and headloss coefficients were derived by a combination of observations of the condition of the channel, theoretical values and calibration of the model against the historical flood events of 5 November, 1993 and 22 July, 1994

4.1.3 Model schematic

The hydrodynamic model extends from downstream of the irrigation weir to the river gauge at Shek Po Tsuen. A total of 99 hydraulic units (or nodes) were used to represent the channel features including: river cross-sections, inflows, Bernoulli headloss units. conduit unit spills and boundary conditions. The upstream boundary condition was provided by the discharge hydrographs generated for subcatchment 1 by the hydrological model. The downstream boundary condition was provided by the telemetered stage hydrographs at Shek Po Tsuen for the calibration events, supplied by DSD. For the design events the water levels at Shek Po Tsuen will not be known so a stage discharge rating curve has been developed at Shek Po Tsuen, based on the results of the calibration exercise and conveyance properties of the trapezoidal channel. Figure 61 gives a copy of the rating curve used for all the design events. The hydraulic model uses this curve to calculate the water level at the downstream based on the discharge at any particular time step during the simulation.

The general cross-sections were taken from sections 1 - 15 of the as built drawings. Unfortunately, these were not at the required locations to identify all the hydraulically significant features of the channel so addition sections were created based on these but with the elevations modified according to the channel slope. The as built sections do not extend to limits of the out-of-bank areas inundated during the past flood events so these were extended using the available contour and spot height information on the as built drawings. This was to ensure that any attenuation due to storage effects and the conveyance of the out-of-bank areas were included within the model. Thus for model test of situations where the out-of-bank areas are separated from the channel by the metal railings, a single computation unit was used to describe both channel and floodplain. The available storage, in both the channel and floodplain, was determined by multiplying the average cross-sectional area at adjacent sections, for a given depth, by the distance between them. Figures 62 to 65 give some sample cross-sections used in the hydraulic model. For situations where the metal railings are replaced by a concrete wall the channel and floodplain were represented by separate computational units. In this case the channel cross-sections were truncated at the top of the concrete wall and the floodplain was represented by reservoir units. The flow between the channel and floodplain was then modelled using spill units which treat the top of the wall as a variable crested weir. The floodplain reservoir units were linked together using spill units to allow flow to pass along the floodplain.

The footbridges and vehicular bridge were included within the model as BERNOULLI HEADLOSS UNITs. This unit calculates the headloss according to Bernoulli's equation:

$$h_1 - h_2 = \frac{Q^2}{2g}\left(\frac{1}{A_1^2} - \frac{1}{A_2^2} \pm \frac{K_{12|21}}{A_{1|2}^2}\right) \qquad \text{Eqn (12)}$$

where h_1 and h_2 are the upstream and downstream water levels, A_1 and A_2 are the upstream and downstream flow areas, Q is the discharge and $K_{12|21}$ is the loss coefficient for forward or reverse flow.

The loss coefficients were set to zero for stages below the bridge soffits to ensure no headloss occurred until each bridge was surcharged. It is not possible to select a standard loss coefficient for the bridges once surcharged because of the complicated flow through the hand rails and around the bridges. Thus these coefficients have been treated as additional calibration parameters. The general layout of the model is shown in Figure 34.

4.1.4 Representation of the channel transitions

Three significant channel transitions occur with the reach of interest: the transition from the steep trapezoidal channel under the TM-YL Highway to the 5.5 m wide rectangular channel (inlet bay); the transition from the 5.5 m wide rectangular channel to the 5.5 m base width trapezoidal channel (T1); and the transition from the trapezoidal channel back to a rectangular channel on the approach to the Castle Peak Road Culverts (T2). It is quite obvious, particularly for lower discharges, that the flow at the inlet to the rectangular channel is super-critical. Thus this transition has been modelled using a SPILL UNIT. The SPILL UNIT calculates the flow over a "jagged" weir, in this case a cross-section of the wing walls and channel bed at the end of the steep, trapezoidal channel. The unit supports free and drowned flow in both the forward and reverse direction by integrating the weir flow equation over each segment of the cross-section. A detailed description of this method can be found in Evans & von Lany (1983). The user must specify a discharge coefficient and the drowning ratio for the unit. Again no standard values are readily available for such an arrangement so these coefficients were treated as calibration parameters.

The effects of transitions T1 and T2 are accommodated in the normal solution of the Saint-Venant Equations. However, additional cross-sections were introduced at each transition to ensure that the change in cross-sectional area between adjacent nodes did not exceed the model stability requirements.

4.1.5 Channel maintenance access ramps

There are two channel maintenance access ramps which enter the rectangular reach of the Hung Shui Kiu Channel. The first, at chainage 89.0 m, enters from the right bank along with the Tan Kwai Tsuen Channel and discussed later. The second (AR1) enters from the left bank at chainage 564.0 m and is a cause of considerable concern.

The ramp is aligned parallel to the channel and the downward slope is from downstream to upstream. Immediately upstream of the end of the ramp the channel has been widened by some 5.5 m to allow maintenance vehicles to manoeuvre off the end of the ramp. Unfortunately, this widening occurs on the left bank at the exit of the sharp right hand bend at chainage 468.0 m. At channel bends the higher velocity flow moves away from the centre and tends to follow the outer bank of the channel. The flow around bends is discussed in greater detail in the following section but of significance here is the fact the high velocity flow will directed on the access ramp. As the ramp rises out of the channel it will force the flow sharply to the left, back into the main channel.

This rapid change in the direction of flow will not only require a considerable force, introducing an additional energy loss, but will also introduce excessive turbulence, flow separation and recirculation. These phenomena will further contribute to the energy loss caused by the access ramp.

Site visits to the channel revealed significant sediment and trash deposits on the right (inside) half of the channel bed at the exit from the bend and also at the left side of the channel adjacent to the end of the access ramp. These deposits indicate lower velocities than in other parts of the channel and support the theory that flow separation and recirculation is occurring. During the second site visit with DSD, representatives of the Local Board, the residents and Legco Members, the separation and recirculation was clearly observed when a passing truck splashed muddy water into the channel just upstream of the bend. Householders living on the right bank opposite the access ramp also reported that during the flood events the flow was directed back across the channel, over the right bank and into their homes by the access ramp.

These phenomena are truly 3-dimensional and cannot be represented explicitly within a 1-dimensional model. However, their effects ie energy loss, can be included by artificially raising the roughness coefficient employed in this reach. There are no easy theoretical methods for determining what this increase should be. However, there is a flood mark on the left bank immediately upstream for the bend, recorded during the July, 1994 event, which was used to select the Manning's n during the calibration exercise.

4.1.6 Energy loss at bends

This section describes some detailed investigations of the energy loss at bends. It has been included to provide justification for the rather high values of Manning's n used in the hydraulic model.

Channel bends introduce a transverse velocity component in addition to the major velocity component normal to the channel cross-section. These transverse velocities result in secondary flow in the plane of the cross-section as illustrated in Figure 35. These secondary currents are mainly due to:

- friction on the channel walls, which causes higher velocities near the centre of the channel than those near the channel walls;
- the centrifugal force which deflects the particles of water from the straight-line motion;
- a vertical velocity distribution which exists in the approach channel and thus initiates spiral motion in the flow; and
- in sharp bends, the separation of the flow and the formation of eddy zones.

This complex flow pattern introduces additional turbulence in the flow which cause an additional energy loss above that which would be expected in a straight channel. These transverse currents do not decay instantly as the channel straightens but may persist for some distance downstream. This may also increase energy loss in the downstream reach. A number of studies have been carried out to investigate the impact of bends on flow resistance using a combination theoretical calculations, experimental testing and analysis of field data. In each of these studies the total energy over a channel reach may expressed in terms of the velocity head of the flow such that:

$$h_f = f\frac{V^2}{2g} \qquad \text{Eqn (13)}$$

where h_f is the head loss, V is the mean velocity and f is the resistance.

For channel reaches with bends f may be considered to have two components f_r, the resistance due the channel bed and bank roughness, and f_b, the additional resistance caused by the bend. The coefficient f may be related to the Manning's roughness coefficient by the following:

$$n = \sqrt{\frac{fR^{\frac{4}{3}}}{2lg}} \qquad \text{Eqn (14)}$$

Two of these methods have been applied to the Hung Shui Kiu Channel to gain an insight into the potential energy losses at the four most severe bends in the rectangular channel. The geometric properties of these four bends are given in Table 9.

Leopold, Bagnold, Wolman & Brush - 1960.

Leopold *et al.*(1960) investigated the effect of channel bends by comparing flow in flumes with straight and sinuous channels. The effect of varying the following parameters was investigated:

- depth - two flow depths were used giving aspect ratios (surface flow width/depth) of 7 and 5.5;
- meander wavelength and amplitude; and
- slope.

Leopold *et al.* showed that for low Froude numbers the increase in resistance due to channel curvature is linearly related to the width to curvature ratio, B/r_c where B is the surface width of the flow and r_c is the radius of curvature of the centre line of the channel. This relationship is reproduced in Figure 36 which compares the ratio f_b/f_r with the ratio B/r_c, where f_b is the energy loss associated with the channel bend and f_r is normal energy loss associated with a similar straight channel. Initial examination of Figure 36 using the values of B/r_2 for the four bends given in Table 9 suggests that there should be no significant additional energy loss at any of the four bends. However, further work by Leopold *et al.* suggests that at Froude numbers above a critical threshold the increase in resistance is significantly greater than that predicted by the linear relationship. The critical Froude number, F_c, for a particular channel may be predicted by the following equation:

$$\frac{\overline{F_c}^2}{2}\left(\frac{B}{r_c} - 1\right) - 1 \cdot 1.5\overline{F_c}^{\frac{2}{3}}\left(1 \cdot \frac{\overline{F_c}^2 B^{\frac{2}{3}}}{2r_c}\right) \cdot 0 \qquad \text{Eqn (15)}$$

The critical Froude numbers for each of the four bends as well as the Froude number associated with the design flow are given in Table 9 and from this it can be seen that although the Froude number for the design flow is marginally less than the critical Froude numbers for each bend, those experienced during the two recent flood events are significantly above the critical threshold. Leopold *et al.* considered a dimensionless factor which accounted for the additional energy loss experienced for flows above the critical Froude number but unfortunately it is not clear how this factor is used to determine a modified roughness coefficient. However this work does indicate the potential for significantly greater energy losses to occur at the channel bends than would be predicted using standard roughness values.

Toebes & Sooky - 1967

Toebes & Sooky undertook an experimental comparison of straight and meandering channels and arrived at similar conclusions to Leopold *et al.*, that the energy loss at bends is linear up to a critical Froude number after which significant additional energy losses occur. However, the linear relationship derived by Toebes & Sooky is based solely on the hydraulic radius R and is as follows:

$$f_b = 2.1\, f_r\, R \qquad \text{Eqn (16)}$$

Where R is the hydraulic radius in feet.

Using Eqn (16) and the design values of depth area and Manning's n, an estimate of f_b for the four most severe bends can made and from this, a modified value of Manning's n determined. These results are given in Table 10 in addition to a comparison of the design headloss around the bend with that predicted by the modified value of Manning's n. This suggests that the total headloss around the bends may well be in excess of 5.0 m rather than just under 0.5 m as assumed in the original design.

Again Toebes & Sooky identified that above a critical threshold the energy losses increased above those predicted by the linear relationship. For all the experiments undertaken by Toebes & Sooky the critical Froude number occurred at slopes of 0.003 ie approximately the same slope as that of the Hung Shui Kiu Channel. This agrees with the estimate of F_c calculated according to Leopold *et al.* and indicates that the energy losses could be higher than those calculated using the linear relationship.

Summary

Although the two approaches described above agree on the broad concepts of energy loss at bends, they do not give similar estimates for the expected additional energy for the geometric configurations bends of the Hung Shui Kiu Channel. In general Leopold *et al.*'s method is preferred because it is simple to use and the geometries of their experimental channels are similar to those found in natural rivers. Toebes & Sooky's work is not preferred as a method for estimating energy losses in natural channels because the slopes and aspect ratios used in their experiments are not normally found in natural channels. However, they are similar to the characteristics of the Hung Shui Kiu Channel and it is therefore appropriate to consider their work in this study. Thus the modified estimates of Manning's n given in Table 10 were used as the initial estimates in the hydraulic model. These estimates were then refined by calibration against observed water levels from the 1993 and 1994 flood events (discussed later).

4.1.7 Tributary and highway inflows

The hydrological investigations have revealed there are six tributary inflows to Hung Shui Kiu Channel in addition to the main inflow over the irrigation weir (sub-catchments 2-7 as shown on Figure 6). Of these, the inflows from sub-catchments 2, 5, 6 and 7 are believed to have no significant effect on water levels in the channel other than that associated with the increase in the channel flow. As such these have been treated as discrete inflows. This reflects the presence of a number of drainage outfalls in the banks of the channel. Inflows from catchment 2 were modelled as a discrete inflow entering the steep trapezoidal channel under the TM-YL highway via the pipe culverts on the right bank. The inflow from catchment 5 was divided equally between the two outfalls on the right bank at approximately chainages 402 m and 1324 m. No data on the location of the outfalls for catchments 6 and 7 were available and no evidence of any outfalls on the left bank was found during the site visits. Thus these inflows were modelled as discrete inflows entering the channel downstream of the Castle Peak Road culverts.

The two that may have some addition impact are the highway runoff, sub-catchment 4 which enters on the right bank just downstream of the TM-YL Highway bridge, and the inflow from the Tan Kwai Tsuen Channel, sub-catchment 3 which enters from the right bank at chainage 89.0 m.

Although these inflows are small in comparison to the main channel flow, they may cause turbulence and loss of momentum as the flow direction changes on entering the main channel. In the case of the highway inflow, the outfall into the channel includes an array of baffles, designed to reduce the energy of the inflow and avoid significant disturbance of the main channel flow. Given this, and the fact that there is ample channel capacity at the location of the inflow, no further energy loss was included in the model for this inflow.

As mentioned previously, the Tan Kwai Tsuen Channel joins the Hung Shui Kiu Channel at the same location as the first access ramp. The total width of the combined channel and access ramp is some 9.0 m, significantly wider than the main channel, and is almost perpendicular to the orientation of the main channel. This may lead to two potential sources of energy loss. Firstly, high velocity flow from the tributary may enter the main channel unchecked causing significant turbulence and secondary currents. Secondly, flow from the main channel may expand into the entrance to the tributary inducing flow separation and recirculation. Both occurrences are likely to result in higher energy losses than would be expected from solely considering the channel roughness.

The site visits revealed that a sizable mound of sediment and trash had accumulated at the entrance to the tributary. The residents stated that the mound was much larger but local people had been removing the sand for their own use. The presence of the sediments and trash confirms that there is some reduction in the flow velocities at this location, possibly due to recirculation. Discussions with the residents also revealed that there was much turbulence at this location during the two flood events and they believed that this was the location where the water level first rose above the bank top level. Thus a BERNOULLI HEAD LOSS unit was included in the model to simulate these additional energy losses. The loss coefficients were treated as calibration parameters and determined by comparison of the model results with observed levels recorded during the July, 1994 flood event.

4.1.8 Effects of sediment and blockage

The effects of sediment and blockage of the channel by rocks and trash is difficult to assess with knowledge of the size and concentration of the actual particles being transported. The Hung Shui Hang Reservoir and the irrigation weir, immediately upstream of the rectangular channel, could be expected to trap significant proportions of mobile material and reduce potential problems downstream. However, the villages and DSD have reported large boulders moving down the channel during the flood events which will have had some impact on the hydraulic performance of the channel. The site inspections have revealed some build up of sediments and trash, particularly at the channel expansions for the access ramp and tributary confluence and on the inside of the channel bends. Unfortunately, it is not possible to make any clear judgements as this limited information might affect the value of Manning's n for the channels. Thus the approach adopted in this study has been to treat the Manning's n as an all embracing energy loss coefficient. By selecting n with reference to the performance of the channel during the 1993 and 1994 flood events it may be assumed that additional losses due to sediment and blockage have been accounted for.

During the 1994 event a truck was washed into the channel and became wedge upstream of a footbridge. This undoubtably caused a major blockage and will have significantly raised water levels immediately upstream. Unfortunately no information is available as to the time that this occurred so it is not possible to assess whether this incident influenced maximum water levels. However, the blockage caused by the truck can not be considered a cause of the flooding as water levels must have been significantly above bank top to be able to wash the truck into the channel.

4.1.9 Modelling of Castle Peak Road culverts

The Hung Shui Kiu Channel passes under Castle Peak Road through two parallel 4.0 m wide box culverts. The upstream entry to the culverts includes a transition from the 5.5 m base width trapezoidal channel to a rectangular channel which has been discussed earlier. These have been modelled using a CONDUIT unit, designed for closed conduits, which incorporates full solution of the Saint-Venant Equations. Pressurised, or surcharged, flow is accommodated by the model introducing an infinitesimally thin, frictionless slot in the soffit of the culvert. These means that the water level calculated by the program becomes the piezometric level but the conduit area and conveyance remain unchanged.

4.2 Simulation of historic events

Two historic flood events were simulated as part of the model calibration and verification process. The calibration parameters discussed above were adjusted to fine tune the model such that it closely reproduced the observed water level data for each of the two events. No calibration criteria were specified in the Study Brief but it is common practice to try and match the observed water levels to within +/- 0.300 m. This standard was discussed with DSD and has been adopted for the calibration of the hydraulic model of the Hung Shui Kiu Channel.

4.2.1 The 22 July, 1994 event

There was much more observed data for the 22 July, 1994 flood thus this was used as the principle calibration event. The first estimates of the various calibration parameters discussed above were adjusted to enable the model to reproduce the observed water levels at six locations along the rectangular, to within the required calibration limits. The observed levels were taken from the flood marks provided by DSD and these were assumed to be the maximum water levels that occurred during the event. Table 11 gives a comparison of the maxima simulated by the model with the observed maximum water levels.

All the observations are within the calibration tolerance of +/- 0.3 m, the maximum being -0.25 m, the mean is -0.10 m and the standard deviation of the errors is 0.12 m. The Manning's n values used to achieve these results are given in Table 12 and the headloss coefficients for the bridges and the junction with the Tan Kwai Tsuen Channel are given in Table 13. A long profile of the maximum and minimum water levels for the event is given in Figure 37.

4.2.2 The 5 November, 1993 event

Only one flood mark was observed within the limits of the hydraulic model for 5 November, 1993 so it was of little use in calibrating the model. The event was simulated using the calibration parameters determined using the 22 July, 1994 event and the results are given in Table 14. Without the need for any further adjustment the model reproduced the one observed level with an error of 0.06 m, well within the calibration tolerance. A long profile of the maximum and minimum water levels for the 5 November event is given in Figure 38. The villagers observed that the channel was approximately bankfull at 08:00 hrs but that the water level rose by 1.5 m between 08:00 and 08:10. These observations must be treated with caution because they are based on a visual assessment, no actual measurements of water levels were taken, and the exact times of the observation can not be verified. Figure 67 gives the stage hydrograph at node hskc03 for the November, 1993 event and shows that the model predicated the water level to be approximately 0.2 m above bank top at 08:00, rising to about 0.75 m above bank top by 08:00. The water level continues to rise very rapidly reaching 1.5 m above bank top at about 08:20. Although this does not exactly recreate the observations made by the villagers, which are themselves questionable, the model does simulate the extremely rapid increase in water level which occurred at about 08:00 and this is perhaps further evidence to support the choice of hydrological and hydraulic parameters used in this study.

4.2.3 Discussion of the calibration

The generally good fit between the model predictions and the observations taken during the 1993 and 1994 flood events suggests that high level of confidence might be attached to the accuracy of the model. However, in an ideal modelling study much more calibration data would be required to allow independent verification of the selected calibration parameters. Both calibration events have only offered observations of maximum water levels, in situations where the channel has been dramatically overtopped. Thus it has not been possible to separate the effects of various sources of energy loss other than by using the Engineer's judgement. Ideally an inbank event should have been used to determine the inbank roughness coefficients and the energy loss induced by the channel bends. The two out-of-bank events would then have been used solely to determine the out-of-bank calibration parameters.

Unfortunately, observations for an inbank event were not available at the time of this study so the division of energy losses between inbank and out-of-bank phenomena has not been verified. Whilst accepting that the calibrated model now offers the best way of examining future engineering works on the Hung Shui Kiu Channel, it is recommended that additional water level data is collected for future in and out-of-bank events to enable the calibration to be verified and confidence in the model results improved.

The Consultants recommend that five temporary water level records are to be installed along the existing rectangular channel. These should be at the following locations:

(i) the confluence with the Tan Kwai Tsuen Channel

(ii) upstream of bend 2

(iii) between downstream of the access ramp (AR 1) and bend 3

(iv) upstream of bend 4

(v) downstream of bend 4 at the start of the channel transition.

Data from the first gauge will provide more information about the energy losses occurring at the confluence. Difference in the water levels measured between gauges 4 and 5 will provide an improved assessment of the energy loss around bend 4. Similarly difference between levels at gauges 3 and 4 will provide a better assessment of the energy loss at bend 3. This may also be used to relate energy loss to bend radius and thus enable more confident predictions to made for the energy loss associated with bend 1 and 2. Gauges 2 and 3 , along with new estimates for the energy loss for bend 2 will provide further information on the impact of the access ramp (AR 1).

These gauges should be installed before the 1995 monsoon season and remain in place for at least one monsoon or until at least three reasonably large events have been recorded. A reasonably large event might be one during which the water level in the channel exceeds 2 m. Ideally the gauges should record the water level at intervals of no more than 5 minutes over a range of channel depths from say 1m to 4.6m. Continuing to record the water level once the flow is out of bank will provide useful calibration data for the flood plain roughness coefficients

4.3 Simulation of the design events.

The design inflows for the 1 in 2, 10, 50 and 200 year rainfall events determined during the hydrological study were simulated using the calibrated hydraulic model. Tables 17 to 20 give the model results for each event and Figures 41 to 44 show the maximum and minimum water level profiles for each event. These show that for even the 1 in 2 year event, the rectangular channel will be overtopped, the most serious flooding occurring between the upstream limit and the junction with the Tan Kwai Tsuen Channel. The depth of flooding (negative freeboard) gets progressively worse for the 1 in 10, 50 and 100 year events. It is interesting to note that there is only minor flooding, 0.29 m deep, from the trapezoidal channel, occurring immediately downstream of the transition from the rectangular channel. The over topping at the downstream boundary may be due to errors in the rating curve but it is encouraging to see that this does not propagate upstream and thus should have no impact on the water levels in the area of interest.

It is obvious that the original design is not performing to the desired standard and some remedial measures are required to improve the level of protection against flooding offered to the surrounding properties.

5 RECOMMENDATIONS FOR REMEDIAL MEASURES

5.1 Review of the original channel design

5.1.1 Design assumptions and method

The design assumption for the original channel are contained within calculation sheets (DSD, 1985 onwards) and were as follows:

1. Storage effects of the Lam Tei Reservoirs and the irrigation dams at Hung Shui Kiu are neglected;

2. The Bransby -William's equation was adopted for the determination of the times of entry in to the lined concrete channels. The velocity of the flow was assumed to be 2 ms^{-1} in estimating the time of travel along the lined concrete channels;

3. The Rational Method was used to estimate runoff quantities;

4. Extreme depths corresponding to rainfall of various return periods were abstracted for Table II of the RO Technical Note No 58;

5. The runoff coefficient for the urban and future development (upgrading) areas was assumed to be 1.0;

6. The runoff coefficient for the Agricultural Priority Areas (APA) and the Countryside Conservation Area (CCA) was assumed to be 0.6;

7. The runoff coefficient for the land bank area within the Tin Shui Wai landholdings was assumed to be 0.6;

8. The Manning's n roughness for the concrete channel and the box culverts was assumed to be 0.015 and the Manning's formula was used to size the channels; and

9. The freeboard will be a minimum of 300 mm above the flood levels of the relevant design storm:

5.1.2 Discussion of the design parameters

The use of a Manning's n of 0.015 in the design of the rectangular channel has been based on the flow resistance expected from the channel bed and sides. Recommended values of Manning's for concrete channels fall between 0.011 and 0.020 depending on the quality of the finish to the concrete surface (Chow, 1959). For smooth finishes such as that of the Hung Shui Kiu Channel the range is given as 0.013 to 0.017. Given the potential for sediments and trash to build up in the channel and the possibility of additional resistance caused by the joints in the channel segments it might be argued that the upper value of 0.017 should be selected but otherwise the value of 0.015 appears a reasonable. However, bed and bank resistance are not the only source of resistance to flow. Vegetation, channel irregularity, obstructions and the channel alignment may also have an impact on the flow resistance which can be accounted for by modifying the value of Manning's n (USSCS, 1963). The US Soil Conservation Service suggest the following procedure for determining the modified value of Manning's n.

1. Select the basic value of Manning's n based on the nature of the channel bed and banks, in this case 0.017.

2. Determine any modification required to account for vegetation, in this none as there is no evidence of vegetation growth in the rectangular channel.

3. Determine any modification required for channel irregularity. The USSCS offers three classifications: changes in size and shape occurring gradually; large and small sections alternating occasionally or shape changes causing occasional shifting of the main flow from side to side; and large and small sections alternating frequently or shape changes causing frequent shifting of the main flow from side to side. Of these classifications the second is perhaps the most representative of the rectangular channel for which a modifying value of 0.005 is offered.

4. Determine any modification required for obstructions. The smooth sides to the channel and the clear span of the bridges suggest that there is unlikely to be any significant blockage of the channel (neglecting the possibility of a truck being washed in to it). The minor build up of sediment and trash has been considered in the selection of the basic Manning's n therefore no further modification is required.

5. Modification for channel alignment. The effect of bends has been discussed in detail earlier. However, the USSCS offers a further method for accounting for the energy loss at bends, based on the ration of meander length to straight line length of the channel. Three classifications are offered, minor, appreciable and severe. The meander length of the Hung Shui Kiu Channel is 863.0 m and the straight line length is 670.0 m, giving a ration of 1.29. This suggests that the effects of the channel alignment will be "appreciable" and a modifying value of 0.15 n' is offered, where n' is the sum of the basic n and all the previous modifying values. Thus the modifying value for channel alignment is 0.0033

6. The final value a Manning's n is found by summing the basic values and the modified values, giving at total n of 0.0253

Substituting this value of n into Manning's equation for the design discharge of 76.0 m^3s^{-1} gives a depth of 4.29 m, considerably higher than the 3.6 m depth of the as built channel.

The original design flow was simulated using the calibrated hydraulic model. The results are tabulated in Table 15 and the water level profile is given in Figure 39. This shows that the rectangular channel would be overtopped by some 0.6 to 0.8 m for virtually its whole length. Figure 40 and Table 16 give the results for a steady flow of 41.25 m^3s^{-1}. In this case the flow is entirely with in the channel, with a minimum freeboard of 0.02 m. Thus the actual channel capacity is only some 54% of the design value. The reasons for this reduction in capacity are the additional energy losses associated with the channel bends, the access ramp and the junction with the Tan Kwai Tsuen Channel.

5.1.3 Causes of the flooding

There appear to have been three contributing factors to the recent flood experienced from the Hung Shui Kiu Channel. Firstly, the return period of the rainfall for both the November, 1993 and July, 1994 events was greater than the 1 in 10 year return period for which the channel was designed. The November, 1993 event rainfall had a return period of approximately 1 in 13 years and the July, 1994 event was approximately 1 in 13 years based on peak intensity or 1 in 38 years based on total rainfall. The severe flooding in November 1993 was largely the result of a very rare short duration storm, with the central one hour core rainfall having a return period of about 136 years. Since the catchment is very sensitive to storms of one hour upwards, the high observed flood levels should be expected following such rainfall. For the July 1994 event, flooding was the result of a more sustained storm having three separate sub-storms nested within the overall storm which lasted almost 15 hours. The peak rainfall intensities of these sub-storms was not as extreme as those experienced during the November 1993 event, with average one hour storms having a return period of only about 14 years. However, the overall storm had a return period of almost 40 years, and so again the observed flooding was inevitable. In both cases, the designed channel capacity was exceeded, but for different reasons. The November 1993 event resulted from a single short duration high intensity rainstorm, whilst that on July 1994 was caused by a prolonged, three peaked rainstorm such that the channel capacity was exceeded by prolonged and recurring flood runoff.

The second contributing factor to the flooding was the higher than expected runoff observed during both events. The runoff coefficients determined during the hydrological study are some 30% higher than those used in the original design. Thus the 1 in 10 year flow is now estimated to be 95 m^3s^{-1} at the upstream limit of the rectangular channel and 104 m^3s^{-1} downstream of the junction with the Tan Kwai Tsuen Channel instead of 70 m^3s^{-1} and 76 m^3s^{-1} respectively.

The third contributing factor was the lower than expected capacity of the rectangular channel. Energy loss from the bends and access ramps has reduced the channel capacity from the 76 m^3s^{-1} determined during the original design to approximately 41 m^3s^{-1}.

Assessment of the impact of energy loss at bends and changes in the channel geometry, such as those at access ramps can not be easily undertaken without using a sophisticated hydrodynamic model such as the one used in this study. Thus modelling should be adopted as part of DSD design procedures for all but the most simple, straight channels.

There is a suggestion from some of the local residents that some of the flooding was caused by overtopping of a water supply reservoir somewhere near Chung Uk Tsuen. This reservoir is supposedly supplied from the Tai Lam Chung Reservoir. The Consultants can find no evidence of this reservoir on any of the aerial photographs of the area, nor any knowledge of such a reservoir within WSD, and thus reject the suggestion that there was any significant contribution to the past flooding from water supply sources.

5.2 Review of the existing remedial measures

The calibrated hydraulic model was updated to include the remedial measures undertaken by DSD. The introduction of the syphons at the Hung Shui Hang reservoir and at the irrigation weir will have little impact because the volumes of storage created by these are insignificant in comparison to the total volume of runoff. Thus they were not modelled. The cross-sections between the upstream limit of the rectangular channel and downstream of the Tan Kwai Tsuen Channel were truncated at the channel limits and the banks raised by 1.0 m to represent replacement of the railings with the concrete wall. The loss unit representing the first footbridge was updated to reflect the raising of the bridge soffit. The left and right floodplain were modelled as reservoir units, the left bank draining back into the channel downstream of the end of the concrete wall and right bank draining into the junction with the Tan Kwai Tsuen Channel.

The 1 in 2, 10, 50 and 200 year rain storm events were simulated and the results are presented in Tables 21 to 24 and Figures 45 to 48. For the 1 in 2 year event the raised banks almost contain the flow, reducing the flooding at the upstream end of the channel. However, they have little impact on the flooding further downstream. For the higher return period events the raised concrete wall is still overtopped and serious flooding occurs along the full length of the channel.

5.3 Potential future remedial measures

Discussions with DSD have identified five potential remedial measures which could lead to an improved standard of flood protection for the properties adjacent to the Hung Shui Kiu Channel. These are:

- Option 1 - widening the existing channel;
- Option 2 - deepening the existing channel by increasing the height of the concrete side walls;
- Option 3 - use of a two stage channel;
- Option 4 - straightening the channel bends; and
- Option 5 - providing flood storage upstream of the channel to reduce peak flood flows.

5.3.1 Option 1 - widening the existing channel

The existing channel cross-section were widened to accommodate the design flow. Two scenarios were investigated, firstly, accommodating the 1 in 200 year flow and secondly, reducing the width to accommodate just the 1 in 50 year flow. The loss coefficients for all the bridges were set to zero as these would now be above the water levels and the channel access ramp removed. The banks were raised by 1.0 m, assuming that the rest of railings would be replaced by concrete walls. For Option 1A the channel was widened to 11.50 m and this was found to be sufficient to pass the 1 in 200 year flood with only minor overtopping. Figures 49 to 52 and Tables 25 to 28 give the results of the 1 in 2, 10, 50 and 200 year events respectively. For this situation the minimum freeboard along the whole channel for the 1 in 10 year event is 0.30m, at the downstream limit of the steep trapezoidal channel under the TM-YL highway. The minimum freeboard for the rectangular channel was 0.71 m. For the 1 in 50 year event the minimum freeboard for the rectangular channel is 0.23 m immediately upstream of the junction of with the Tan Kwai Tsuen Channel, but greater than 0.30 m elsewhere. These tables also show that there may be some minor flooding from the entry and exit from the rectangular channel. The banks should be raised at these locations to over come this.

Option 1B only increased the channel width to 10.50 m, providing sufficient capacity to accommodate the 1 in 50 year flood with only minor overtopping. Table 29 and Figure 53 give the full model results. Given that the reducing the capacity from 1 in 200 years to only 1 in 50 years only reduces the required width by 1.0 m, it is not recommended that this option be considered further.

5.3.2 Option 2 - deepening the existing channel

This option is not considered practical because of the large amount of additional capacity required. One test was undertaken to demonstrate this. The channel banks were raised by 3.0 m and the 1 in 10 year flood was simulated using this new geometry. From the results in Table 30 and Figure 54 it can be seen that the banks would have to be raised by at least 2.5 m to provide the required 300 mm freeboard for this event. Thus it is not recommended that this option be considered further.

5.3.3 Option 3 - using an two stage channel

Two stage channels are often considered for flood alleviation works because the upper stage may provide some amenity value when not required to carry flood waters. However, a two stage channel requires a greater width to provide the same capacity as a simple rectangular channel because the full depth is not utilised across the whole section. It would be inappropriate to consider using the upper stage of a two stage channel for any amenity purpose on the Hung Shui Kiu Channel because the extremely rapid increase in water levels experienced during flood events would make warning and evacuation impractical. Given this and the fact that there is little available land adjacent to the existing channel, this option was rejected without undertaking any modelling work.

5.3.4 Option 4 - straightening the channel

A significant contribution to the lower than expected capacity of the existing channel is the effect of the bends. Thus it is logical to explore removing the bends to improve the performance of the channel. The current land bank adjacent to the channel is not wide enough to allow the bend radii to be significantly increased whilst maintaining the existing channel alignment. However, if the channel alignment is moved all the bends can be straightened. This option proposes that a new channel is constructed following a straight line from the junction with the Tan Kwai Tsuen Channel to the upstream limit of the trapezoidal channel. Unfortunately, there is little vacant land along this route. Thus a number of properties, including a large industrial building would need to be demolished to build this channel.

This route is some 114 m shorter than the existing route and the model cross-section spacings between nodes hskc01 and hskc07 were reduced to reflect this. The same labels were used for convenience only. A rectangular shape was selected for the new channel to minimise the land requirements and the gradient was selected to match the inverts of the existing channels upstream and downstream of the new one. Thus the depth of the new channel was set to be 4.6 m, comprising 3.6 m from the bottom of the channel to ground level and 1.0 m provided by a concrete wall. A Manning's n of 0.017 was selected to represent the roughness of the straight concrete channel. The width of the channel was selected to provide sufficient capacity to pass the 1 in 50 year flood with at least 300 mm of freeboard. Table 33 gives the model results for a channel width of 6.5 m, showing a minimum freeboard of 0.46 m occurring in the modified channel. The maximum and minimum water levels are given in Figure 57. Tables 31, 32 and 34 and Figures 55, 56 and 58 give the results for the 1 in 2, 10 and 200 year events and from Table 34 it can be seen that there is only minor overtopping of this channel for the 1 in 200 year event. The overtopping occurs at the downstream limit of the steep trapezoidal channel and at the bank low spots at the start of the 5.5 m wide trapezoidal channel at chainage 788.6 m. This could be eliminated by small increases to the bank heights at these locations and this would also provide the required 0.30 m freeboard for the 1 in 50 year design event. Thus a straight, 6.5 m wide by 4.6 m deep, rectangular concrete channel appears to offer an adequate solution to the current flood problems. Figure 66 gives a schematic layout of the proposed new channel.

5.3.5 Option 5 - using flood storage

DSD's current remedial measures include the construction of two 400 mm diameter syphons over the Hung Shui Hang Dam to lower the water level upstream of the dam by 5.0 m. This creates storage behind the dam which may be used to reduce the flood peak of future events. However, the capacity of the syphons is small in comparison to the downstream channel capacity and the runoff flows are likely to induce flooding and so this new storage volume will be filled at the start of the event. Thus it will have little impact on the peak flows. In this Option the Consultants have explored the possibility of increasing the syphon capacity to approximately that of the downstream channel such that the storage behind the dam only starts to be filled once the downstream channel capacity is exceeded.

Unfortunately, there are few reliable methods available for calculating the discharge through a syphon. The discharge characteristics of many of the syphons examined in the past by HR Wallingford were determined by physical modelling, before the syphons were constructed. However, in this case no modelling was undertaken as part of the syphon design. Some efforts have been made by DSD to measure the discharge through the syphons since they were constructed and the Consultants recommend that this work is continued. In the meantime, if the syphon is treated as a simple orifice then the discharge may be calculated from the following:

$$Q \quad = \quad A\,(2\,g\,h)^{0.5} \qquad \text{Eqn (17)}$$

where Q is the discharge, A is the area of the outlet and h is head across the syphon. This equation estimates the maximum discharge through the two 400 mm diameter syphons to be 1.24 m^3s^{-1}.

To evaluate the potential for the use of storage for flood alleviation the Consultants have tried to maximise the use of the Hung Shui Hang Reservoir to provide the most significant improvement. Thus for Option 5 the maximum syphon capacity has been increased to 50 m^3s^{-1}, corresponding to a flow area of 2.91 m^2, and the water level upstream of the dam drawn down to 70 m PD, almost the lowest draw off level. In addition the remaining metal railings along the rectangular channel were replaced with concrete walls, increasing the channel depth by 1.0 m. Table 35 and Figure 59 give the results for the 1 in 10 year rain storm event, it can be seen that this option eliminates nearly all the flooding except for a short reach at the upper end of the rectangular channel. Figure 60 shows the impact of storage on the discharge over the irrigation weir. reducing the peak flow by some 20 m^3s^{-1} and lengthening the recession of the event.

These results demonstrate how effective the use of storage may be in alleviating flooding but unfortunately also show that there is not quite sufficient storage available in the Hung Shui Hang Reservoir to provide protection for the 1 in 10 year event. This implies that the use of storage is not a viable option for the long term alleviation of flooding from the Hung Shui Kiu Channel. If the additional siphon capacity can be supplied quickly and economically along with completing the replacement of the railings, then Option 5 may provide some useful short term improvements.

5.3.6 Other short term measures

DSD (1995b) suggested two short term measures to improve the performance of the channel. The first, Measure A, was to remove the access ramp AR1. Two options were proposed, either filling over the access ramp to create a 5.5 m wide channel or breaking half the ramp out to create a wider section of channel. Removing the access ramp is fully supported by the Consultants as at present it induces considerable energy loss raising water levels upstream causing flood flows to be diverted across the channel into adjacent properties. Of the two suggested new channel layouts, SK1 is preferred because it avoids an unnecessary expansion and contraction of the flow, provides a much smoother bank alignment between the two bends and is compatible with potential future plans to widen the channel by constructing box culverts either side of the existing channel banks.

The second, Measure B, involves widening and smoothing the transition from the rectangular channel to the trapezoidal channel. This again is supported as it is likely to reduce the energy loss at this transition and thus reduce upstream water levels. The magnitude of this improvement however, is likely to be a lot less than that offered by Measure A. Given that this transition may require extensive reconstruction when the long term measures are implemented, it may not be economically justified to invest in improvements now that can not be preserved in the long term.

In addition to the above measures DSD are also planning to install a flood siren along the Hung Shui Kiu Channel. This work is strongly supported. Flood warning is often the most effective method for avoiding one of the most unacceptable consequences of flooding, the loss of human life.

Finally, all the model results show significant energy losses occurring at the junction with the Tan Kwai Tsuen Channel. These are believed to represent the turbulence and associated energy loss caused by the high velocity inflow joining the main flow. The construction of energy dissipators on the Tan Kwai Tsuen Channel and a realignment of the junction may reduce this problem.

Whilst these measures, either individually or combined, will not provide the desired improvement to the standard of protection to the 1 in 2 to 10 year level for the whole channel, the Consultants support their implementation where compatible with the long term measures. These measures will provide local improvement to what are perhaps the most hydraulically inefficient features of the channel and therefore some local benefit.

5.4 The need for further work

The Consultants have examined five options for providing long term improvements to the standard of flood defence offered by the Hung Shui Kiu Channel. Of these only two, Option 1 - channel widening and Option 4 - a new straight channel, are worthy of further investigation. Initial comparisons between the two suggest that Option 1 is more favourable as it does not appear to require the purchase of any additional land. However, there will be substantial construction costs associated with providing an extra 6.0m of channel width. Though it is of shorter length, construction cost may be higher for Option 4 than Option 1, as the new channel proposed by Option 4will pass through higher grounds, involving deeper excavetion. Option 4 is less attractive because it would require considerable land purchases and a number of existing properties to be demolished. There may be additional revenue made available by the release of land occupied by the existing channel once the new channel is completed, however, this may be limited, as local drainage are required along the existing channel. Detailed estimates for each option should be prepared to allow a proper evaluation to be undertaken.

DSD have suggested that the additional 6.0m of width required for Option 1 could be provided by constructing box culverts either side of the existing channel with connections at appropriate intervals. The Consultants believe this could be an extremely efficient method for gaining the required capacity but are concerned that the additional wetted perimeter and perhaps more significantly, the turbulence associated with the inter-connections between the channel and the box culverts may lead to a higher energy loss than that predicted by the model. As such, a channel and culvert system is likely to display very complex flow patterns the Consultants recommend that the design is tested using a physical model before being approved for construction.

6 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are made on the basis of the work undertaken as part of this study:

- Detailed hydrological and 1-D hydraulic models of the Hung Shui Kiu Catchment and channel network to Shek Po Tsuen were constructed and calibrated against the flooding events of 5 November, 1993 and 22 July, 1994. Although the comparison between observed and modelled water levels was good, it is recommended that further data is collected, by installing water level records and undertaking current metering, to enable the calibration to be verified.

- The original channel design was reviewed and found to be extremely thorough and entirely consistent with the design guideline prevailing at the time of the design.

- Three causes of the past flooding incidents were found to be: (i) higher return period rainfall than the original channel design; (ii) greater runoff flows from a given rainfall due to a higher than expected percentage runoff for rural catchments; and (iii) a lower than expected channel capacity due to unforseen energy losses at the channel bends and access ramps.

- The existing remedial measures were reviewed and whilst these offer some improvement, additional works are required to obtain an acceptable standard of flood protection.

- The short term measures proposed by DSD were reviewed and whilst not providing the desired improvement in the standard of flood protect to the 1 in 2 to 10 year level, these will provide some improvement and, where compatible with the long term remedial measures, are supported by the Consultants.

- Five options for further remedial measures were investigated. Deepening the channel, use of a two stage channel and use of the Hung Shui Hang Reservoir for flood storage were found to be impractical but widening the existing channel to 11.5 m and completing the replacement of the metal rails or building a new 6.5 m by 4.6 m channel to straighten the bends were found to provide the required standard of flood protection. The Consultants recommend that detailed cost estimates for the two viable options are prepared to enable them to be further evaluated.

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Tables

Table 1 Autographic raingauge data available for 5 November 1993

Raingauge	5-min data	15-min data	60-min data
R27	-	-	00:00 - 07:00
No17	-	00:00 - 13:00	00:00 - 13:00
No174	-	00:00 - 24:00	00:00 - 24:00
N07	00:00 - 13:00	00:00 - 13:00	00:00 - 13:00
N12	00:00 - 13:00	00:00 - 13:00	00:00 - 13:00

Table 2 Autographic raingauge data available for 22 July 1994

Raingauge	5-min data	15-min data	60-min data
R27	00:00 - 01:35	00:00 - 01:30	00:00 - 01:00
No17	-	00:06 - 09:00	00:60 - 09:00
No174	-	00:00 - 24:00	00:00 - 24:00
N07	00:00 - 24:00	00:00 - 24:00	00:00 - 24:00
N12	00:00 - 24:00	00:00 - 24:00	00:00 - 24:00

Table 3 Hourly raingauge data used for constructing spatial and temporal rainfall patterns

Raingauge	Name	5 Nov 93	22 Jul 94
R01	RO	-	Y
R11	Ngong Ping	Y	Y
R12	Discovery Bay TW	Y	Y
R17	Green Island	Y	M
R21	Tap Shek Kok PS	Y	P
R22	Tsim Bei Tsui	M	P
R23	Wong Shiu Chi	-	Y
R24	Sha Tau Kok	-	Y
R26	Shek Kong	Y	P
R27	Yuen Long RG Filter	P	M
R28	Au Tau	Y	P
R29	Lok Ma Chau	Y	Y
R31	Tai Mei Tuk	-	Y
R33	Tai O	-	Y
R42	Yuen Long	-	P
No17	Yuen Long RG Filter	Y	P
No174	Tin Shui Wai	Y	Y
N01	Shatin TW	-	Y
N02	Wo Che Estate	-	Y
N03	Tsuen Wan RG Filter	-	Y
N04	Cho Yiu Estate	-	Y
N05	Cheung Wah Estate	-	Y
N06	Shek Lei	-	Y
N07	Tuen Mun	Y	Y
N09	Shatin	-	Y
N10	Sham Tseng	Y	Y
N11	Tsing Li	-	Y
N12	Yuen Long	Y	Y
N14	Tai Mo Shan	Y	P
N17	Tung Chung	Y	Y
N18	Mui Wo	Y	Y
D01	Hang Tau Tai Po	P	P
D03	Lo Wu	Y	P
D62	Lo Shue Ling	M	M
D64	Kam Tin	Y	M

where: 'Y' means data available for all or most of period, 'P' means significant part of data missing, 'M' means data missing for entire period and '-' means data not collected for this gauge.

Table 4 Catchment characteristics of Hung Shui Kiu subcatchments

Catchment	Area (km^2)	MSL (km)	Lt (km)	S ($m\ m^{-1}$)	Sl ($m\ 100\ m^{-1}$)
1	3.93	3.85	3.90	0.104	10.40
1(a)	2.85	2.70	2.75	0.131	13.10
2	0.07	0.35	0.40	0.151	15.10
3	0.38	0.75	1.20	0.023	2.33
4	0.24	0.40	0.70	0.021	2.14
5	0.42	0.65	0.65	0.038	3.85
6	0.34	n/a	n/a	n/a	n/a
7	0.22	n/a	n/a	n/a	n/a

where: Area is sub-catchment area (km^2),

MSL is mainstream length (km): length measured to top of stream on the map,

Lt is total drainage length (km): length measured along mainstream but extended to watershed,

S is slope of mainstream (dimensionless): height difference between watershed and subcatchment outfall divided by Lt,

Sl is slope of mainstream ($m\ 100\ m^{-1}$): computed as described above for S, but divided by Lt * 0.01. Sl is required for the Bransby-Williams formula (Williams, 1922).

n/a means catchment characteristic not applicable. Hydrological response will be estimated by proportioning flows from subcatchment number 5.

Table 5 Estimates of Tc and LAG of Hung Shui Kiu subcatchments using various methods

Method	Catchment 1	Catchment 2	Catchment 3	Catchment 4	Catchment 5
Bransby-Williams:					
Tc (min)	77	11	41	25	20
Kirpich:					
Tc (min)	27	3.7	14	8.7	10
WSD Eqn (3):					
LAG (hr)	0.99	0.11	0.28	0.24	0.30
LAG (min)	59	6.6	17	14	18
Watt & Chow:					
LAG (hr)	0.55	0.08	0.39	0.27	0.20
LAG (min)	33	4.7	24	16	12

Table 6 *Hydrological model parameters for Hung Shui Kiu subcatchments*

Catchment	Time-to-peak (hr)	Percentage runoff (%)	Loss rate (mm $5min^{-1}$)	Baseflow ($m^3s^{-1}km^{-2}$)
1	0.55	95	0.25	0.24
2	0.08	90	0.25	0.24
3	0.39	90	0.25	0.24
4	0.27	90	0.25	0.24
5	0.20	80	0.50	0.24

Table 7 *Basic data to be used in the areal rainfall algorithm*

Site	Distance from catchment centroid (km)	Average annual rainfall (mm)
Hung Shui Kiu catchment	-	1800
R27 Yuen Long RG Filter	0.56	1650
No17 Yuen Long RG Filter	0.56	1650
No174 Tin Shui Wai	2.38	1600
N07 Tuen Mun	1.75	1900
N12 Yuen Long	1.94	1600

Table 8 *Extreme rainfall depths for given durations and return periods derived from Yuen Long RG Filters data compared with corresponding figures as given for RO Headquarters (Lam & Leung, 1994)*

	2-year (mm)		10-year (mm)		50-year (mm)		200-year (mm)	
	YLRGF	RO	YLRGF	RO	YLRGF	RO	YLRGF	RO
1-hr	52.05	56.8	81.01	82.0	105.64	102.0	126.16	116.00
2-hr	74.86	86.3	128.65	130.0	174.75	163.0	213.28	187.00
4-hr	107.60	115.0	204.31	186.0	288.98	254.0	360.44	315.00
6-hr	133.04	131.0	267.80	218.0	387.82	315.0	489.91	414.00

Table 9 Geometric and hydraulic properties of channel bends

Bend No	Chainage (m)	r_c (m)	B/r_c	F_c	F_r for design flow	Approx F_r for 1994 flow
1	270.0	60.0	0.0917	0.748	0.679	0.792
2	468.0	30.0	0.1833	0.681	0.679	0.786
3	558.0	50.0	0.1100	0.732	0.679	0.808
4	722.0	30.0	0.1833	0.681	0.679	0.896

Table 10 Head loss at channel bends for the design flow after Toebes & Sooky

Bend No	R (ft)	l (m)	f_r	f_b	f	Modified n	Design head loss (m)	Modified head loss (m)
1	5.04	70.0	0.174	1.842	2.016	0.051	0.140	1.619
2	5.04	43.5	0.108	1.143	1.251	0.051	0.087	1.005
3	5.04	66.6	0.166	1.757	1.923	0.051	0.133	1.545
4	5.04	37.9	0.094	0.995	1.089	0.051	0.076	0.875
Totals							0.435	5.044

Table 11 *Calibration results for the 22 July, 1994 flood event.*

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Observed (m PD)	Diff (m)
yltm-us	85.97	13.94	1.003	2.767		
yltm-01	85.99	13.93	0.915	2.295	14.05	-0.12
yltm-02	86.02	13.79	1.117	2.817		
yltm-03	86.07	13.85	0.606	1.752		
yltm-04	86.11	13.77	1.022	2.257		
yltm-ds	92.75	13.77	0.000	0.000		
hskc01u	92.75	13.48	1.008	2.763		
hskc01ub	92.75	13.31	0.777	2.313		
hskc01	92.99	13.23	0.582	2.068	13.47	-0.24
hskc01ju	93.03	13.21	0.577	2.001		
hskc01jl	93.03	12.89	0.000	0.000		
hskc01jd	102.15	12.89	0.894	2.959		
hskc02	103.62	12.81	0.790	2.682	13.06	-0.25
hskc03	105.66	12.61	0.812	2.679		
hskc03b	105.66	12.50	0.927	2.993		
hskc03b1	105.19	12.46	0.998	3.109		
hskc03b2	105.19	12.20	0.924	3.303	12.28	-0.08
hskc04	105.42	12.05	0.867	3.345		
hskc04d	105.39	11.98	0.792	3.148	11.89	0.09
hskc05	106.02	12.10	0.773	1.983		
hskc06u	106.86	11.96	0.760	2.714		
hskc06ub	106.86	11.83	0.834	3.025		
hskc06b	107.36	11.81	0.784	2.778		
hskc06b1	107.36	11.50	0.857	3.011	11.52	-0.02
hskc06	101.07	11.38	0.894	3.228		
hskc06b2	101.07	10.25	1.085	3.825		
hskc06d	101.08	9.18	0.887	5.206		
hskc07	101.09	8.89	0.874	3.628		
hskc07b	101.09	8.64	0.970	4.128		
hskc08u	101.09	8.81	0.573	2.609		
hskc08	100.79	8.47	0.648	2.873		
hskc09	100.78	8.13	0.765	3.272		
hskc10	100.79	7.59	0.952	3.845		
cpr-us	100.82	7.68	0.540	2.716		
cpr-ds	112.87	7.69	0.501	2.614		
hskc11	112.83	7.48	0.745	3.237		
hskc12u	112.44	7.01	0.920	3.807		
hskc12	112.46	6.69	1.215	4.338		
hskc12m	112.46	6.61	0.952	4.022		
hskc12d	112.47	6.25	1.285	4.337		
hskc13u	112.47	6.17	0.905	3.730		
hskc13	112.48	5.82	1.463	4.346		
hskc13d	112.49	5.77	0.897	3.671		
hskc14	112.93	5.41	0.963	3.857		
hskc15	113.94	5.16	1.173	4.386		
hskc15d	114.98	5.20	1.962	5.425		
Mean						-0.10
SD						0.12

Table 12 Calibrated values of Manning's n.

Label	Left floodplain	Channel	Right floodplain
yltm-us		0.045	
yltm-01		0.045	
yltm-02		0.045	
yltm-03		0.045	
yltm-04		0.045	
hskc01ub	0.080	0.025	0.080
hskc01	0.080	0.025	0.080
hskc01ju	0.080	0.025	0.080
hskc01jl	0.080	0.025	0.080
hskc01jd	0.080	0.025	0.080
hskc02	0.060	0.025	0.060
hskc03	0.060	0.035	0.050
hskc03b	0.060	0.025	0.050
hskc03b1	0.060	0.025	0.050
hskc03b2	0.060	0.025	0.050
hskc04	0.060	0.025	0.050
hskc04d	0.060	0.020	0.060
hskc05	0.060	0.020	0.060
hskc06u	0.060	0.020	0.060
hskc06ub	0.060	0.020	0.060
hskc06b	0.060	0.020	0.060
hskc06b1	0.060	0.030	0.060
hskc06	0.060	0.020	0.060
hskc06b2	0.060	0.030	0.060
hskc06d	0.060	0.030	0.060
hskc07		0.025	
hskc07b		0.025	
hskc08u		0.025	
hskc08		0.025	
hskc09		0.025	
hskc10		0.025	
cpr-us		0.025	
cpr-ds		0.025	
hskc11		0.025	
hskc12u		0.025	
hskc12		0.025	
hskc12m		0.025	
hskc12d		0.025	
hskc13u		0.025	
hskc13		0.025	
hskc13d		0.025	
hskc14		0.025	
hskc15		0.025	
hskc15d		0.025	

Table 13 Calibrated headloss coefficients based on the 22 July, 1994 flood event.

Upstream label	Downstream label	Location	Chainage (m)	Elevation (m PD)	Headloss coefficient
hskc01u	hskc01ub	Footbridge CF1	0.0	11.75	0.30
				12.75	0.30
				17.00	0.60
hskc01ju	hskc01jl	TKTC confluence	89.0	7.85	2.90
				12.35	2.90
hskc03	hskc03b	Footbridge CF2	288.0	10.75	0.30
				11.75	0.30
				16.20	0.60
hskc03b1	hskc03b2	Footbridge CF3	384.0	10.47	0.30
				11.48	0.30
				15.92	0.60
hskc06u	hskc06ub	Footbridge CF4	573.0	9.87	0.30
				10.87	0.30
				15.00	0.60
hskc06b	hskc06b1	Footbridge CF5	624.0	9.75	0.30
				10.75	0.30
				15.00	0.60
hskc06	hskc06b2	Vehicle bridge CV1	696.0	9.55	0.30
				10.55	0.30
				15.00	0.60
hskc07	hskc07b	Footbridge FB1	828.0	9.47	0.30
				10.47	0.30
				12.47	0.60

Table 14 Calibration results for the 5 November, 1993 flood event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Observed (m PD)	Diff (m)
yltm-us	119.55	14.61	0.999	0.999		
yltm-01	119.57	14.63	0.915	2.274		
yltm-02	119.59	14.51	1.117	2.803		
yltm-03	119.64	14.57	0.606	1.747		
yltm-04	119.67	14.47	1.031	2.255		
yltm-ds	129.45	14.47	0.000	0.000		
hskc01u	129.45	14.12	1.008	2.763	14.04	0.06
hskc01ub	129.45	13.98	0.594	2.400		
hskc01	129.59	13.92	0.509	2.094		
hskc01ju	129.63	13.91	0.516	2.029		
hskc01jl	129.63	13.60	0.000	0.000		
hskc01jd	141.57	13.60	0.838	2.941		
hskc02	141.94	13.52	0.800	2.668		
hskc03	142.42	13.31	0.820	2.684		
hskc03b	142.42	13.21	0.919	2.975		
hskc03b1	142.71	13.11	0.995	3.100		
hskc03b2	142.71	13.02	0.906	3.125		
hskc04	142.86	12.85	0.863	3.132		
hskc04d	143.00	12.76	0.769	3.090		
hskc05	143.05	12.92	0.782	1.964		
hskc06u	143.11	12.71	0.760	2.713		
hskc06ub	143.11	12.60	0.789	3.017		
hskc06b	143.13	12.58	0.788	2.768		
hskc06b1	143.13	12.48	0.831	3.008		
hskc06	144.30	12.40	0.890	3.160		
hskc06b2	144.30	10.85	1.048	3.750		
hskc06d	143.36	9.89	1.363	5.089		
hskc07	143.43	9.57	0.869	3.739		
hskc07b	143.43	9.19	0.988	4.470		
hskc08u	143.45	9.37	0.604	2.927		
hskc08	143.55	8.98	0.696	3.261		
hskc09	143.59	8.65	0.795	3.614		
hskc10	143.67	8.13	0.960	4.103		
cpr-us	143.69	8.18	0.590	3.239		
cpr-ds	165.54	8.19	0.595	3.243		
hskc11	165.54	8.01	0.791	3.675		
hskc12u	165.53	7.52	0.964	4.267		
hskc12	165.50	7.20	1.221	4.793		
hskc12m	165.50	7.11	1.014	4.593		
hskc12d	165.46	6.73	1.281	4.851		
hskc13u	165.45	6.65	0.961	4.271		
hskc13	165.41	6.27	1.448	4.890		
hskc13d	165.40	6.21	0.953	4.251		
hskc14	165.37	5.79	1.029	4.493		
hskc15	165.50	5.24	1.270	5.127		
hskc15d	165.67	4.83	2.849	8.825		

Table 15 Original design flow simulated using the calibrated hydraulic model.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)
yltm-us	70.00	13.59	0.74	2.75	0.00	16.33
yltm-01	70.00	13.55	0.49	1.99	28.80	16.63
yltm-02	70.00	13.38	0.58	2.35	46.00	15.75
yltm-03	70.00	13.45	0.31	1.42	78.60	16.11
yltm-04	70.00	13.37	0.34	1.67	102.60	13.14
yltm-ds	70.00	13.37	0.00	0.00	102.60	13.00
hskc01u	70.00	13.15	1.01	2.76	102.60	13.00
hskc01ub	70.00	12.67	0.50	2.11	102.60	13.00
hskc01	70.00	12.59	0.47	1.92	182.60	11.48
hskc01ju	70.00	12.57	0.46	1.87	201.60	11.45
hskc01jl	70.00	12.10	0.00	0.00	201.60	11.42
hskc01jd	76.00	12.10	0.67	2.15	201.60	11.42
hskc02	76.00	11.96	0.64	1.98	282.60	11.15
hskc03	76.00	11.64	0.53	1.94	402.60	10.75
hskc03b	76.00	11.41	0.67	2.28	402.60	10.75
hskc03b1	76.00	11.20	0.67	2.27	486.60	10.47
hskc03b2	76.00	10.97	0.74	2.53	486.60	10.47
hskc04	76.00	10.76	0.74	2.69	542.60	10.28
hskc04d	76.00	10.64	0.66	2.45	622.60	10.01
hskc05	76.00	10.87	0.36	1.41	642.60	9.95
hskc06u	76.00	10.62	0.61	2.37	666.60	9.87
hskc06ub	76.00	10.39	0.71	2.70	666.60	9.87
hskc06b	76.00	10.37	0.67	2.55	702.60	9.75
hskc06b1	76.00	10.14	0.77	2.92	702.60	9.75
hskc06	76.00	9.81	0.86	3.20	762.60	9.55
hskc06b2	76.00	9.58	1.07	3.79	762.60	9.55
hskc06d	76.00	8.62	0.86	4.64	862.60	9.21
hskc07	76.00	8.41	0.85	3.53	902.60	9.47
hskc07b	76.00	8.25	0.95	3.86	902.60	9.47
hskc08u	76.00	8.43	0.54	2.36	922.60	9.61
hskc08	76.00	8.10	0.63	2.63	1062.60	9.47
hskc09	76.00	7.79	0.73	2.97	1142.60	9.94
hskc10	76.00	7.27	0.91	3.54	1276.60	9.39
cpr-us	76.00	7.39	0.48	2.30	1324.60	11.10
cpr-ds	87.00	7.40	0.45	2.26	1381.35	10.50
hskc11	87.00	7.19	0.71	2.93	1397.35	10.50
hskc12u	87.00	6.72	0.89	3.50	1522.35	8.67
hskc12	87.00	6.41	1.07	4.03	1578.35	8.59
hskc12m	87.00	6.32	0.92	3.69	1593.35	9.15
hskc12d	87.00	5.97	1.07	4.03	1656.35	8.13
hskc13u	87.00	5.89	0.87	3.42	1671.33	7.80
hskc13	87.00	5.55	1.05	3.97	1734.33	7.69
hskc13d	87.00	5.51	0.82	3.28	1749.33	7.36
hskc14	87.00	5.22	0.80	3.23	1928.33	7.05
hskc15	87.00	4.99	0.74	3.01	2108.33	6.59
hskc15d	87.00	5.04	0.58	2.40	2219.33	5.66

Table 16 41.25 m^3s^{-1} flow simulated using the calibrated hydraulic model.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	38.00	12.97	0.88	2.59	0.00	16.33	3.36
yltm-01	38.00	12.55	0.77	2.35	28.80	16.63	4.08
yltm-02	38.00	12.29	0.93	2.90	46.00	15.75	3.46
yltm-03	38.00	12.07	0.48	1.73	78.60	16.11	4.05
yltm-04	38.00	11.91	0.53	1.96	102.60	13.14	1.24
yltm-ds	38.00	11.91	0.00	0.00	102.60	13.00	1.10
hskc01u	38.00	11.61	1.01	2.76	102.60	13.00	1.39
hskc01ub	38.00	11.54	0.36	2.06	102.60	13.00	1.46
hskc01	38.00	11.45	0.33	1.96	182.60	11.48	0.03
hskc01ju	38.00	11.43	0.33	1.95	201.60	11.45	0.02
hskc01jl	38.00	10.79	0.00	0.00	201.60	11.42	0.63
hskc01jd	41.25	10.79	0.48	2.56	201.60	11.42	0.63
hskc02	41.25	10.61	0.45	2.48	282.60	11.15	0.54
hskc03	41.25	10.19	0.46	2.50	402.60	10.75	0.56
hskc03b	41.25	10.09	0.49	2.59	402.60	10.75	0.66
hskc03b1	41.25	9.75	0.57	2.89	486.60	10.47	0.72
hskc03b2	41.25	9.63	0.53	2.75	486.60	10.47	0.84
hskc04	41.25	9.47	0.52	2.72	542.60	10.28	0.81
hskc04d	41.25	9.31	0.49	2.62	622.60	10.01	0.70
hskc05	41.25	9.52	0.30	1.51	642.60	9.95	0.43
hskc06u	41.25	9.29	0.46	2.51	666.60	9.87	0.58
hskc06ub	41.25	9.19	0.49	2.60	666.60	9.87	0.68
hskc06b	41.25	9.15	0.47	2.54	702.60	9.75	0.61
hskc06b1	41.25	9.04	0.50	2.63	702.60	9.75	0.71
hskc06	41.25	8.78	0.51	2.68	762.60	9.55	0.77
hskc06b2	41.25	8.66	0.55	2.80	762.60	9.55	0.89
hskc06d	41.25	7.85	0.73	3.40	862.60	9.21	1.36
hskc07	41.25	7.63	0.84	3.14	902.60	9.47	1.85
hskc07b	41.25	7.57	0.88	3.27	902.60	9.47	1.91
hskc08u	41.25	7.73	0.50	1.93	922.60	9.61	1.88
hskc08	41.25	7.45	0.57	2.14	1062.60	9.47	2.02
hskc09	41.25	7.22	0.65	2.37	1142.60	9.94	2.72
hskc10	41.25	6.70	0.84	2.89	1276.60	9.39	2.69
cpr-us	41.25	6.85	0.39	1.63	1324.60	11.10	4.25
cpr-ds	52.25	6.86	0.38	1.71	1381.35	10.50	3.64
hskc11	52.25	6.70	0.63	2.37	1397.35	10.50	3.81
hskc12u	52.25	6.23	0.84	2.96	1522.35	8.67	2.44
hskc12	52.25	5.90	1.10	3.63	1578.35	8.59	2.69
hskc12m	52.25	5.82	0.86	3.10	1593.35	9.15	3.34
hskc12d	52.25	5.46	1.11	3.64	1656.35	8.13	2.67
hskc13u	52.25	5.38	0.82	2.91	1671.33	7.80	2.42
hskc13	52.25	5.01	1.12	3.67	1734.33	7.69	2.68
hskc13d	52.25	4.96	0.80	2.86	1749.33	7.36	2.40
hskc14	52.25	4.67	0.78	2.81	1928.33	7.05	2.38
hskc15	52.25	4.44	0.71	2.60	2108.33	6.59	2.15
hskc15d	52.25	4.44	0.56	2.10	2219.33	5.66	1.22

Table 17 Model results - 1 in 2 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	59.11	13.36	1.00	2.76	0.00	16.33	2.97
yltm-01	58.99	13.19	0.97	2.27	28.80	16.63	3.45
yltm-02	58.91	12.91	1.12	2.80	46.00	15.75	2.84
yltm-03	58.74	12.99	0.61	1.73	78.60	16.11	3.12
yltm-04	58.77	12.90	1.05	2.15	102.60	13.14	0.24
yltm-ds	62.97	12.90	0.00	0.00	102.60	13.00	0.10
hskc01u	62.97	12.64	1.01	2.76	102.60	13.00	0.36
hskc01ub	62.97	12.49	0.56	2.19	102.60	13.00	0.51
hskc01	62.64	12.41	0.49	1.94	182.60	11.48	-0.93
hskc01ju	62.56	12.39	0.50	1.92	201.60	11.45	-0.94
hskc01jl	62.56	11.87	0.00	0.00	201.60	11.42	-0.45
hskc01jd	68.42	11.87	0.82	2.94	201.60	11.42	-0.45
hskc02	67.86	11.70	0.74	2.60	282.60	11.15	-0.55
hskc03	67.73	11.35	0.75	2.50	402.60	10.75	-0.60
hskc03b	69.33	11.16	0.91	2.94	402.60	10.75	-0.41
hskc03b1	69.20	10.92	0.98	3.08	486.60	10.47	-0.45
hskc03b2	69.20	10.73	0.90	3.21	486.60	10.47	-0.26
hskc04	69.11	10.55	0.85	3.19	542.60	10.28	-0.27
hskc04d	68.96	10.40	0.79	3.10	622.60	10.01	-0.39
hskc05	68.91	10.65	0.77	1.96	642.60	9.95	-0.70
hskc06u	68.87	10.40	0.76	2.71	666.60	9.87	-0.53
hskc06ub	68.87	10.21	0.81	3.02	666.60	9.87	-0.34
hskc06b	68.83	10.20	0.79	2.76	702.60	9.75	-0.45
hskc06b1	68.83	10.01	0.84	3.01	702.60	9.75	-0.26
hskc06	68.78	9.71	0.88	3.15	762.60	9.55	-0.16
hskc06b2	68.78	9.50	0.61	3.56	762.60	9.55	0.05
hskc06d	68.75	8.48	0.84	4.41	862.60	9.21	0.73
hskc07	68.73	8.26	0.87	3.49	902.60	9.47	1.21
hskc07b	68.73	8.12	0.95	3.78	902.60	9.47	1.35
hskc08u	68.72	8.30	0.55	2.31	902.60	9.47	1.18
hskc08	68.72	7.98	0.62	2.54	1062.60	9.47	1.49
hskc09	68.76	7.68	0.72	2.88	1142.60	9.94	2.26
hskc10	68.81	7.12	1.04	3.56	1276.60	9.39	2.27
cpr-us	68.84	7.22	0.49	2.25	1324.60	11.10	3.88
cpr-ds	75.27	7.22	0.43	2.09	1381.35	10.50	3.28
hskc11	75.27	7.03	0.69	2.77	1397.35	10.50	3.48
hskc12u	75.26	6.56	0.89	3.37	1522.35	8.67	2.11
hskc12	75.26	6.24	1.44	3.91	1578.35	8.59	2.35
hskc12m	75.25	6.16	0.91	3.52	1593.35	9.15	2.99
hskc12d	75.25	5.81	1.44	3.90	1656.35	8.13	2.32
hskc13u	75.24	5.73	0.87	3.29	1671.33	7.80	2.07
hskc13	75.23	5.39	1.66	3.86	1734.33	7.69	2.30
hskc13d	75.23	5.35	0.84	3.15	1749.33	7.36	2.01
hskc14	75.23	5.05	0.82	3.12	1928.33	7.05	2.00
hskc15	75.25	4.81	0.75	2.91	2108.33	6.59	1.78
hskc15d	75.26	4.85	0.57	2.32	2219.33	5.66	0.81

Table 18 Model results - 1 in 10 year rain storm event.

Label	Flow (m³s⁻¹)	Stage (m PD)	Froude Number	Velocity (ms⁻¹)	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	88.84	13.98	1.00	2.75	0.00	16.33	2.35
yltm-01	88.76	13.97	0.97	2.29	28.80	16.63	2.66
yltm-02	88.71	13.83	1.12	2.82	46.00	15.75	1.92
yltm-03	88.70	13.90	0.60	1.72	78.60	16.11	2.22
yltm-04	88.72	13.80	1.05	2.09	102.60	13.14	-0.66
yltm-ds	95.45	13.80	0.00	0.00	102.60	13.00	-0.80
hskc01u	95.45	13.51	1.01	2.76	102.60	13.00	-0.51
hskc01ub	95.45	13.33	0.57	2.26	102.60	13.00	-0.33
hskc01	95.27	13.26	0.49	1.99	182.60	11.48	-1.78
hskc01ju	95.22	13.24	0.50	1.94	201.60	11.45	-1.79
hskc01jl	95.22	12.92	0.00	0.00	201.60	11.42	-1.50
hskc01jd	103.87	12.92	0.83	2.89	201.60	11.42	-1.50
hskc02	103.48	12.84	0.74	2.58	282.60	11.15	-1.69
hskc03	103.15	12.62	0.77	2.54	402.60	10.75	-1.87
hskc03b	106.41	12.49	0.91	2.95	402.60	10.75	-1.74
hskc03b1	106.59	12.38	0.99	3.09	486.60	10.47	-1.91
hskc03b2	106.59	12.25	0.88	3.17	486.60	10.47	-1.78
hskc04	105.97	12.10	0.89	3.16	542.60	10.28	-1.82
hskc04d	105.89	12.03	0.81	3.12	622.60	10.01	-2.02
hskc05	105.86	12.17	0.77	1.96	642.60	9.95	-2.22
hskc06u	105.83	12.00	0.76	2.73	666.60	9.87	-2.13
hskc06ub	105.83	11.85	0.81	3.05	666.60	9.87	-1.98
hskc06b	105.79	11.83	0.78	2.78	702.60	9.75	-2.08
hskc06b1	105.79	11.67	0.87	3.03	702.60	9.75	-1.92
hskc06	105.71	11.55	0.90	3.21	762.60	9.55	-2.00
hskc06b2	105.71	10.27	1.08	3.79	762.60	9.55	-0.72
hskc06d	105.65	9.32	1.41	5.17	862.60	9.21	-0.11
hskc07	105.63	8.98	0.86	3.63	902.60	9.47	0.49
hskc07b	105.63	8.71	0.97	4.15	902.60	9.47	0.76
hskc08u	105.62	8.88	0.58	2.63	902.60	9.47	0.59
hskc08	105.53	8.53	0.65	2.90	1062.60	9.47	0.94
hskc09	105.56	8.20	0.76	3.30	1142.60	9.94	1.74
hskc10	105.64	7.66	1.04	3.88	1276.60	9.39	1.73
cpr-us	105.68	7.75	0.54	2.77	1324.60	11.10	3.35
cpr-ds	118.14	7.76	0.51	2.67	1381.35	10.50	2.75
hskc11	118.15	7.54	0.75	3.27	1397.35	10.50	2.96
hskc12u	118.15	7.07	0.92	3.85	1522.35	8.67	1.60
hskc12	118.14	6.77	1.44	4.36	1578.35	8.59	1.82
hskc12m	118.14	6.68	0.95	4.06	1593.35	9.15	2.47
hskc12d	118.12	6.34	1.44	4.33	1656.35	8.13	1.79
hskc13u	118.12	6.26	0.89	3.74	1671.33	7.80	1.54
hskc13	118.08	5.95	1.66	4.21	1734.33	7.69	1.75
hskc13d	118.07	5.91	0.84	3.58	1749.33	7.36	1.45
hskc14	117.91	5.62	0.82	3.52	1928.33	7.05	1.44
hskc15	117.82	5.37	0.77	3.31	2108.33	6.59	1.22
hskc15d	117.86	5.45	0.59	2.61	2219.33	5.66	0.21

Table 19 Model results - 1 in 50 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	114.41	14.53	1.00	2.77	0.00	16.33	1.80
yltm-01	114.38	14.55	0.97	2.28	28.80	16.63	2.08
yltm-02	114.38	14.42	1.12	2.80	46.00	15.75	1.33
yltm-03	114.44	14.49	0.60	1.75	78.60	16.11	1.63
yltm-04	114.49	14.39	1.05	2.22	102.60	13.14	-1.25
yltm-ds	123.34	14.39	0.00	0.00	102.60	13.00	-1.39
hskc01u	123.34	14.05	1.01	2.76	102.60	13.00	-1.05
hskc01ub	123.34	13.92	0.58	2.34	102.60	13.00	-0.92
hskc01	123.44	13.85	0.49	2.04	182.60	11.48	-2.37
hskc01ju	123.46	13.84	0.50	1.98	201.60	11.45	-2.39
hskc01jl	123.46	13.53	0.00	0.00	201.60	11.42	-2.11
hskc01jd	134.50	13.53	0.84	2.87	201.60	11.42	-2.11
hskc02	134.95	13.46	0.76	2.57	282.60	11.15	-2.31
hskc03	135.58	13.28	0.74	2.53	402.60	10.75	-2.53
hskc03b	140.06	13.19	0.92	2.97	402.60	10.75	-2.44
hskc03b1	140.50	13.11	0.96	3.12	486.60	10.47	-2.64
hskc03b2	140.50	13.03	0.91	3.14	486.60	10.47	-2.56
hskc04	140.75	12.88	0.87	3.12	542.60	10.28	-2.60
hskc04d	141.00	12.80	0.78	3.06	622.60	10.01	-2.79
hskc05	141.08	12.94	0.79	1.98	642.60	9.95	-2.99
hskc06u	141.17	12.76	0.75	2.66	666.60	9.87	-2.89
hskc06ub	141.17	12.64	0.82	2.88	666.60	9.87	-2.77
hskc06b	141.24	12.61	0.80	2.78	702.60	9.75	-2.86
hskc06b1	141.24	12.48	0.83	3.02	702.60	9.75	-2.73
hskc06	141.27	12.38	0.87	3.21	762.60	9.55	-2.83
hskc06b2	141.27	10.83	1.08	3.81	762.60	9.55	-1.28
hskc06d	141.52	9.87	1.39	5.12	862.60	9.21	-0.66
hskc07	141.60	9.55	0.87	3.72	902.60	9.47	-0.08
hskc07b	141.60	9.17	0.98	4.42	902.60	9.47	0.30
hskc08u	141.61	9.34	0.59	2.88	902.60	9.47	0.13
hskc08	141.58	8.95	0.68	3.21	1062.60	9.47	0.52
hskc09	141.56	8.62	0.79	3.59	1142.60	9.94	1.32
hskc10	141.55	8.10	1.04	4.10	1276.60	9.39	1.29
cpr-us	141.54	8.15	0.59	3.22	1324.60	11.10	2.95
cpr-ds	160.85	8.15	0.59	3.18	1381.35	10.50	2.35
hskc11	160.83	7.98	0.78	3.62	1397.35	10.50	2.52
hskc12u	160.71	7.50	0.94	4.18	1522.35	8.67	1.17
hskc12	160.63	7.20	1.44	4.66	1578.35	8.59	1.40
hskc12m	160.61	7.11	0.98	4.46	1593.35	9.15	2.04
hskc12d	160.50	6.76	1.44	4.64	1656.35	8.13	1.37
hskc13u	160.46	6.68	0.90	4.06	1671.33	7.80	1.12
hskc13	160.28	6.37	1.66	4.50	1734.33	7.69	1.32
hskc13d	160.23	6.34	0.85	3.89	1749.33	7.36	1.02
hskc14	159.42	6.05	0.83	3.82	1928.33	7.05	1.00
hskc15	158.46	5.81	0.78	3.61	2108.33	6.59	0.78
hskc15d	158.61	5.90	0.60	2.87	2219.33	5.66	-0.24

Table 20 Model results - 1 in 200 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	135.81	14.89	1.00	2.76	0.00	16.33	1.44
yltm-01	135.78	14.92	0.92	2.26	28.80	16.63	1.71
yltm-02	135.80	14.79	1.12	2.80	46.00	15.75	0.95
yltm-03	135.83	14.87	0.60	1.74	78.60	16.11	1.25
yltm-04	135.85	14.75	1.00	2.21	102.60	13.14	-1.61
yltm-ds	146.19	14.75	0.00	0.00	102.60	13.00	-1.75
hskc01u	146.19	14.37	1.01	2.76	102.60	13.00	-1.37
hskc01ub	146.19	14.23	0.60	2.52	102.60	13.00	-1.23
hskc01	146.00	14.17	0.49	2.18	182.60	11.48	-2.69
hskc01ju	145.95	14.15	0.48	2.11	201.60	11.45	-2.70
hskc01jl	145.95	13.82	0.00	0.00	201.60	11.42	-2.40
hskc01jd	158.98	13.82	0.82	2.95	201.60	11.42	-2.40
hskc02	158.62	13.73	0.76	2.57	282.60	11.15	-2.58
hskc03	158.66	13.50	0.75	2.52	402.60	10.75	-2.75
hskc03b	163.63	13.38	0.93	3.00	402.60	10.75	-2.63
hskc03b1	163.72	13.28	0.99	3.11	486.60	10.47	-2.81
hskc03b2	163.72	13.17	0.87	3.29	486.60	10.47	-2.70
hskc04	163.77	12.95	0.91	3.10	542.60	10.28	-2.67
hskc04d	163.82	12.85	0.77	3.03	622.60	10.01	-2.84
hskc05	163.84	13.06	0.70	1.97	642.60	9.95	-3.11
hskc06u	163.86	12.80	0.76	2.69	666.60	9.87	-2.93
hskc06ub	163.86	12.62	0.80	2.96	666.60	9.87	-2.75
hskc06b	163.88	12.61	0.78	2.74	702.60	9.75	-2.86
hskc06b1	163.88	12.50	0.83	2.98	702.60	9.75	-2.75
hskc06	163.91	12.43	0.88	3.19	762.60	9.55	-2.88
hskc06b2	163.91	11.12	1.07	3.80	762.60	9.55	-1.57
hskc06d	163.93	10.10	1.44	5.21	862.60	9.21	-0.89
hskc07	163.93	9.76	0.87	3.95	902.60	9.47	-0.29
hskc07b	163.93	9.42	0.99	4.56	902.60	9.47	0.05
hskc08u	163.93	9.59	0.61	3.02	902.60	9.47	-0.12
hskc08	163.93	9.19	0.69	3.35	1062.60	9.47	0.28
hskc09	163.93	8.86	0.79	3.73	1142.60	9.94	1.08
hskc10	163.96	8.34	0.98	4.24	1276.60	9.39	1.05
cpr-us	163.99	8.35	0.62	3.50	1324.60	11.10	2.75
cpr-ds	185.59	8.36	0.62	3.43	1381.35	10.50	2.14
hskc11	185.58	8.21	0.80	3.77	1397.35	10.50	2.30
hskc12u	185.56	7.72	0.96	4.34	1522.35	8.67	0.95
hskc12	185.58	7.43	1.20	4.79	1578.35	8.59	1.16
hskc12m	185.58	7.35	1.00	4.62	1593.35	9.15	1.80
hskc12d	185.61	7.01	1.26	4.75	1656.35	8.13	1.12
hskc13u	185.61	6.94	0.92	4.19	1671.33	7.80	0.86
hskc13	185.64	6.64	1.46	4.59	1734.33	7.69	1.05
hskc13d	185.64	6.60	0.87	4.00	1749.33	7.36	0.76
hskc14	185.68	6.31	0.85	3.95	1928.33	7.05	0.74
hskc15	185.69	6.06	0.79	3.76	2108.33	6.59	0.53
hskc15d	185.70	6.15	0.60	3.03	2219.33	5.66	-0.49

Table 21 Existing remedial measures - 1 in 2 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	59.11	13.38	1.00	2.71	0.00	16.33	2.95
yltm-01	59.02	13.26	0.97	2.26	28.80	16.63	3.37
yltm-02	58.96	13.07	1.12	2.81	46.00	15.75	2.68
yltm-03	58.87	13.13	0.61	1.74	78.60	16.11	2.98
yltm-04	58.79	13.05	1.05	2.17	102.60	13.14	0.09
yltm-ds	63.19	13.05	0.00	0.00	102.60	13.00	-0.05
hskc01u	63.19	12.82	1.01	2.76	102.60	13.00	0.18
hskc01ub	63.19	12.72	0.54	2.54	102.60	13.00	0.28
hskc01	61.56	12.64	0.45	2.38	182.60	12.48	-0.16
hskc01ju	60.27	12.64	0.45	2.31	201.60	12.45	-0.19
hskc01jl	60.27	11.86	0.00	0.00	201.60	12.42	0.56
hskc01jd	67.66	11.86	0.81	2.95	201.60	12.42	0.56
hskc02	67.42	11.68	0.74	2.61	282.60	11.15	-0.53
hskc03	67.24	11.33	0.75	2.49	402.60	10.75	-0.58
hskc03b	68.84	11.14	0.91	2.94	402.60	10.75	-0.39
hskc03b1	68.71	10.90	0.98	3.08	486.60	10.47	-0.43
hskc03b2	68.71	10.71	0.89	3.19	486.60	10.47	-0.24
hskc04	68.62	10.53	0.85	3.17	542.60	10.28	-0.25
hskc04d	68.48	10.39	0.79	3.10	622.60	10.01	-0.38
hskc05	68.44	10.63	0.77	1.96	642.60	9.95	-0.68
hskc06u	68.40	10.38	0.76	2.71	666.60	9.87	-0.51
hskc06ub	68.40	10.19	0.81	3.02	666.60	9.87	-0.32
hskc06b	68.35	10.19	0.79	2.75	702.60	9.75	-0.44
hskc06b1	68.35	10.00	0.85	3.02	702.60	9.75	-0.25
hskc06	68.31	9.70	0.88	3.15	762.60	9.55	-0.15
hskc06b2	68.31	9.49	0.61	3.55	762.60	9.55	0.06
hskc06d	68.28	8.47	0.84	4.40	862.60	9.21	0.74
hskc07	68.30	8.25	0.87	3.48	902.60	9.47	1.22
hskc07b	68.30	8.12	0.95	3.77	902.60	9.47	1.35
hskc08u	68.31	8.29	0.55	2.30	902.60	9.47	1.18
hskc08	68.35	7.97	0.61	2.53	1062.60	9.47	1.50
hskc09	68.37	7.67	0.72	2.88	1142.60	9.94	2.27
hskc10	68.41	7.11	1.04	3.55	1276.60	9.39	2.28
cpr-us	68.43	7.21	0.49	2.25	1324.60	11.10	3.89
cpr-ds	74.86	7.22	0.43	2.09	1381.35	10.50	3.28
hskc11	74.86	7.02	0.69	2.77	1397.35	10.50	3.48
hskc12u	74.87	6.56	0.89	3.36	1522.35	8.67	2.12
hskc12	74.87	6.24	1.44	3.91	1578.35	8.59	2.35
hskc12m	74.87	6.15	0.91	3.51	1593.35	9.15	3.00
hskc12d	74.87	5.80	1.44	3.90	1656.35	8.13	2.33
hskc13u	74.87	5.72	0.87	3.28	1671.33	7.80	2.08
hskc13	74.86	5.39	1.66	3.86	1734.33	7.69	2.30
hskc13d	74.86	5.35	0.84	3.15	1749.33	7.36	2.01
hskc14	74.84	5.04	0.81	3.12	1928.33	7.05	2.01
hskc15	74.82	4.80	0.75	2.91	2108.33	6.59	1.79
hskc15d	74.83	4.84	0.57	2.31	2219.33	5.66	0.82

Table 22 Existing remedial measures - 1 in 10 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	88.84	14.02	1.00	2.81	0.00	16.33	2.31
yltm-01	88.71	14.02	0.97	2.31	28.80	16.63	2.62
yltm-02	88.66	13.88	1.12	2.83	46.00	15.75	1.87
yltm-03	88.68	13.94	0.61	1.77	78.60	16.11	2.17
yltm-04	88.70	13.85	1.05	2.26	102.60	13.14	-0.71
yltm-ds	95.30	13.85	0.00	0.00	102.60	13.00	-0.85
hskc01u	95.30	13.57	1.01	2.76	102.60	13.00	-0.57
hskc01ub	95.30	13.35	0.54	3.42	102.60	13.00	-0.35
hskc01	89.05	13.33	0.45	2.99	182.60	12.48	-0.85
hskc01ju	80.73	13.48	0.45	2.62	201.60	12.45	-1.03
hskc01jl	80.73	12.91	0.00	0.00	201.60	12.42	-0.49
hskc01jd	103.65	12.91	0.83	2.89	201.60	12.42	-0.49
hskc02	103.72	12.83	0.74	2.56	282.60	11.15	-1.68
hskc03	103.21	12.62	0.77	2.54	402.60	10.75	-1.87
hskc03b	106.55	12.47	0.91	2.94	402.60	10.75	-1.72
hskc03b1	105.95	12.39	0.99	3.10	486.60	10.47	-1.92
hskc03b2	105.95	12.24	0.90	3.16	486.60	10.47	-1.77
hskc04	105.88	12.10	0.86	3.17	542.60	10.28	-1.82
hskc04d	105.79	12.03	0.80	3.12	622.60	10.01	-2.02
hskc05	105.76	12.17	0.77	1.96	642.60	9.95	-2.22
hskc06u	105.73	12.00	0.77	2.72	666.60	9.87	-2.13
hskc06ub	105.73	11.85	0.83	3.05	666.60	9.87	-1.98
hskc06b	105.69	11.83	0.77	2.78	702.60	9.75	-2.08
hskc06b1	105.69	11.66	0.87	3.03	702.60	9.75	-1.91
hskc06	105.60	11.55	0.89	3.18	762.60	9.55	-2.00
hskc06b2	105.60	10.27	1.07	3.76	762.60	9.55	-0.72
hskc06d	105.55	9.31	1.42	5.11	862.60	9.21	-0.10
hskc07	105.52	8.98	0.86	3.64	902.60	9.47	0.49
hskc07b	105.52	8.71	0.97	4.16	902.60	9.47	0.76
hskc08u	105.51	8.88	0.58	2.64	902.60	9.47	0.59
hskc08	105.42	8.53	0.65	2.90	1062.60	9.47	0.94
hskc09	105.45	8.20	0.77	3.30	1142.60	9.94	1.75
hskc10	105.53	7.66	1.04	3.88	1276.60	9.39	1.73
cpr-us	105.56	7.75	0.54	2.77	1324.60	11.10	3.35
cpr-ds	117.97	7.75	0.51	2.67	1381.35	10.50	2.75
hskc11	117.96	7.54	0.75	3.28	1397.35	10.50	2.96
hskc12u	117.94	7.07	0.92	3.84	1522.35	8.67	1.60
hskc12	117.91	6.76	1.44	4.36	1578.35	8.59	1.83
hskc12m	117.91	6.68	0.95	4.06	1593.35	9.15	2.47
hskc12d	117.87	6.33	1.44	4.33	1656.35	8.13	1.80
hskc13u	117.86	6.26	0.89	3.73	1671.33	7.80	1.54
hskc13	117.80	5.94	1.66	4.21	1734.33	7.69	1.75
hskc13d	117.78	5.91	0.84	3.57	1749.33	7.36	1.45
hskc14	117.59	5.61	0.82	3.51	1928.33	7.05	1.44
hskc15	117.64	5.37	0.77	3.31	2108.33	6.59	1.22
hskc15d	117.67	5.45	0.59	2.61	2219.33	5.66	0.21

Table 23 Existing remedial measures - 1 in 50 year rainstorm event.

Label –	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	114.42	15.08	1.00	2.79	0.00	16.33	1.26
yltm-01	114.35	15.09	0.97	2.29	28.80	16.63	1.54
yltm-02	114.30	15.02	1.12	2.81	46.00	15.75	0.72
yltm-03	114.20	15.06	0.60	1.76	78.60	16.11	1.05
yltm-04	114.14	14.99	1.05	2.24	102.60	13.14	-1.85
yltm-ds	123.16	14.99	0.00	0.00	102.60	13.00	-1.99
hskc01u	123.16	14.75	1.01	2.76	102.60	13.00	-1.75
hskc01ub	123.16	13.93	0.56	4.14	102.60	13.00	-0.93
hskc01	119.05	13.81	0.49	3.68	182.60	12.48	-1.33
hskc01ju	109.53	14.02	0.45	3.20	201.60	12.45	-1.57
hskc01jl	109.53	13.56	0.00	0.00	201.60	12.42	-1.14
hskc01jd	134.42	13.56	0.83	2.87	201.60	12.42	-1.14
hskc02	135.19	13.47	0.75	2.57	282.60	11.15	-2.32
hskc03	136.32	13.26	0.74	2.53	402.60	10.75	-2.51
hskc03b	140.80	13.16	0.91	2.96	402.60	10.75	-2.41
hskc03b1	141.67	13.08	0.96	3.12	486.60	10.47	-2.61
hskc03b2	141.67	13.01	0.91	3.13	486.60	10.47	-2.54
hskc04	142.20	12.88	0.87	3.12	542.60	10.28	-2.60
hskc04d	142.76	12.81	0.78	3.06	622.60	10.01	-2.80
hskc05	142.93	12.94	0.79	1.98	642.60	9.95	-2.99
hskc06u	143.12	12.78	0.75	2.66	666.60	9.87	-2.91
hskc06ub	143.12	12.68	0.82	2.88	666.60	9.87	-2.81
hskc06b	143.30	12.66	0.79	2.77	702.60	9.75	-2.91
hskc06b1	143.30	12.55	0.87	3.02	702.60	9.75	-2.80
hskc06	143.55	12.46	0.89	3.21	762.60	9.55	-2.91
hskc06b2	143.55	10.85	1.08	3.81	762.60	9.55	-1.30
hskc06d	143.71	9.89	1.43	5.08	862.60	9.21	-0.68
hskc07	143.76	9.57	0.87	3.74	902.60	9.47	-0.10
hskc07b	143.76	9.18	0.98	4.45	902.60	9.47	0.29
hskc08u	143.73	9.35	0.60	2.91	902.60	9.47	0.12
hskc08	143.26	8.95	0.69	3.24	1062.60	9.47	0.52
hskc09	142.96	8.63	0.79	3.62	1142.60	9.94	1.31
hskc10	142.61	8.10	1.04	4.13	1276.60	9.39	1.29
cpr-us	142.51	8.15	0.59	3.25	1324.60	11.10	2.96
cpr-ds	161.70	8.15	0.59	3.21	1381.35	10.50	2.35
hskc11	161.63	7.98	0.79	3.65	1397.35	10.50	2.53
hskc12u	160.97	7.49	0.94	4.21	1522.35	8.67	1.18
hskc12	160.65	7.19	1.44	4.68	1578.35	8.59	1.40
hskc12m	160.57	7.10	0.98	4.48	1593.35	9.15	2.05
hskc12d	160.21	6.76	1.44	4.65	1656.35	8.13	1.37
hskc13u	160.12	6.68	0.91	4.08	1671.33	7.80	1.12
hskc13	159.70	6.38	1.66	4.53	1734.33	7.69	1.31
hskc13d	159.59	6.34	0.86	3.91	1749.33	7.36	1.02
hskc14	158.69	6.06	0.84	3.83	1928.33	7.05	0.99
hskc15	159.07	5.81	0.79	3.60	2108.33	6.59	0.78
hskc15d	159.32	5.90	0.60	2.87	2219.33	5.66	-0.24

Table 24 *Existing remedial measures - 1 in 200 year rain storm event.*

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	135.82	15.18	1.00	2.83	0.00	16.33	1.16
yltm-01	135.79	15.20	0.97	2.25	28.80	16.63	1.43
yltm-02	135.76	15.11	1.12	2.81	46.00	15.75	0.64
yltm-03	135.74	15.17	0.60	1.74	78.60	16.11	0.95
yltm-04	135.79	15.06	1.05	2.21	102.60	13.14	-1.92
yltm-ds	146.31	15.06	0.00	0.00	102.60	13.00	-2.06
hskc01u	146.31	14.78	1.01	2.76	102.60	13.00	-1.78
hskc01ub	146.31	14.22	0.62	4.74	102.60	13.00	-1.22
hskc01	142.46	14.03	0.56	4.28	182.60	12.48	-1.55
hskc01ju	132.18	14.29	0.47	3.69	201.60	12.45	-1.84
hskc01jl	132.18	13.82	0.00	0.00	201.60	12.42	-1.40
hskc01jd	159.01	13.82	0.81	2.96	201.60	12.42	-1.40
hskc02	158.67	13.73	0.75	2.58	282.60	11.15	-2.58
hskc03	158.75	13.50	0.75	2.51	402.60	10.75	-2.75
hskc03b	163.76	13.38	0.93	3.00	402.60	10.75	-2.63
hskc03b1	163.76	13.28	0.99	3.10	486.60	10.47	-2.81
hskc03b2	163.76	13.17	1.01	3.27	486.60	10.47	-2.70
hskc04	163.81	12.95	0.90	3.11	542.60	10.28	-2.67
hskc04d	163.86	12.85	0.77	3.04	622.60	10.01	-2.84
hskc05	163.88	13.06	0.70	1.98	642.60	9.95	-3.11
hskc06u	163.90	12.80	0.76	2.69	666.60	9.87	-2.93
hskc06ub	163.90	12.61	0.80	2.97	666.60	9.87	-2.74
hskc06b	163.91	12.58	0.77	2.73	702.60	9.75	-2.83
hskc06b1	163.91	12.47	0.83	2.99	702.60	9.75	-2.72
hskc06	163.92	12.40	0.87	3.20	762.60	9.55	-2.85
hskc06b2	163.92	11.12	1.07	3.81	762.60	9.55	-1.57
hskc06d	163.94	10.10	1.43	5.21	862.60	9.21	-0.89
hskc07	163.95	9.76	0.87	3.95	902.60	9.47	-0.29
hskc07b	163.95	9.42	0.99	4.56	902.60	9.47	0.05
hskc08u	163.95	9.59	0.61	3.02	902.60	9.47	-0.12
hskc08	163.97	9.19	0.69	3.35	1062.60	9.47	0.28
hskc09	164.00	8.86	0.79	3.73	1142.60	9.94	1.08
hskc10	164.05	8.34	1.04	4.24	1276.60	9.39	1.05
cpr-us	164.08	8.35	0.62	3.50	1324.60	11.10	2.75
cpr-ds	185.95	8.36	0.62	3.44	1381.35	10.50	2.14
hskc11	185.95	8.21	0.80	3.78	1397.35	10.50	2.29
hskc12u	185.94	7.73	0.95	4.35	1522.35	8.67	0.95
hskc12	185.94	7.43	1.44	4.80	1578.35	8.59	1.16
hskc12m	185.94	7.35	0.99	4.63	1593.35	9.15	1.80
hskc12d	185.93	7.01	1.44	4.75	1656.35	8.13	1.12
hskc13u	185.93	6.94	0.91	4.21	1671.33	7.80	0.86
hskc13	185.91	6.64	1.66	4.60	1734.33	7.69	1.05
hskc13d	185.91	6.61	0.86	4.02	1749.33	7.36	0.75
hskc14	185.88	6.31	0.84	3.95	1928.33	7.05	0.74
hskc15	185.95	6.06	0.79	3.76	2108.33	6.59	0.53
hskc15d	185.98	6.16	0.60	3.03	2219.33	5.66	-0.50

Table 25 Option 1A - 1 in 2 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	59.11	13.27	1.00	2.96	0.00	16.33	3.06
yltm-01	59.01	12.90	0.92	2.63	28.80	16.63	3.73
yltm-02	58.96	12.66	1.12	3.25	46.00	15.75	3.09
yltm-03	58.85	12.41	0.61	2.12	78.60	16.11	3.71
yltm-04	58.77	12.13	1.05	2.63	102.60	13.14	1.01
yltm-ds	63.17	12.13	0.00	0.00	102.60	13.00	0.87
hskc01u	63.17	10.89	1.01	2.76	102.60	13.00	2.12
hskc01ub	63.17	10.89	0.77	2.03	102.60	13.00	2.12
hskc01	62.95	10.83	0.58	1.87	182.60	12.48	1.65
hskc01ju	62.90	10.82	0.61	1.86	201.60	12.45	1.63
hskc01jl	62.90	10.28	0.00	0.00	201.60	12.42	2.14
hskc01jd	68.76	10.28	0.75	2.46	201.60	12.42	2.14
hskc02	68.62	10.19	0.56	2.29	282.60	12.15	1.96
hskc03	68.45	10.00	0.41	2.12	402.60	11.75	1.75
hskc03b	70.41	10.00	0.41	2.17	402.60	11.75	1.75
hskc03b1	70.30	9.88	1.14	2.45	486.60	11.47	1.59
hskc03b2	70.30	9.12	0.84	2.76	486.60	11.47	2.35
hskc04	70.31	9.02	0.81	2.66	542.60	11.28	2.26
hskc04d	70.44	8.93	0.68	2.46	622.60	11.01	2.08
hskc05	70.49	9.05	0.62	1.85	642.60	10.95	1.90
hskc06u	70.54	8.92	0.54	2.34	666.60	10.87	1.95
hskc06ub	70.54	8.81	0.57	2.45	666.60	10.87	2.06
hskc06b	70.54	8.80	0.47	2.35	702.60	10.75	1.95
hskc06b1	70.54	8.80	0.47	2.35	702.60	10.75	1.95
hskc06	70.54	8.69	0.44	2.27	762.60	10.55	1.86
hskc06b2	70.54	8.69	0.44	2.27	762.60	10.55	1.86
hskc06d	70.52	8.54	0.40	2.12	862.60	10.21	1.67
hskc07	70.52	8.30	0.87	3.50	902.60	9.47	1.17
hskc07b	70.52	8.15	0.95	3.80	902.60	9.47	1.32
hskc08u	70.52	8.33	0.55	2.32	902.60	9.47	1.14
hskc08	70.50	8.01	0.62	2.56	1062.60	9.47	1.47
hskc09	70.47	7.71	0.73	2.90	1142.60	9.94	2.24
hskc10	70.46	7.15	0.95	3.56	1276.60	9.39	2.24
cpr-us	70.48	7.25	0.49	2.27	1324.60	11.10	3.85
cpr-ds	77.40	7.26	0.43	2.12	1381.35	10.50	3.24
hskc11	77.40	7.06	0.69	2.80	1397.35	10.50	3.44
hskc12u	77.44	6.59	0.89	3.40	1522.35	8.67	2.08
hskc12	77.45	6.28	1.25	3.93	1578.35	8.59	2.32
hskc12m	77.45	6.19	0.91	3.55	1593.35	9.15	2.96
hskc12d	77.46	5.84	1.30	3.92	1656.35	8.13	2.29
hskc13u	77.46	5.76	0.87	3.32	1671.33	7.80	2.04
hskc13	77.46	5.42	1.51	3.87	1734.33	7.69	2.27
hskc13d	77.46	5.38	0.85	3.18	1749.33	7.36	1.98
hskc14	77.44	5.08	0.82	3.15	1928.33	7.05	1.97
hskc15	77.38	4.84	0.75	2.93	2108.33	6.59	1.75
hskc15d	77.33	4.88	0.57	2.33	2219.33	5.66	0.78

Table 26 Option 1A - 1 in 10 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	88.84	13.68	1.00	3.28	0.00	16.33	2.65
yltm-01	88.76	13.34	0.92	2.88	28.80	16.63	3.29
yltm-02	88.71	13.12	1.12	3.50	46.00	15.75	2.63
yltm-03	88.66	12.99	0.61	2.28	78.60	16.11	3.12
yltm-04	88.69	12.70	1.05	2.88	102.60	13.14	0.44
yltm-ds	95.44	12.70	0.00	0.00	102.60	13.00	0.30
hskc01u	95.44	11.81	1.01	2.76	102.60	13.00	1.19
hskc01ub	95.44	11.81	0.76	2.29	102.60	13.00	1.19
hskc01	95.31	11.76	0.58	2.16	182.60	12.48	0.72
hskc01ju	95.28	11.74	0.60	2.15	201.60	12.45	0.71
hskc01jl	95.28	11.03	0.00	0.00	201.60	12.42	1.39
hskc01jd	103.93	11.03	0.74	2.84	201.60	12.42	1.39
hskc02	103.86	10.93	0.56	2.70	282.60	12.15	1.22
hskc03	103.79	10.69	0.44	2.58	402.60	11.75	1.06
hskc03b	107.13	10.69	0.45	2.66	402.60	11.75	1.06
hskc03b1	107.10	10.52	1.14	2.77	486.60	11.47	0.95
hskc03b2	107.10	9.93	0.83	3.08	486.60	11.47	1.54
hskc04	107.05	9.83	0.81	2.98	542.60	11.28	1.45
hskc04d	106.97	9.75	0.68	2.82	622.60	11.01	1.26
hskc05	106.95	9.93	0.62	2.03	642.60	10.95	1.02
hskc06u	106.92	9.74	0.54	2.71	666.60	10.87	1.13
hskc06ub	106.92	9.64	0.57	2.80	666.60	10.87	1.23
hskc06b	106.86	9.62	0.47	2.71	702.60	10.75	1.13
hskc06b1	106.86	9.62	0.47	2.71	702.60	10.75	1.13
hskc06	106.79	9.50	0.45	2.65	762.60	10.55	1.05
hskc06b2	106.79	9.50	0.45	2.65	762.60	10.55	1.05
hskc06d	106.89	9.33	0.42	2.53	862.60	10.21	0.88
hskc07	106.93	9.01	0.86	3.62	902.60	9.47	0.46
hskc07b	106.93	8.73	0.97	4.16	902.60	9.47	0.74
hskc08u	106.94	8.90	0.57	2.64	902.60	9.47	0.57
hskc08	107.03	8.55	0.65	2.91	1062.60	9.47	0.92
hskc09	107.08	8.22	0.76	3.31	1142.60	9.94	1.72
hskc10	107.18	7.68	0.95	3.88	1276.60	9.39	1.71
cpr-us	107.23	7.78	0.54	2.78	1324.60	11.10	3.32
cpr-ds	120.47	7.78	0.51	2.70	1381.35	10.50	2.72
hskc11	120.47	7.57	0.75	3.28	1397.35	10.50	2.93
hskc12u	120.47	7.10	0.92	3.86	1522.35	8.67	1.57
hskc12	120.47	6.79	1.24	4.35	1578.35	8.59	1.80
hskc12m	120.47	6.71	0.95	4.07	1593.35	9.15	2.44
hskc12d	120.46	6.37	1.29	4.32	1656.35	8.13	1.77
hskc13u	120.45	6.29	0.89	3.74	1671.33	7.80	1.51
hskc13	120.44	5.98	1.49	4.19	1734.33	7.69	1.71
hskc13d	120.43	5.94	0.85	3.58	1749.33	7.36	1.42
hskc14	120.36	5.65	0.82	3.53	1928.33	7.05	1.40
hskc15	120.41	5.41	0.77	3.33	2108.33	6.59	1.18
hskc15d	120.44	5.49	0.59	2.63	2219.33	5.66	0.17

Table 27 Option 1A - 1 in 50 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	114.41	14.00	1.00	3.43	0.00	16.33	2.33
yltm-01	114.33	13.76	0.92	2.90	28.80	16.63	2.87
yltm-02	114.34	13.51	1.12	3.59	46.00	15.75	2.24
yltm-03	114.36	13.47	0.61	2.31	78.60	16.11	2.64
yltm-04	114.38	13.17	1.05	2.98	102.60	13.14	-0.03
yltm-ds	123.10	13.17	0.00	0.00	102.60	13.00	-0.17
hskc01u	123.10	12.31	1.01	2.76	102.60	13.00	0.69
hskc01ub	123.10	12.31	0.76	2.60	102.60	13.00	0.69
hskc01	122.96	12.24	0.58	2.47	182.60	12.48	0.24
hskc01ju	122.93	12.22	0.60	2.47	201.60	12.45	0.23
hskc01jl	122.93	11.35	0.00	0.00	201.60	12.42	1.07
hskc01jd	133.97	11.35	0.74	3.33	201.60	12.42	1.07
hskc02	133.91	11.20	0.56	3.22	282.60	12.15	0.95
hskc03	133.86	10.81	0.54	3.21	402.60	11.75	0.94
hskc03b	138.34	10.81	0.56	3.32	402.60	11.75	0.94
hskc03b1	138.35	10.57	1.14	3.61	486.60	11.47	0.90
hskc03b2	138.35	10.43	0.83	3.41	486.60	11.47	1.04
hskc04	138.30	10.32	0.81	3.33	542.60	11.28	0.96
hskc04d	138.22	10.22	0.68	3.18	622.60	11.01	0.79
hskc05	138.20	10.46	0.62	2.23	642.60	10.95	0.49
hskc06u	138.17	10.22	0.54	3.07	666.60	10.87	0.65
hskc06ub	138.17	10.22	0.57	3.07	666.60	10.87	0.65
hskc06b	138.14	10.20	0.48	2.99	702.60	10.75	0.55
hskc06b1	138.14	10.20	0.48	2.99	702.60	10.75	0.55
hskc06	138.09	10.06	0.47	2.94	762.60	10.55	0.49
hskc06b2	138.09	10.06	0.47	2.94	762.60	10.55	0.49
hskc06d	138.02	9.86	0.44	2.85	862.60	10.21	0.35
hskc07	137.98	9.52	0.87	3.68	902.60	9.47	-0.05
hskc07b	137.98	9.13	0.97	4.39	902.60	9.47	0.34
hskc08u	137.96	9.31	0.59	2.84	902.60	9.47	0.16
hskc08	137.97	8.92	0.68	3.17	1062.60	9.47	0.55
hskc09	138.06	8.59	0.78	3.56	1142.60	9.94	1.35
hskc10	138.22	8.06	0.95	4.08	1276.60	9.39	1.33
cpr-us	138.28	8.13	0.58	3.18	1324.60	11.10	2.97
cpr-ds	156.99	8.13	0.58	3.13	1381.35	10.50	2.37
hskc11	156.99	7.95	0.77	3.58	1397.35	10.50	2.55
hskc12u	157.03	7.47	0.94	4.14	1522.35	8.67	1.20
hskc12	157.04	7.17	1.24	4.61	1578.35	8.59	1.42
hskc12m	157.05	7.09	0.97	4.41	1593.35	9.15	2.06
hskc12d	157.06	6.75	1.29	4.57	1656.35	8.13	1.39
hskc13u	157.06	6.67	0.90	4.01	1671.33	7.80	1.13
hskc13	157.06	6.36	1.49	4.45	1734.33	7.69	1.33
hskc13d	157.06	6.32	0.85	3.83	1749.33	7.36	1.04
hskc14	157.04	6.03	0.83	3.77	1928.33	7.05	1.02
hskc15	157.00	5.79	0.78	3.59	2108.33	6.59	0.80
hskc15d	156.97	5.88	0.60	2.86	2219.33	5.66	-0.22

Table 28 Option 1A - 1 in 200 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	135.82	14.27	1.00	3.52	0.00	16.33	2.06
yltm-01	135.77	14.11	0.92	2.89	28.80	16.63	2.52
yltm-02	135.78	13.82	1.12	3.60	46.00	15.75	1.93
yltm-03	135.81	13.86	0.61	2.33	78.60	16.11	2.25
yltm-04	135.84	13.57	1.05	3.01	102.60	13.14	-0.43
yltm-ds	146.26	13.57	0.00	0.00	102.60	13.00	-0.57
hskc01u	146.26	12.70	1.01	2.76	102.60	13.00	0.30
hskc01ub	146.26	12.70	0.76	2.81	102.60	13.00	0.30
hskc01	146.13	12.63	0.57	2.70	182.60	12.48	-0.15
hskc01ju	146.10	12.61	0.60	2.69	201.60	12.45	-0.16
hskc01jl	146.10	11.75	0.00	0.00	201.60	12.42	0.67
hskc01jd	159.13	11.75	0.74	3.55	201.60	12.42	0.67
hskc02	159.02	11.59	0.56	3.45	282.60	12.15	0.56
hskc03	158.88	11.15	0.56	3.48	402.60	11.75	0.60
hskc03b	164.31	11.15	0.58	3.60	402.60	11.75	0.60
hskc03b1	164.22	10.78	1.14	3.94	486.60	11.47	0.69
hskc03b2	164.22	10.78	0.83	3.69	486.60	11.47	0.69
hskc04	164.17	10.65	0.81	3.63	542.60	11.28	0.64
hskc04d	164.10	10.53	0.68	3.49	622.60	11.01	0.48
hskc05	164.08	10.83	0.62	2.41	642.60	10.95	0.12
hskc06u	164.05	10.52	0.54	3.38	666.60	10.87	0.35
hskc06ub	164.05	10.52	0.57	3.38	666.60	10.87	0.35
hskc06b	164.02	10.50	0.51	3.31	702.60	10.75	0.25
hskc06b1	164.02	10.50	0.51	3.31	702.60	10.75	0.25
hskc06	163.98	10.33	0.50	3.28	762.60	10.55	0.22
hskc06b2	163.98	10.33	0.50	3.28	762.60	10.55	0.22
hskc06d	163.91	10.07	0.49	3.22	862.60	10.21	0.14
hskc07	163.91	9.76	0.87	3.95	902.60	9.47	-0.29
hskc07b	163.91	9.42	0.98	4.56	902.60	9.47	0.05
hskc08u	163.93	9.60	0.61	3.02	902.60	9.47	-0.13
hskc08	164.06	9.19	0.69	3.35	1062.60	9.47	0.28
hskc09	164.14	8.86	0.79	3.73	1142.60	9.94	1.08
hskc10	164.29	8.34	0.97	4.24	1276.60	9.39	1.05
cpr-us	164.35	8.36	0.62	3.51	1324.60	11.10	2.74
cpr-ds	186.93	8.37	0.62	3.44	1381.35	10.50	2.13
hskc11	186.93	8.22	0.79	3.78	1397.35	10.50	2.28
hskc12u	186.95	7.73	0.95	4.35	1522.35	8.67	0.94
hskc12	186.96	7.44	1.24	4.78	1578.35	8.59	1.15
hskc12m	186.96	7.36	0.99	4.63	1593.35	9.15	1.79
hskc12d	186.96	7.02	1.29	4.73	1656.35	8.13	1.11
hskc13u	186.96	6.95	0.91	4.19	1671.33	7.80	0.85
hskc13	186.95	6.65	1.49	4.56	1734.33	7.69	1.04
hskc13d	186.95	6.62	0.86	4.00	1749.33	7.36	0.75
hskc14	186.91	6.32	0.84	3.96	1928.33	7.05	0.73
hskc15	186.85	6.07	0.79	3.77	2108.33	6.59	0.52
hskc15d	186.81	6.17	0.60	3.03	2219.33	5.66	-0.51

Table 29 Option 1B - 1 in 50 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	114.42	14.04	1.00	3.35	0.00	16.33	2.29
yltm-01	114.33	13.88	0.92	2.82	28.80	16.63	2.76
yltm-02	114.34	13.58	1.12	3.46	46.00	15.75	2.17
yltm-03	114.37	13.62	0.61	2.23	78.60	16.11	2.49
yltm-04	114.40	13.38	1.05	2.78	102.60	13.14	-0.24
yltm-ds	123.06	13.38	0.00	0.00	102.60	13.00	-0.38
hskc01u	123.06	12.69	1.01	2.76	102.60	13.00	0.31
hskc01ub	123.06	12.69	0.71	2.60	102.60	13.00	0.31
hskc01	122.90	12.62	0.56	2.49	182.60	12.48	-0.14
hskc01ju	122.87	12.60	0.58	2.48	201.60	12.45	-0.15
hskc01jl	122.87	11.57	0.00	0.00	201.60	12.42	0.85
hskc01jd	133.91	11.57	0.70	3.43	201.60	12.42	0.85
hskc02	133.79	11.40	0.56	3.34	282.60	12.15	0.75
hskc03	133.63	10.95	0.56	3.38	402.60	11.75	0.80
hskc03b	138.11	10.95	0.58	3.49	402.60	11.75	0.80
hskc03b1	138.01	10.56	1.23	3.87	486.60	11.47	0.92
hskc03b2	138.01	10.56	0.77	3.60	486.60	11.47	0.92
hskc04	137.95	10.41	0.81	3.55	542.60	11.28	0.87
hskc04d	137.87	10.29	0.70	3.42	622.60	11.01	0.72
hskc05	137.85	10.59	0.65	2.31	642.60	10.95	0.36
hskc06u	137.82	10.28	0.57	3.31	666.60	10.87	0.59
hskc06ub	137.82	10.28	0.57	3.31	666.60	10.87	0.59
hskc06b	137.78	10.25	0.51	3.23	702.60	10.75	0.50
hskc06b1	137.78	10.25	0.51	3.23	702.60	10.75	0.50
hskc06	137.78	10.08	0.51	3.21	762.60	10.55	0.48
hskc06b2	137.78	10.08	0.51	3.21	762.60	10.55	0.48
hskc06d	137.84	9.79	0.50	3.17	862.60	10.21	0.42
hskc07	137.86	9.52	0.87	3.68	902.60	9.47	-0.05
hskc07b	137.86	9.13	0.97	4.39	902.60	9.47	0.34
hskc08u	137.87	9.31	0.59	2.84	902.60	9.47	0.17
hskc08	137.97	8.92	0.68	3.17	1062.60	9.47	0.55
hskc09	138.03	8.59	0.78	3.56	1142.60	9.94	1.35
hskc10	138.14	8.06	0.95	4.08	1276.60	9.39	1.33
cpr-us	138.19	8.12	0.58	3.18	1324.60	11.10	2.98
cpr-ds	156.53	8.13	0.58	3.12	1381.35	10.50	2.37
hskc11	156.53	7.95	0.77	3.57	1397.35	10.50	2.55
hskc12u	156.55	7.47	0.94	4.14	1522.35	8.67	1.20
hskc12	156.56	7.17	1.24	4.60	1578.35	8.59	1.42
hskc12m	156.56	7.09	0.97	4.40	1593.35	9.15	2.07
hskc12d	156.56	6.74	1.29	4.57	1656.35	8.13	1.39
hskc13u	156.56	6.67	0.90	4.00	1671.33	7.80	1.14
hskc13	156.56	6.36	1.49	4.44	1734.33	7.69	1.33
hskc13d	156.56	6.32	0.85	3.83	1749.33	7.36	1.04
hskc14	156.53	6.03	0.84	3.77	1928.33	7.05	1.02
hskc15	156.47	5.79	0.78	3.59	2108.33	6.59	0.80
hskc15d	156.44	5.87	0.60	2.86	2219.33	5.66	-0.21

Table 30 Option 2 - 1 in 10 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	88.84	14.18	1.00	2.79	0.00	16.33	2.16
yltm-01	88.68	14.18	0.97	2.44	28.80	16.63	2.45
yltm-02	88.72	14.07	1.12	2.98	46.00	15.75	1.68
yltm-03	88.79	14.12	0.61	1.86	78.60	16.11	1.99
yltm-04	88.85	14.05	1.05	2.34	102.60	13.14	-0.91
yltm-ds	95.04	14.05	0.00	0.00	102.60	13.00	-1.05
hskc01u	95.04	13.80	1.01	2.76	102.60	13.00	-0.80
hskc01ub	95.04	13.80	0.58	3.09	102.60	13.00	-0.80
hskc01	95.06	13.61	0.52	3.04	182.60	14.48	0.87
hskc01ju	95.07	13.55	0.54	3.05	201.60	14.45	0.90
hskc01jl	95.07	13.35	0.00	0.00	201.60	14.42	1.07
hskc01jd	103.53	13.35	0.58	3.43	201.60	14.42	1.07
hskc02	103.40	13.09	0.53	3.42	282.60	14.15	1.06
hskc03	103.39	12.39	0.54	3.62	402.60	13.75	1.36
hskc03b	106.57	12.39	0.58	3.74	402.60	13.75	1.36
hskc03b1	106.46	11.83	1.01	4.18	486.60	13.47	1.64
hskc03b2	106.46	11.83	0.65	3.94	486.60	13.47	1.64
hskc04	106.42	11.54	0.69	4.03	542.60	13.28	1.74
hskc04d	106.46	11.20	0.67	4.09	622.60	13.01	1.81
hskc05	106.48	11.79	0.78	2.11	642.60	12.95	1.16
hskc06u	106.50	11.15	0.59	4.00	666.60	12.87	1.72
hskc06ub	106.50	11.15	0.59	4.00	666.60	12.87	1.72
hskc06b	106.51	11.05	0.58	3.99	702.60	12.75	1.70
hskc06b1	106.51	11.05	0.58	3.99	702.60	12.75	1.70
hskc06	106.54	10.45	0.66	4.35	762.60	12.55	2.11
hskc06b2	106.54	10.45	0.66	4.35	762.60	12.55	2.11
hskc06d	106.58	9.30	0.89	5.31	862.60	10.21	0.91
hskc07	106.60	9.00	0.87	3.62	902.60	9.47	0.47
hskc07b	106.60	8.72	0.97	4.16	902.60	9.47	0.75
hskc08u	106.61	8.90	0.57	2.64	902.60	9.47	0.57
hskc08	106.65	8.55	0.65	2.91	1062.60	9.47	0.92
hskc09	106.68	8.22	0.76	3.30	1142.60	9.94	1.73
hskc10	106.75	7.68	1.04	3.87	1276.60	9.39	1.71
cpr-us	106.79	7.77	0.54	2.78	1324.60	11.10	3.33
cpr-ds	119.82	7.77	0.51	2.69	1381.35	10.50	2.73
hskc11	119.82	7.56	0.75	3.28	1397.35	10.50	2.94
hskc12u	119.81	7.09	0.92	3.85	1522.35	8.67	1.58
hskc12	119.81	6.79	1.44	4.35	1578.35	8.59	1.80
hskc12m	119.80	6.70	0.95	4.06	1593.35	9.15	2.45
hskc12d	119.79	6.36	1.44	4.31	1656.35	8.13	1.77
hskc13u	119.79	6.28	0.89	3.74	1671.33	7.80	1.52
hskc13	119.77	5.97	1.66	4.19	1734.33	7.69	1.72
hskc13d	119.77	5.94	0.85	3.57	1749.33	7.36	1.43
hskc14	119.84	5.64	0.82	3.52	1928.33	7.05	1.41
hskc15	119.87	5.40	0.77	3.32	2108.33	6.59	1.19
hskc15d	119.88	5.48	0.59	2.62	2219.33	5.66	0.18

Table 31 Option 4 - 1 in 2 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	59.11	13.27	1.00	2.96	0.00	16.33	3.06
yltm-01	59.02	12.90	0.92	2.63	28.80	16.63	3.73
yltm-02	58.96	12.66	1.12	3.25	46.00	15.75	3.09
yltm-03	58.85	12.41	0.61	2.12	78.60	16.11	3.71
yltm-04	58.77	12.13	1.02	2.63	102.60	13.14	1.01
yltm-ds	63.17	12.13	0.00	0.00	102.60	13.00	0.87
hskc01u	63.17	10.52	1.01	2.76	102.60	13.00	2.48
hskc01ub	63.17	10.52	1.09	4.17	102.60	13.00	2.48
hskc01	63.09	10.31	1.01	4.05	170.60	12.48	2.17
hskc01ju	63.07	10.26	1.07	4.09	186.75	12.45	2.19
hskc01jl	63.07	10.26	0.00	0.00	186.75	12.42	2.16
hskc01jd	68.93	10.26	1.02	4.42	186.75	12.42	2.16
hskc02	68.85	10.01	0.94	4.37	255.60	12.15	2.14
hskc03	68.75	9.67	0.87	4.26	357.60	11.75	2.08
hskc03b	70.71	9.67	0.92	4.38	357.60	11.75	2.08
hskc03b1	70.64	9.38	1.83	4.93	429.00	11.47	2.09
hskc03b2	70.64	9.38	1.08	4.39	429.00	11.47	2.09
hskc04	70.60	9.22	1.04	4.33	476.60	11.28	2.06
hskc04d	70.53	9.01	0.96	4.23	544.60	11.01	2.00
hskc05	70.51	8.96	0.95	4.22	561.60	10.95	1.99
hskc06u	70.49	8.90	0.91	4.17	582.00	10.87	1.97
hskc06ub	70.49	8.90	0.91	4.17	582.00	10.87	1.97
hskc06b	70.46	8.83	0.85	4.10	612.60	10.75	1.92
hskc06b1	70.46	8.83	0.85	4.10	612.60	10.75	1.92
hskc06	70.41	8.73	0.76	3.95	663.60	10.55	1.82
hskc06b2	70.41	8.73	0.76	3.95	663.60	10.55	1.82
hskc06d	70.34	8.42	0.88	4.61	748.60	10.21	1.79
hskc07	70.29	8.29	0.87	3.51	788.60	9.47	1.18
hskc07b	70.29	8.15	0.95	3.80	788.60	9.47	1.32
hskc08u	70.26	8.33	0.55	2.32	788.60	9.47	1.15
hskc08	70.25	8.00	0.62	2.55	948.60	9.47	1.47
hskc09	70.33	7.71	0.72	2.90	1028.60	9.94	2.23
hskc10	70.47	7.16	0.94	3.56	1162.60	9.39	2.23
cpr-us	70.54	7.27	0.49	2.26	1210.60	11.10	3.83
cpr-ds	78.30	7.27	0.43	2.14	1267.35	10.50	3.23
hskc11	78.31	7.07	0.70	2.82	1283.35	10.50	3.43
hskc12u	78.34	6.61	0.89	3.41	1408.35	8.67	2.07
hskc12	78.35	6.29	1.20	3.95	1464.35	8.59	2.30
hskc12m	78.35	6.20	0.91	3.57	1479.35	9.15	2.95
hskc12d	78.36	5.85	1.26	3.93	1542.35	8.13	2.28
hskc13u	78.36	5.77	0.87	3.34	1557.33	7.80	2.03
hskc13	78.37	5.43	1.42	3.89	1620.33	7.69	2.26
hskc13d	78.37	5.39	0.85	3.20	1635.33	7.36	1.97
hskc14	78.35	5.09	0.82	3.16	1814.33	7.05	1.96
hskc15	78.29	4.86	0.75	2.94	1994.33	6.59	1.73
hskc15d	78.27	4.90	0.57	2.34	2105.33	5.66	0.76

Table 32 Option 4 - 1 in 10 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	88.84	13.68	1.00	3.28	0.00	16.33	2.65
yltm-01	88.76	13.34	0.92	2.89	28.80	16.63	3.30
yltm-02	88.71	13.12	1.12	3.50	46.00	15.75	2.63
yltm-03	88.66	12.98	0.61	2.29	78.60	16.11	3.14
yltm-04	88.68	12.67	1.02	2.90	102.60	13.14	0.47
yltm-ds	95.45	12.67	0.00	0.00	102.60	13.00	0.33
hskc01u	95.45	11.36	1.01	2.76	102.60	13.00	1.64
hskc01ub	95.45	11.36	1.09	4.63	102.60	13.00	1.64
hskc01	95.38	11.17	1.02	4.51	170.60	12.48	1.31
hskc01ju	95.36	11.11	1.09	4.56	186.75	12.45	1.34
hskc01jl	95.36	11.11	0.00	0.00	186.75	12.42	1.31
hskc01jd	104.01	11.11	1.02	4.92	186.75	12.42	1.31
hskc02	103.94	10.86	0.95	4.88	255.60	12.15	1.29
hskc03	103.84	10.52	0.87	4.79	357.60	11.75	1.23
hskc03b	107.18	10.52	0.91	4.94	357.60	11.75	1.23
hskc03b1	107.12	10.24	1.89	5.37	429.00	11.47	1.23
hskc03b2	107.12	10.24	1.10	4.94	429.00	11.47	1.23
hskc04	107.08	10.08	1.06	4.90	476.60	11.28	1.20
hskc04d	107.02	9.86	0.97	4.83	544.60	11.01	1.15
hskc05	107.00	9.80	0.96	4.82	561.60	10.95	1.15
hskc06u	106.99	9.74	0.92	4.79	582.00	10.87	1.13
hskc06ub	106.99	9.74	0.92	4.79	582.00	10.87	1.13
hskc06b	106.96	9.66	0.86	4.74	612.60	10.75	1.10
hskc06b1	106.96	9.66	0.86	4.74	612.60	10.75	1.10
hskc06	106.92	9.52	0.79	4.66	663.60	10.55	1.03
hskc06b2	106.92	9.52	0.79	4.66	663.60	10.55	1.03
hskc06d	106.86	9.18	0.93	5.50	748.60	10.21	1.03
hskc07	106.82	9.00	0.87	3.62	788.60	9.47	0.47
hskc07b	106.82	8.73	0.97	4.16	788.60	9.47	0.74
hskc08u	106.79	8.90	0.57	2.64	788.60	9.47	0.57
hskc08	106.66	8.55	0.65	2.90	948.60	9.47	0.92
hskc09	106.75	8.22	0.76	3.30	1028.60	9.94	1.72
hskc10	106.91	7.69	0.94	3.87	1162.60	9.39	1.70
cpr-us	106.98	7.78	0.54	2.78	1210.60	11.10	3.32
cpr-ds	120.85	7.79	0.51	2.71	1267.35	10.50	2.71
hskc11	120.86	7.57	0.75	3.29	1283.35	10.50	2.93
hskc12u	120.89	7.11	0.92	3.87	1408.35	8.67	1.57
hskc12	120.90	6.80	1.22	4.36	1464.35	8.59	1.79
hskc12m	120.91	6.72	0.95	4.08	1479.35	9.15	2.43
hskc12d	120.91	6.37	1.28	4.32	1542.35	8.13	1.76
hskc13u	120.92	6.30	0.89	3.75	1557.33	7.80	1.51
hskc13	120.92	5.98	1.47	4.20	1620.33	7.69	1.71
hskc13d	120.92	5.95	0.85	3.58	1635.33	7.36	1.41
hskc14	120.90	5.65	0.82	3.53	1814.33	7.05	1.40
hskc15	120.87	5.41	0.77	3.33	1994.33	6.59	1.18
hskc15d	120.84	5.49	0.59	2.63	2105.33	5.66	0.17

Table 33 Option 4 - 1 in 50 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chainage (m)	Bank (m PD)	Freeboard (m)
yltm-us	114.42	13.99	1.00	3.46	0.00	16.33	2.35
yltm-01	114.33	13.72	0.92	2.96	28.80	16.63	2.92
yltm-02	114.33	13.48	1.12	3.64	46.00	15.75	2.27
yltm-03	114.36	13.42	0.60	2.37	78.60	16.11	2.70
yltm-04	114.38	13.09	1.01	3.09	102.60	13.14	0.06
yltm-ds	123.10	13.09	0.00	0.00	102.60	13.00	-0.09
hskc01u	123.10	12.03	1.01	2.76	102.60	13.00	0.97
hskc01ub	123.10	12.03	1.09	4.93	102.60	13.00	0.97
hskc01	123.03	11.85	1.02	4.81	170.60	12.48	0.63
hskc01ju	123.01	11.78	1.09	4.86	186.75	12.45	0.67
hskc01jl	123.01	11.78	0.00	0.00	186.75	12.42	0.64
hskc01jd	134.05	11.78	1.03	5.26	186.75	12.42	0.64
hskc02	133.98	11.54	0.95	5.22	255.60	12.15	0.61
hskc03	133.90	11.19	0.88	5.15	357.60	11.75	0.56
hskc03b	138.38	11.19	0.93	5.32	357.60	11.75	0.56
hskc03b1	138.32	10.90	1.87	5.71	429.00	11.47	0.57
hskc03b2	138.32	10.90	1.09	5.33	429.00	11.47	0.57
hskc04	138.29	10.73	1.06	5.30	476.60	11.28	0.55
hskc04d	138.24	10.49	0.97	5.26	544.60	11.01	0.52
hskc05	138.23	10.43	0.96	5.26	561.60	10.95	0.52
hskc06u	138.22	10.36	0.92	5.24	582.00	10.87	0.51
hskc06ub	138.22	10.36	0.92	5.24	582.00	10.87	0.51
hskc06b	138.20	10.26	0.86	5.22	612.60	10.75	0.49
hskc06b1	138.20	10.26	0.86	5.22	612.60	10.75	0.49
hskc06	138.17	10.10	0.82	5.17	663.60	10.55	0.46
hskc06b2	138.17	10.10	0.82	5.17	663.60	10.55	0.46
hskc06d	138.13	9.71	0.98	6.19	748.60	10.21	0.50
hskc07	138.10	9.52	0.87	3.68	788.60	9.47	-0.05
hskc07b	138.10	9.13	0.97	4.39	788.60	9.47	0.34
hskc08u	138.07	9.31	0.59	2.85	788.60	9.47	0.17
hskc08	137.91	8.92	0.68	3.17	948.60	9.47	0.55
hskc09	137.93	8.59	0.78	3.56	1028.60	9.94	1.35
hskc10	138.08	8.07	0.95	4.08	1162.60	9.39	1.33
cpr-us	138.14	8.13	0.58	3.18	1210.60	11.10	2.97
cpr-ds	157.09	8.13	0.58	3.13	1267.35	10.50	2.37
hskc11	157.10	7.95	0.77	3.58	1283.35	10.50	2.55
hskc12u	157.14	7.48	0.94	4.14	1408.35	8.67	1.20
hskc12	157.15	7.18	1.21	4.61	1464.35	8.59	1.42
hskc12m	157.16	7.09	0.97	4.41	1479.35	9.15	2.06
hskc12d	157.17	6.75	1.27	4.58	1542.35	8.13	1.38
hskc13u	157.17	6.67	0.90	4.01	1557.33	7.80	1.13
hskc13	157.18	6.36	1.46	4.45	1620.33	7.69	1.33
hskc13d	157.18	6.33	0.85	3.83	1635.33	7.36	1.03
hskc14	157.18	6.04	0.83	3.77	1814.33	7.05	1.01
hskc15	157.17	5.79	0.78	3.59	1994.33	6.59	0.80
hskc15d	157.16	5.88	0.60	2.86	2105.33	5.66	-0.22

Table 34 Option 4 - 1 in 200 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	135.82	14.25	1.00	3.55	0.00	16.33	2.08
yltm-01	135.77	14.06	0.92	2.96	28.80	16.63	2.57
yltm-02	135.78	13.79	1.12	3.66	46.00	15.75	1.96
yltm-03	135.81	13.80	0.60	2.39	78.60	16.11	2.31
yltm-04	135.84	13.49	1.00	3.12	102.60	13.14	-0.35
yltm-ds	146.27	13.49	0.00	0.00	102.60	13.00	-0.49
hskc01u	146.27	12.55	1.01	2.76	102.60	13.00	0.45
hskc01ub	146.27	12.55	1.08	5.16	102.60	13.00	0.45
hskc01	146.20	12.36	1.02	5.06	170.60	12.48	0.12
hskc01ju	146.18	12.29	1.09	5.11	186.75	12.45	0.16
hskc01jl	146.18	12.29	0.00	0.00	186.75	12.42	0.13
hskc01jd	159.21	12.29	1.02	5.52	186.75	12.42	0.13
hskc02	159.15	12.04	0.94	5.50	255.60	12.15	0.11
hskc03	159.06	11.67	0.87	5.46	357.60	11.75	0.08
hskc03b	164.49	11.67	0.93	5.65	357.60	11.75	0.08
hskc03b1	164.44	11.37	1.86	6.04	429.00	11.47	0.11
hskc03b2	164.44	11.37	1.09	5.68	429.00	11.47	0.11
hskc04	164.41	11.18	1.05	5.67	476.60	11.28	0.10
hskc04d	164.37	10.91	0.97	5.67	544.60	11.01	0.10
hskc05	164.36	10.84	0.96	5.68	561.60	10.95	0.11
hskc06u	164.34	10.76	0.92	5.68	582.00	10.87	0.11
hskc06ub	164.34	10.76	0.92	5.68	582.00	10.87	0.11
hskc06b	164.33	10.64	0.86	5.68	612.60	10.75	0.11
hskc06b1	164.33	10.64	0.86	5.68	612.60	10.75	0.11
hskc06	164.30	10.43	0.86	5.68	663.60	10.55	0.12
hskc06b2	164.30	10.43	0.86	5.68	663.60	10.55	0.12
hskc06d	164.26	9.97	1.06	6.91	748.60	10.21	0.24
hskc07	164.24	9.76	0.88	3.96	788.60	9.47	-0.29
hskc07b	164.24	9.43	0.98	4.56	788.60	9.47	0.05
hskc08u	164.22	9.60	0.61	3.02	788.60	9.47	-0.13
hskc08	164.05	9.19	0.69	3.34	948.60	9.47	0.28
hskc09	164.01	8.86	0.79	3.73	1028.60	9.94	1.08
hskc10	164.18	8.35	0.96	4.24	1162.60	9.39	1.04
cpr-us	164.24	8.37	0.62	3.50	1210.60	11.10	2.74
cpr-ds	187.34	8.37	0.62	3.45	1267.35	10.50	2.13
hskc11	187.35	8.22	0.79	3.79	1283.35	10.50	2.28
hskc12u	187.39	7.74	0.95	4.35	1408.35	8.67	0.93
hskc12	187.40	7.44	1.20	4.79	1464.35	8.59	1.15
hskc12m	187.40	7.36	0.99	4.63	1479.35	9.15	1.79
hskc12d	187.41	7.03	1.26	4.73	1542.35	8.13	1.10
hskc13u	187.41	6.95	0.91	4.19	1557.33	7.80	0.85
hskc13	187,42	6.66	1.46	4.57	1620.33	7.69	1.03
hskc13d	187.42	6.62	0.86	4.01	1635.33	7.36	0.74
hskc14	187.41	6.33	0.84	3.96	1814.33	7.05	0.72
hskc15	187.40	6.08	0.79	3.77	1994.33	6.59	0.51
hskc15d	187.39	6.17	0.60	3.04	2105.33	5.66	-0.51

Table 35 Option5 - 1 in 10 year rain storm event.

Label	Flow (m^3s^{-1})	Stage (m PD)	Froude Number	Velocity (ms^{-1})	Chain-age (m)	Bank (m PD)	Free-board (m)
yltm-us	66.33	13.69	1.00	2.70	0.00	16.33	2.64
yltm-01	66.33	13.67	0.92	2.35	28.80	16.63	2.97
yltm-02	66.34	13.56	1.13	2.90	46.00	15.75	2.19
yltm-03	66.34	13.61	0.62	1.80	78.60	16.11	2.51
yltm-04	66.34	13.55	1.05	2.22	102.60	13.14	-0.41
yltm-ds	72.47	13.55	0.00	0.00	102.60	13.00	-0.55
hskc01u	72.47	13.35	1.01	2.76	102.60	13.00	-0.35
hskc01ub	72.47	13.35	0.58	2.55	102.60	13.00	-0.35
hskc01	72.42	13.24	0.52	2.48	182.60	12.48	-0.76
hskc01ju	72.41	13.21	0.53	2.48	201.60	12.45	-0.76
hskc01jl	72.41	12.51	0.00	0.00	201.60	12.42	-0.09
hskc01jd	80.37	12.51	0.59	3.15	201.60	12.42	-0.09
hskc02	80.37	12.28	0.52	3.12	282.60	12.15	-0.13
hskc03	80.39	11.70	0.53	3.26	402.60	11.75	0.05
hskc03b	85.52	11.70	0.58	3.46	402.60	11.75	0.05
hskc03b1	85.53	11.18	1.03	3.89	486.60	11.47	0.29
hskc03b2	85.53	11.18	0.66	3.64	486.60	11.47	0.29
hskc04	85.54	10.93	0.70	3.70	542.60	11.28	0.35
hskc04d	85.54	10.64	0.67	3.71	622.60	11.01	0.37
hskc05	85.55	10.58	0.64	3.70	642.60	10.95	0.37
hskc06u	85.55	10.52	0.60	3.69	666.60	10.87	0.35
hskc06ub	85.55	10.52	0.60	3.69	666.60	10.87	0.35
hskc06b	85.55	10.44	0.57	3.66	702.60	10.75	0.31
hskc06b1	85.55	10.44	0.57	3.66	702.60	10.75	0.31
hskc06	85.55	9.92	0.64	3.96	762.60	10.55	0.63
hskc06b2	85.55	9.92	0.64	3.96	762.60	10.55	0.63
hskc06d	85.56	8.84	0.87	4.87	862.60	10.21	1.37
hskc07	85.56	8.61	0.86	3.55	902.60	9.47	0.86
hskc07b	85.56	8.42	0.96	3.96	902.60	9.47	1.05
hskc08u	85.55	8.60	0.55	2.43	902.60	9.47	0.87
hskc08	85.51	8.29	0.63	2.66	1062.60	9.47	1.18
hskc09	85.50	7.99	0.74	3.02	1142.60	9.94	1.95
hskc10	85.50	7.48	0.95	3.59	1276.60	9.39	1.91
cpr-us	85.52	7.64	0.49	2.36	1324.60	11.10	3.46
cpr-ds	107.68	7.64	0.49	2.54	1381.35	10.50	2.86
hskc11	107.68	7.42	0.74	3.17	1397.35	10.50	3.08
hskc12u	107.67	6.96	0.91	3.73	1522.35	8.67	1.71
hskc12	107.65	6.65	1.24	4.27	1578.35	8.59	1.94
hskc12m	107.65	6.56	0.94	3.94	1593.35	9.15	2.59
hskc12d	107.62	6.21	1.29	4.25	1656.35	8.13	1.92
hskc13u	107.61	6.13	0.88	3.63	1671.33	7.80	1.67
hskc13	107.56	5.81	1.50	4.15	1734.33	7.69	1.88
hskc13d	107.54	5.77	0.84	3.48	1749.33	7.36	1.59
hskc14	107.31	5.48	0.82	3.42	1928.33	7.05	1.57
hskc15	107.06	5.24	0.76	3.23	2108.33	6.59	1.35
hskc15d	106.88	5.31	0.58	2.54	2219.33	5.66	0.35

Table 36 Flood marks for the 1993 and 1994 events.

Point	Level (mPD)	Remarks	
1993			
A	20.11	Red mark on pig house	
B	17.96	Top of dam	
C	17.21	Red mark on rubble masonary	
D	14.04	Red mark on a tree *	
1994			
A	19.61	Red mark at house wall	
B	19.06	Red mark at dam wall	
D	14.05	Red mark at wall	
E	14.04	Red mark at a tree	* The Concultants believe that this mark is infact mark D from the 1993 event
F	13.47	Red mark at lamp post	
G	13.06	Red mark at house wall	
I	12.28	Red mark at lamp post	
J	11.89	Red mark at lamp post	
L	11.52	Red mark at lamp post	

Table 37 Comparison of catchment areas from the original design and the current study.

Sub-catchment	Original design study Area (km^2)	Current study Area (km^2)	Percentage difference (%)
Catchments 1, 2 and 3	4.61	4.24	-8.0
Catchments 1,2,3 and 4	5.06	4.62	-8.7

Figures

Location map

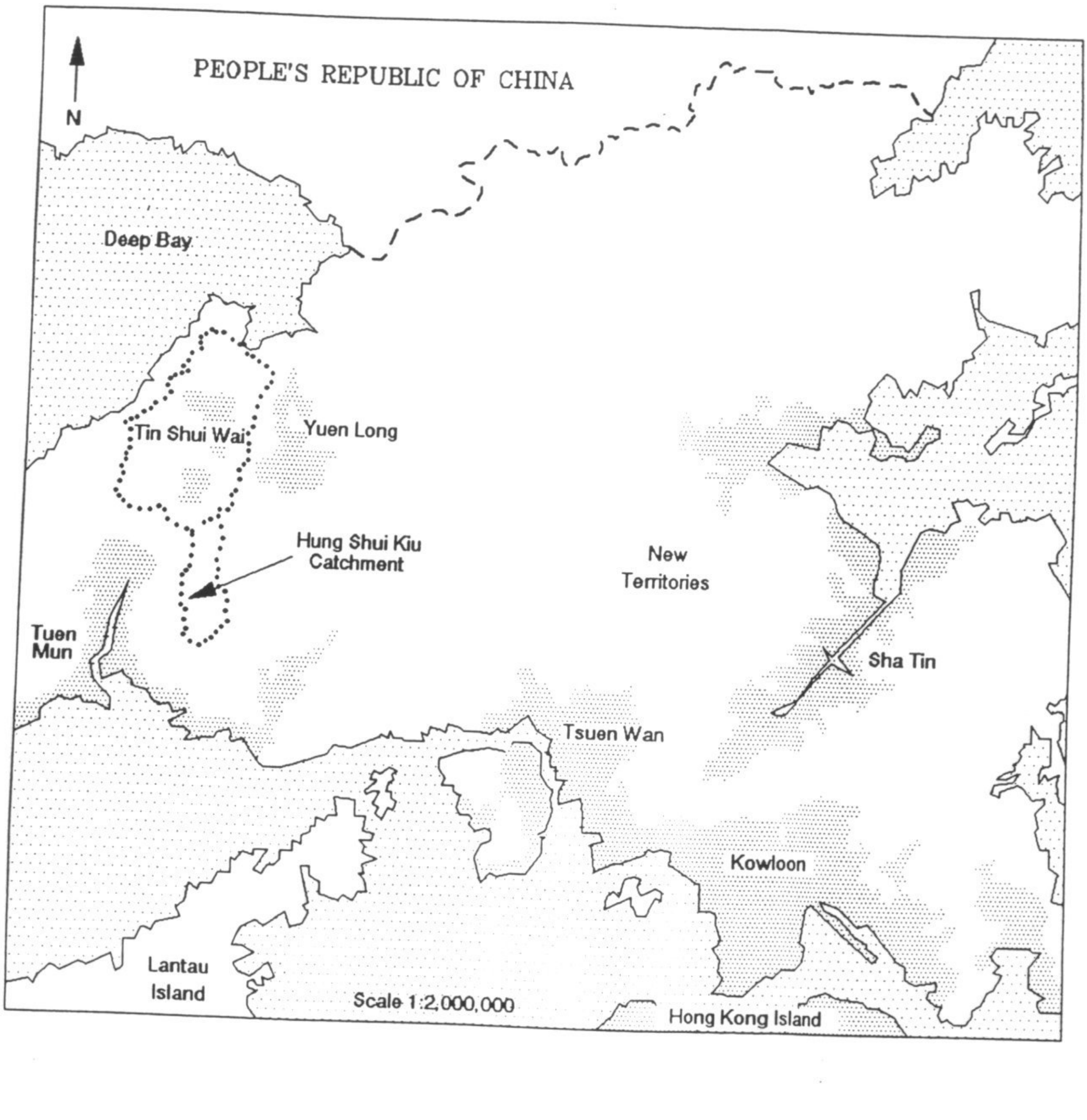


Figure 1

Hung Shui Kiu and Tin Shui Wai catchments

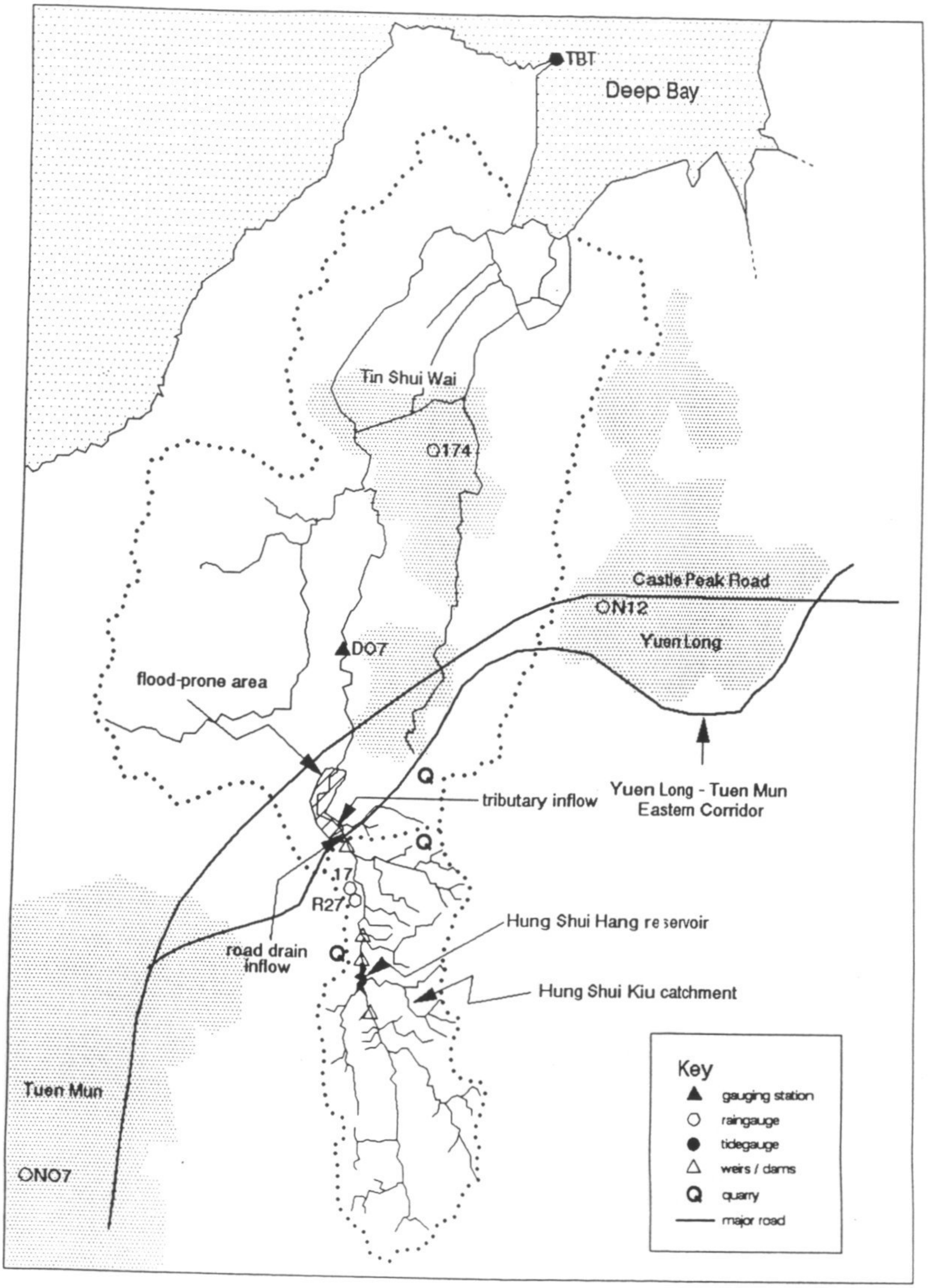


Figure 2

Location map of autographic raingauges

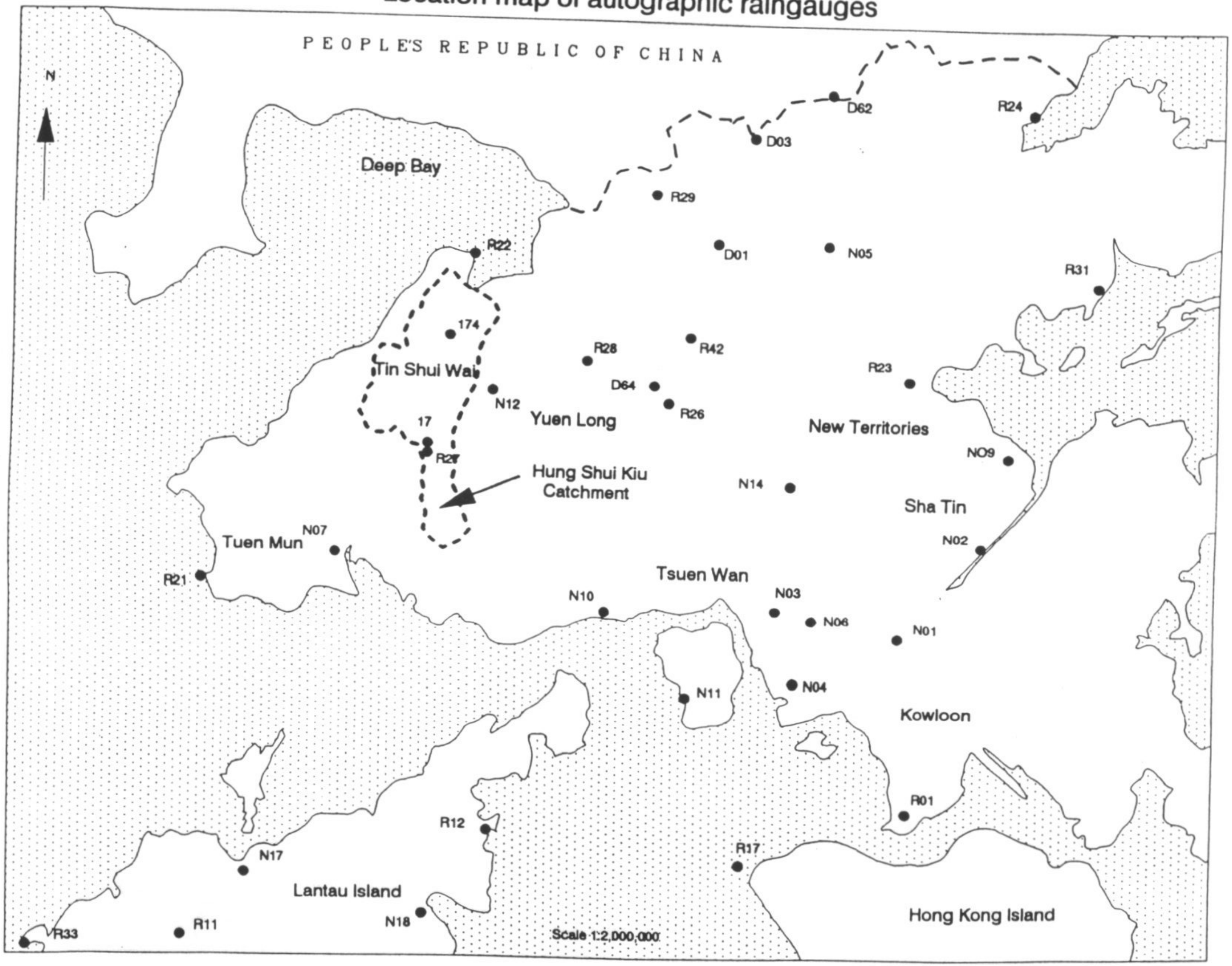
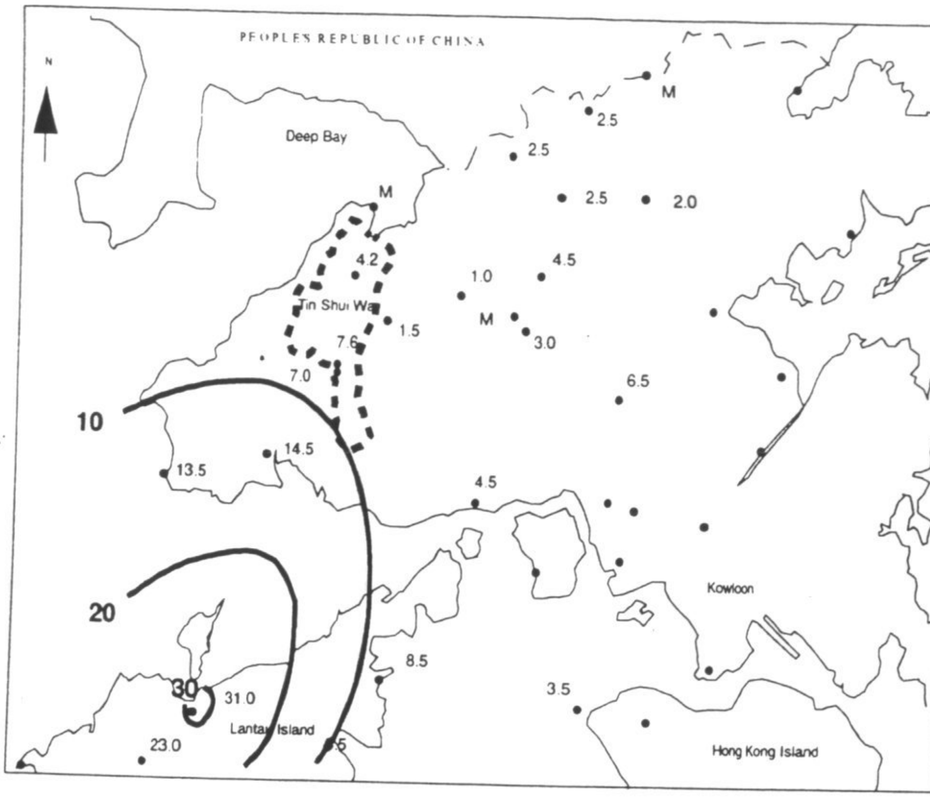


Figure 3

Isohyetal map for 5th November 1993 (mm)

00:00 - 01:00



01:00 - 02:00

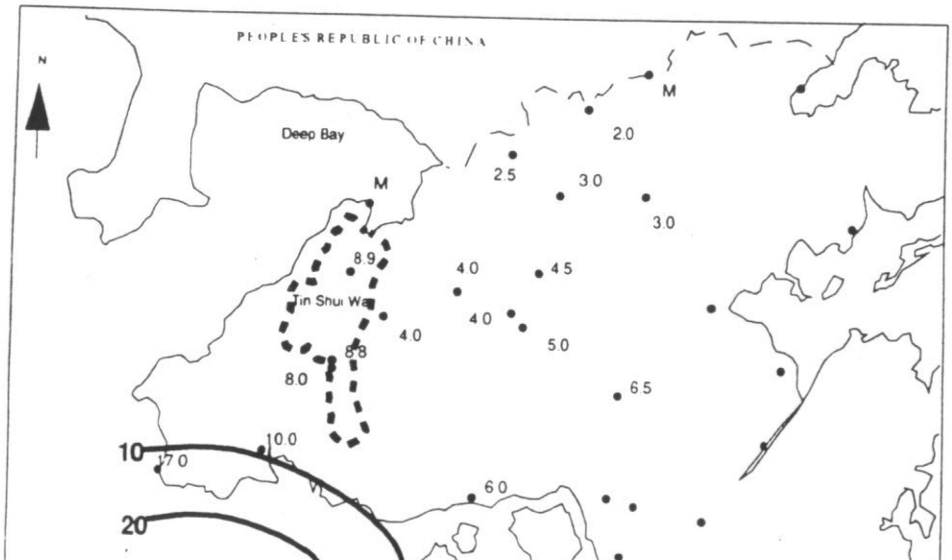


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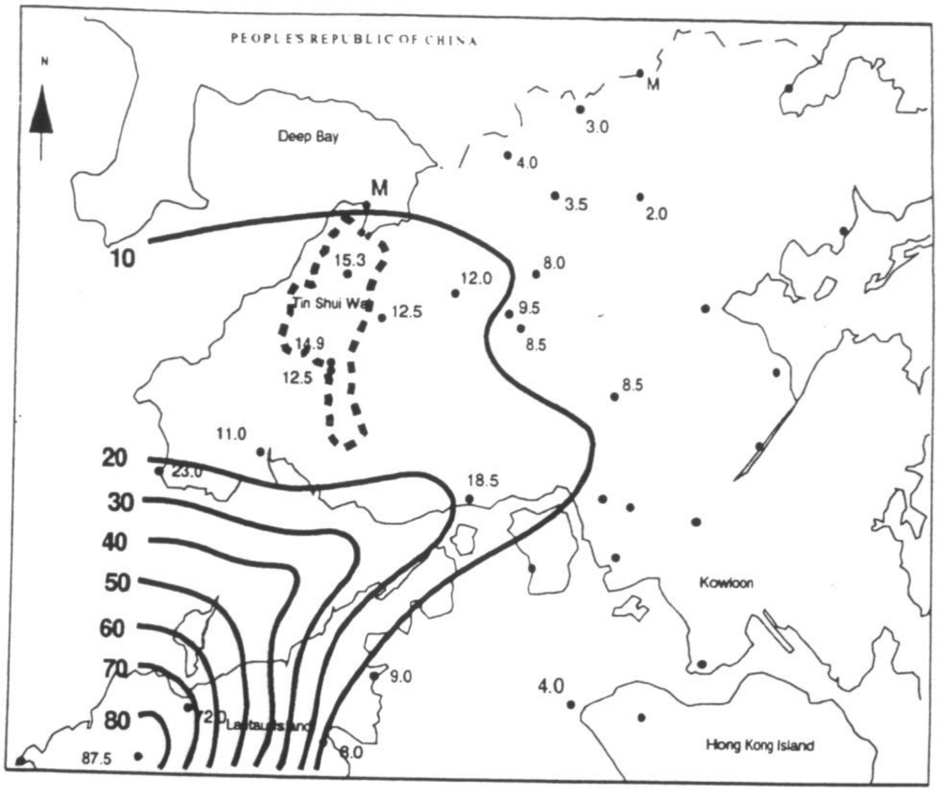
Figure 4

Isohyetal map for 5th November 1993 (mm)

02:00 - 03:00



03:00 - 04:00

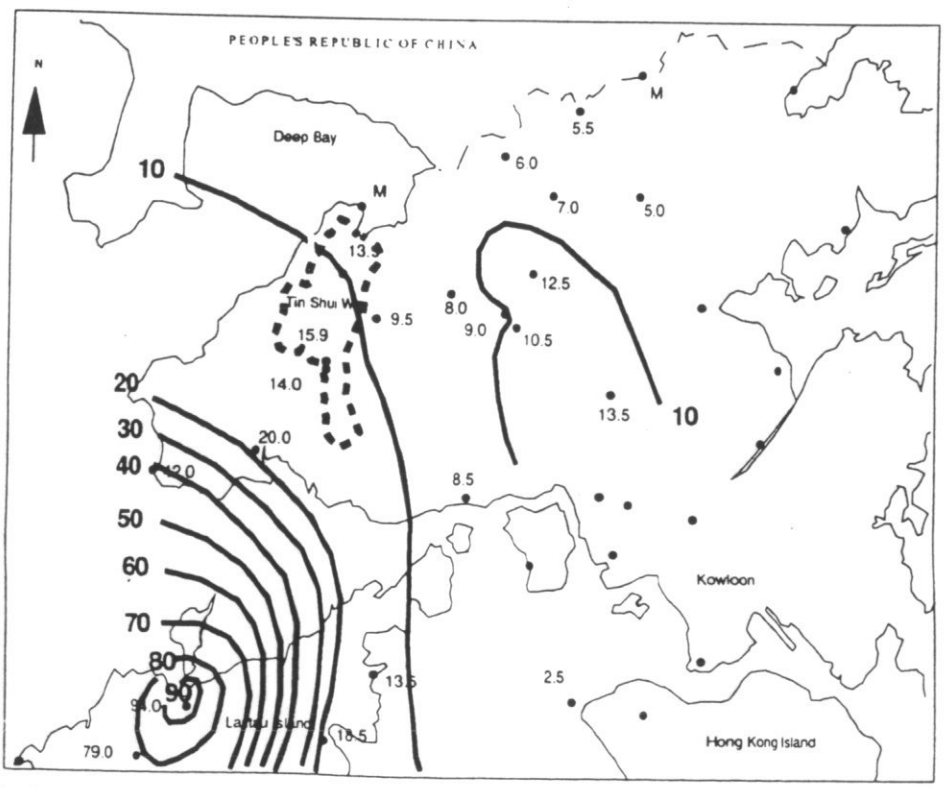


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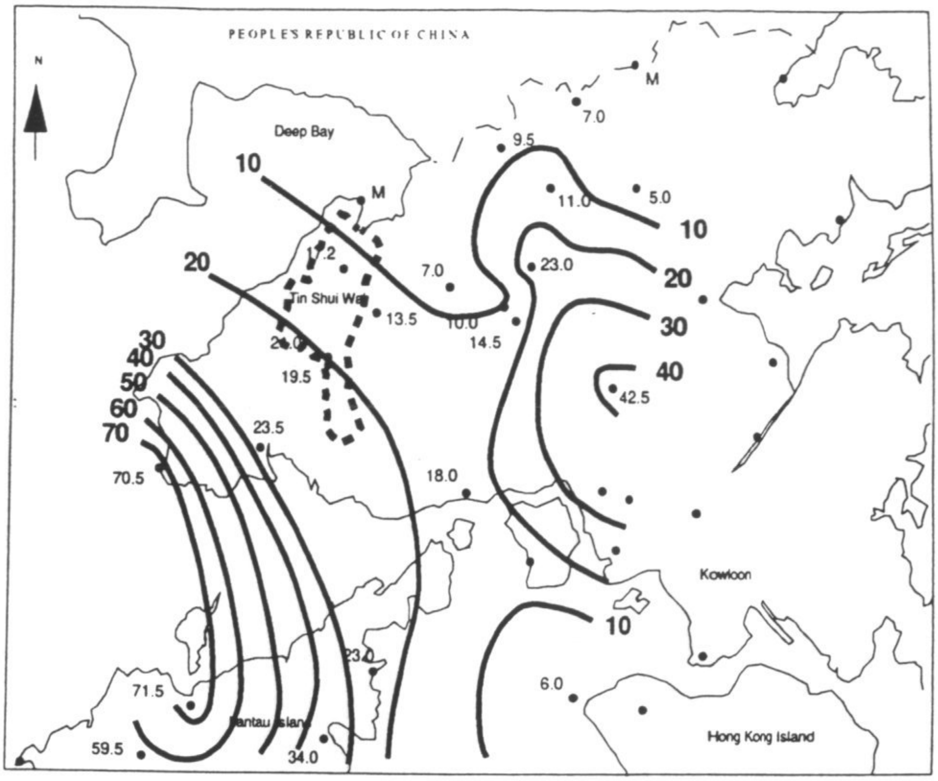
Figure 4

Isohyetal map for 5th November 1993 (mm)

04:00 - 05:00



05:00 - 06:00

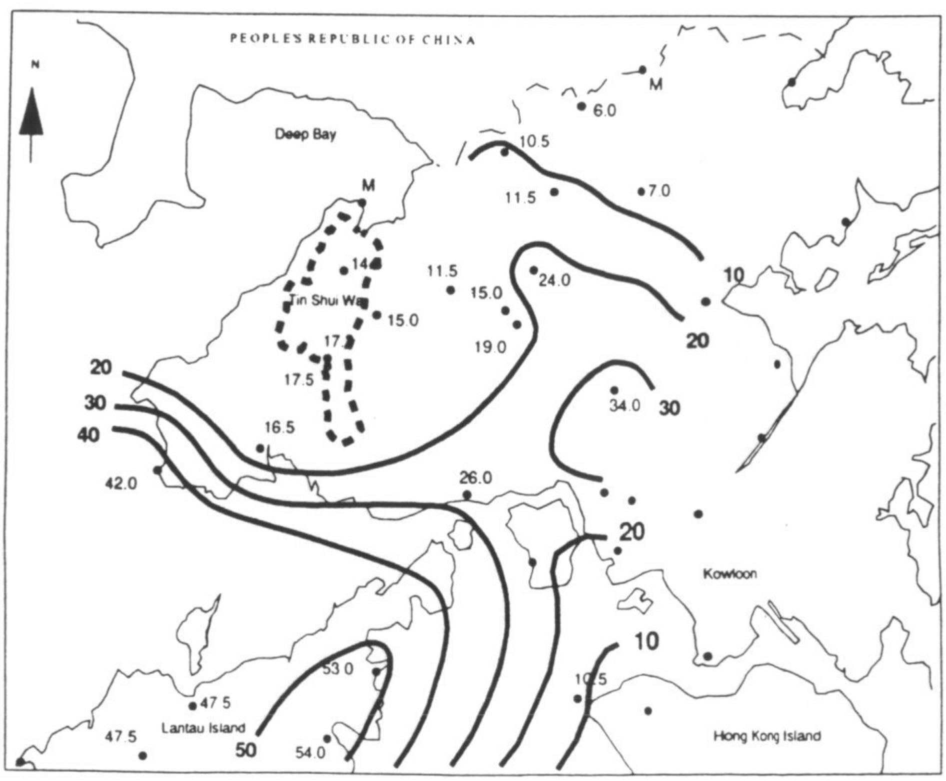


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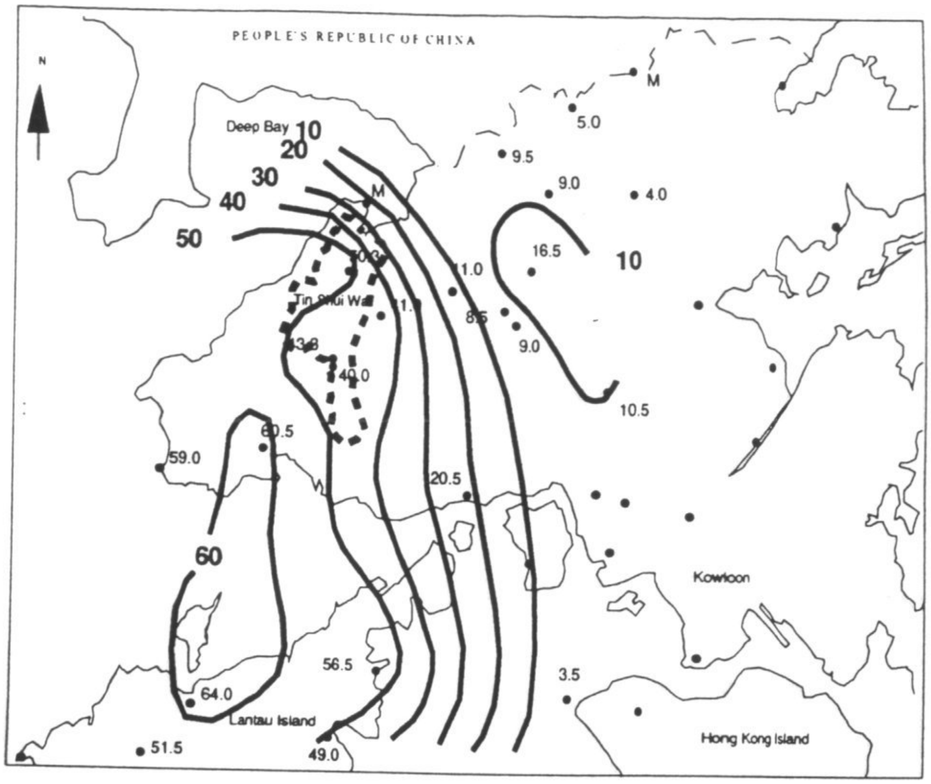
Figure 4

Isohyetal map for 5th November 1993 (mm)

06:00 - 07:00



07:00 - 08:00

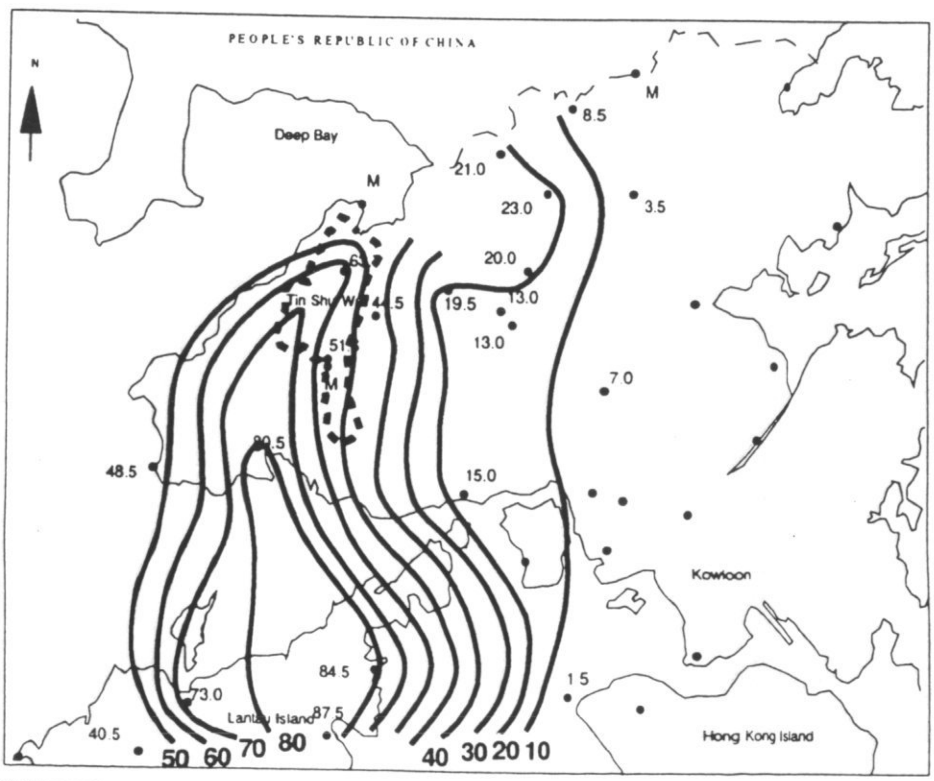


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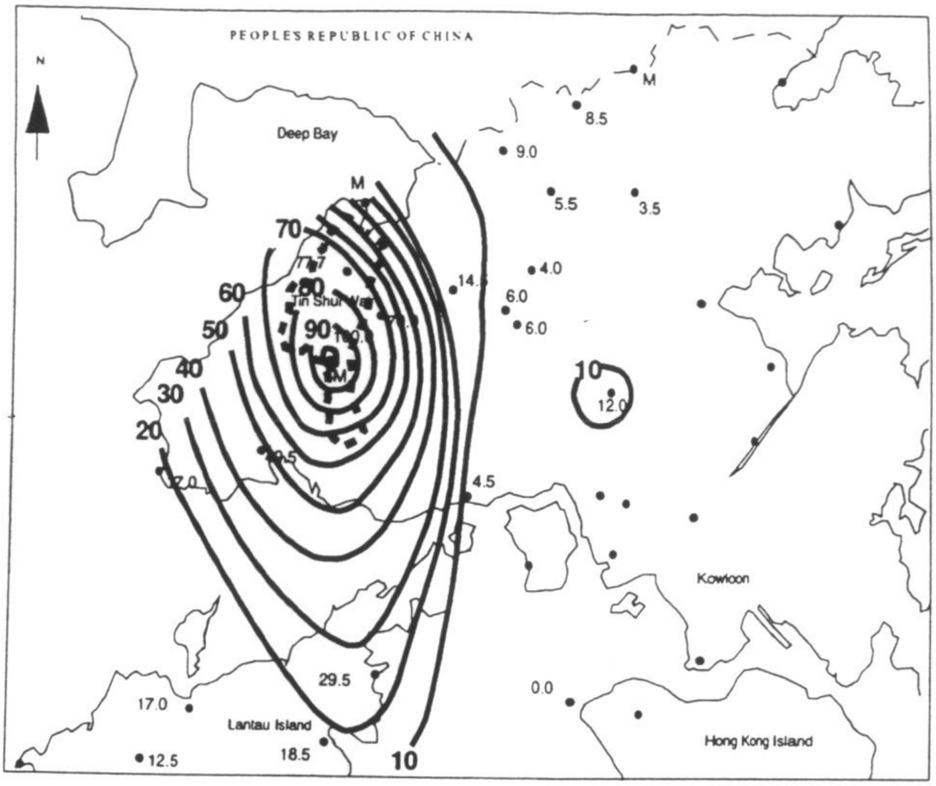
Figure 4

Isohyetal map for 5th November 1993 (mm)

08:00 - 09:00



09:00 - 10:00

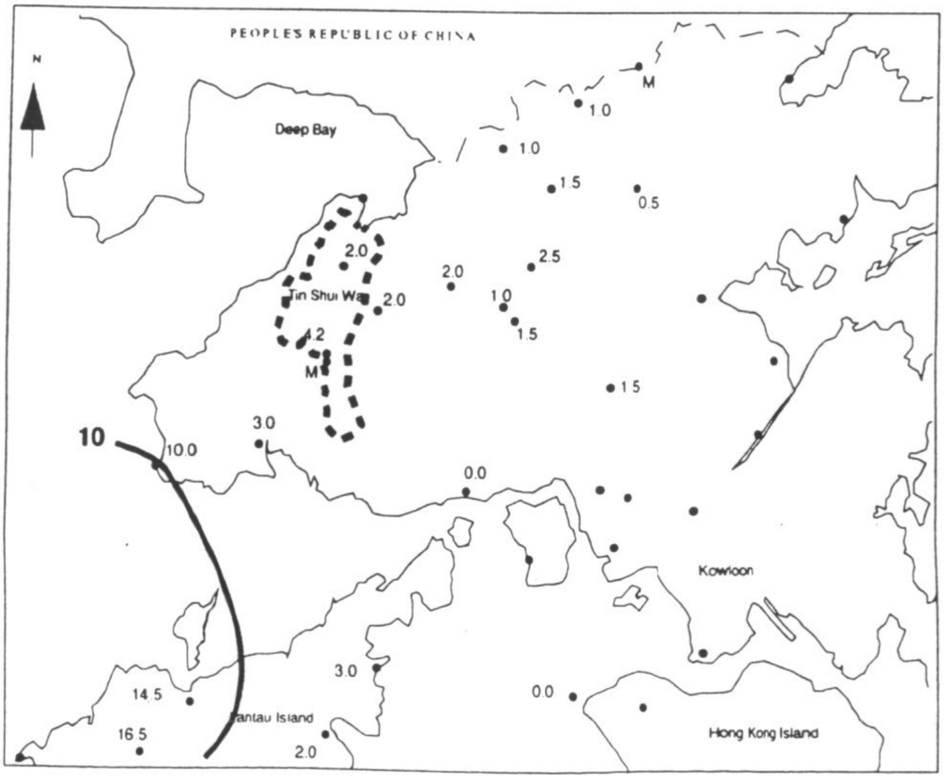


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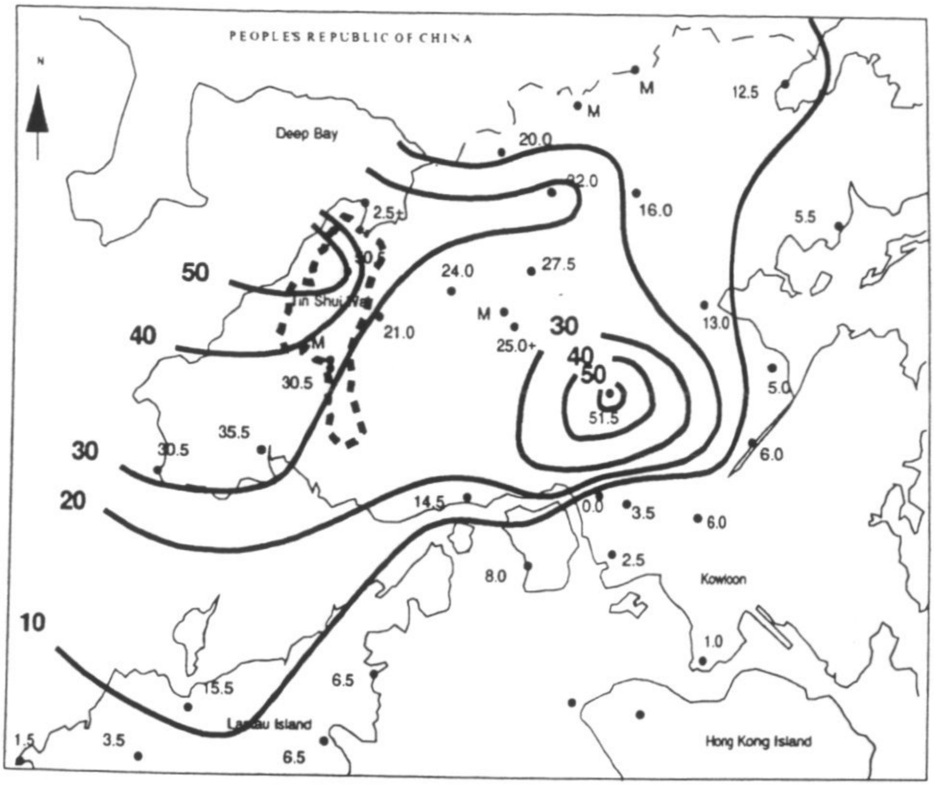
Figure 4

Isohyetal map for 22nd July 1994 (mm)

00:00 - 01:00



01:00 - 02:00

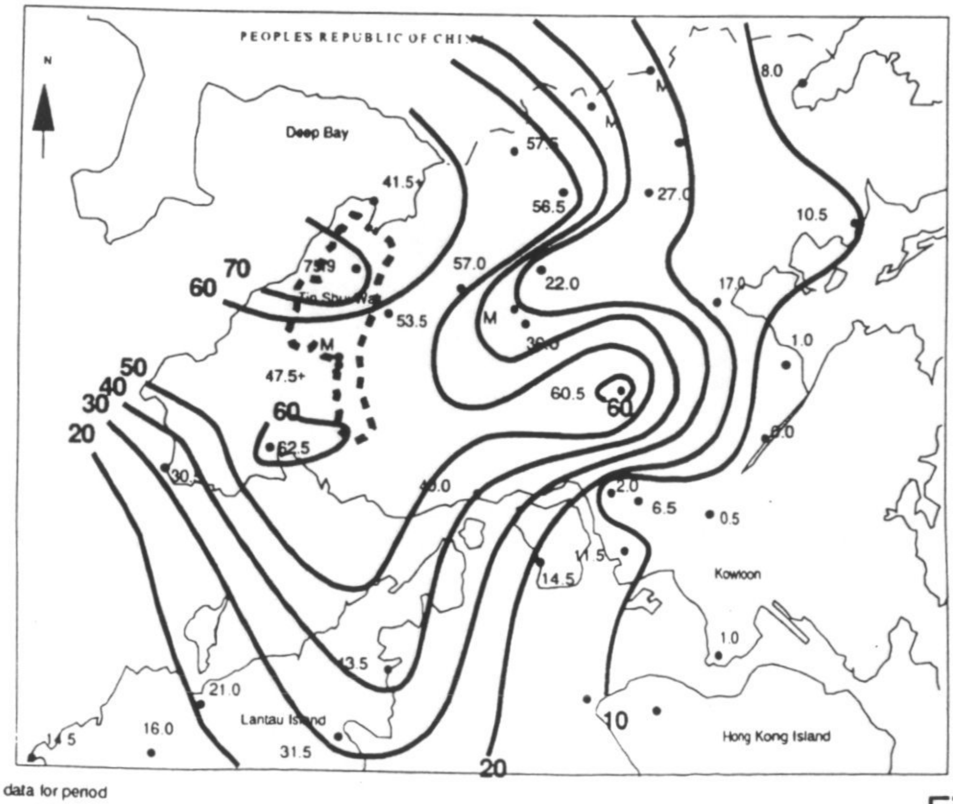


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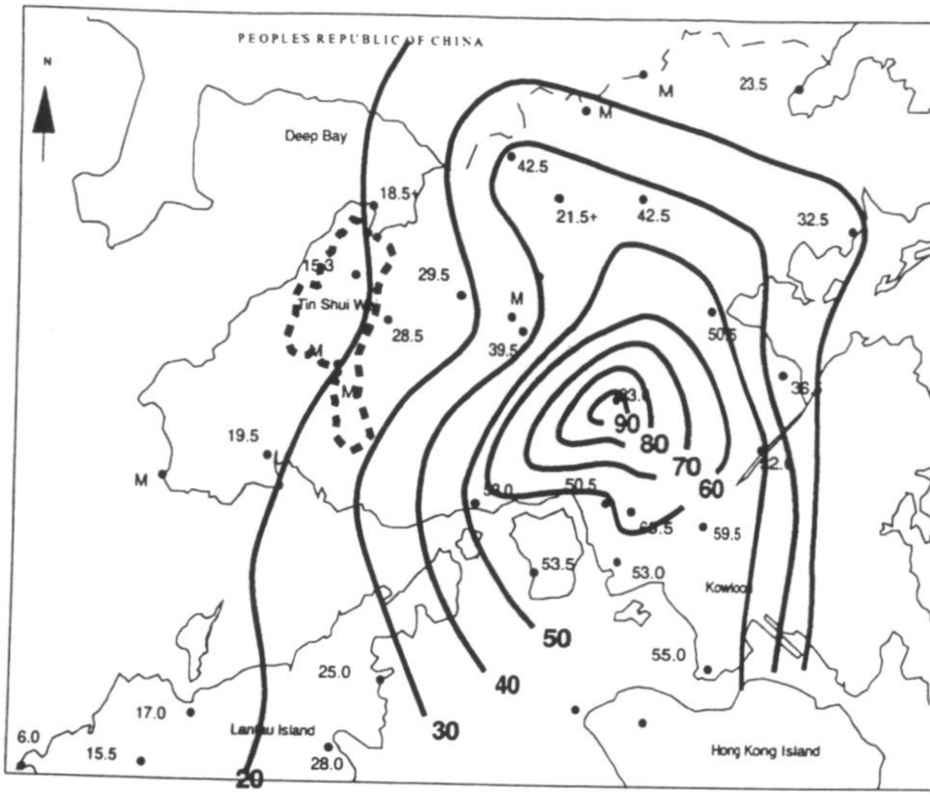
Figure 5

Isohyetal map for 22nd July 1994 (mm)

02:00 - 03:00



03:00 - 04:00

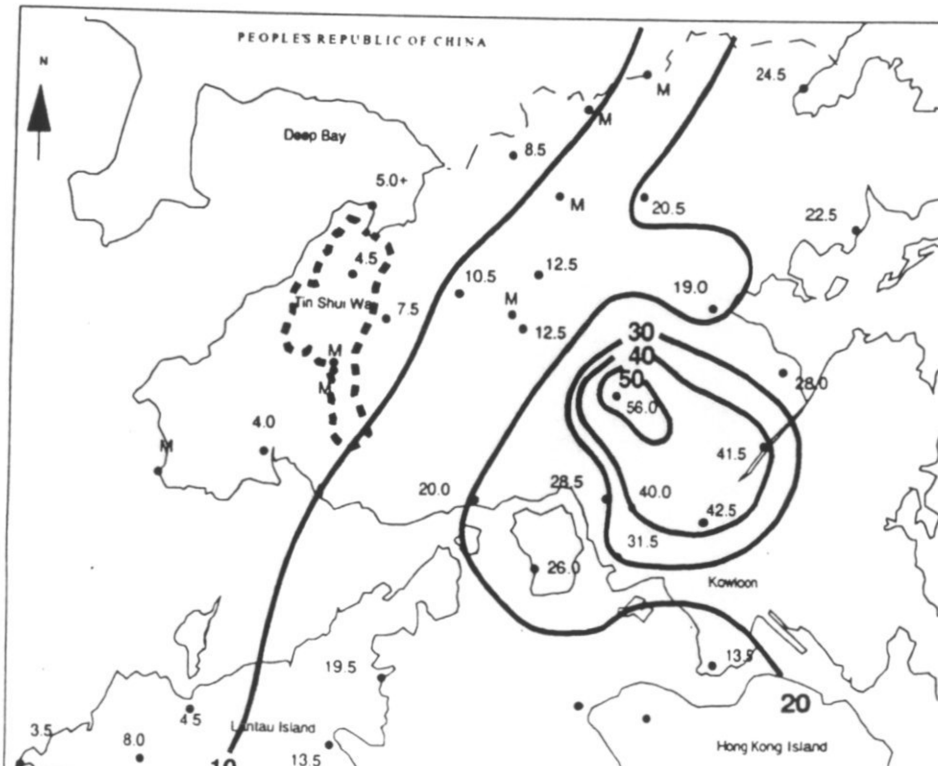


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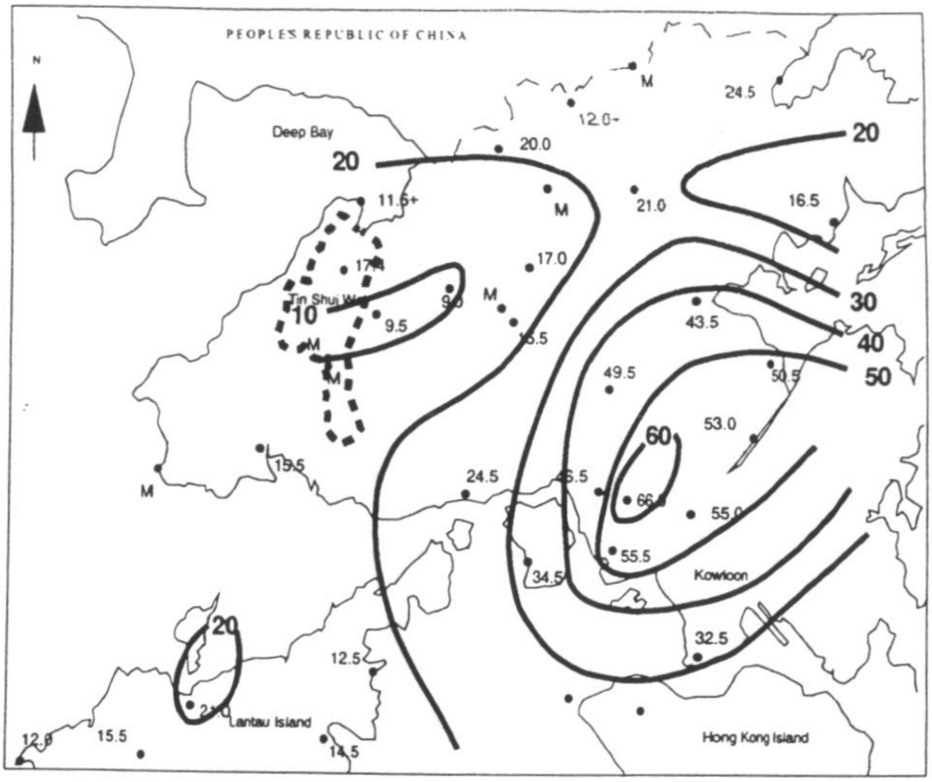
Figure 5

Isohyetal map for 22nd July 1994 (mm)

04:00 - 05:00



05:00 - 06:00

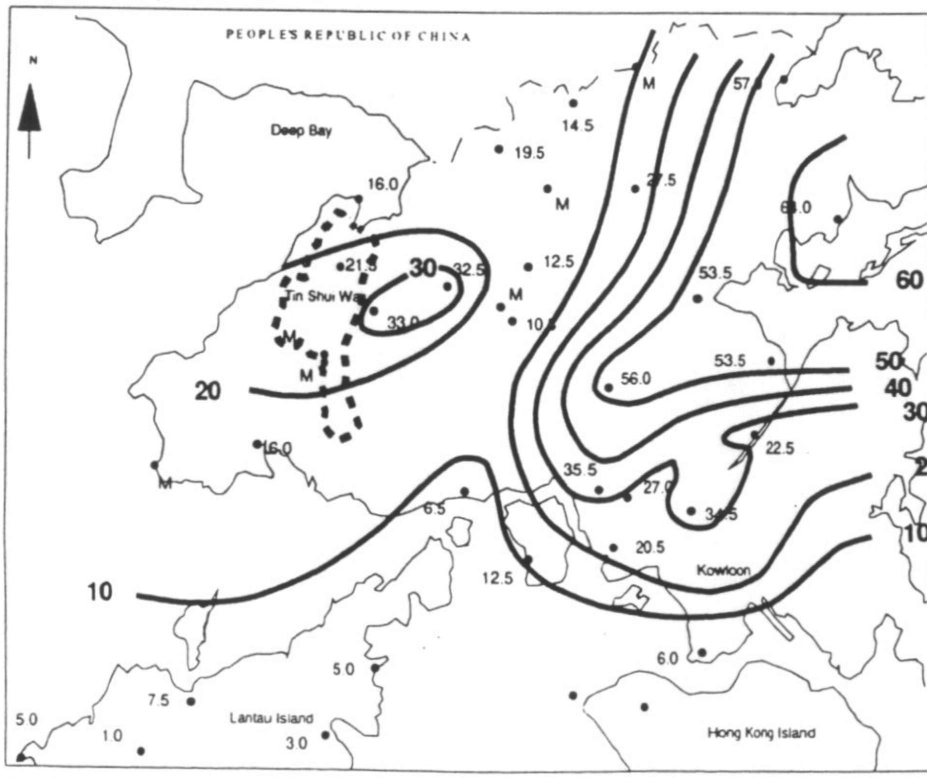


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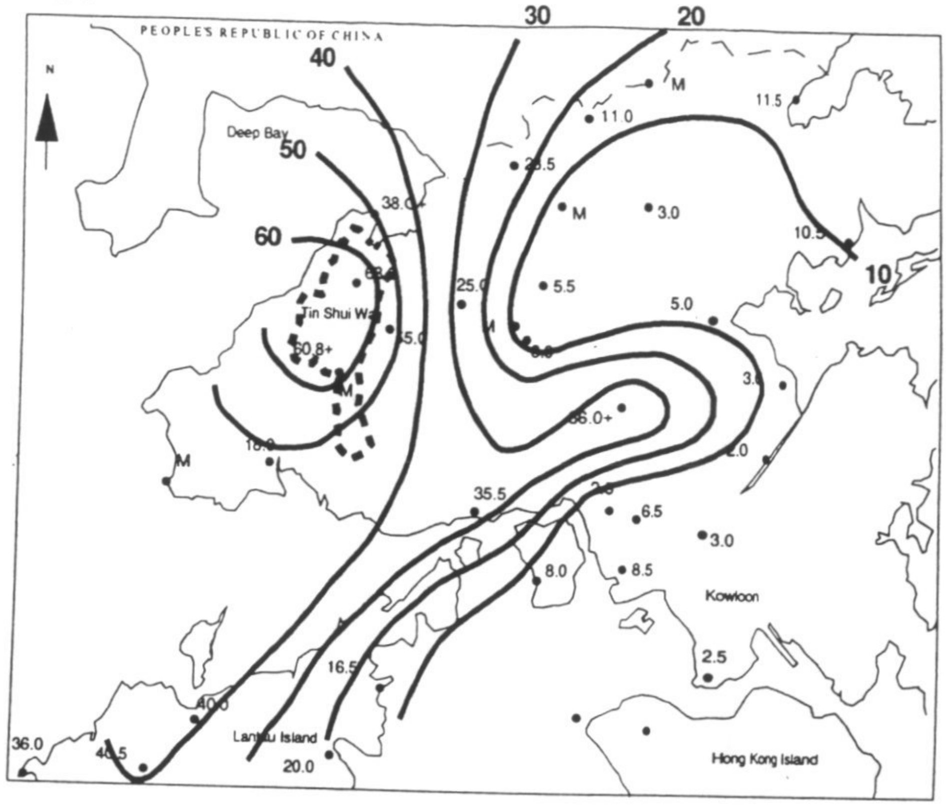
Figure 5

Isohyetal map for 22nd July 1994 (mm)

06:00 - 07:00



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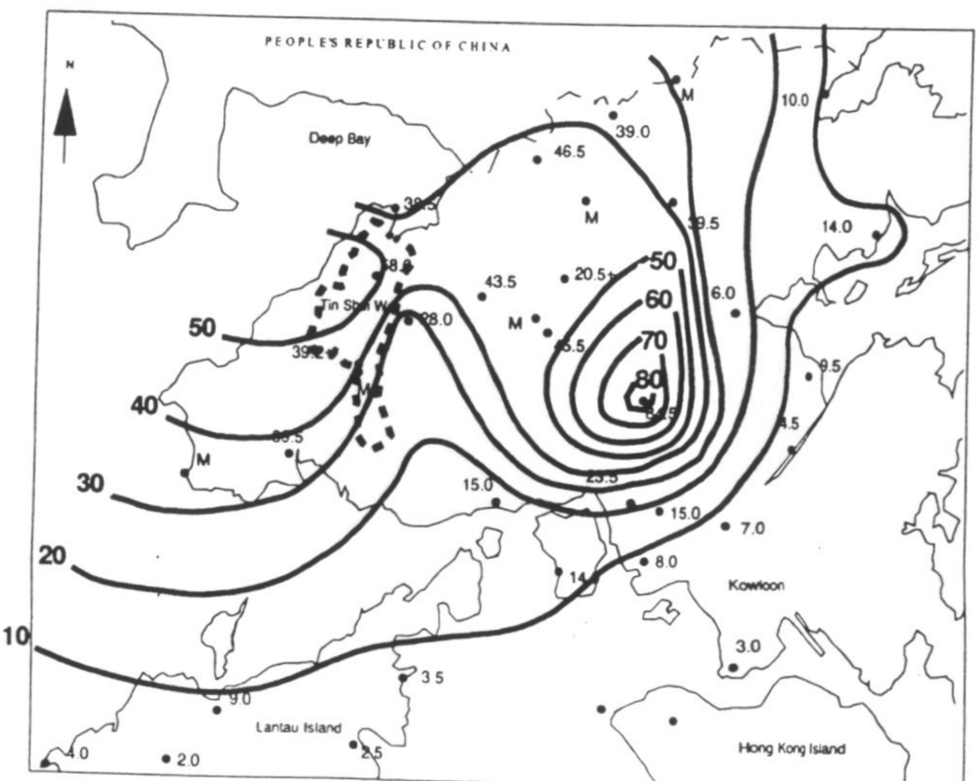

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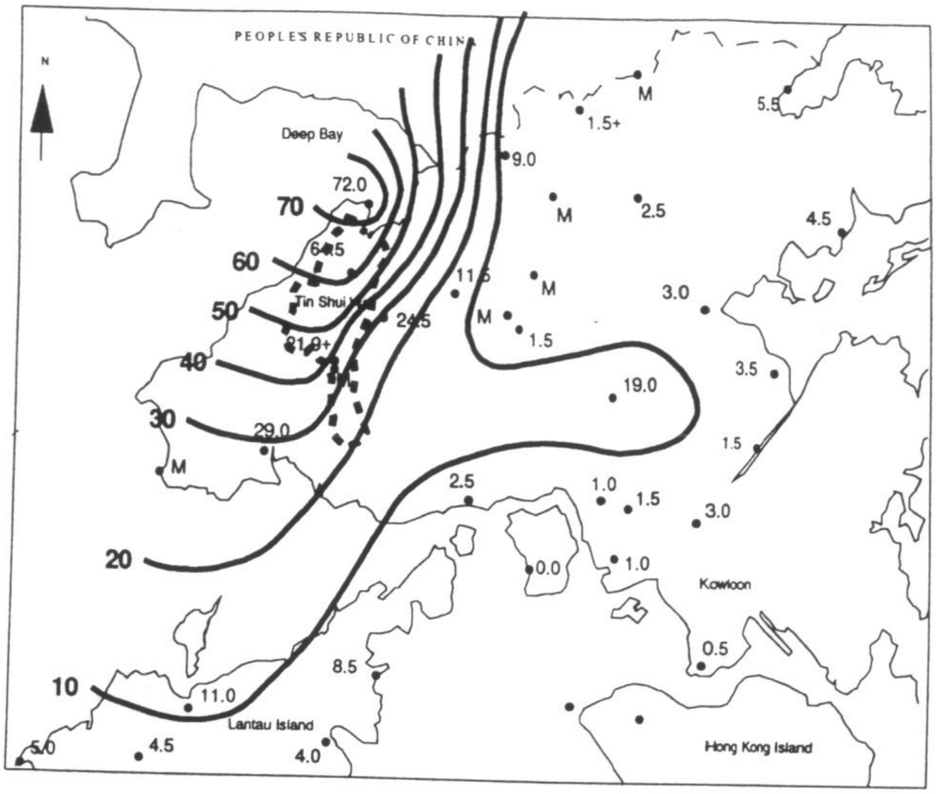
Figure 5

Isohyetal map for 22nd July 1994 (mm)

08:00 - 09:00



09:00 -10:00

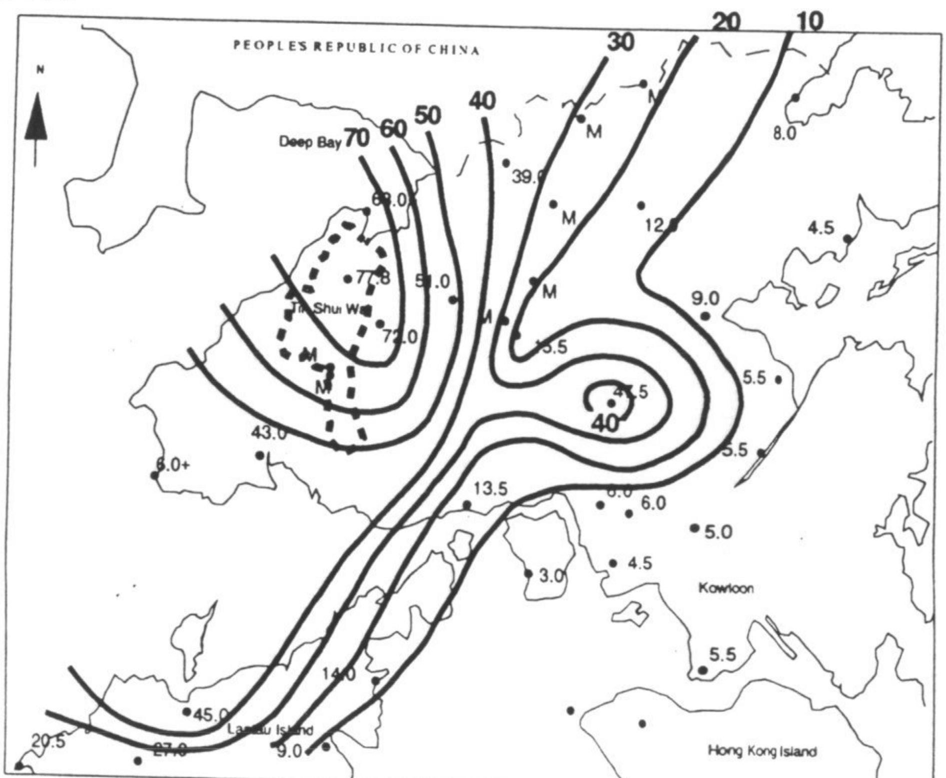


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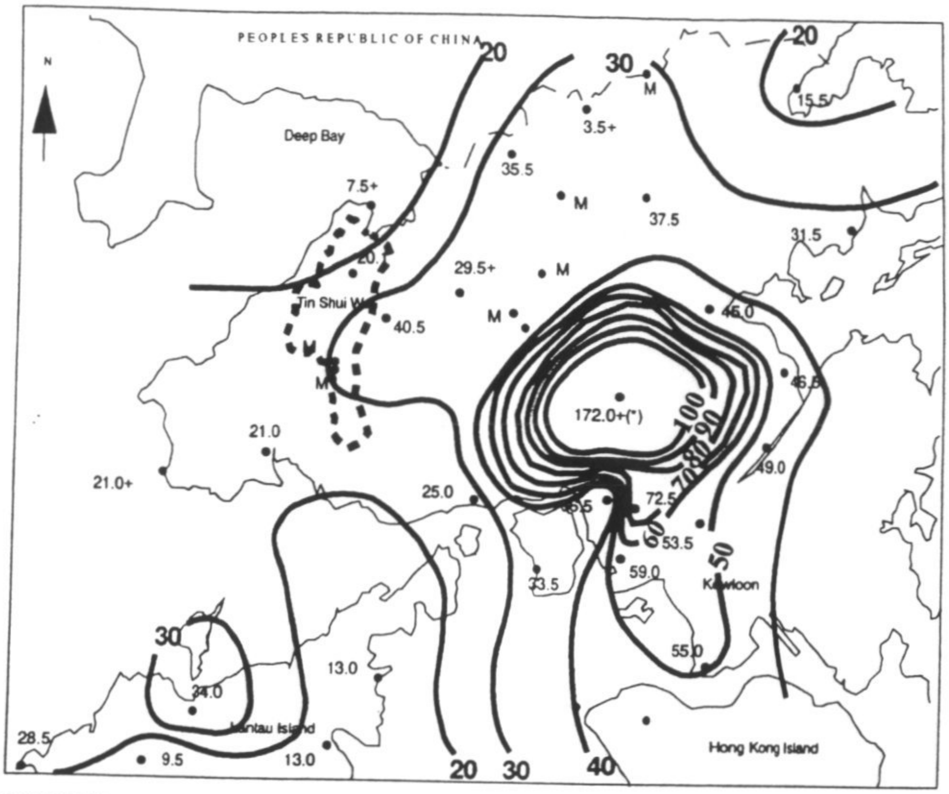
Figure 5

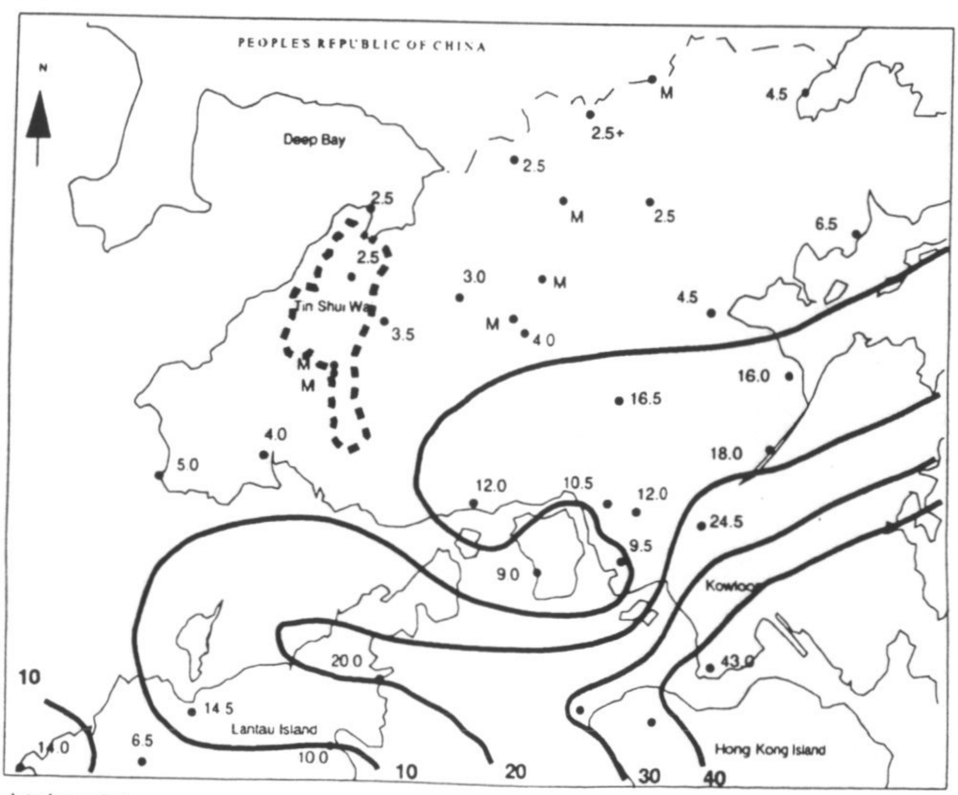
Isohyetal map for 22nd July 1994 (mm)

10:00 - 11:00



* suspect value

11:00 - 12:00



M missing all data for period

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Figure 5

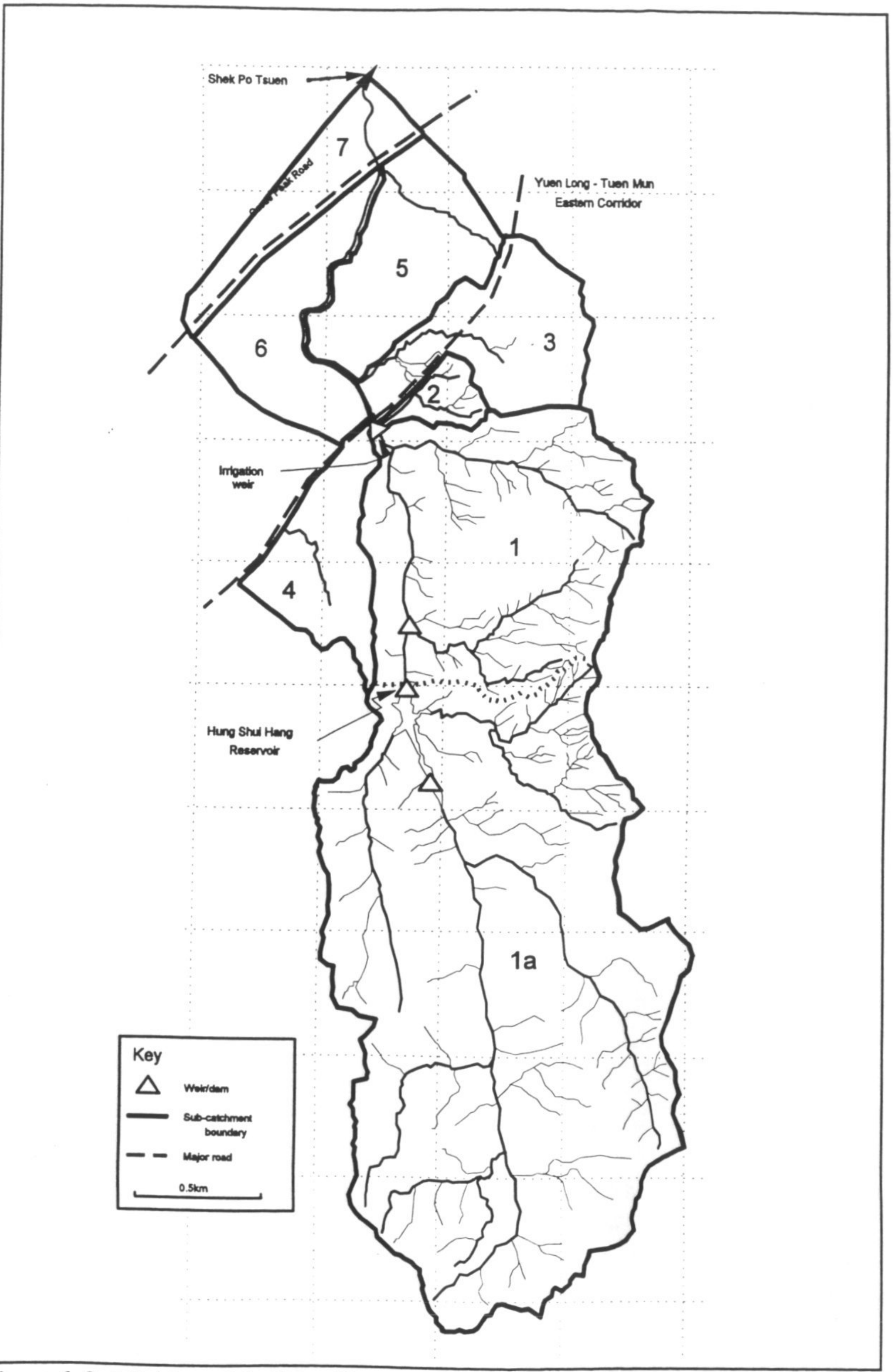


Figure 6 Subdivision of the Hung Shui Kiu catchment for hydrological modelling.

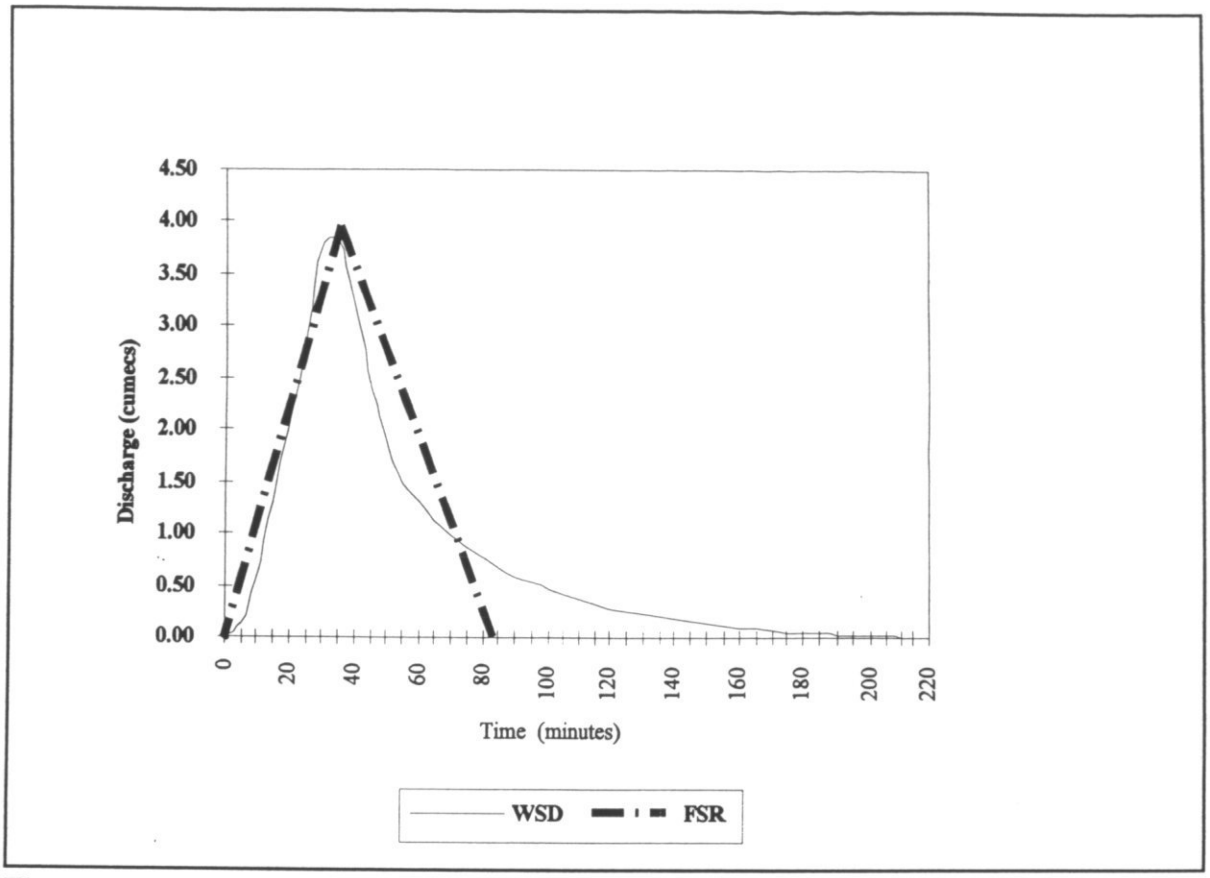


Figure 7 Comparison of the WSD and FSR unit hydrographs.

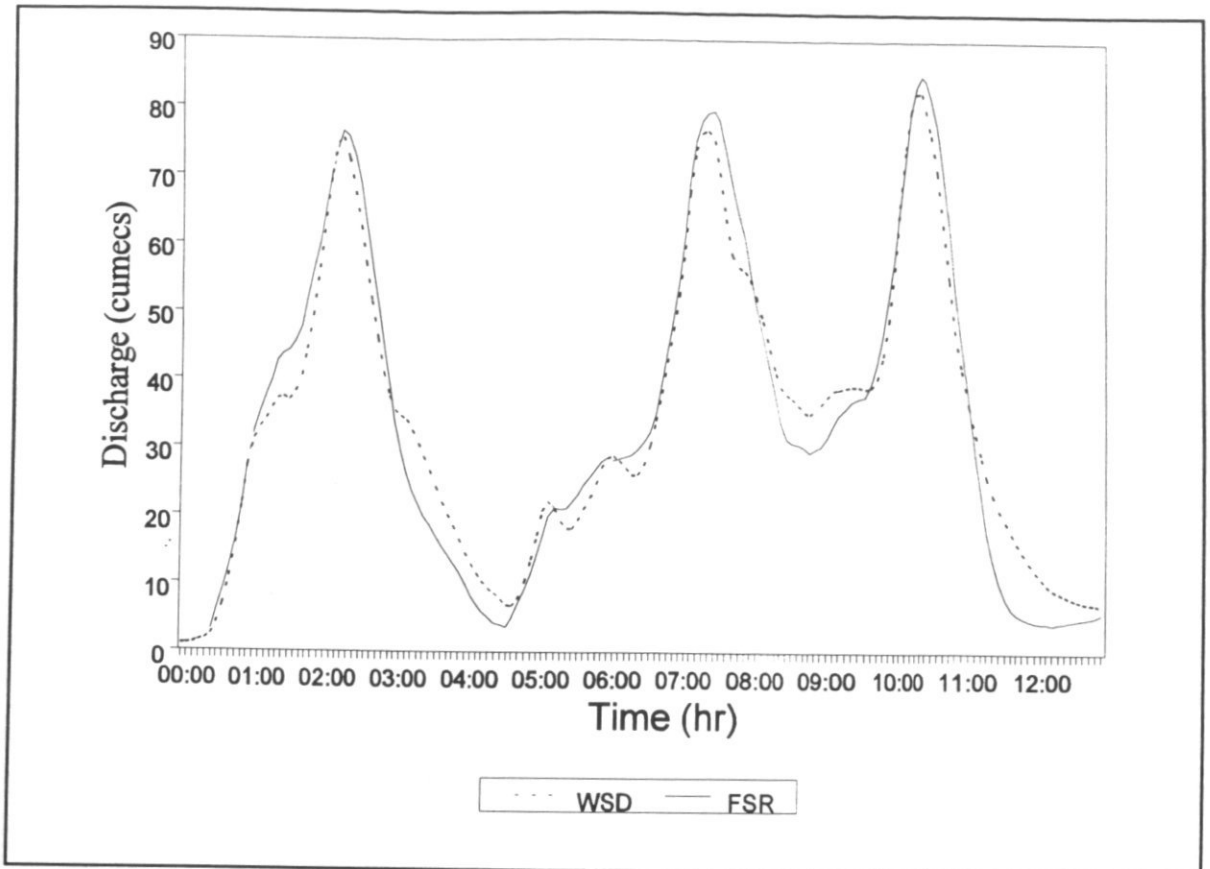


Figure 8 Effects of using the WSD and FSR unit hydrographs - 22 July, 1994 event.

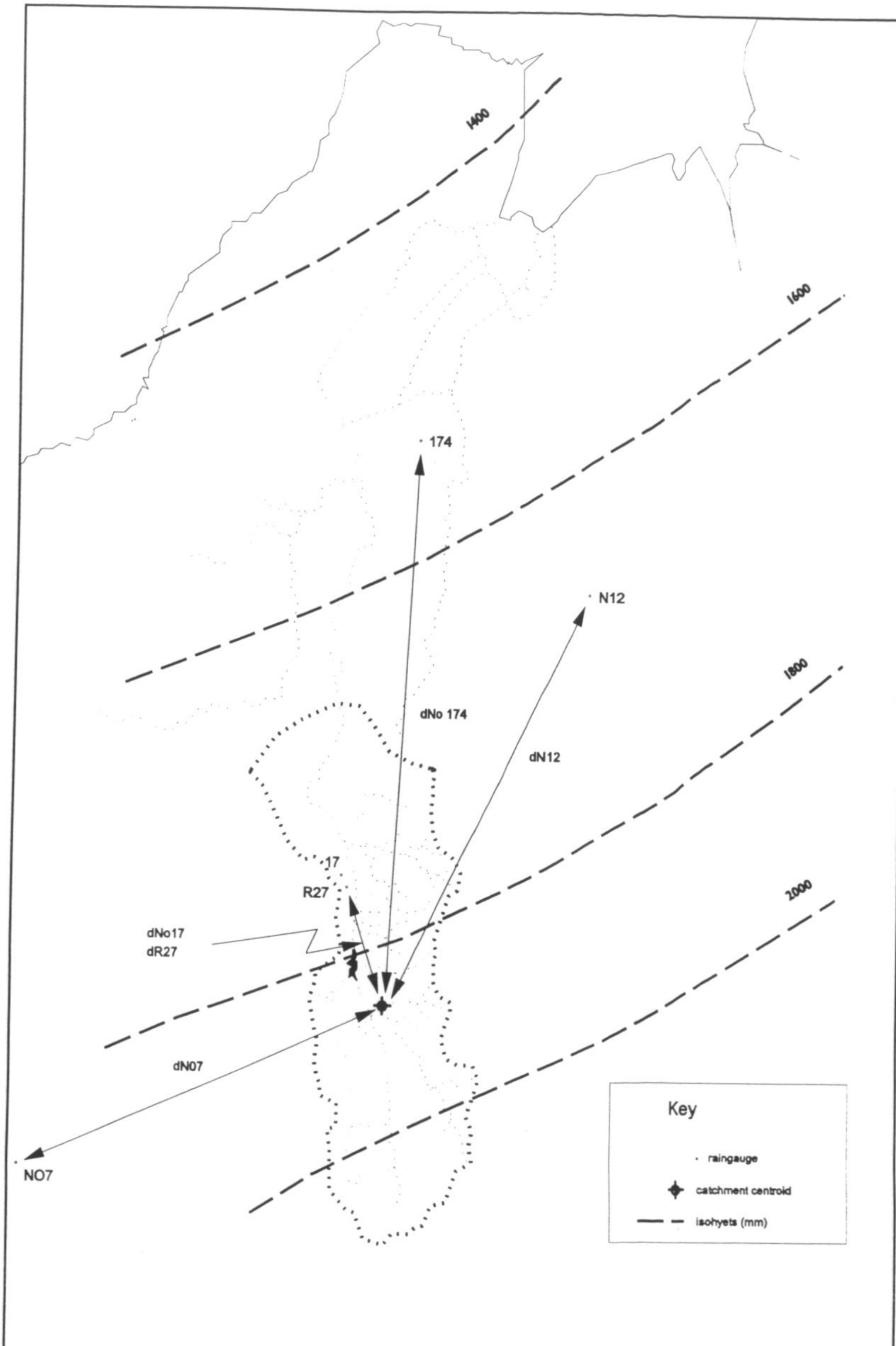


Figure 9 Hung Shui Kiu catchment - areal rainfall algorithm.

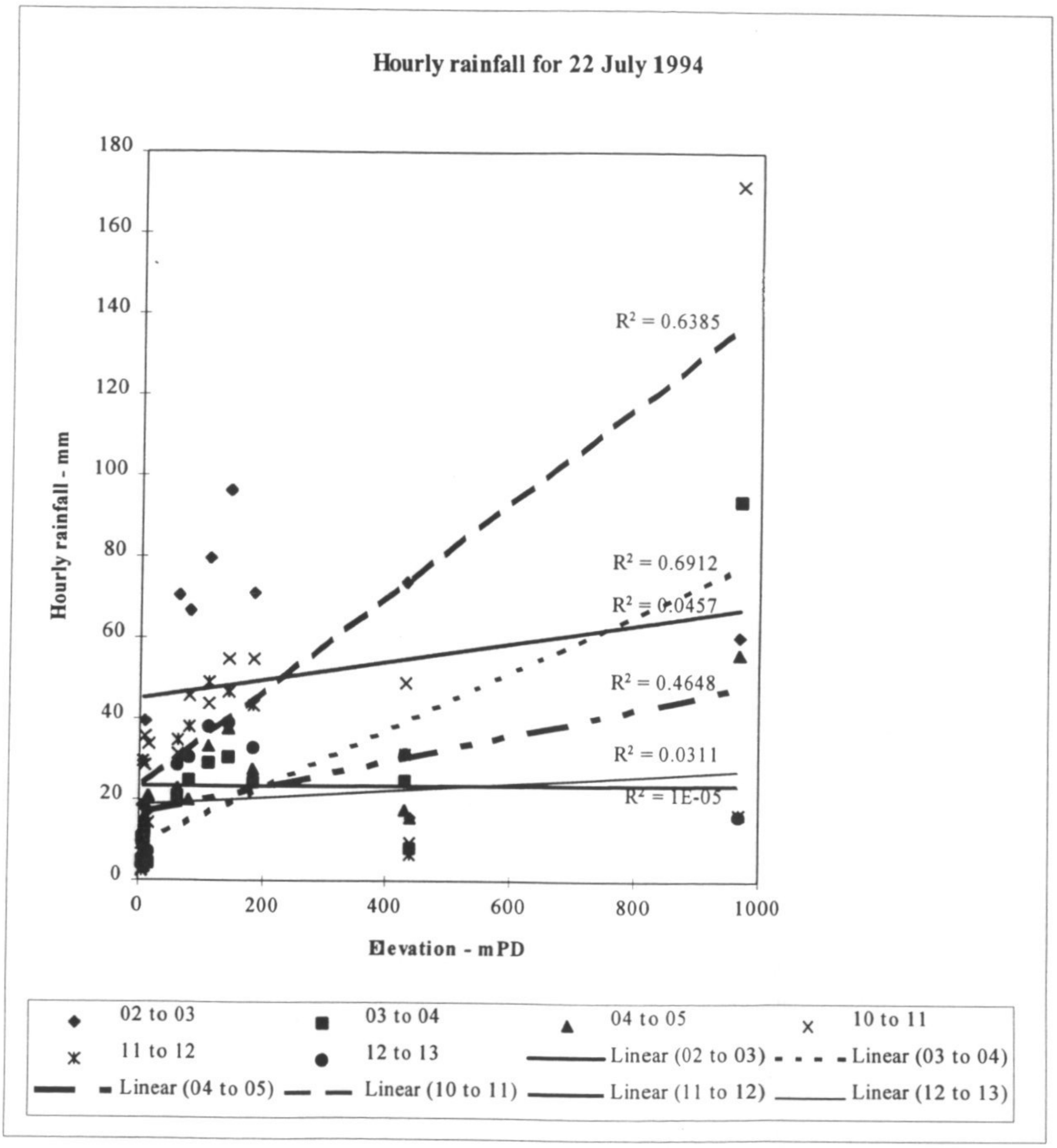


Figure 10 Comparison of hourly rainfall with elevation - 22 July, 1994 event

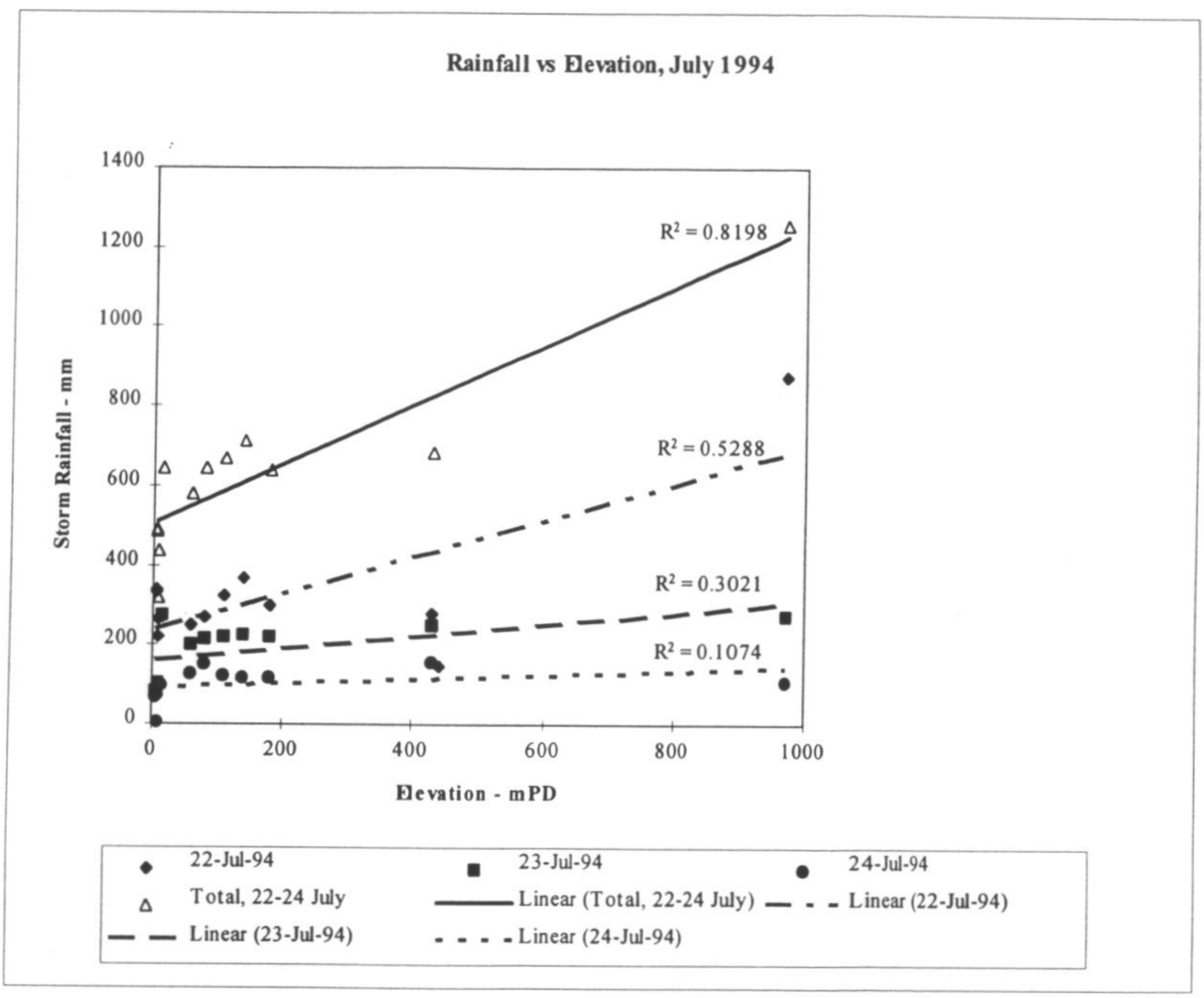


Figure 11 Comparison of total rainfall with elevation - 22 July, 1994 event

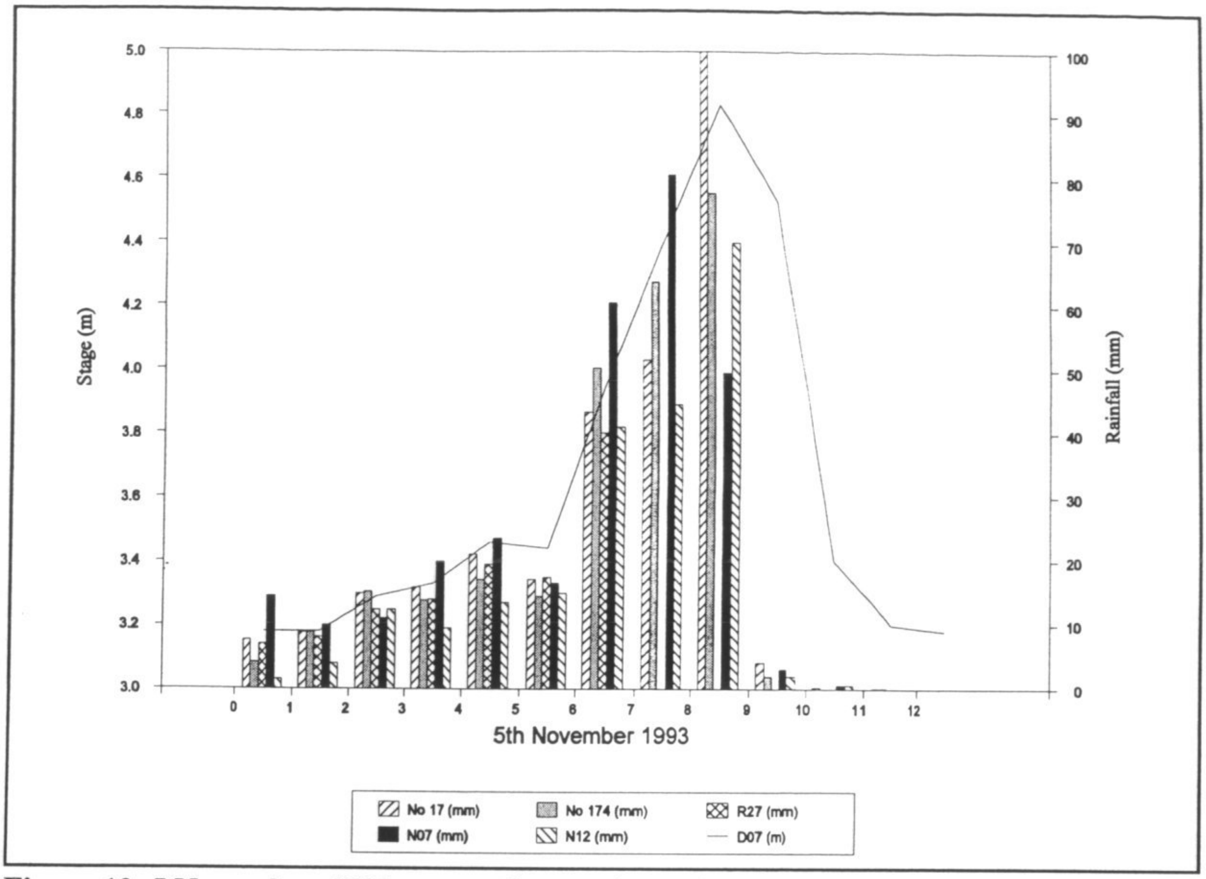


Figure 12 5 November, 1993 event - Comparison of hourly stage and point rainfall data.

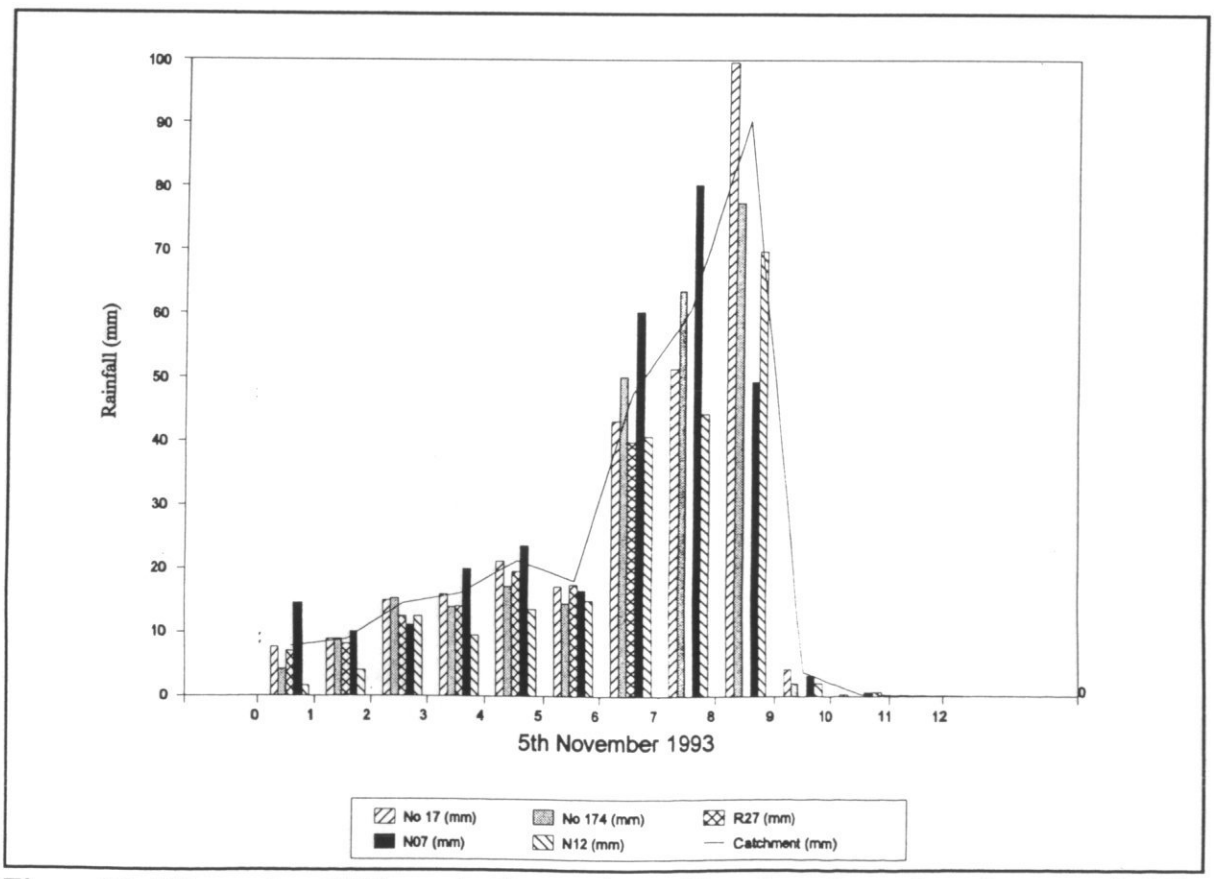


Figure 13 5 November, 1993 event - Comparison of hourly point and catchment rainfalls.

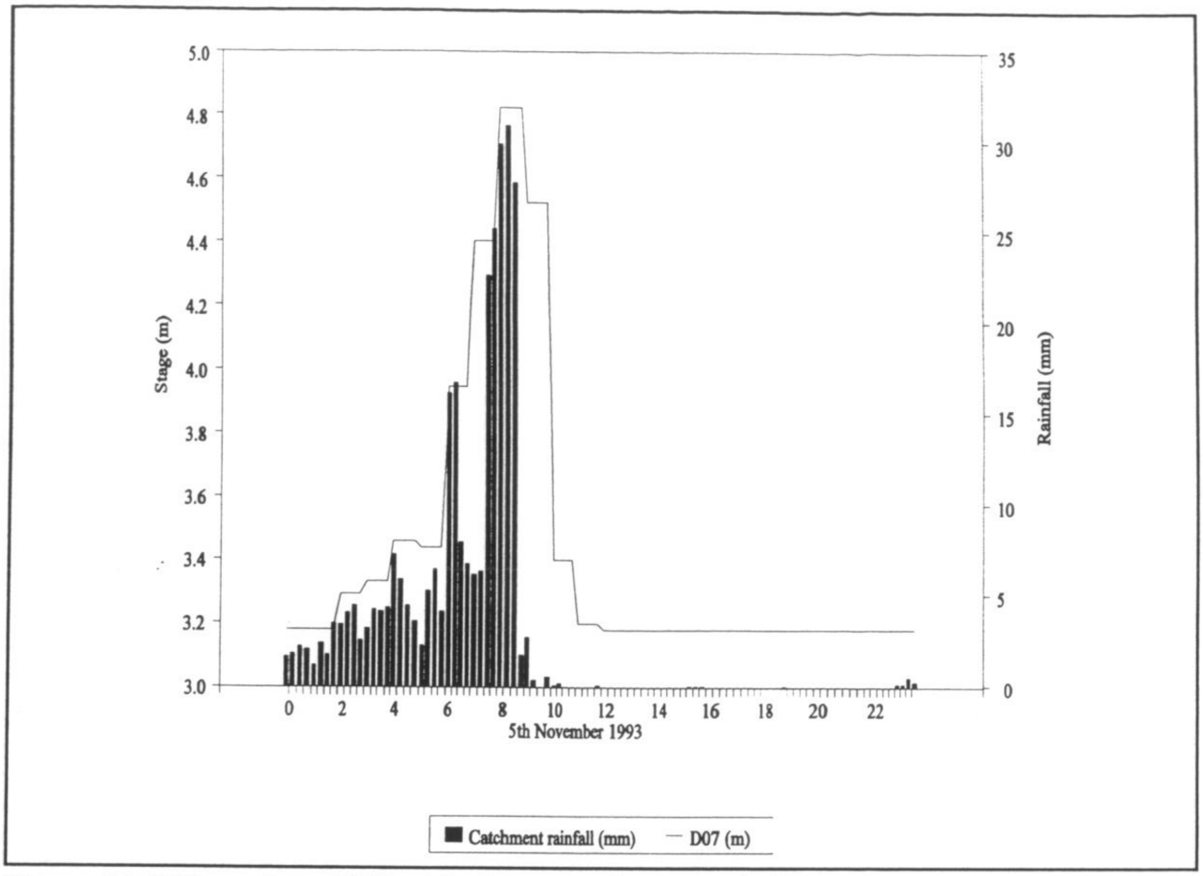


Figure 14 5 November, 1993 - 15 minute catchment rainfall and stage.

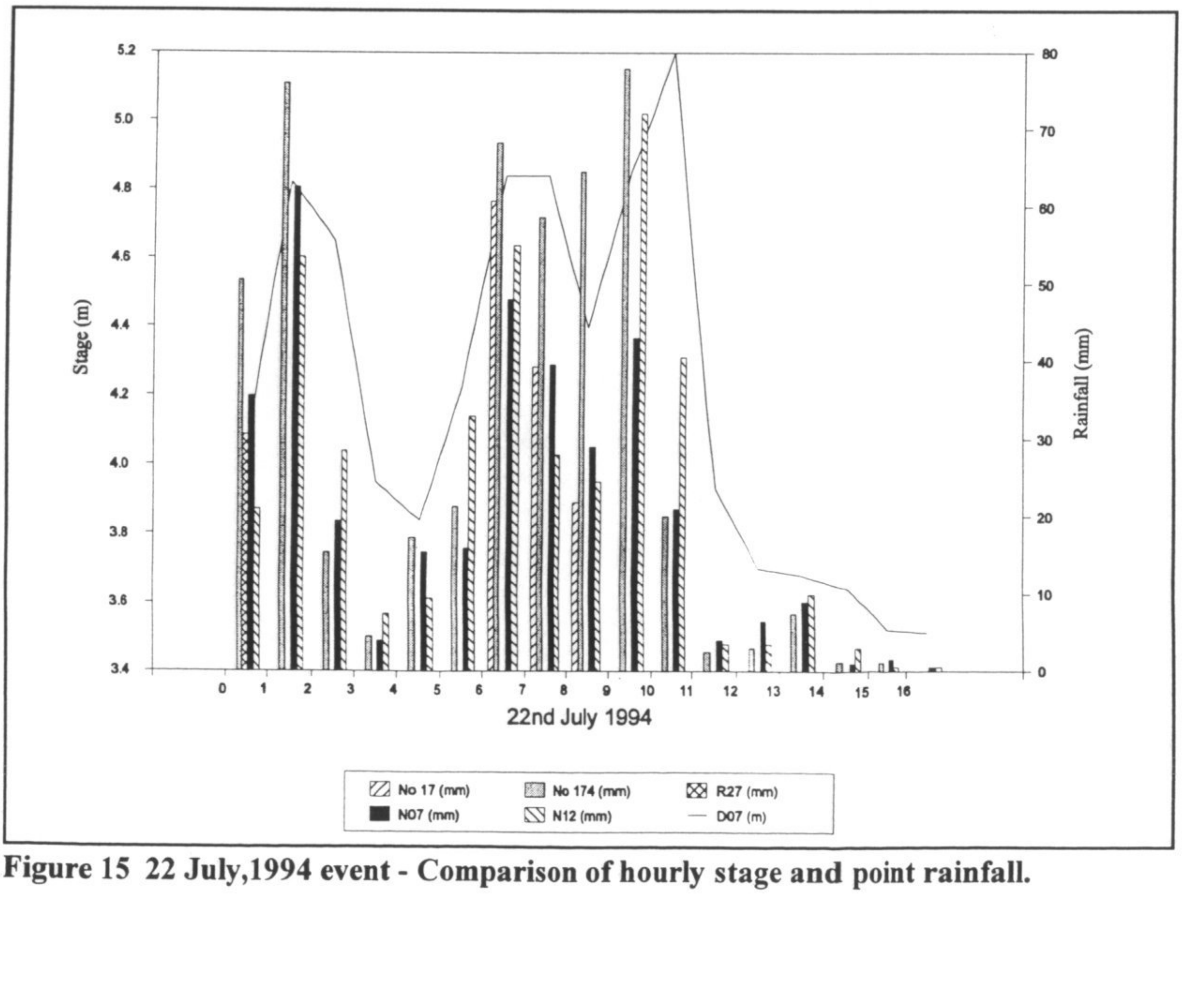


Figure 15 22 July,1994 event - Comparison of hourly stage and point rainfall.

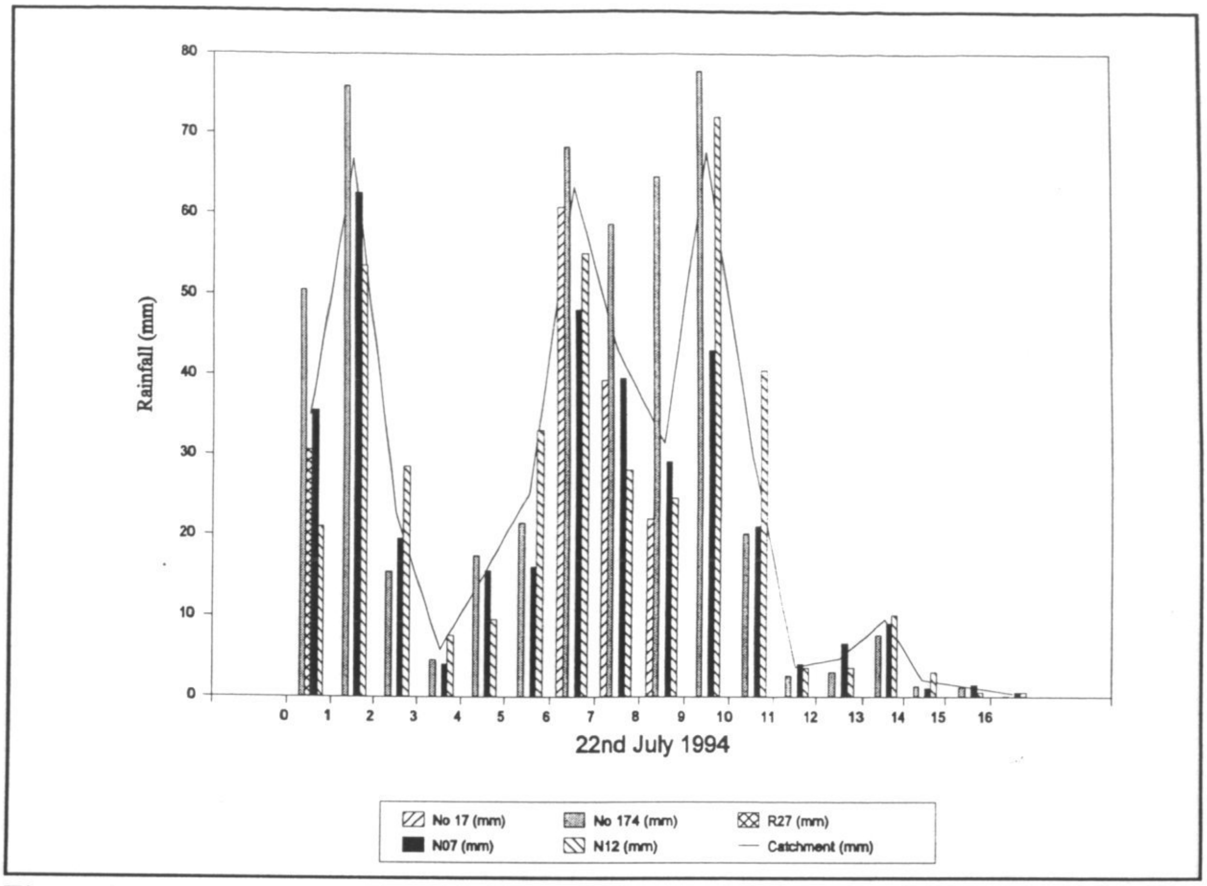


Figure 16 22 July, 1994 event - Comparison of hourly point and catchment rainfalls.

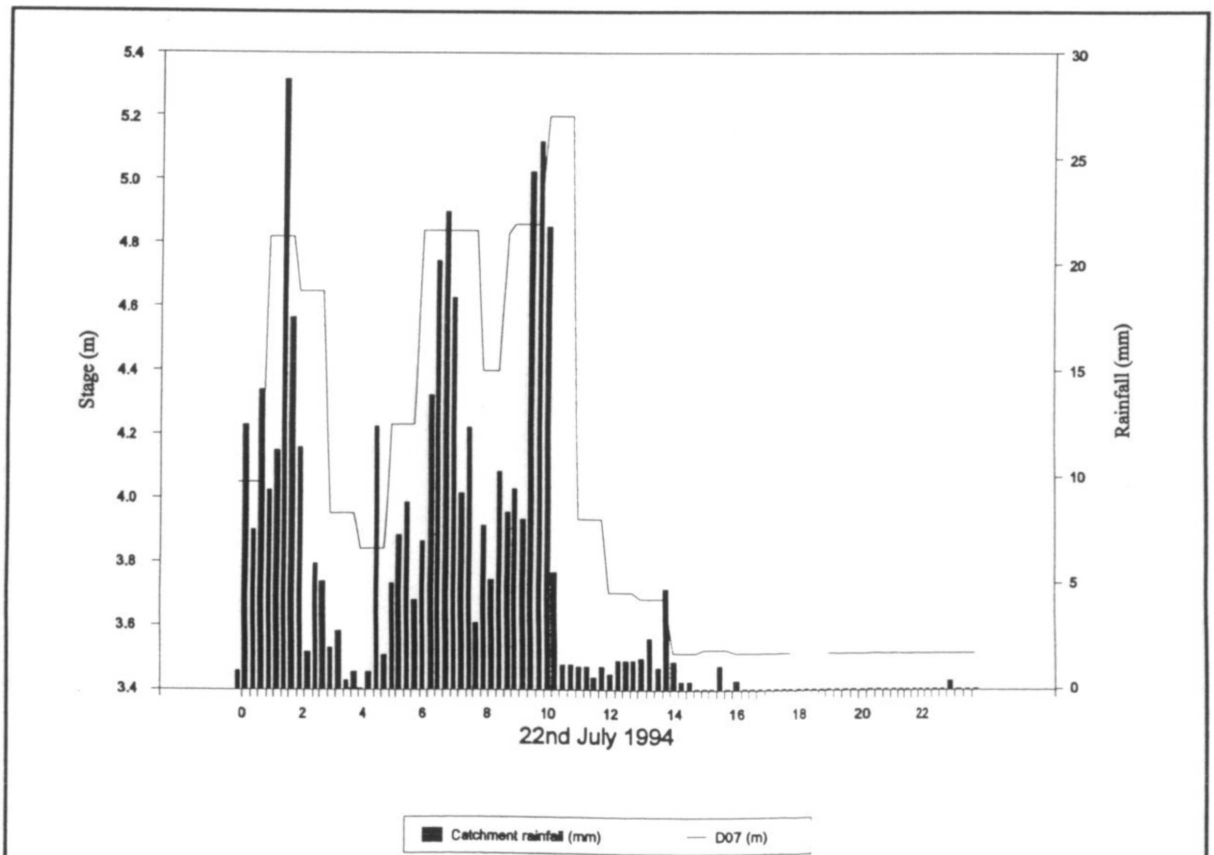


Figure 17 22 July, 1994 event - 15 minute catchment rainfall and stage.

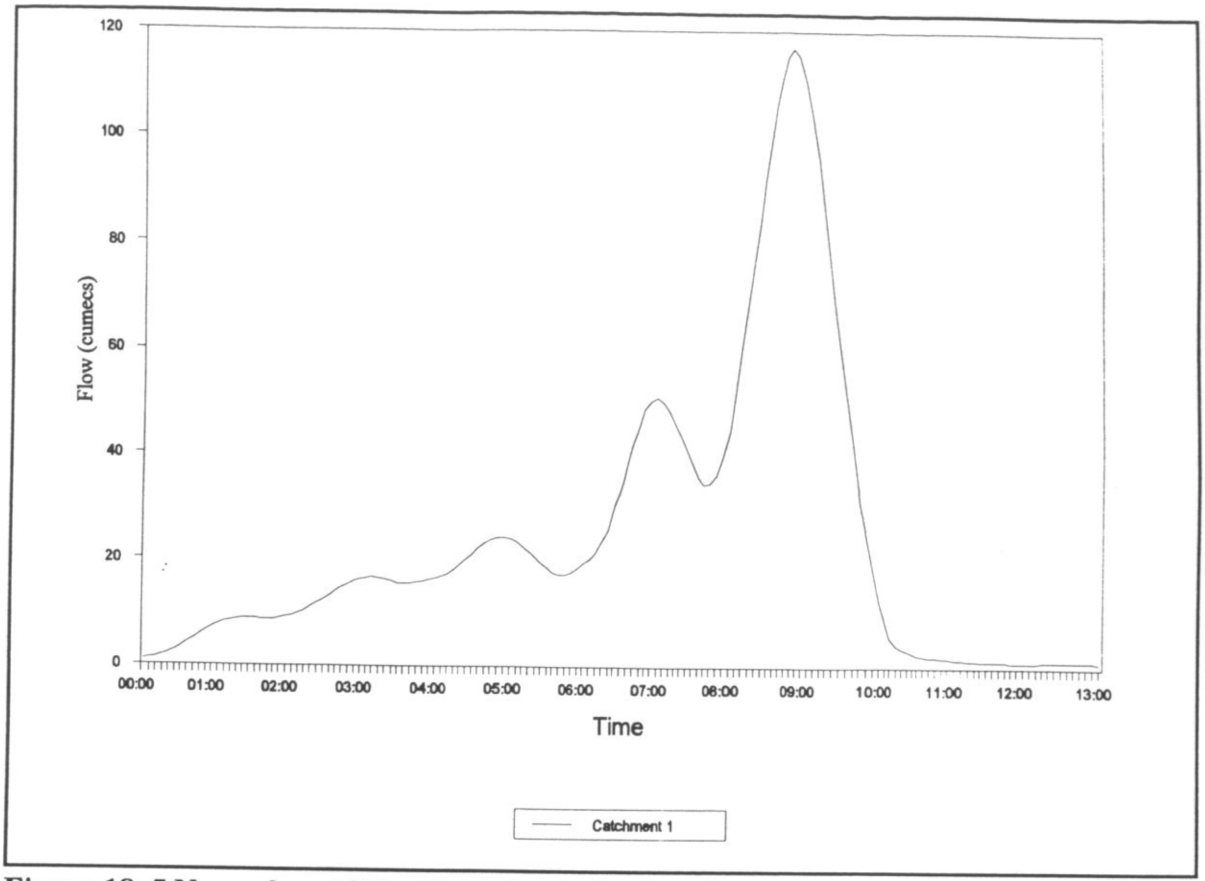


Figure 18 5 November, 1993 - Flow simulation by PR method.

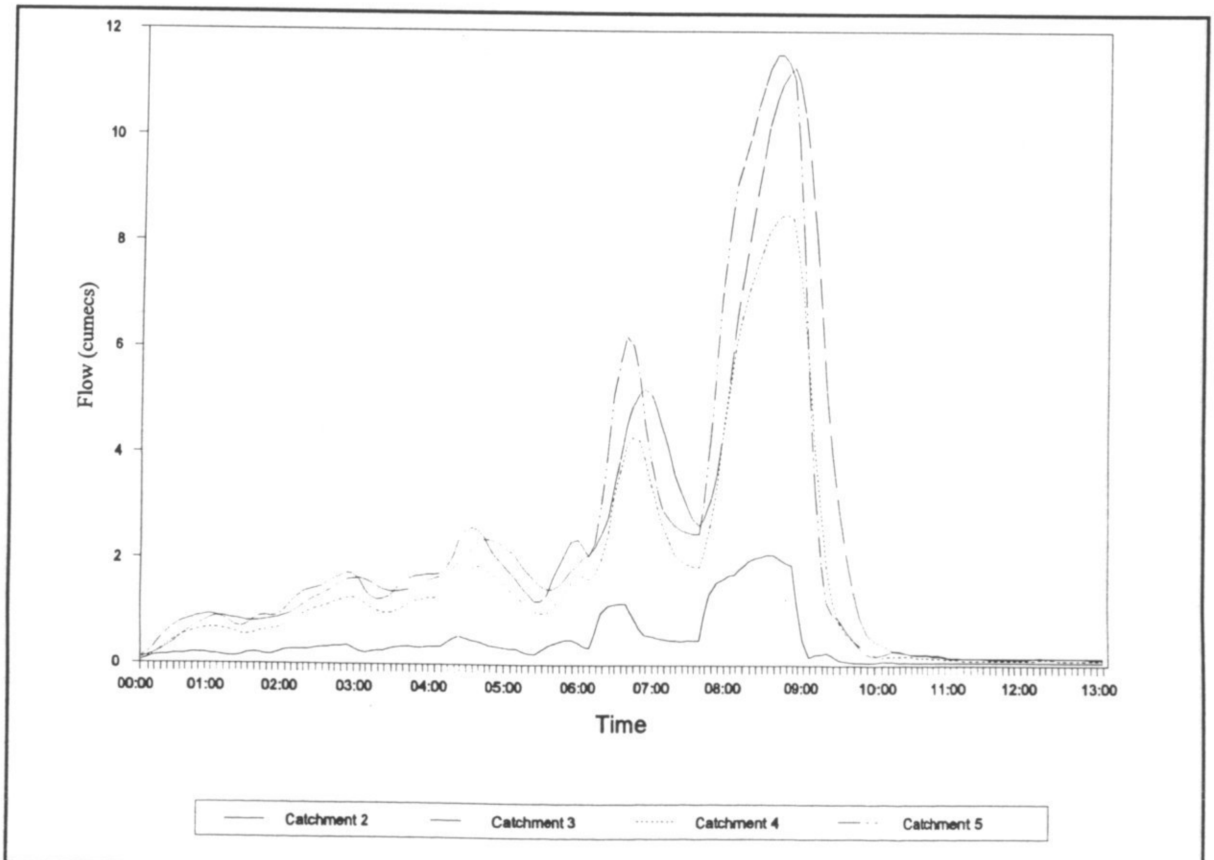


Figure 19 5 November, 1993 event - Flow simulation by PR method.

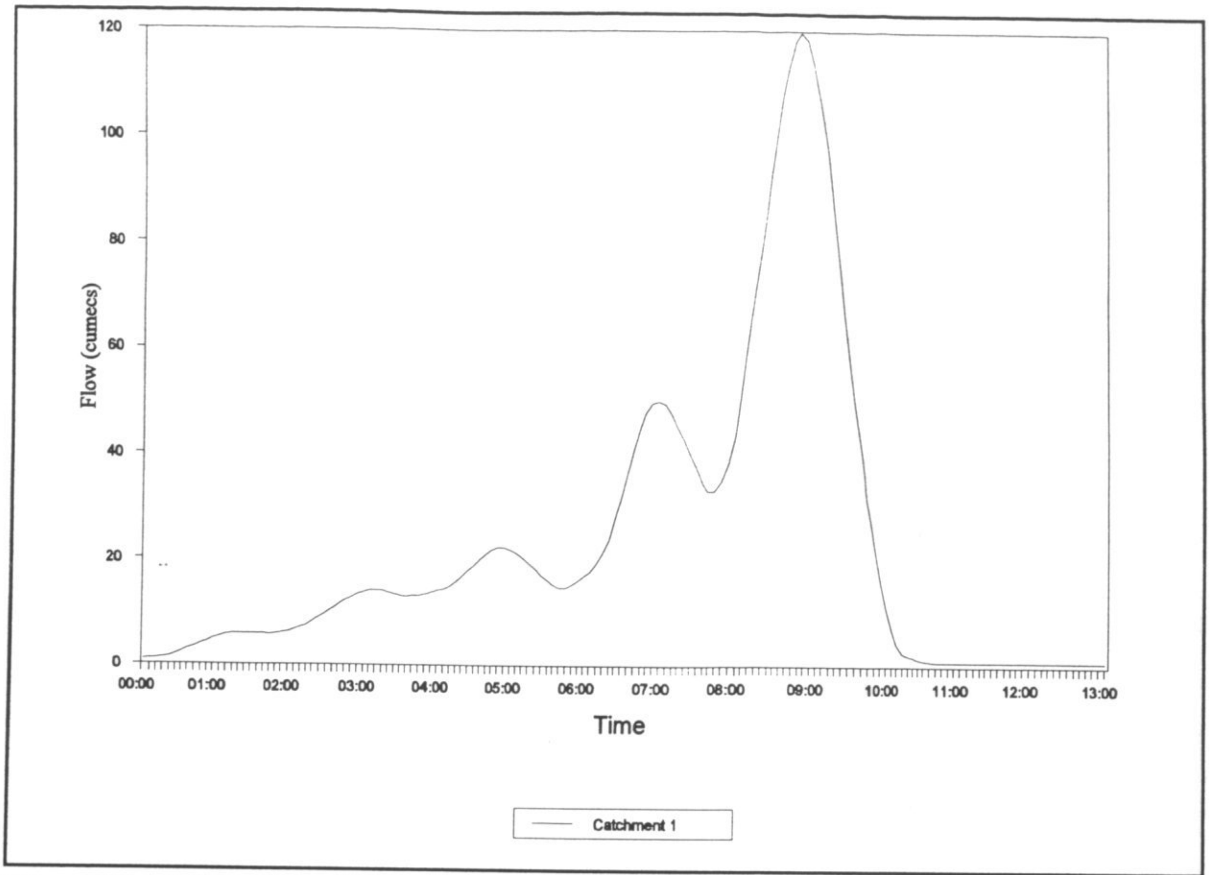


Figure 20 5 November, 1993 event - Flow simulation by loss rate method.

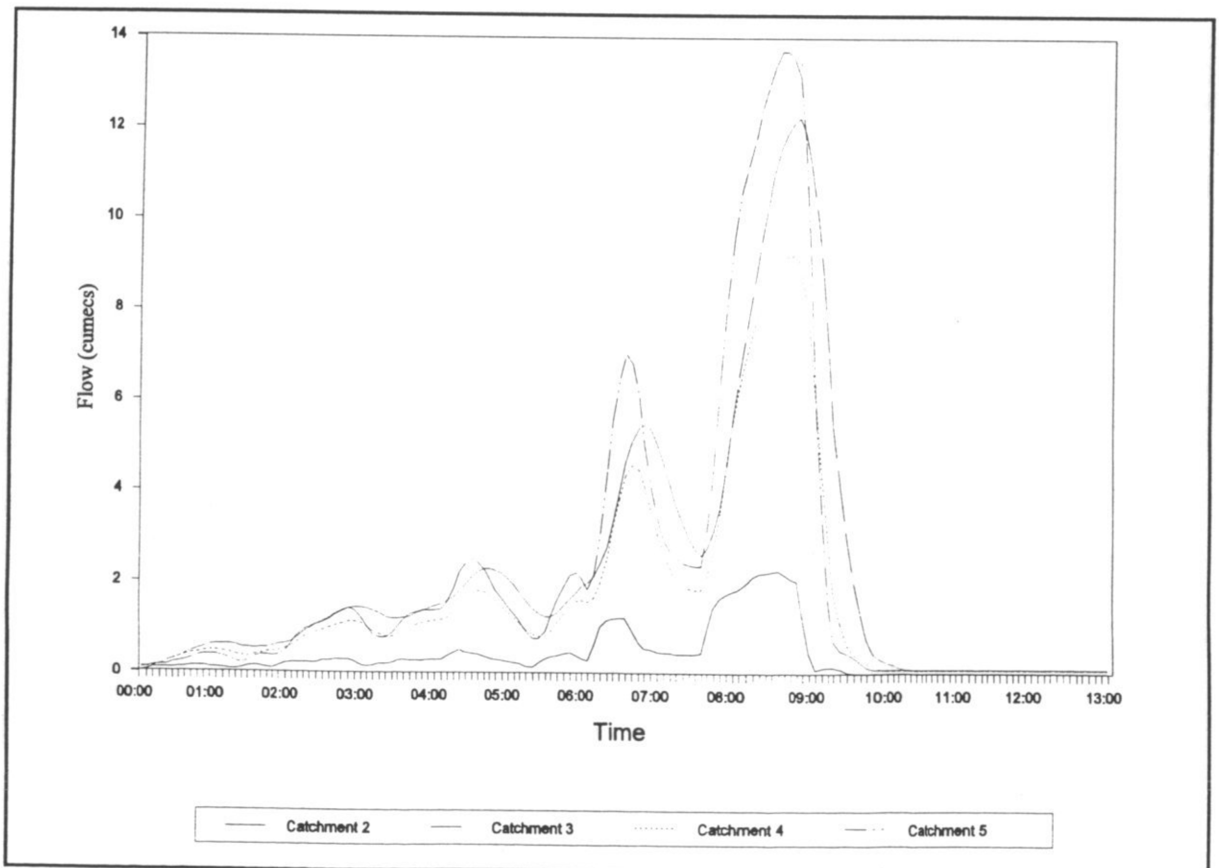


Figure 21 5 November, 1993 event - Flow simulation by loss rate method.

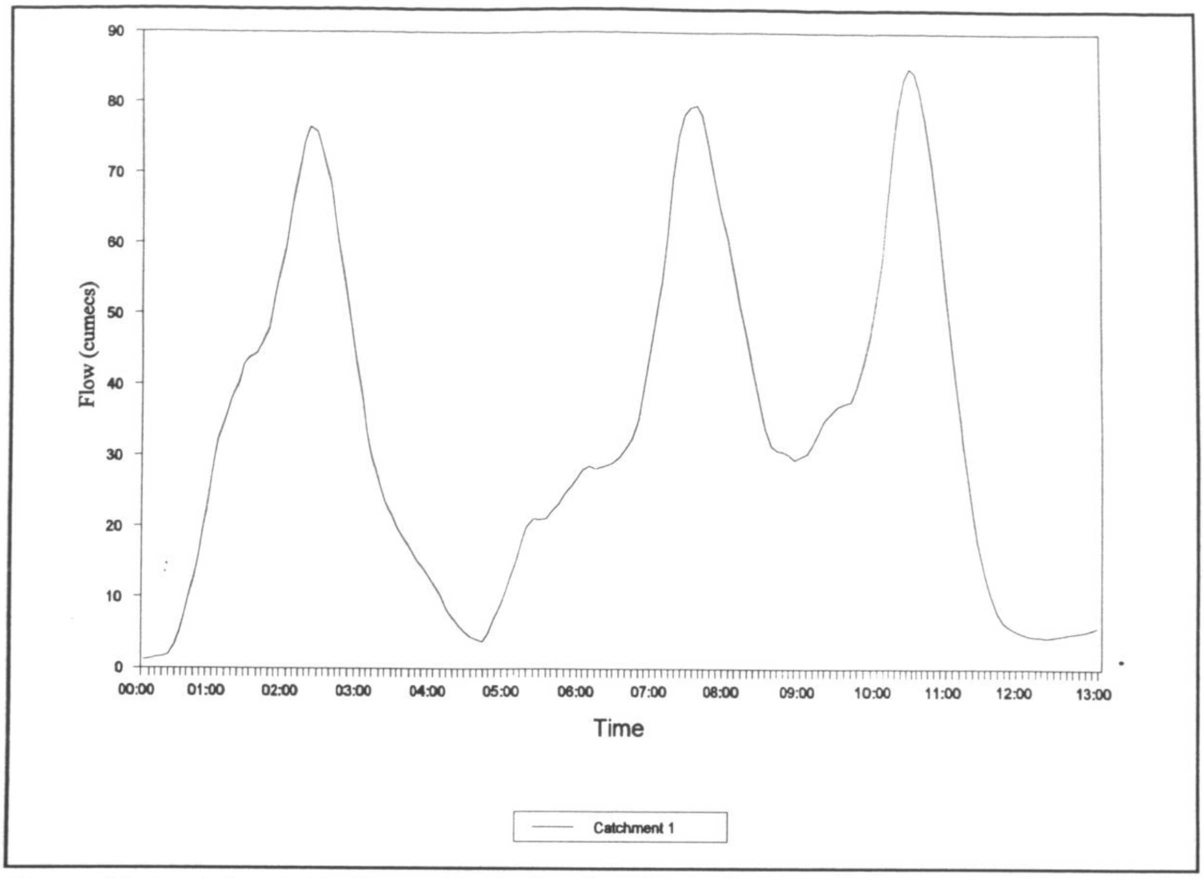


Figure 22 22 July, 1994 - Flow simulation by PR method.

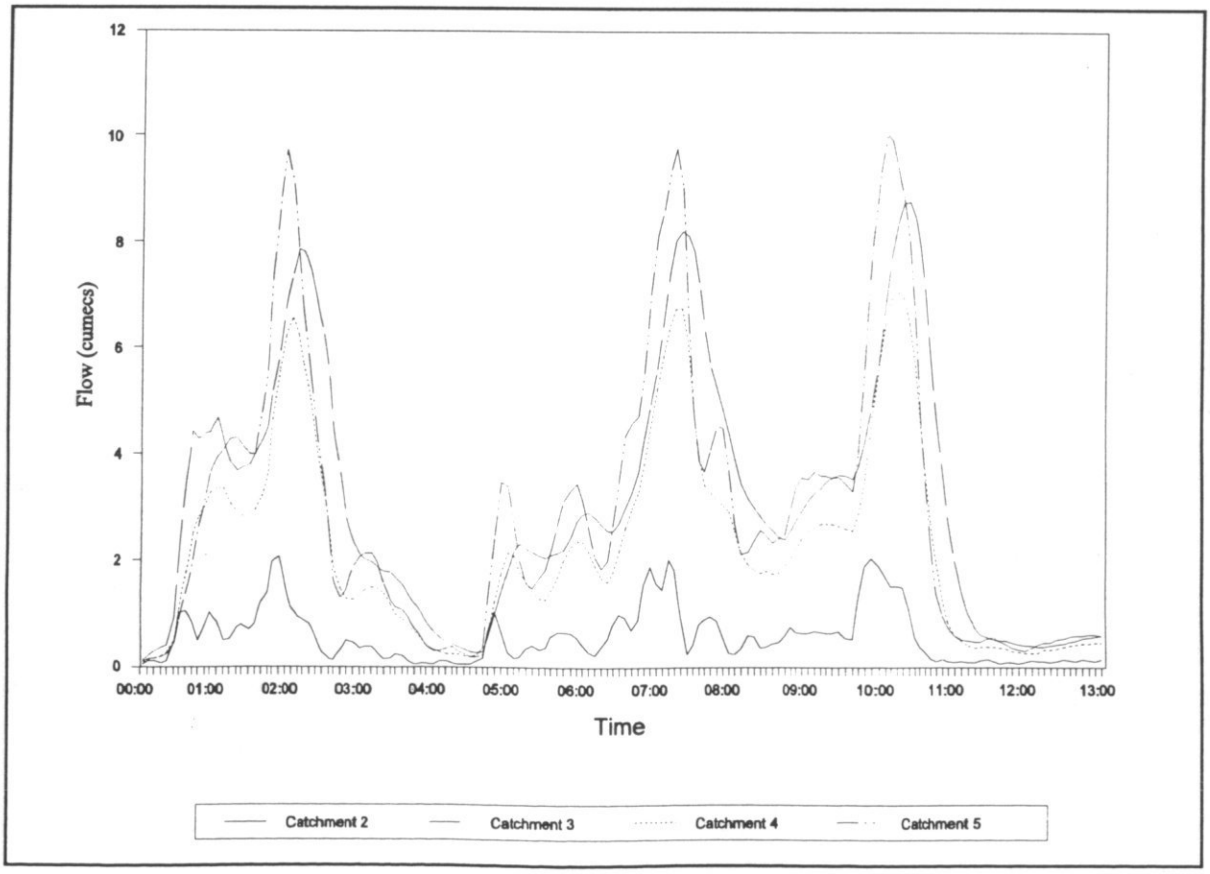


Figure 23 22 July, 1994 event - Flow simulation by PR method.

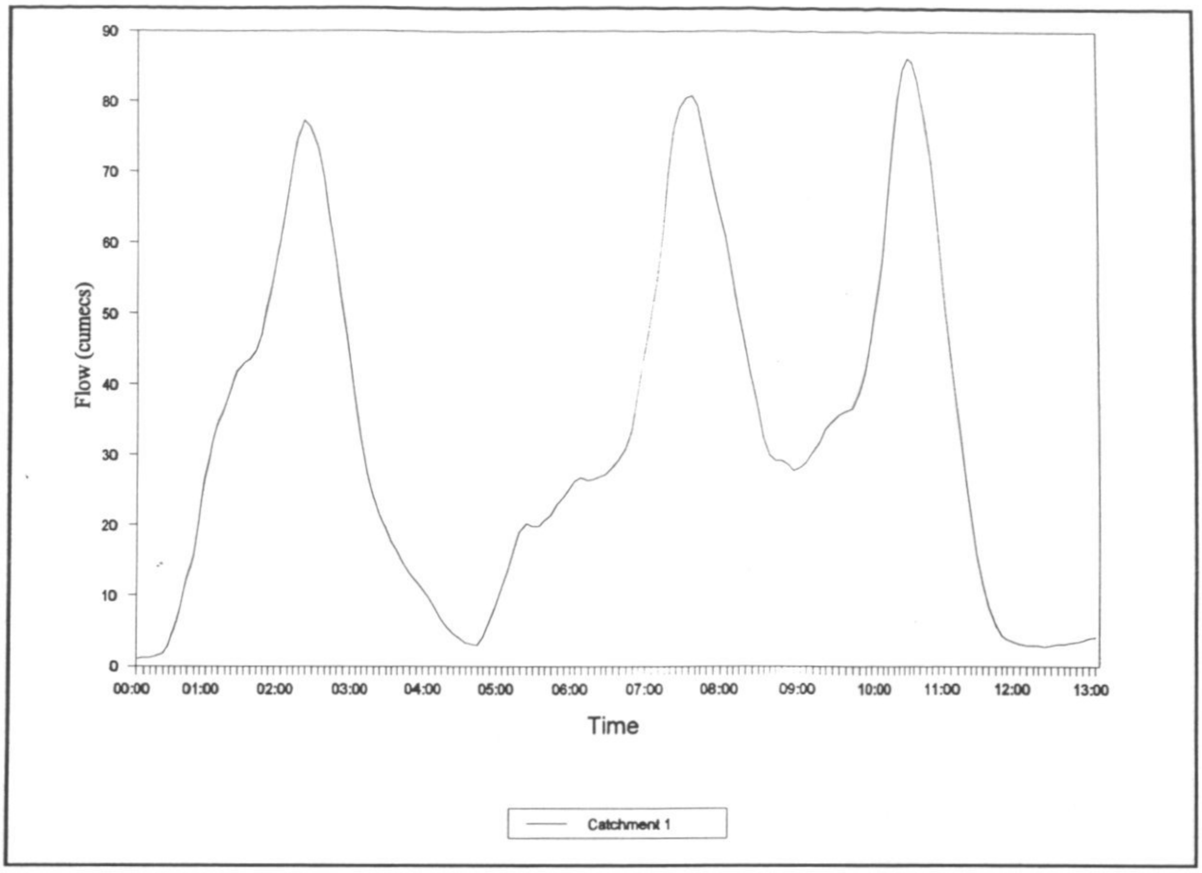


Figure 24 22 July, 1994 event - Flow simulation by loss rate method.

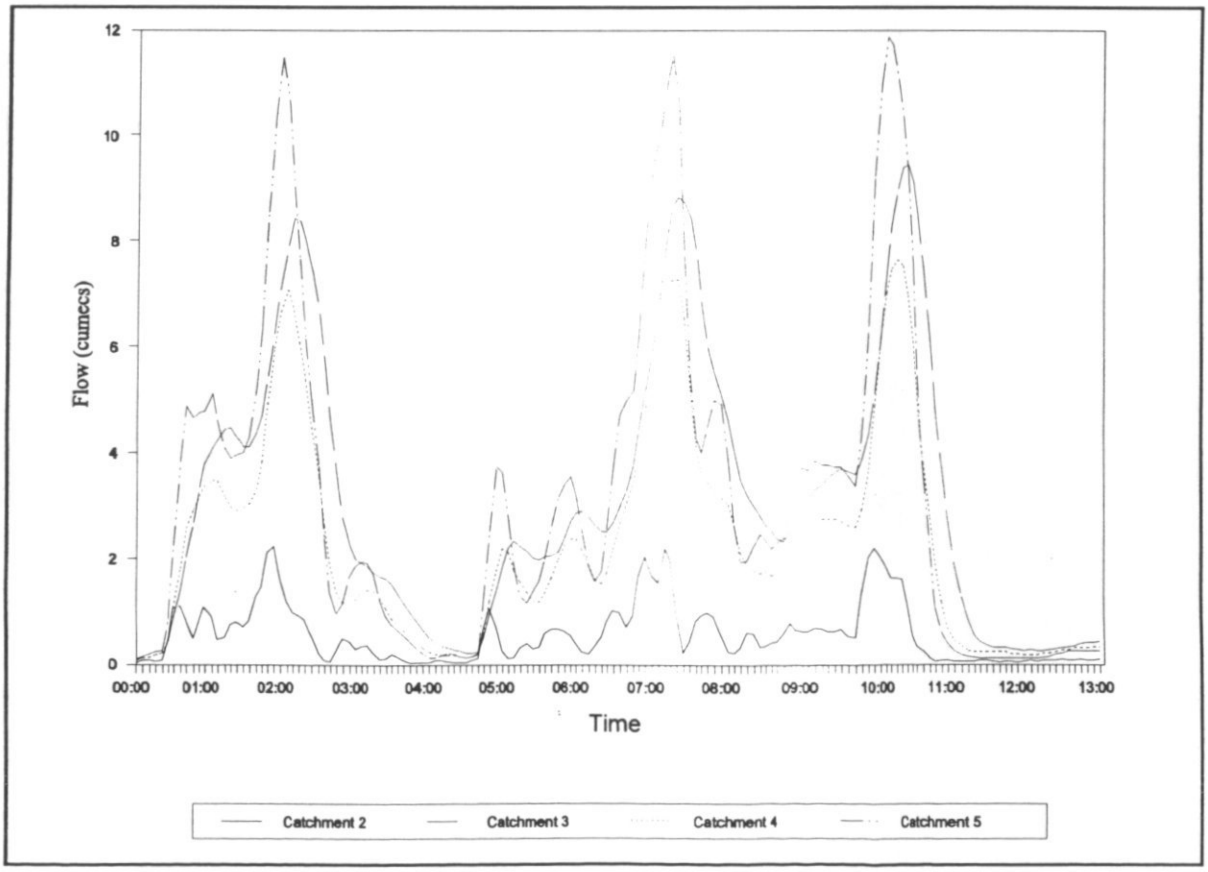


Figure 25 22 July, 1994 event - Flow simulation by loss rate method.

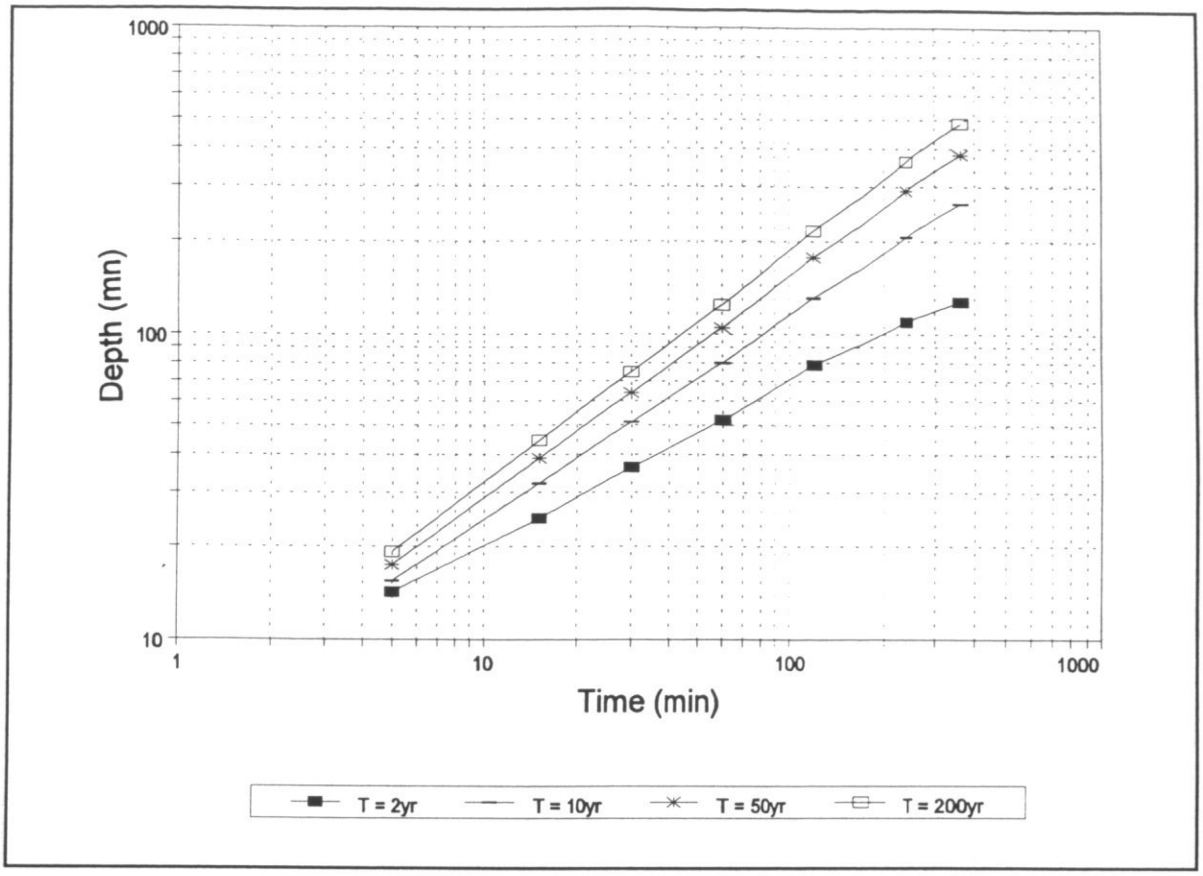


Figure 26 Depth-duration curves for Yuen Long RG Fliters.

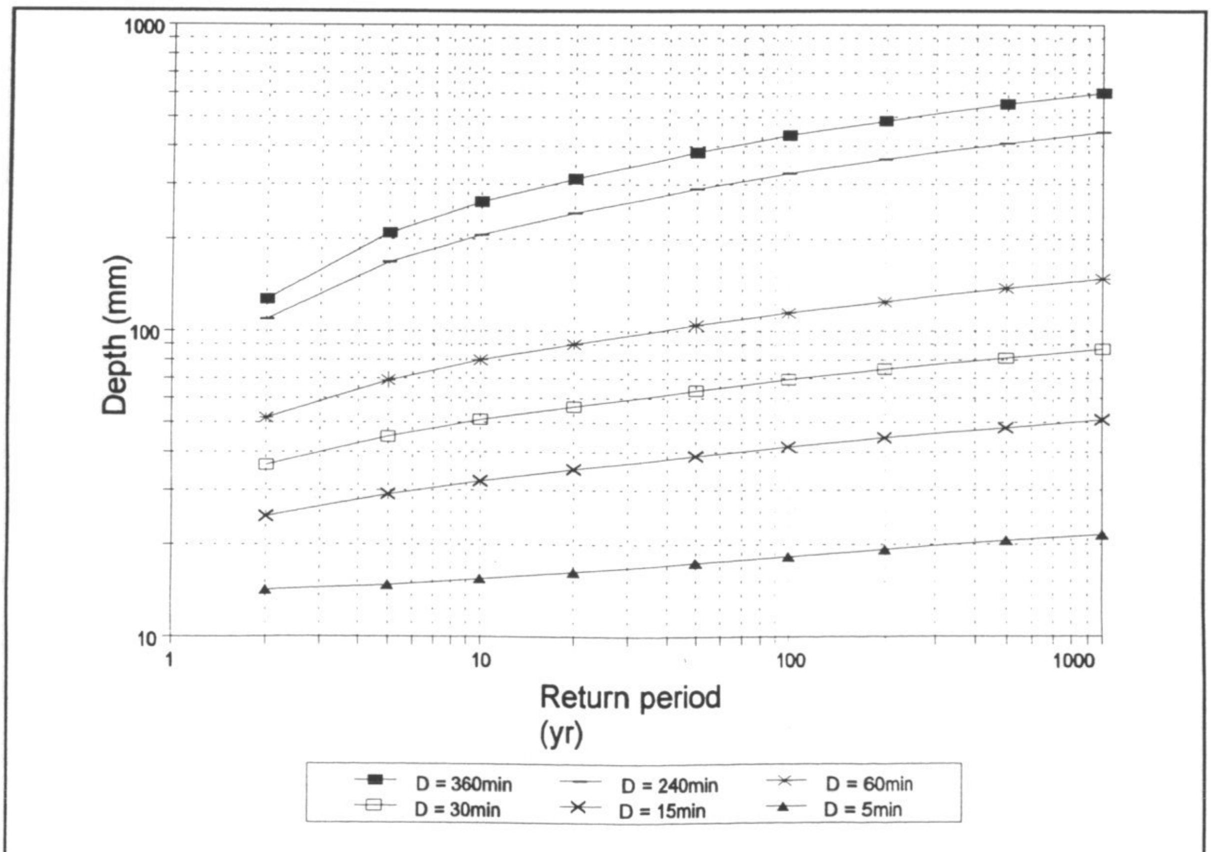


Figure 27 Depth-frequency curves for Yuen Long RG Filters.

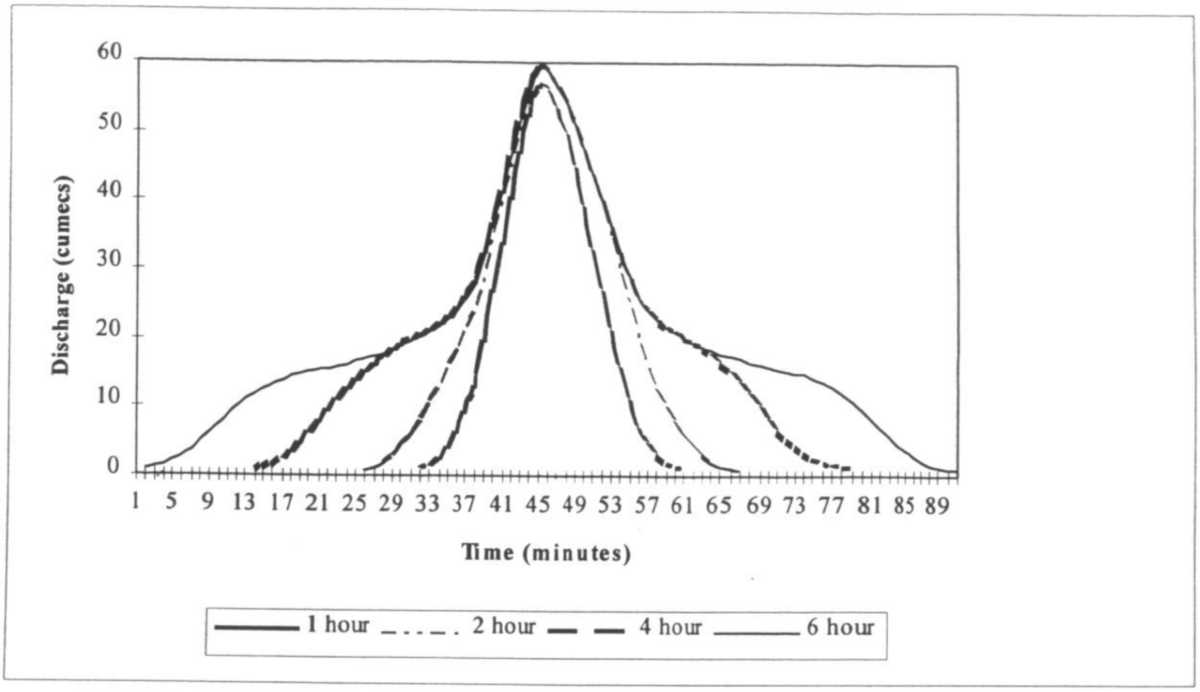


Figure 28 Effect of storm duration - Subcatchment 1, 1 in 2 year storm.

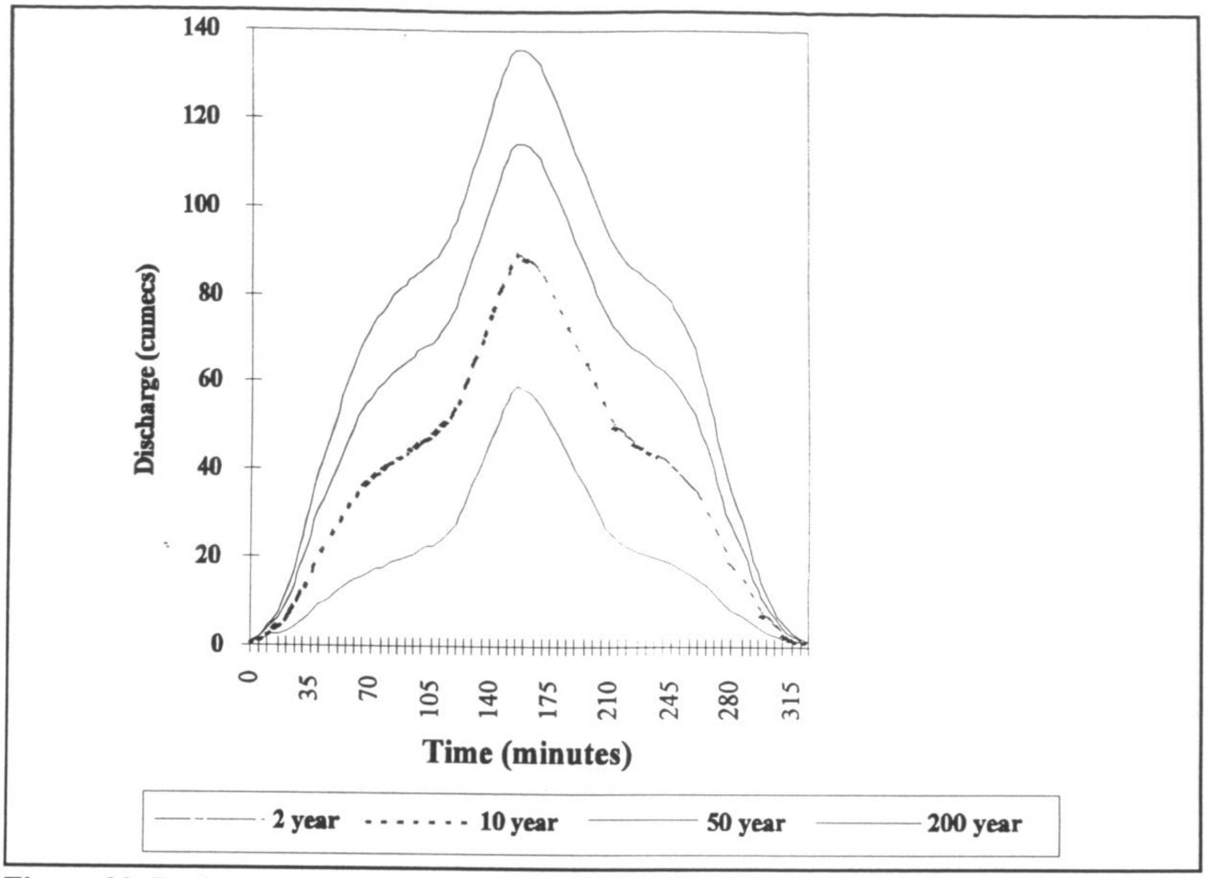


Figure 29 Design hydrographs - Sub-catchment 1.

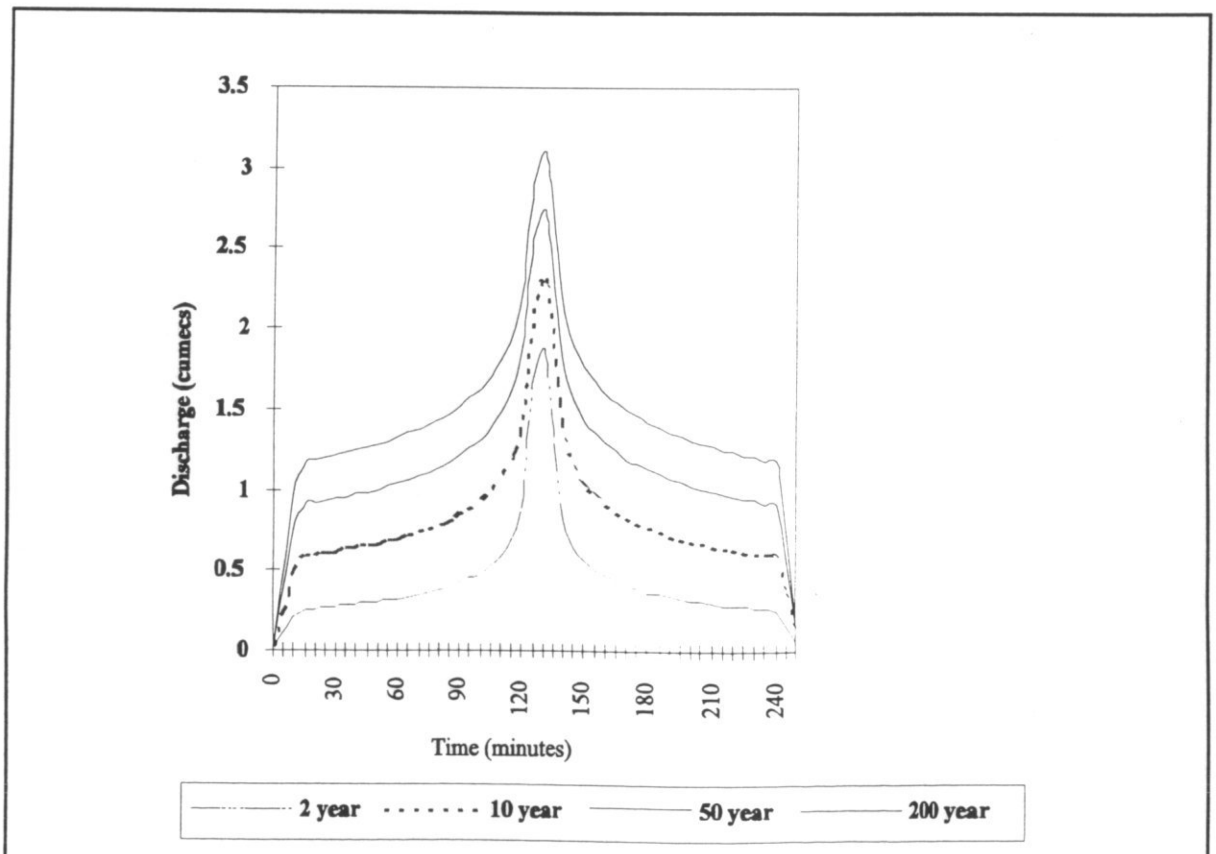


Figure 30 Design hydrographs - Sub-catchment 2.

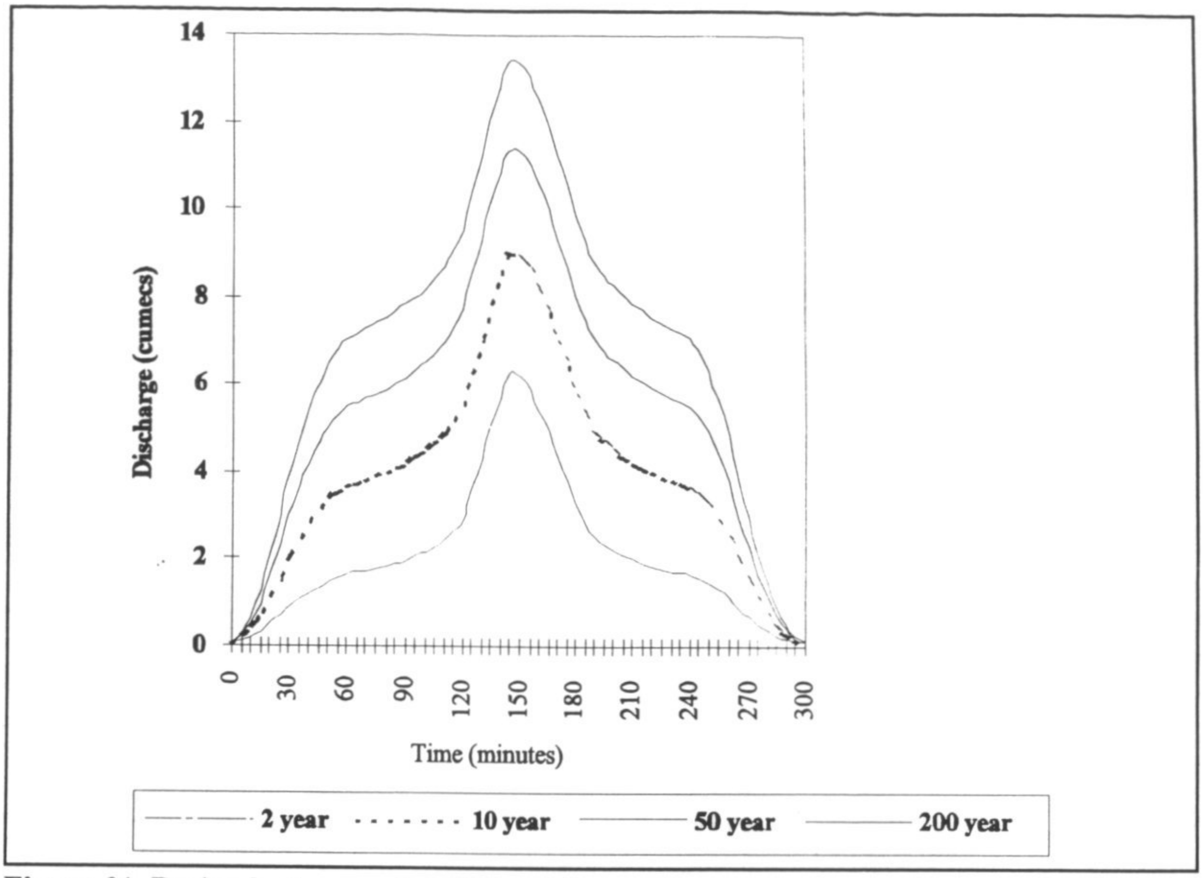


Figure 31 Design hydrographs - Sub-catchment 3.

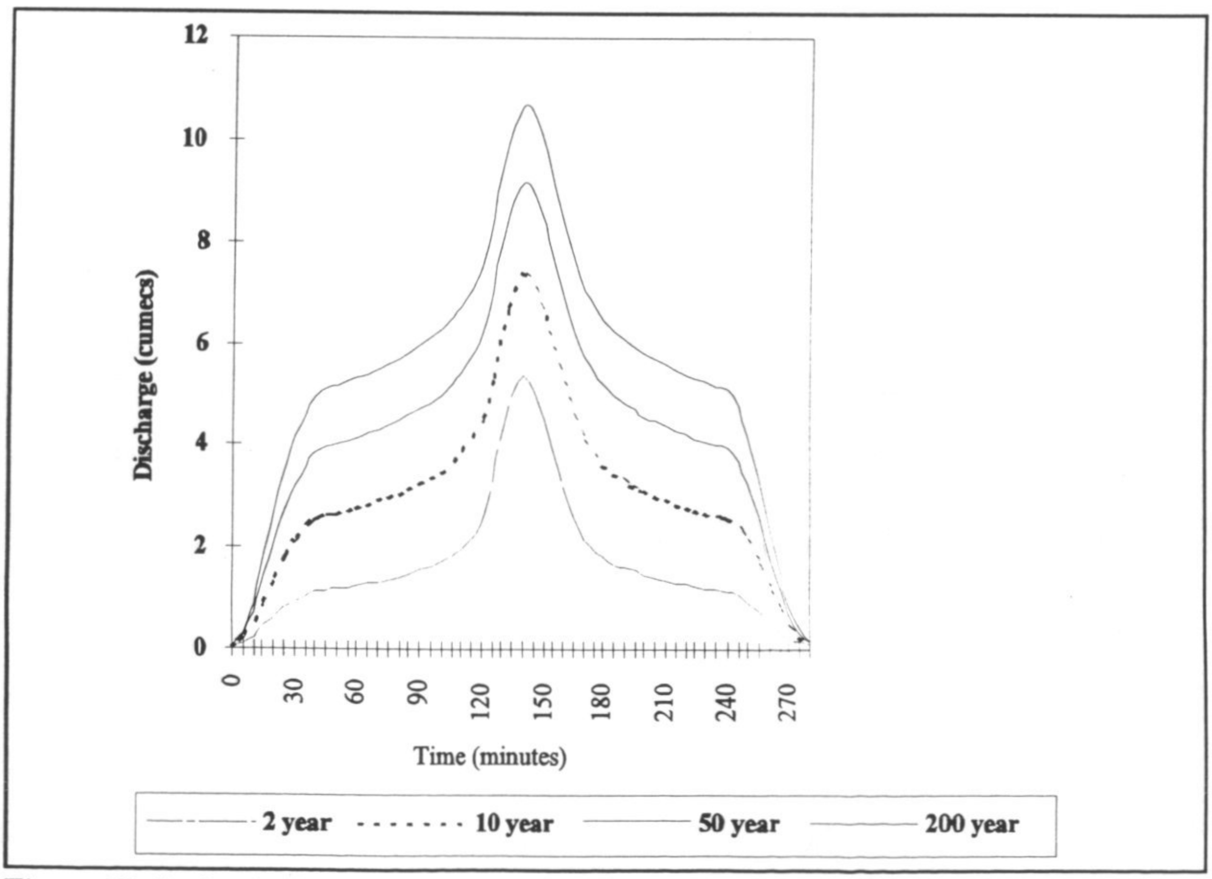


Figure 32 Design hydrographs - Sub-catchment 4.

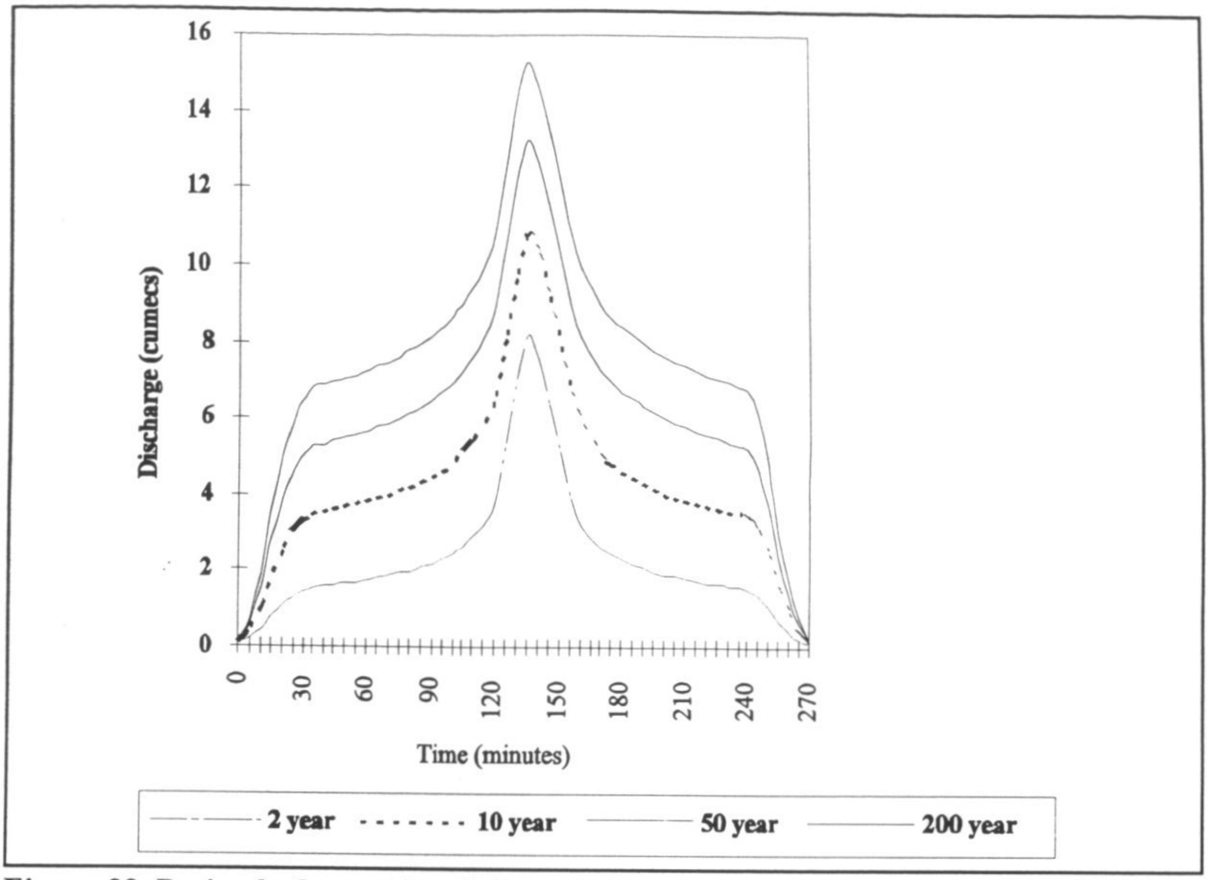


Figure 33 Design hydrographs - Sub-catchment 5.

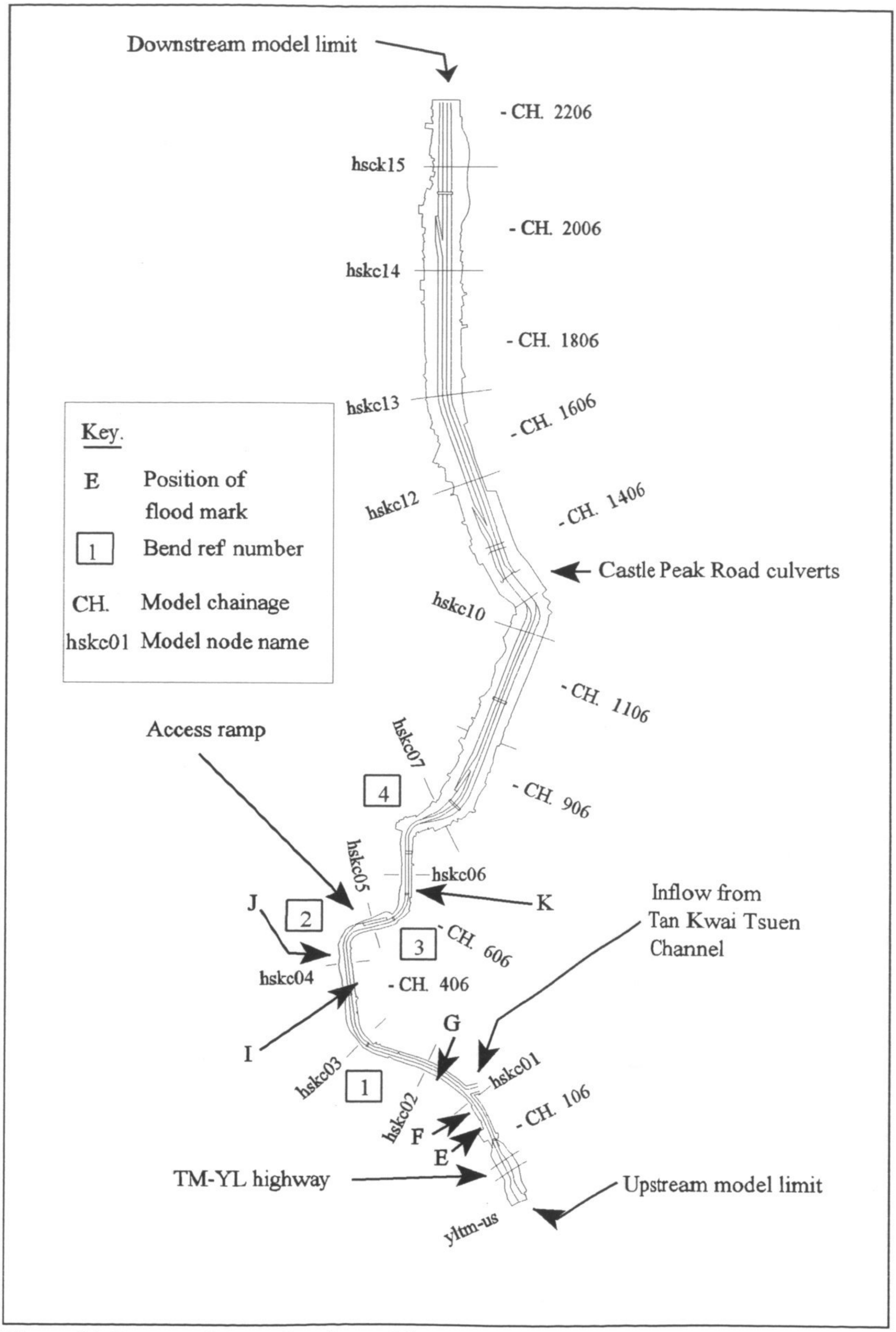


Figure 34 Layout of the hydraulic model.

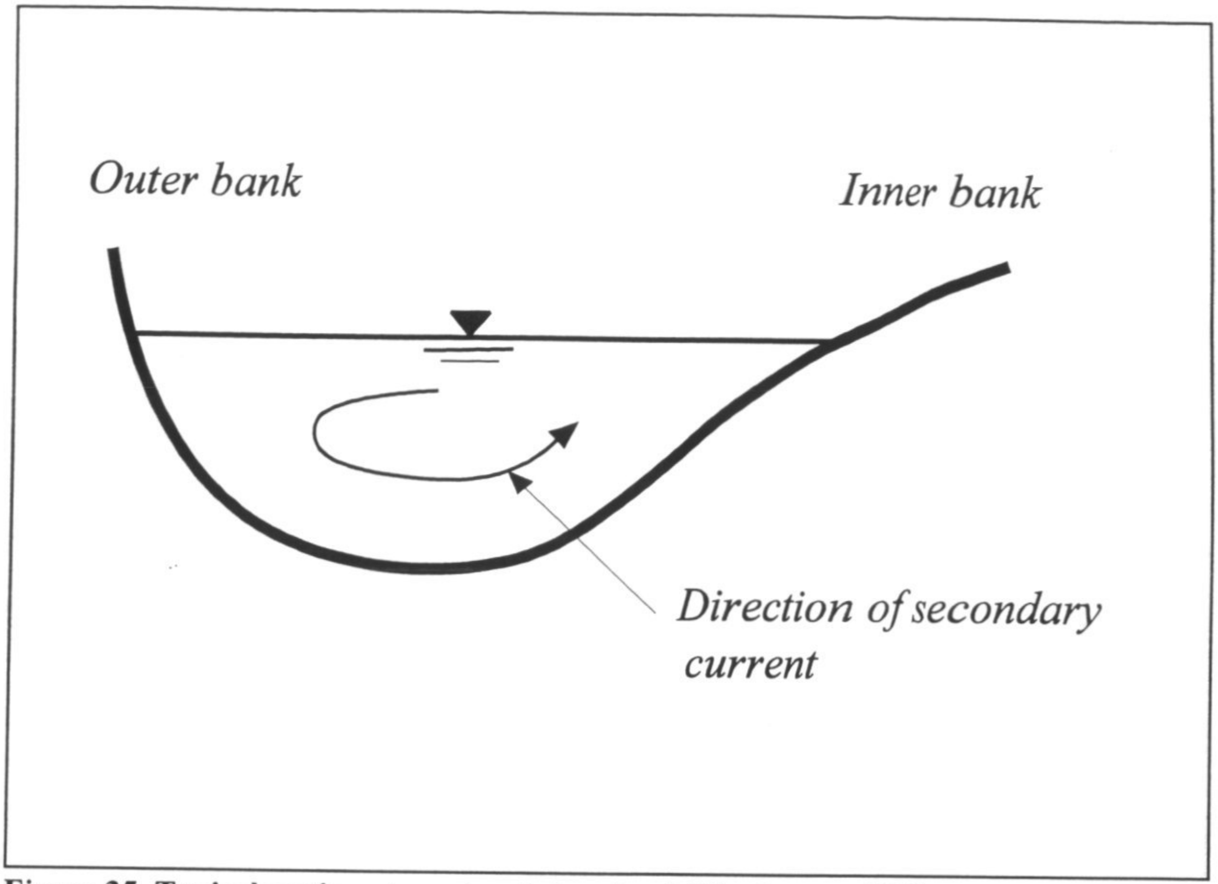


Figure 35 Typical section at a natural river bend (Henderson, 1966)

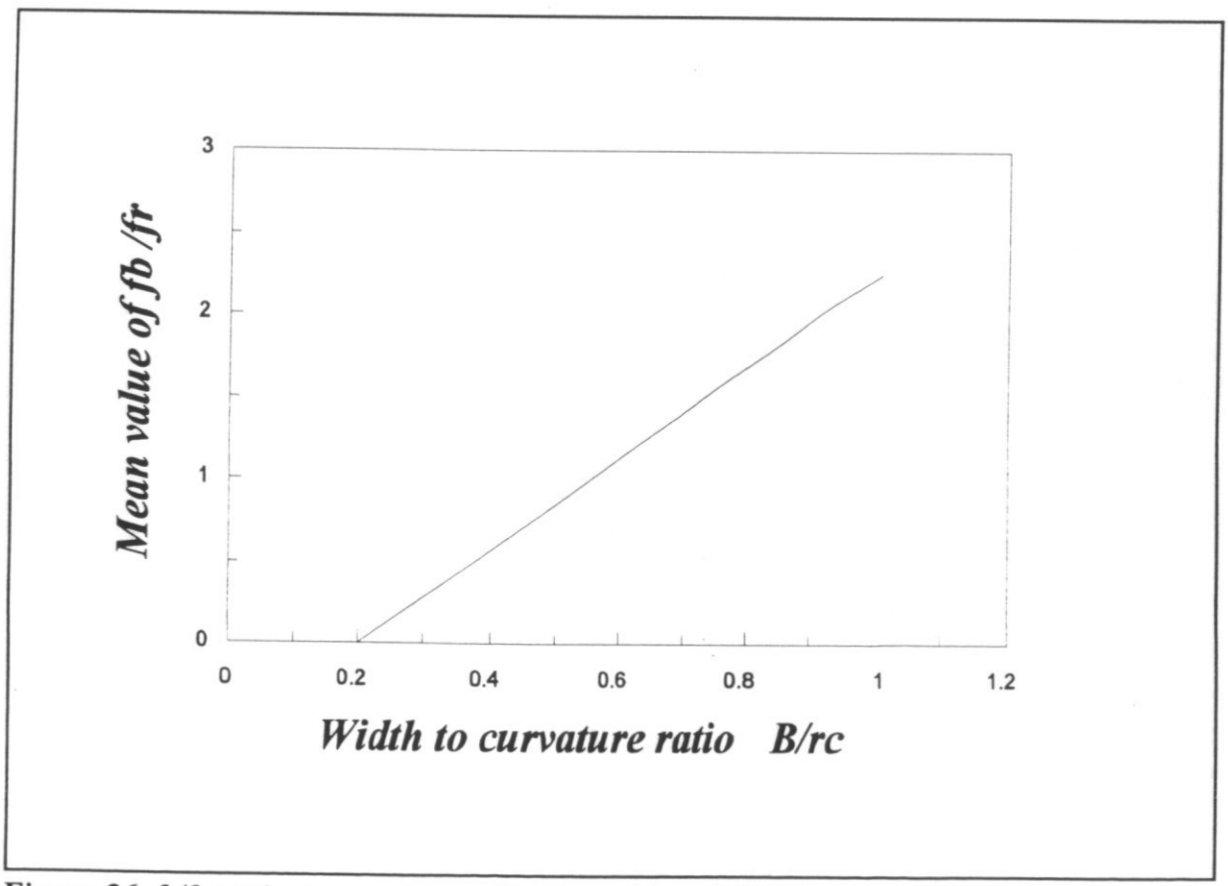
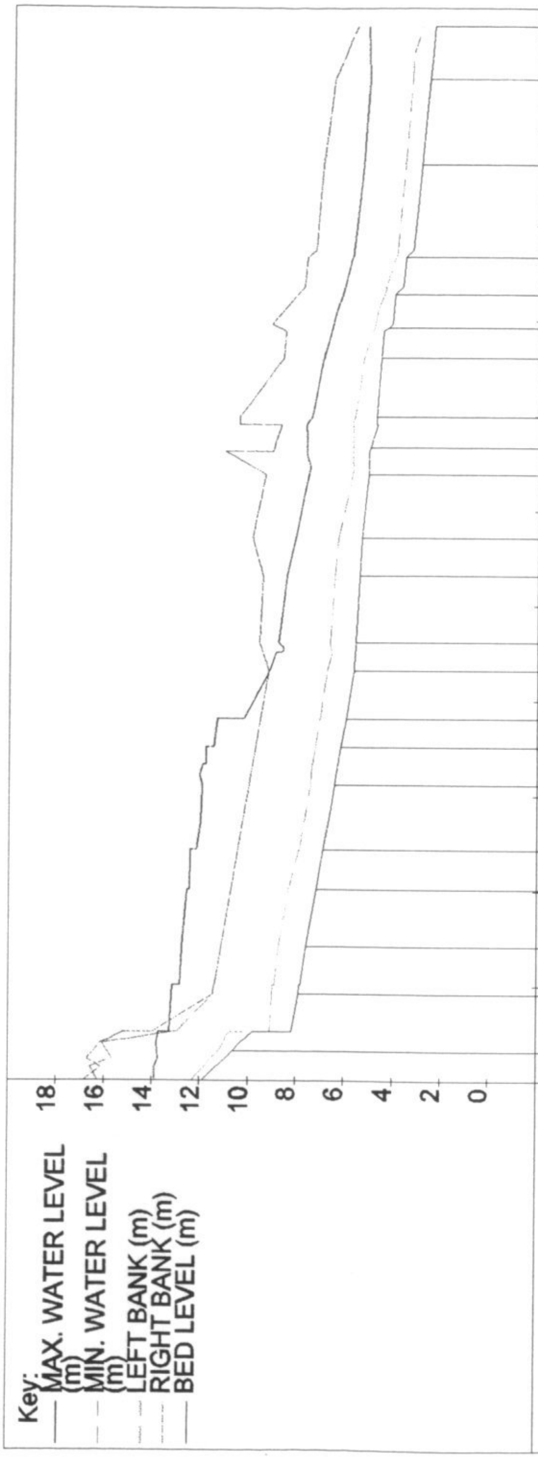
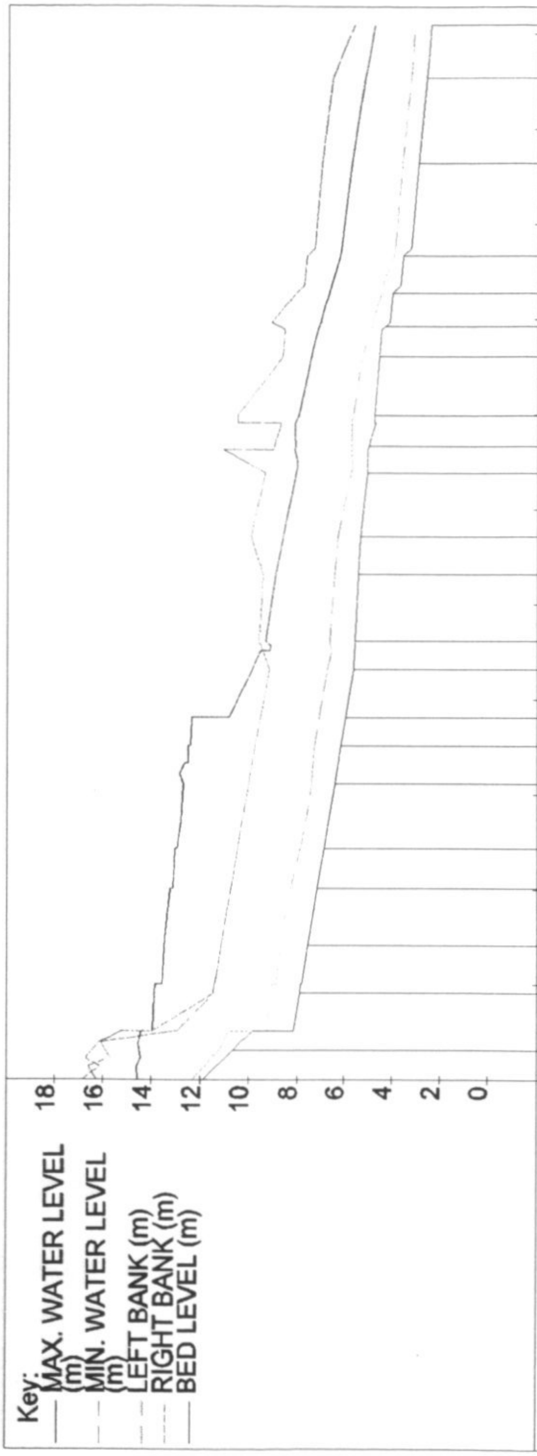


Figure 36 f_b/f_r against curvature ratio (Leopold *et al*, 1960)



MAX. WATER LEVEL	13.94	13.83	13.23	12.81	12.61	12.46	11.98	11.81	11.38	9.18	8.81	8.47	8.13	7.59	7.67	7.47	7.01	6.65	6.25	5.82	5.41	5.16	5.20
MIN. WATER LEVEL	12.37	11.24	8.98	8.71	8.32	7.91	7.43	7.25	7.05	6.75	6.64	6.46	6.33	5.76	5.73	5.68	5.31	4.93	4.52	4.05	3.67	3.37	3.00
LEFT BANK (m)	16.3	15.9	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yttm-us	yttm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-lii1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 37 Maximum and minimum water levels for the 22 July, 1994 event.

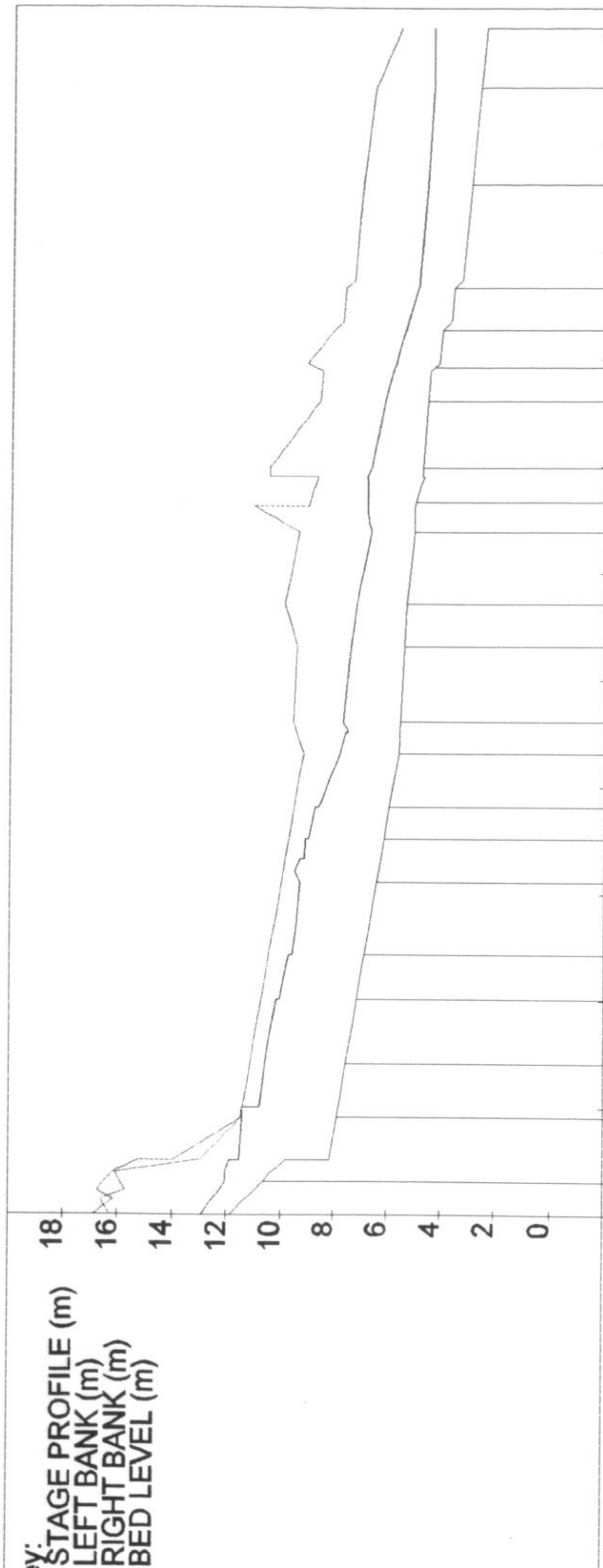


LABEL	yftm-us	yftm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12l	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	14.61	14.55	13.92	13.52	13.31	13.11	12.76	12.58	12.40	9.89	9.37	8.98	8.65	8.13	8.17	8.01	7.52	7.15	6.73	6.27	5.79	5.24	4.83
MIN. WATER LEVEL	12.38	11.24	9.14	8.70	8.32	7.90	7.43	7.32	7.11	6.74	6.64	6.46	6.33	5.76	5.73	5.67	5.31	4.92	4.52	4.04	3.67	3.32	3.18
LEFT BANK (m)	16.3	15.9	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.8	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.8	3.0	2.7	2.5

Figure 38 Maximum and minimum water levels for the 5 November, 1993 event

STAGE PROFILE (m)	13.6	13.4	12.6	12.0	11.6	11.2	10.6	10.4	9.8	8.6	8.4	8.1	7.8	7.3	7.4	6.7	6.4	6.0	5.6	5.2	5.0	5.0
LEFT BANK (m)	16.3	15.9	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yttm-us	yttm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li2	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.6	4.3	4.1	3.6	3.0	2.7	2.5

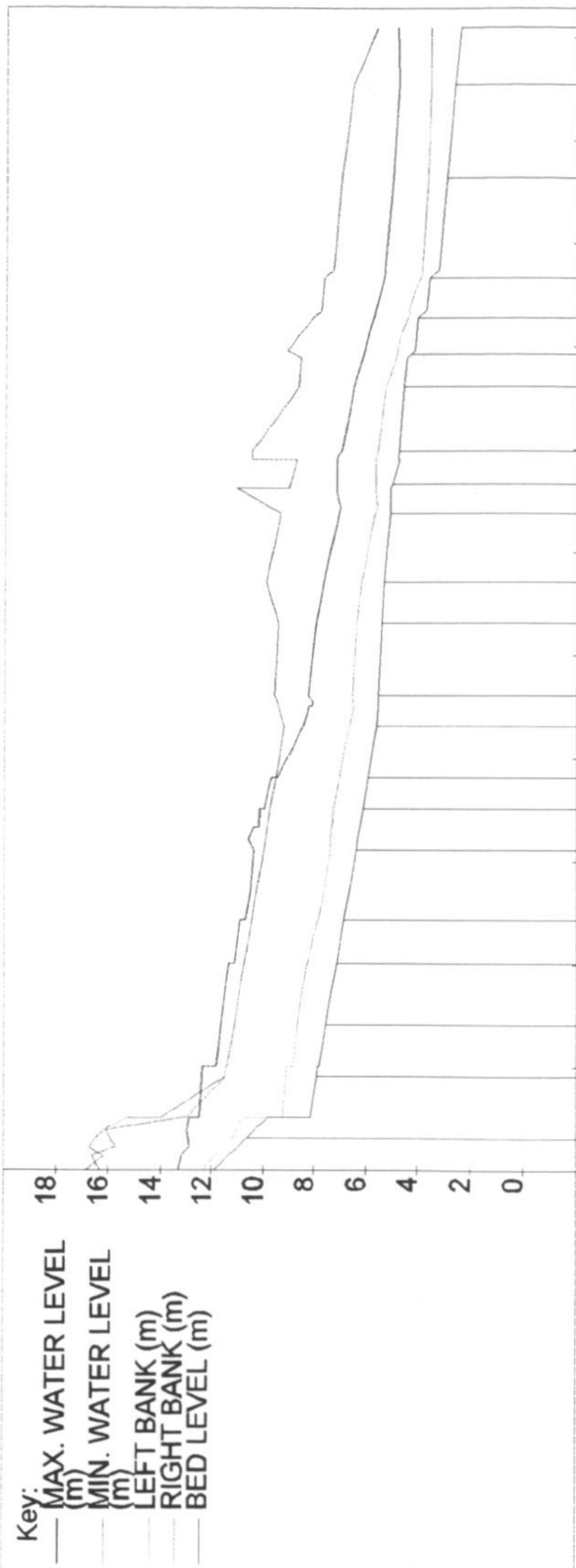
Figure 39 Simulation of the original design flow using the calibrated hydraulic model.



LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08
STAGE PROFILE (m)	13.0	12.1	11.5	10.6	10.2	9.7	9.3	9.1	8.8	7.9	7.7	7.4
LEFT BANK (m)	16.3	15.9	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5
RIGHT BANK (m)	16.9	16.5	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4

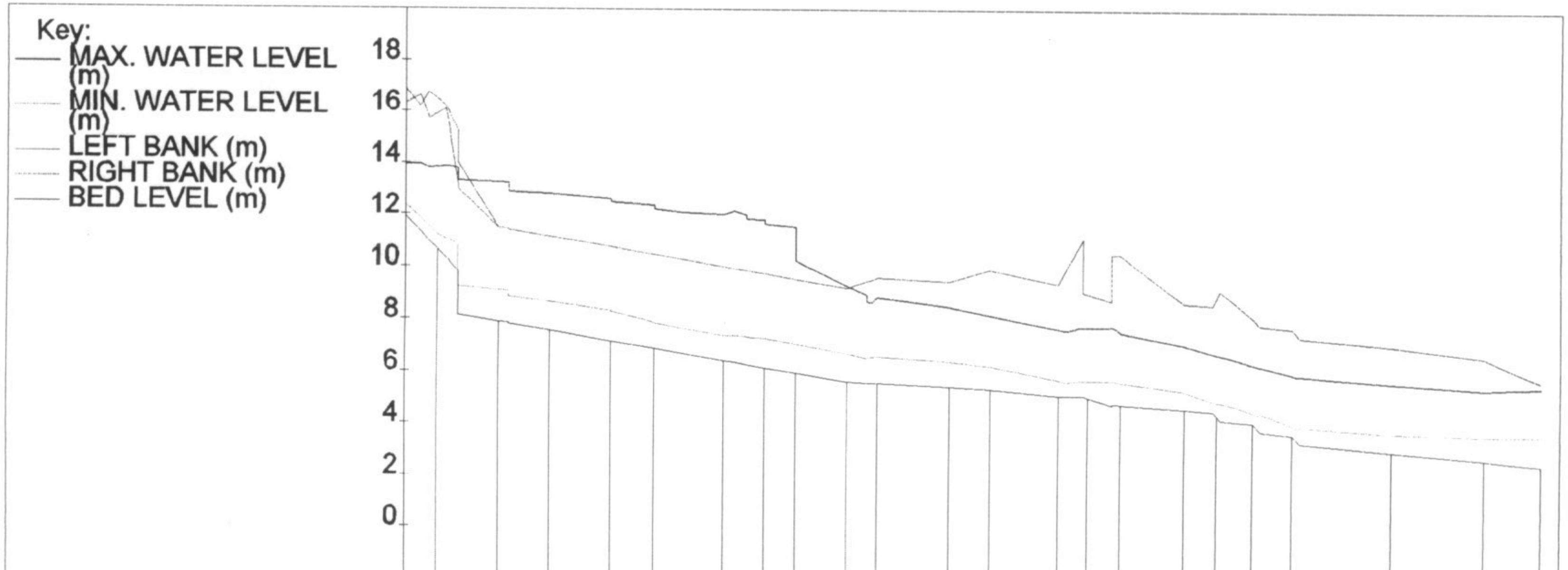
LABEL	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
STAGE PROFILE (m)	7.2	6.7	6.9	6.7	6.2	5.9	5.5	5.0	4.7	4.4	4.4
LEFT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 40 Water level profile for 41.25 cumec flow using calibrated hydraulic model.



LABEL	MAX. WATER LEVEL	MIN. WATER LEVEL	LEFT BANK (m)	RIGHT BANK (m)	BED LEVEL (m)
yftm-us	13.36	12.37	16.3	16.9	11.9
yftm-02i	12.97	11.21	15.9	16.5	10.6
hskc01	12.40	9.10	11.5	11.5	7.9
hskc02	11.70	8.65	11.2	11.2	7.6
hskc03	11.35	8.32	10.8	10.8	7.2
hskc03b1	10.92	7.89	10.5	10.5	6.9
hskc04d	10.40	7.39	10.0	10.0	6.4
hskc06b	10.20	7.29	9.8	9.8	6.2
hskc06	9.71	7.09	9.6	9.6	6.0
hskc06d	8.48	6.71	9.2	9.2	5.6
hskc08u	8.29	6.60	9.6	9.6	5.6
hskc08	7.98	6.40	9.5	9.5	5.4
hskc09	7.68	6.23	9.9	9.9	5.3
hskc10	7.12	5.71	9.4	9.4	5.1
cprc-lit	7.22	5.68	9.0	9.0	5.0
hskc11	7.02	5.61	10.5	10.5	4.8
hskc12u	6.56	5.29	8.7	8.7	4.6
hskc12i	6.20	4.85	8.9	8.9	4.3
hskc12d	5.81	4.48	8.1	8.1	4.1
hskc13	5.39	4.00	7.7	7.7	3.6
hskc14	5.05	3.69	7.1	7.1	3.0
hskc15	4.81	3.60	6.6	6.6	2.7
hskc15d	4.85	3.59	5.7	5.7	2.5

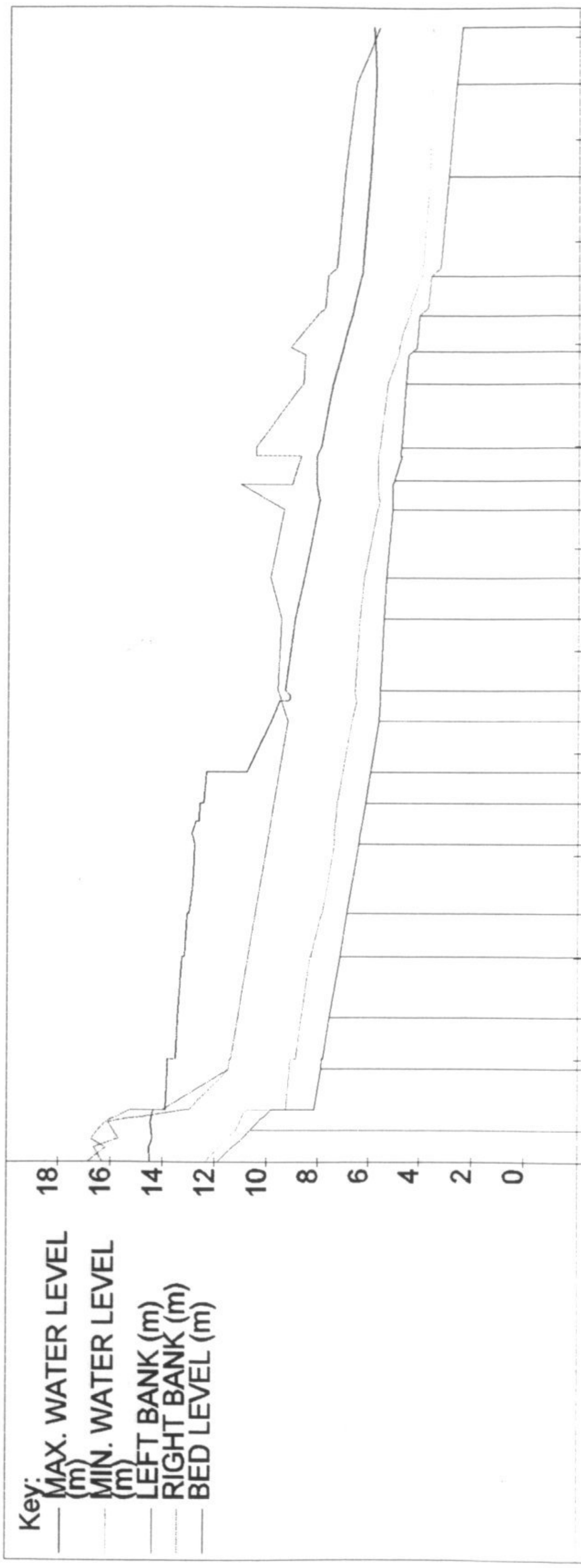
Figure 41 Maximum and minimum water levels for the 1 in 2 year rainfall event.



	yftm-us	yftm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08
MAX. WATER LEVEL	13.98	13.87	13.26	12.84	12.62	12.38	12.03	11.83	11.55	9.32	8.88	8.53
MIN. WATER LEVEL	12.37	11.21	9.10	8.65	8.33	7.89	7.39	7.29	7.09	6.71	6.60	6.40
LEFT BANK (m)	16.3	15.9	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5
RIGHT BANK (m)	16.9	16.5	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4

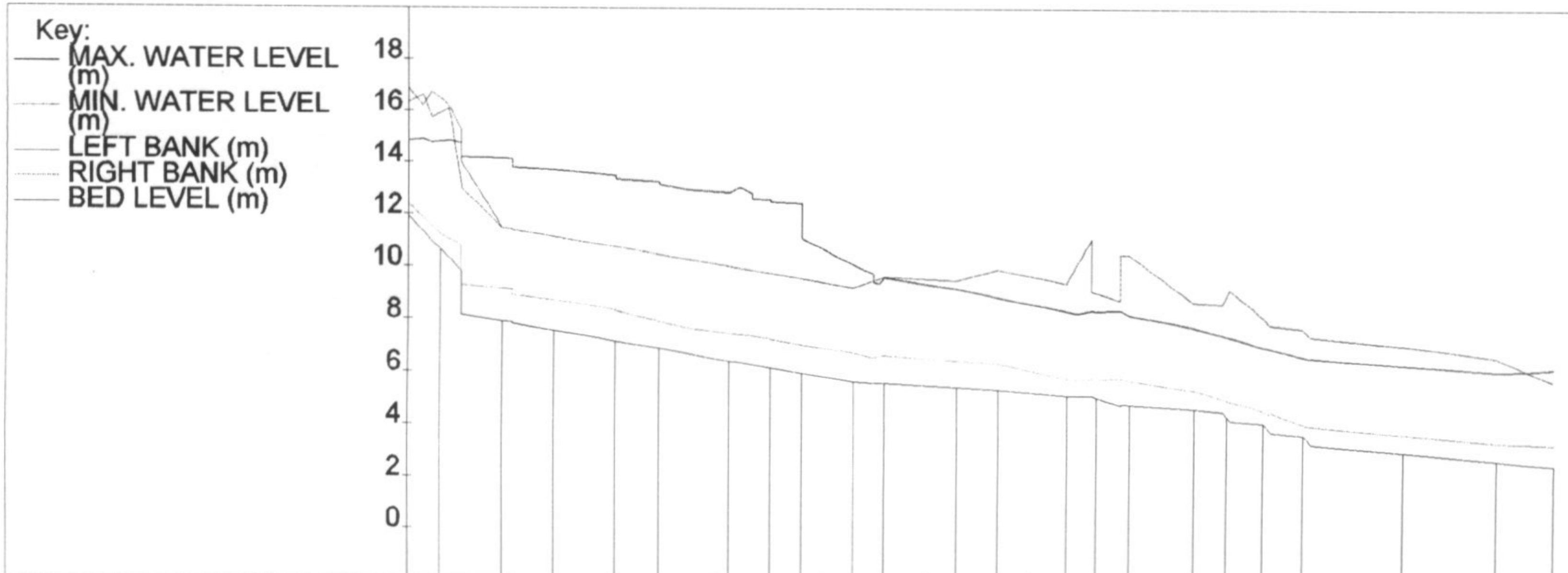
	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	8.20	7.66	7.74	7.54	7.07	6.72	6.34	5.94	5.62	5.37	5.45
MIN. WATER LEVEL	6.23	5.71	5.68	5.61	5.29	4.85	4.48	4.00	3.69	3.60	3.59
LEFT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 42 Maximum and minimum water levels for the 1 in 10 year rainfall event.



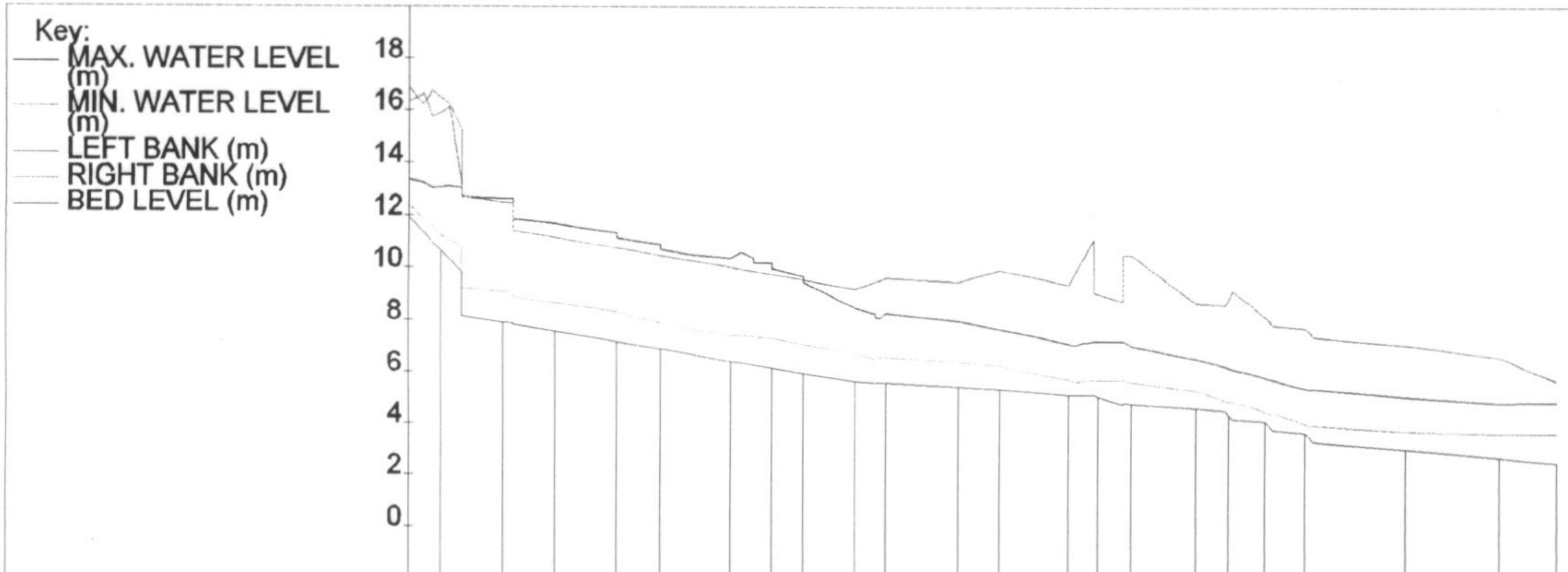
MAX. WATER LEVEL	MIN. WATER LEVEL	LEFT BANK (m)	RIGHT BANK (m)	LABEL	BED LEVEL (m)
14.53	12.37	16.3	16.9	yltm-us	11.9
14.46	11.21	15.9	16.5	yltm-02i	10.6
13.85	9.10	11.5	11.5	hskc01	7.9
13.46	8.65	11.2	11.2	hskc02	7.6
13.28	8.33	10.8	10.8	hskc03	7.2
13.11	7.89	10.5	10.5	hskc03b1	6.9
12.80	7.39	10.0	10.0	hskc04d	6.4
12.61	7.29	9.8	9.8	hskc06b	6.2
12.38	7.09	9.6	9.6	hskc06	6.0
9.87	6.71	9.2	9.2	hskc06d	5.6
9.34	6.60	9.6	9.6	hskc08u	5.6
8.95	6.40	9.5	9.5	hskc08	5.4
8.62	6.23	9.9	9.9	hskc09	5.3
8.10	5.71	9.4	9.4	hskc10	5.1
8.14	5.68	9.0	9.0	cprc-li1	5.0
7.98	5.61	10.5	10.5	hskc11	4.8
7.50	5.30	8.7	8.7	hskc12u	4.6
7.15	4.85	8.9	8.9	hskc12i	4.3
6.76	4.48	8.1	8.1	hskc12d	4.1
6.37	4.00	7.7	7.7	hskc13	3.6
6.05	3.70	7.1	7.1	hskc14	3.0
5.81	3.60	6.6	6.6	hskc15	2.7
5.90	3.59	5.7	5.7	hskc15d	2.5

Figure 43 Maximum and minimum water levels for the 1 in 50 year rainfall event.



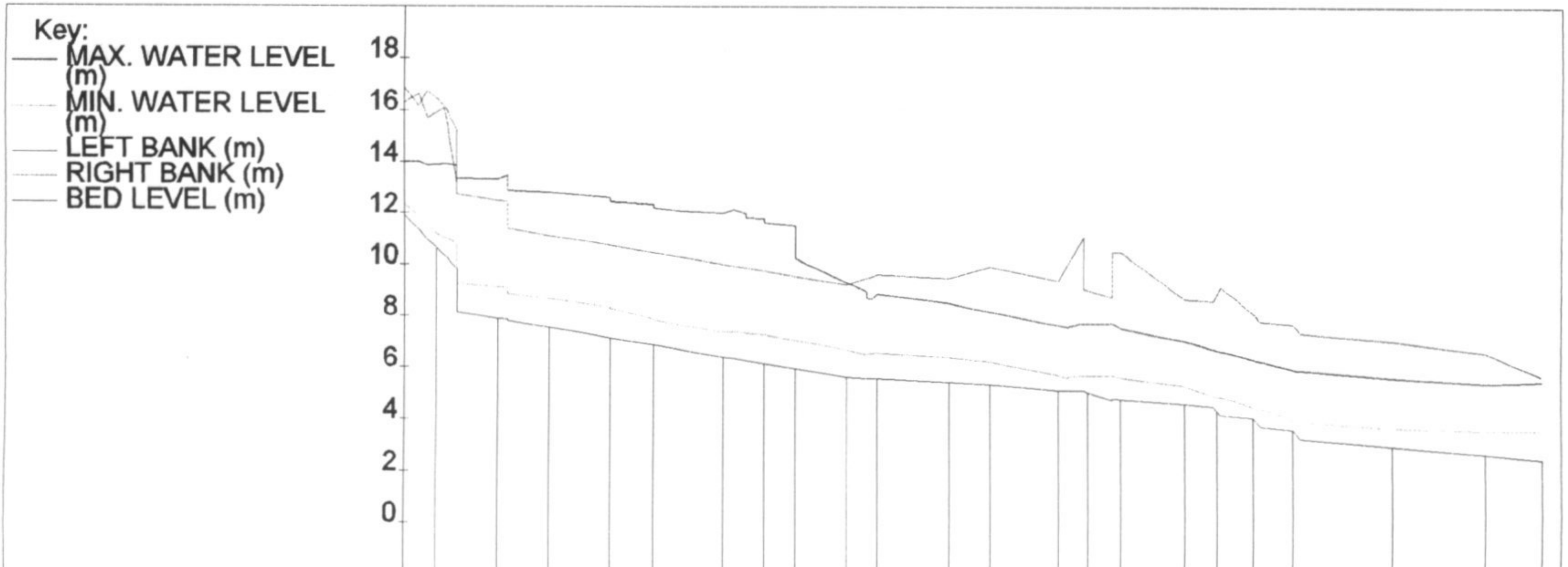
MAX. WATER LEVEL	14.89	14.84	14.17	13.73	13.50	13.28	12.85	12.61	12.43	10.10	9.59	9.19	8.86	8.34	8.34	8.21	7.72	7.39	7.01	6.64	6.31	6.06	6.15
MIN. WATER LEVEL	12.37	11.23	9.14	8.71	8.33	7.91	7.44	7.25	7.05	6.75	6.64	6.46	6.33	5.77	5.73	5.68	5.32	4.94	4.53	4.05	3.67	3.37	3.29
LEFT BANK (m)	16.3	15.9	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	11.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yftm-us	yftm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 44 Maximum and minimum water levels for the 1 in 200 year rainfall event.



LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	13.38	13.11	12.64	11.68	11.33	10.90	10.39	10.19	9.69	8.47	8.29	7.97	7.67	7.11	7.21	7.02	6.55	6.19	5.80	5.39	5.04	4.80	4.84
MIN. WATER LEVEL	12.37	11.21	9.10	8.65	8.32	7.89	7.39	7.29	7.09	6.71	6.60	6.40	6.23	5.71	5.68	5.61	5.28	4.85	4.48	4.00	3.69	3.60	3.59
LEFT BANK (m)	16.3	15.9	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 45 Max & min water levels 1 in 2 year rainfall event - Existing remedial works.

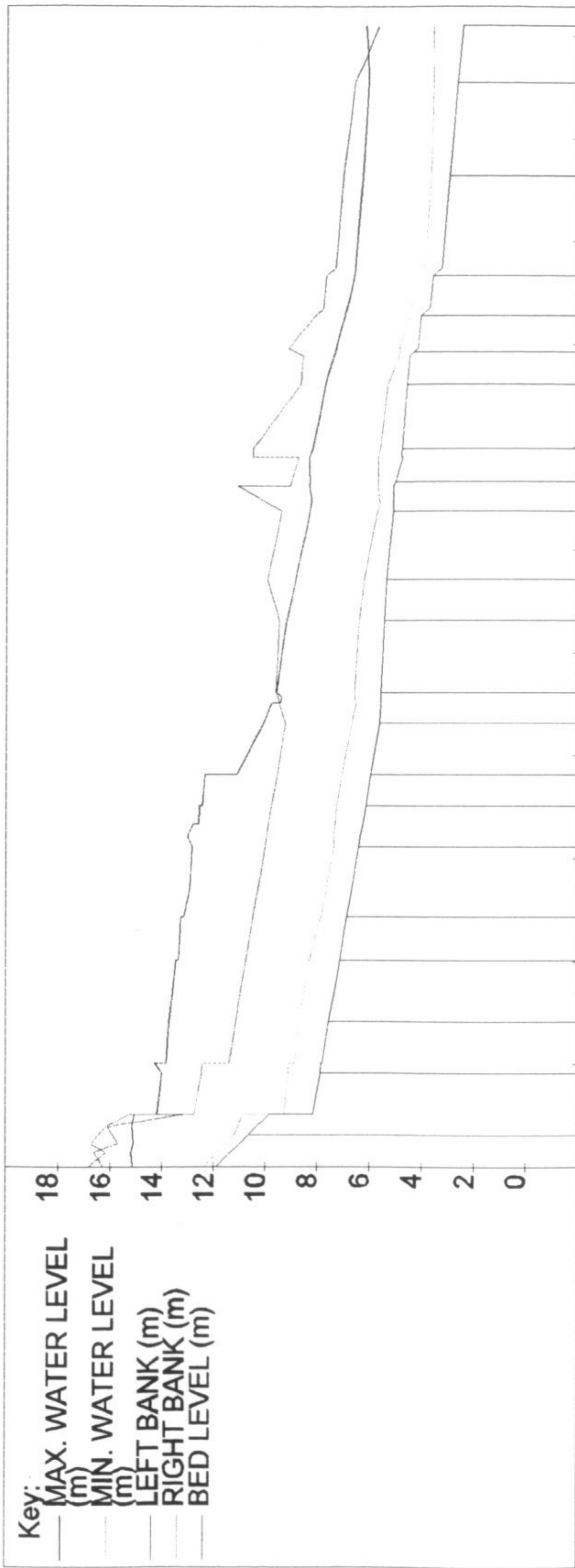


MAX. WATER LEVEL	14.02	13.92	13.33	12.83	12.62	12.39	12.03	11.83	11.55	9.31	8.88	8.53	8.20	7.66	7.73	7.54	7.07	6.72	6.33	5.94	5.61	5.37	5.45
MIN. WATER LEVEL	12.37	11.21	9.10	8.65	8.33	7.89	7.39	7.29	7.09	6.71	6.60	6.40	6.23	5.71	5.68	5.61	5.29	4.85	4.48	4.00	3.69	3.60	3.59
LEFT BANK (m)	16.3	15.9	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yftm-us	yftm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 46 Max & min water levels for 1 in 10 year rainfall event - Existing remedial works.

LABEL	yftm-us	yftm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	15.08	15.05	13.81	13.47	13.26	13.08	12.81	12.66	12.46	9.89	9.35	8.95	8.63	8.10	8.14	7.97	7.49	7.14	6.76	6.38	6.06	5.81	5.90
MIN. WATER LEVEL	12.37	11.21	9.10	8.65	8.33	7.89	7.39	7.29	7.09	6.71	6.60	6.40	6.23	5.71	5.68	5.61	5.30	4.85	4.48	4.00	3.70	3.60	3.59
LEFT BANK (m)	16.3	15.9	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

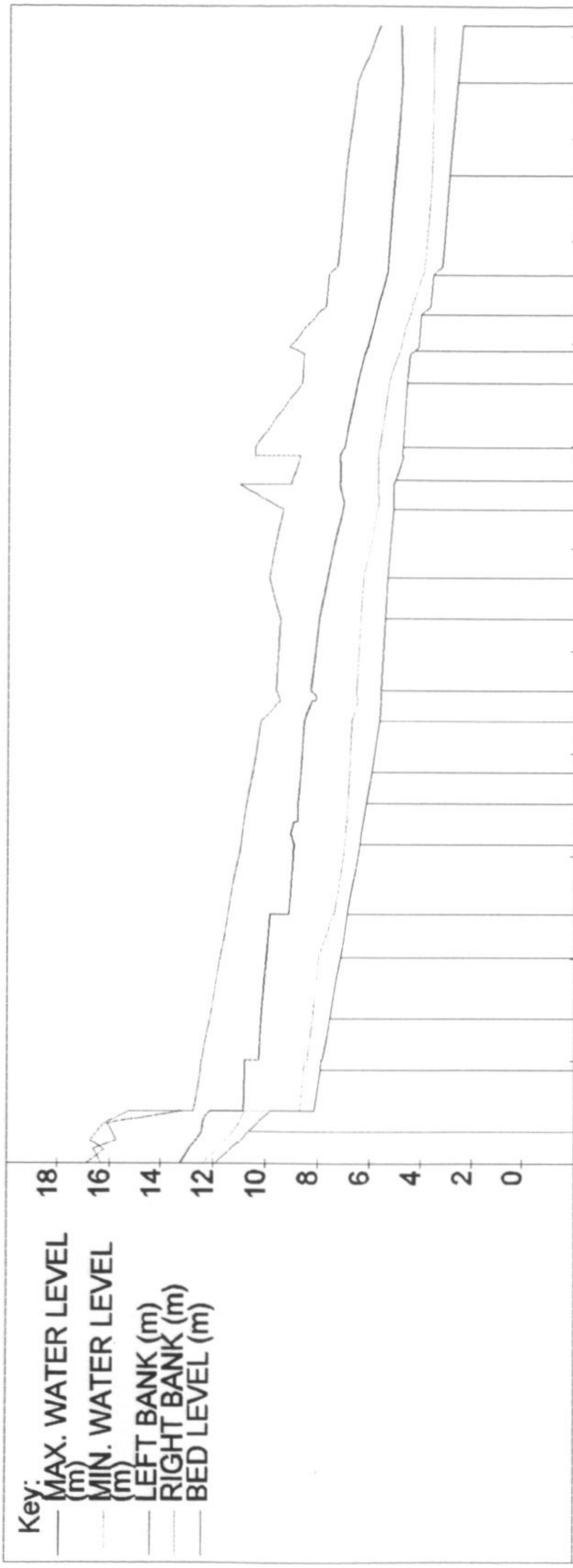
Figure 47 Max & min water levels, 1 in 50 year rainfall event - Existing remedial works.



LABEL	yttm-us	yttm-02i	hskc01	hskc02	hskc03	hskc03d1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08
MAX. WATER LEVEL	15.18	15.15	14.03	13.73	13.50	13.28	12.85	12.58	12.40	10.10	9.59	9.19
MIN. WATER LEVEL	12.37	11.21	9.10	8.65	8.33	7.89	7.39	7.29	7.09	6.71	6.60	6.40
LEFT BANK (m)	16.3	15.9	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5
RIGHT BANK (m)	16.9	16.5	12.5	11.2	10.8	10.5	10.0	9.8	9.6	9.2	9.6	9.5
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4

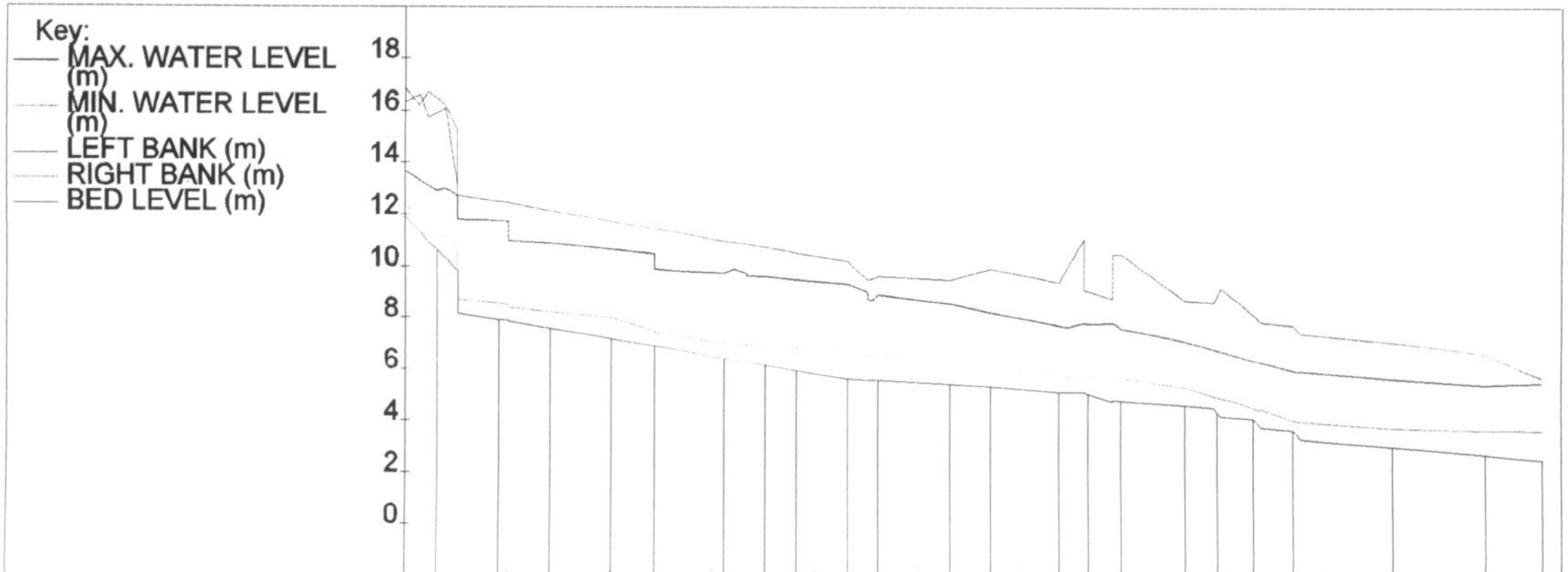
LABEL	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	8.86	8.34	8.35	8.21	7.73	7.39	7.01	6.64	6.31	6.06	6.16
MIN. WATER LEVEL	6.23	5.71	5.68	5.61	5.31	4.85	4.48	4.00	3.70	3.60	3.59
LEFT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 48 Max & min water levels, 1 in 200 year rainfall event - Existing remedial works.



LABEL	MAX. WATER LEVEL	MIN. WATER LEVEL	LEFT BANK (m)	RIGHT BANK (m)	BED LEVEL (m)
yltm-us	13.27	12.37	16.3	16.9	11.9
yltm-02i	12.42	11.23	15.9	16.5	10.6
hskc01	10.83	8.53	12.5	12.5	7.9
hskc02	10.19	8.22	12.2	12.2	7.6
hskc03	10.00	7.97	11.8	11.8	7.2
hskc03b1	9.88	7.46	11.5	11.5	6.9
hskc04d	8.93	7.00	11.0	11.0	6.4
hskc06b	8.80	6.89	10.8	10.8	6.2
hskc06	8.69	6.80	10.6	10.6	6.0
hskc06d	8.54	6.73	10.2	10.2	5.9
hskc08u	8.33	6.58	9.6	9.6	5.6
hskc08	8.00	6.40	9.5	9.5	5.4
hskc09	7.70	6.28	9.9	9.9	5.3
hskc10	7.15	5.74	9.4	9.4	5.1
cpro-lit	7.24	5.72	9.0	9.0	5.0
hskc11	7.06	5.66	10.5	10.5	4.8
hskc12u	6.59	5.30	8.7	8.7	4.6
hskc12i	6.23	4.90	8.9	8.9	4.3
hskc12d	5.84	4.50	8.1	8.1	4.1
hskc13	5.42	4.02	7.7	7.7	3.6
hskc14	5.08	3.70	7.1	7.1	3.0
hskc15	4.84	3.61	6.6	6.6	2.7
hskc15d	4.88	3.60	5.7	5.7	2.5

Figure 49 Max & min water levels, 1 in 2 year rainfall event - Option 1A.



MAX. WATER LEVEL	13.68	12.92	11.76	10.93	10.69	10.52	9.75	9.62	9.50	9.33	8.90	8.55	8.22	7.68	7.76	7.57	7.10	6.75	6.37	5.98	5.65	5.41	5.49
MIN. WATER LEVEL	12.37	11.23	8.54	8.22	7.97	7.46	7.00	6.89	6.81	6.73	6.58	6.40	6.28	5.74	5.72	5.67	5.30	4.91	4.51	4.02	3.71	3.62	3.60
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 50 Max & min water levels, 1 in 10 year rainfall event - Option 1A.

MAX. WATER LEVEL	14.00	13.39	12.24	11.20	10.81	10.57	10.22	10.20	10.06	9.86	9.31	8.92
MIN. WATER LEVEL	12.37	11.23	8.54	8.22	7.97	7.46	7.00	6.89	6.81	6.74	6.58	6.40
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5
LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4

MAX. WATER LEVEL	8.59	8.06	8.12	7.95	7.47	7.13	6.75	6.36	6.03	5.79	5.88
MIN. WATER LEVEL	6.28	5.74	5.72	5.67	5.30	4.91	4.51	4.02	3.71	3.62	3.60
LEFT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 51 Max & min water levels, 1 in 50 year rainfall event - Option 1A.

LABEL	MAX. WATER LEVEL	MIN. WATER LEVEL	LEFT BANK (m)	RIGHT BANK (m)	BED LEVEL (m)
yttm-us	14.27	12.37	16.3	16.9	11.9
yttm-02i	13.79	11.23	15.9	16.5	10.6
hskc01	12.63	8.54	12.5	12.5	7.9
hskc02	11.59	8.22	12.2	12.2	7.6
hskc03	11.15	7.97	11.8	11.8	7.2
hskc03b1	10.78	7.46	11.5	11.5	6.9
hskc04d	10.53	7.00	11.0	11.0	6.4
hskc06b	10.50	6.89	10.8	10.8	6.2
hskc06	10.33	6.81	10.6	10.6	6.0
hskc06d	10.07	6.74	10.2	10.2	5.6
hskc08u	9.60	6.58	9.6	9.6	5.6
hskc08	9.19	6.40	9.5	9.5	5.4
hskc09	8.86	6.27	9.9	9.9	5.3
hskc10	8.34	5.74	9.4	9.4	5.1
cprc-li1	8.35	5.72	9.0	9.0	5.0
hskc11	8.22	5.67	10.5	10.5	4.8
hskc12u	7.73	5.30	8.7	8.7	4.6
hskc12i	7.40	4.91	8.9	8.9	4.3
hskc12d	7.02	4.51	8.1	8.1	4.1
hskc13	6.65	4.02	7.7	7.7	3.6
hskc14	6.32	3.71	7.1	7.1	3.0
hskc15	6.07	3.62	6.6	6.6	2.7
hskc15d	6.16	3.60	5.7	5.7	2.5

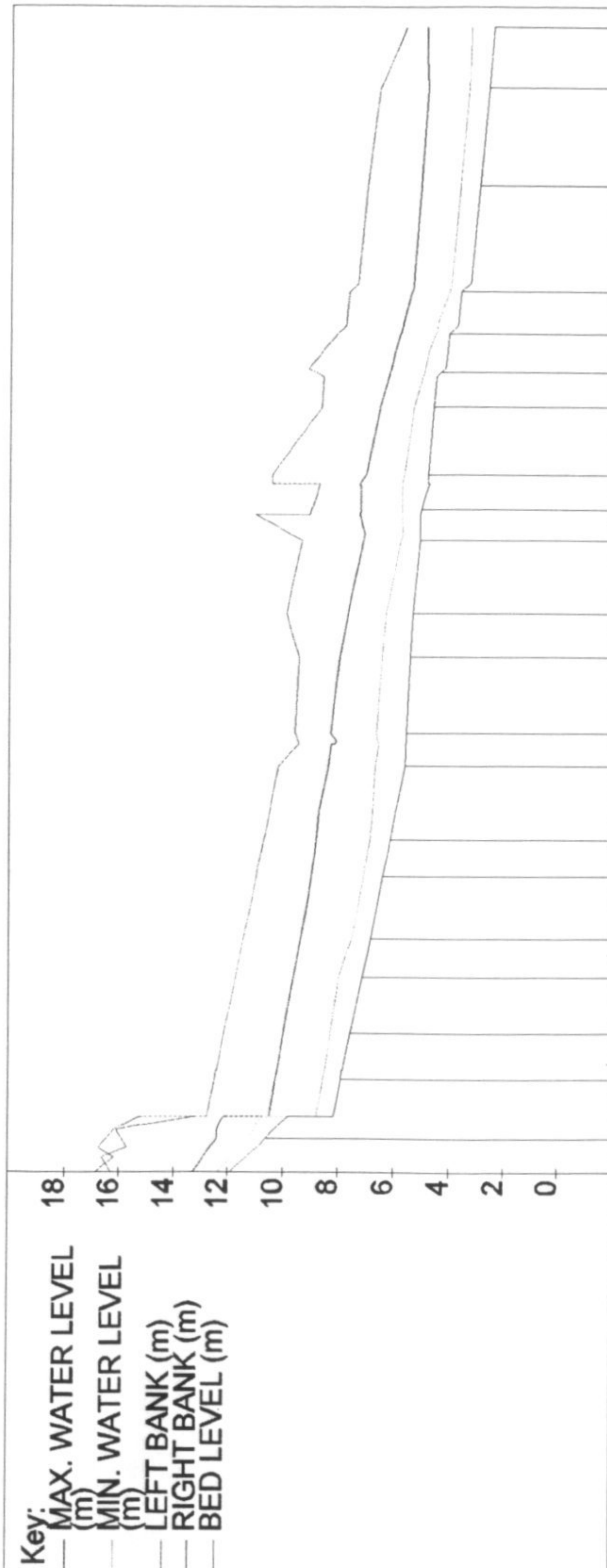
Figure 52 Max & min water levels, 1 in 200 year rainfall event - Option 1A.

LABEL	yftm-us	yftm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	14.04	13.56	12.62	11.40	10.95	10.56	10.29	10.25	10.07	9.79	9.31	8.92	8.59	8.06	8.11	7.95	7.47	7.12	6.74	6.36	6.03	5.79	5.87
MIN. WATER LEVEL	12.37	11.23	8.59	8.27	8.02	7.50	7.05	6.98	6.88	6.78	6.63	6.44	6.31	5.76	5.71	5.67	5.30	4.91	4.51	4.02	3.71	3.62	3.60
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 53 Max & min water levels, 1 in 50 year rainfall event - Option 1B.

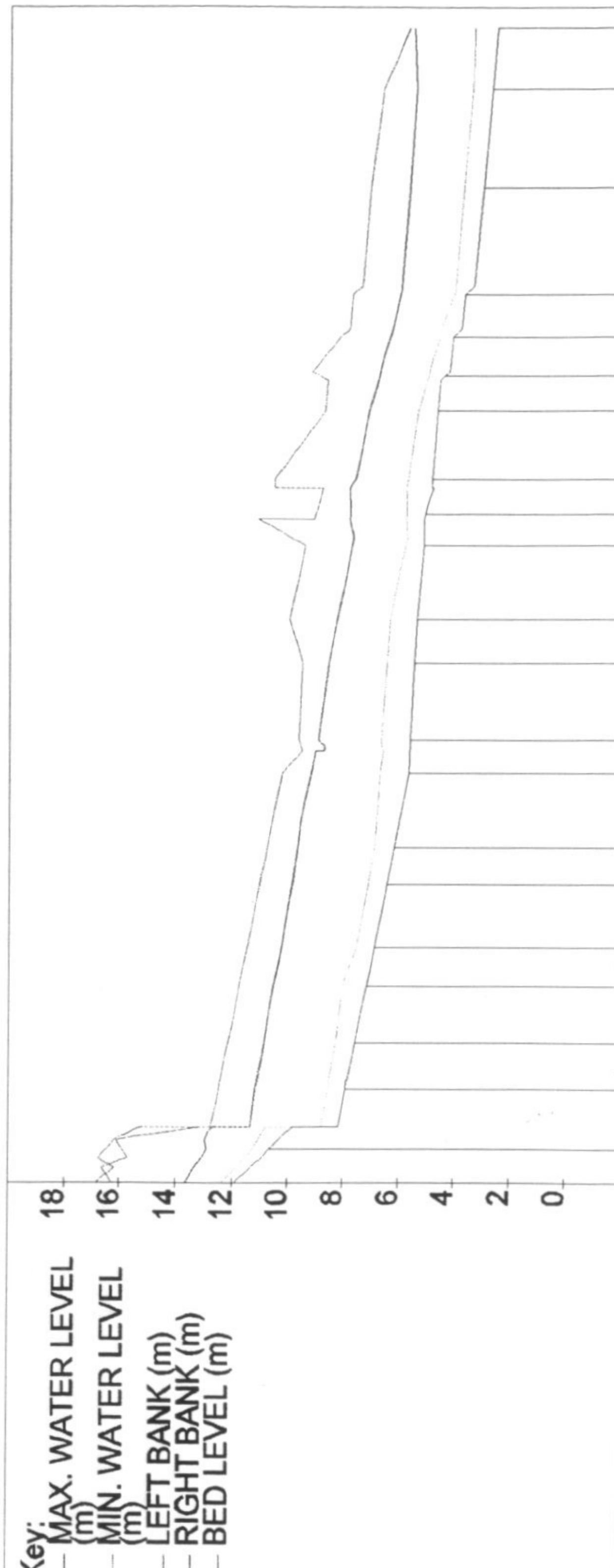
MAX. WATER LEVEL	14.18	14.10	13.61	13.09	12.39	11.83	11.20	11.05	10.44	9.30	8.90	8.55	8.21	7.68	7.77	7.56	7.09	6.74	6.36	5.97	5.64	5.40	5.48
MIN. WATER LEVEL	12.37	11.21	8.99	8.66	8.33	7.86	7.38	7.28	7.07	6.71	6.60	6.40	6.23	5.71	5.68	5.61	5.29	4.85	4.48	4.00	3.69	3.60	3.59
LEFT BANK (m)	16.3	15.9	14.5	14.2	13.8	13.5	13.0	12.8	12.6	12.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	14.5	14.2	13.8	13.5	13.0	12.8	12.6	12.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 54 Max & min water levels, 1 in 10 year rainfall event - Option 2.



MAX. WATER LEVEL	13.27	12.42	10.31	10.01	9.67	9.38	9.01	8.83	8.42	8.32	8.00	7.71	7.16	7.25	7.07	6.60	6.24	5.85	5.43	5.09	4.86	4.90
MIN. WATER LEVEL	12.37	11.23	8.55	8.26	8.01	7.52	7.12	6.92	6.71	6.66	6.47	6.32	5.79	5.75	5.69	5.33	4.95	4.54	4.06	3.67	3.37	3.29
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yltm-us	yltm-02l	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 55 Max & min water levels, 1 in 2 year rainfall event - Option 4.



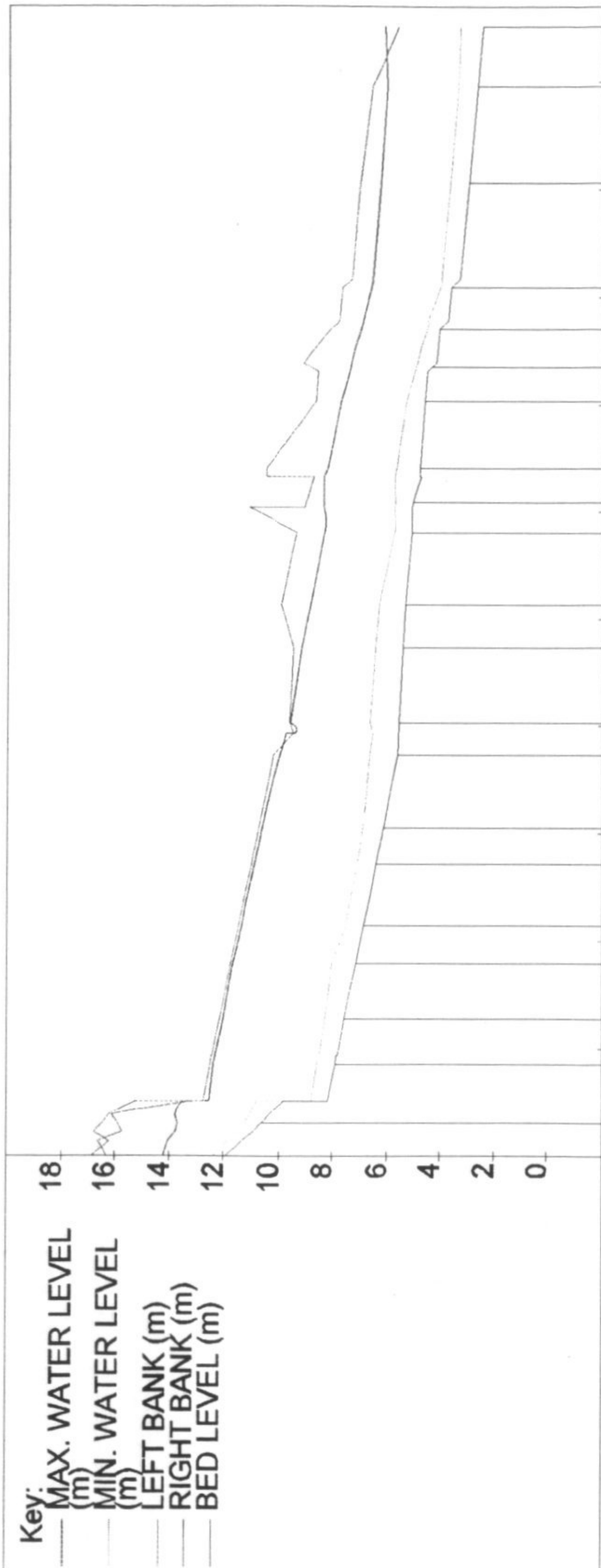
MAX. WATER LEVEL	13.68	12.91	11.17	10.86	10.52	10.24	9.86	9.66	9.18	8.90	8.55	8.22	7.69	7.77	7.57	7.11	6.75	6.37	5.98	5.65	5.41	5.49
MIN. WATER LEVEL	12.37	11.23	8.53	8.26	8.01	7.50	7.09	6.89	6.68	6.63	6.45	6.32	5.76	5.72	5.67	5.31	4.92	4.52	4.04	3.67	3.37	3.29
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	ytm-us	ytm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 56 Max & min water levels, 1 in 10 year rainfall event - Option 4.

MAX. WATER LEVEL	13.98	13.32	11.85	11.54	11.19	10.90	10.49	10.26	9.71	9.31	8.92
MIN. WATER LEVEL	12.37	11.23	8.54	8.26	8.02	7.51	7.10	6.90	6.68	6.64	6.45
LEFT BANK (m)	16.3	15.9	12.5	14.0	16.0	14.5	13.0	10.8	10.2	9.6	9.5
RIGHT BANK (m)	16.9	16.5	12.5	14.0	16.0	14.5	13.0	10.8	10.2	9.6	9.5
LABEL	yftm-us	yftm-02i	hskc01	hskc16	hskc17	hskc18	hskc19	hskc06b	hskc06d	hskc08u	hskc08
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	5.6	5.6	5.4

MAX. WATER LEVEL	8.59	8.06	8.12	7.95	7.48	7.13	6.75	6.36	6.04	5.79	5.88
MIN. WATER LEVEL	6.32	5.77	5.73	5.68	5.32	4.93	4.53	4.04	3.67	3.37	3.29
LEFT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	hskc09	hskc10	cprc-ll1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 57 Max & min water levels, 1 in 50 year rainfall event - Option 4.



LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-lil	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
MAX. WATER LEVEL	14.25	13.72	12.36	12.04	11.67	11.36	10.91	10.64	9.97	9.60	9.19	8.86	8.35	8.36	8.22	7.74	7.40	7.03	6.66	6.33	6.08	6.17
MIN. WATER LEVEL	12.37	11.23	8.54	8.26	8.03	7.51	7.10	6.90	6.69	6.64	6.45	6.32	5.77	5.73	5.68	5.32	4.94	4.53	4.05	3.67	3.37	3.29
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 58 Max & min water levels, 1 in 200 year rainfall event - Option 4.

MAX. WATER LEVEL	13.69	13.59	13.24	12.28	11.70	11.18	10.64	10.44	9.92	8.84	8.60	8.29	7.99	7.48	7.62	7.42	6.96	6.60	6.21	5.81	5.48	5.24	5.31
MIN. WATER LEVEL	12.34	11.17	9.00	8.66	8.32	7.85	7.37	7.27	7.07	6.73	6.62	6.44	6.30	5.75	5.72	5.67	5.30	4.91	4.51	4.02	3.71	3.62	3.60
LEFT BANK (m)	16.3	15.9	12.5	12.2	11.8	11.5	11.0	10.8	10.6	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
RIGHT BANK (m)	16.9	16.5	12.5	12.2	11.8	11.5	11.0	10.8	10.8	10.2	9.6	9.5	9.9	9.4	9.0	10.5	8.7	8.9	8.1	7.7	7.1	6.6	5.7
LABEL	yltm-us	yltm-02i	hskc01	hskc02	hskc03	hskc03b1	hskc04d	hskc06b	hskc06	hskc06d	hskc08u	hskc08	hskc09	hskc10	cprc-li1	hskc11	hskc12u	hskc12i	hskc12d	hskc13	hskc14	hskc15	hskc15d
BED LEVEL (m)	11.9	10.6	7.9	7.6	7.2	6.9	6.4	6.2	6.0	5.6	5.6	5.4	5.3	5.1	5.0	4.8	4.6	4.3	4.1	3.6	3.0	2.7	2.5

Figure 59 Max & min water levels, 1 in 10 year rainfall event - Option 5.

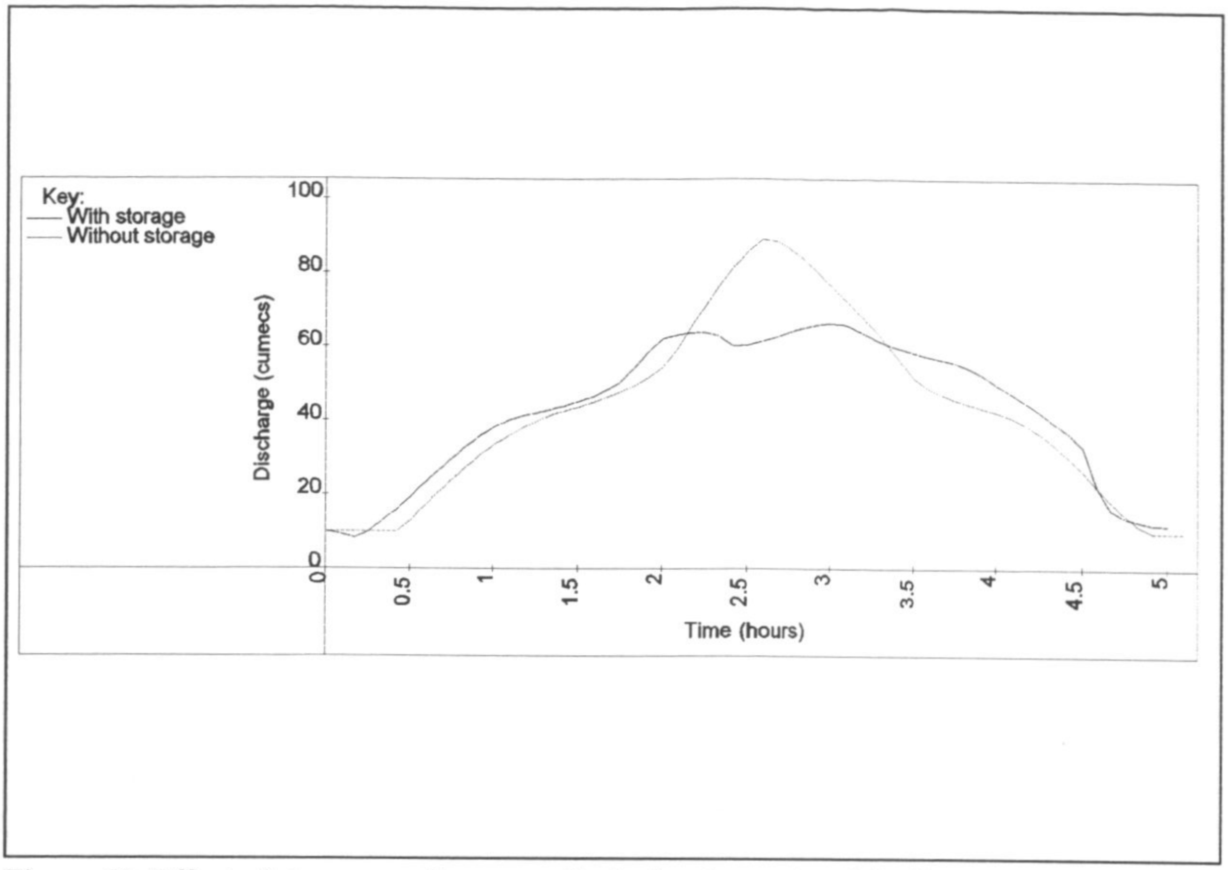


Figure 60 Effect of storage on flow over the irrigation weir - 1 in 10 year event.

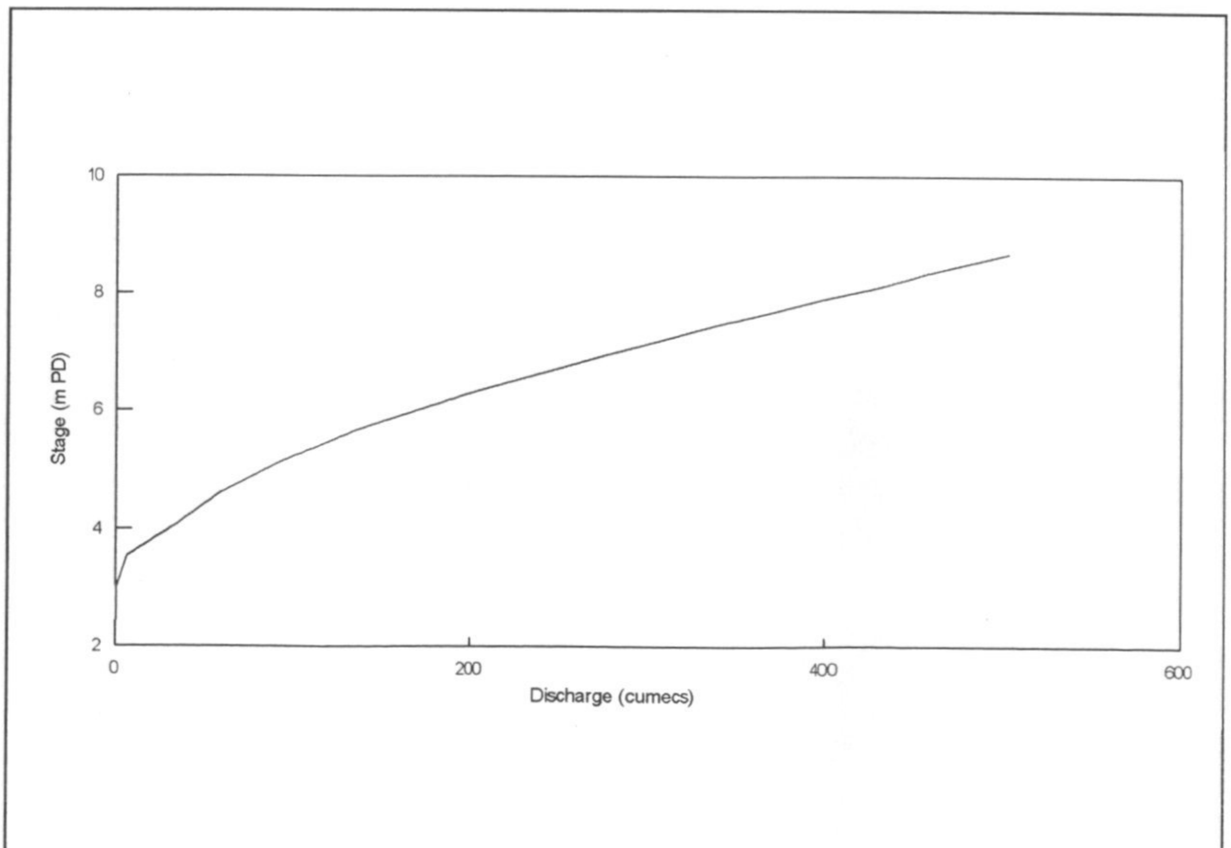


Figure 61 Rating curve for the downstream boundary condition.

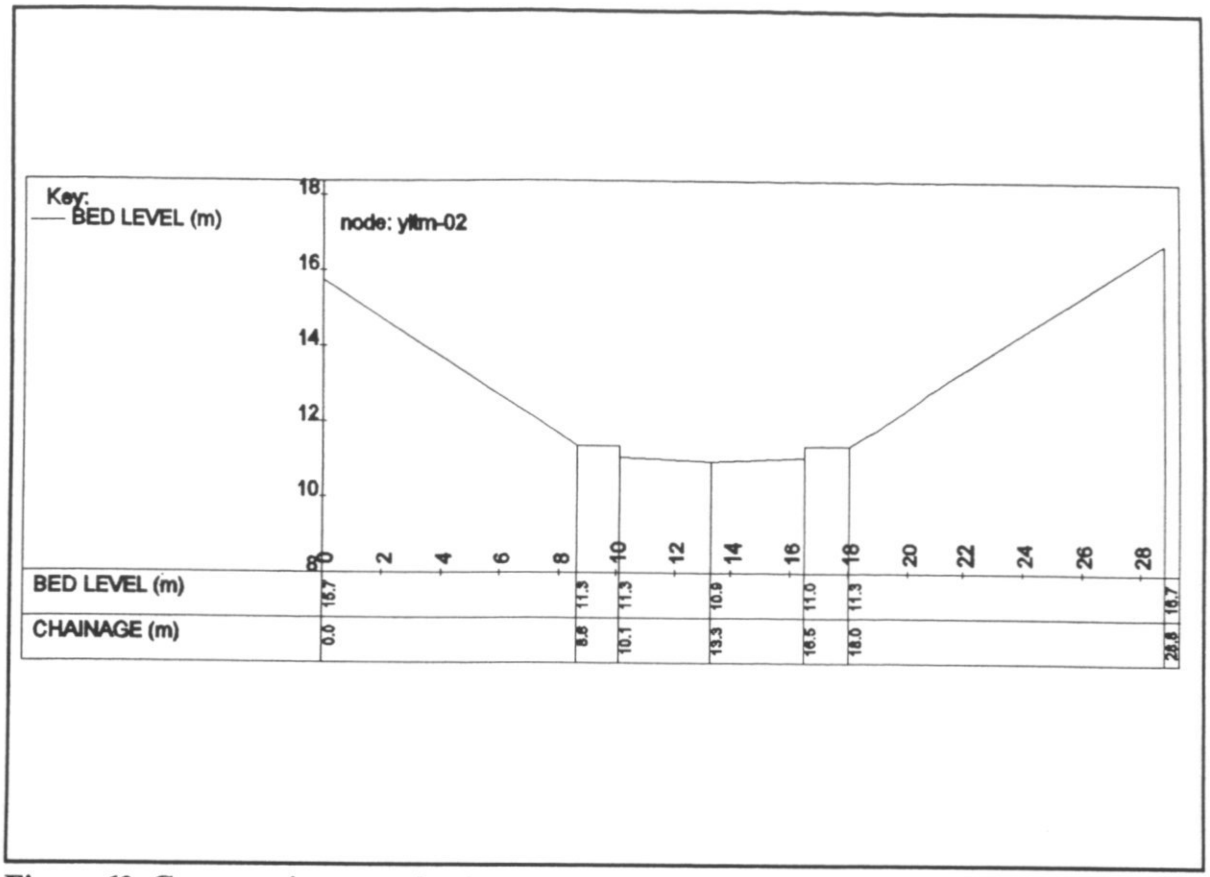


Figure 62 Cross-section at node yltm-02

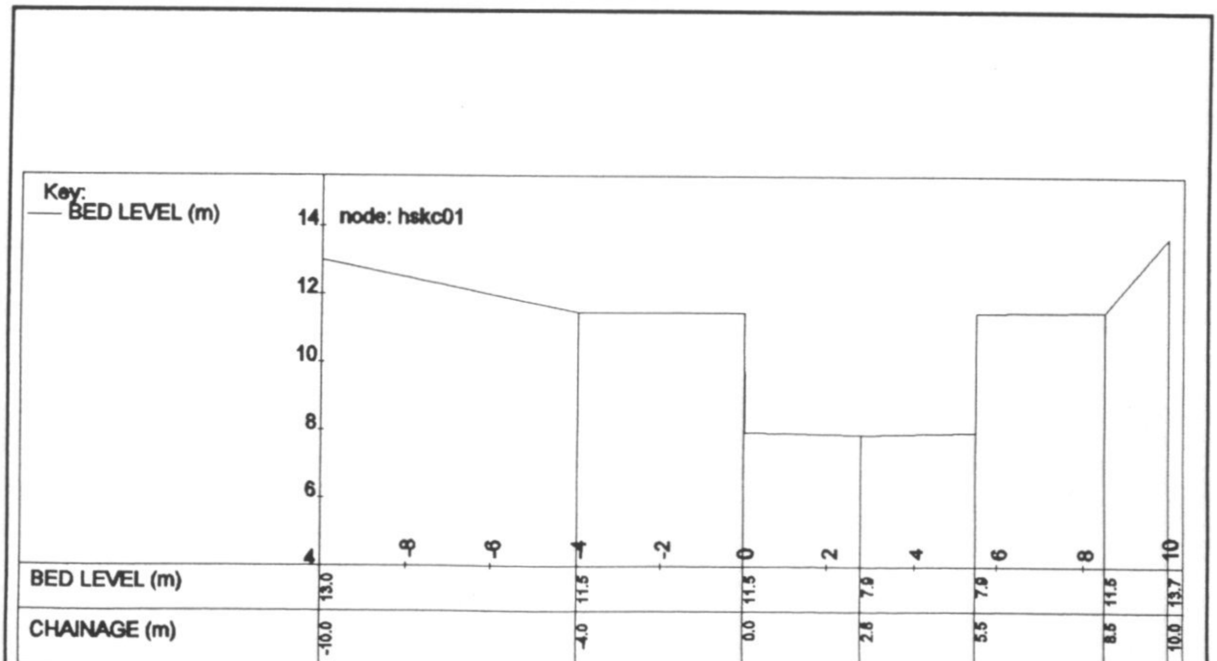


Figure 63 Cross-section at node hskc01

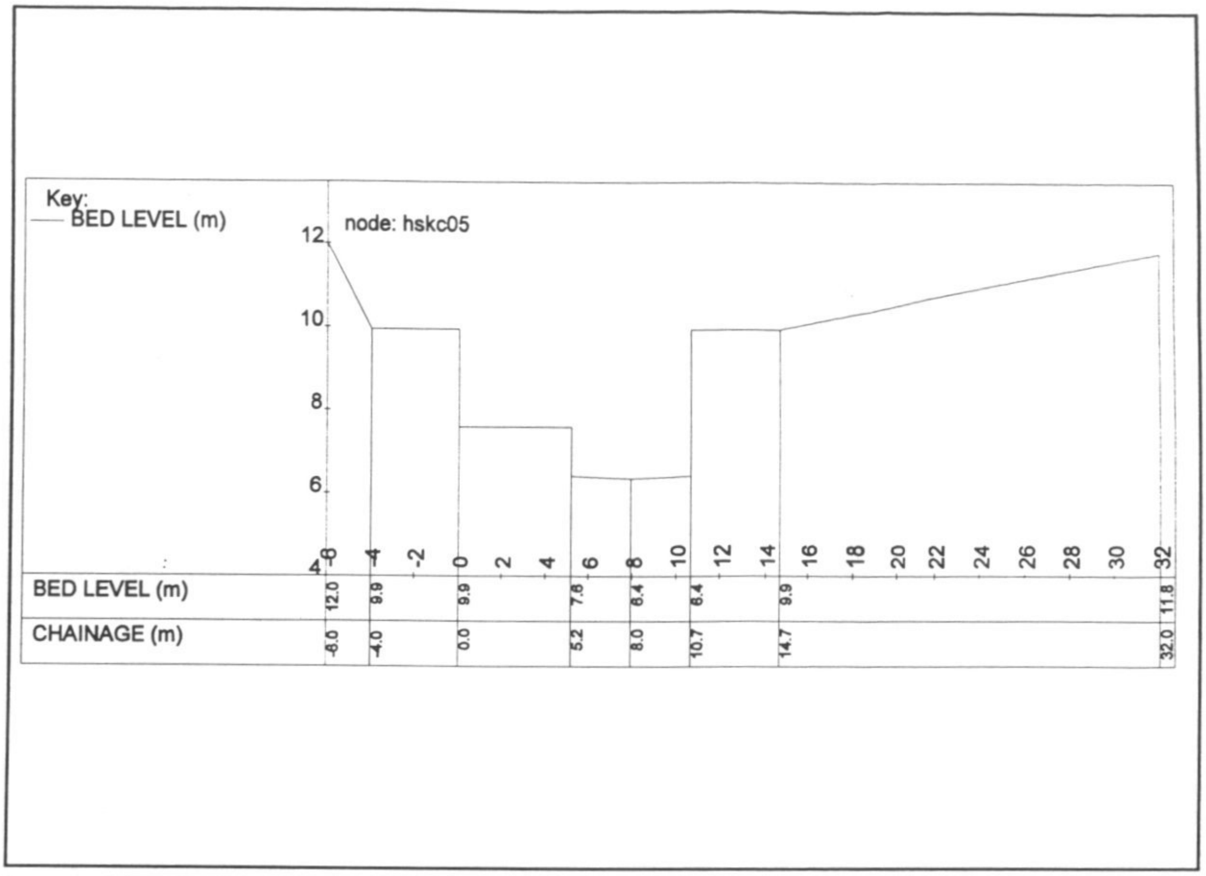


Figure 64 Cross-section at node hskc05

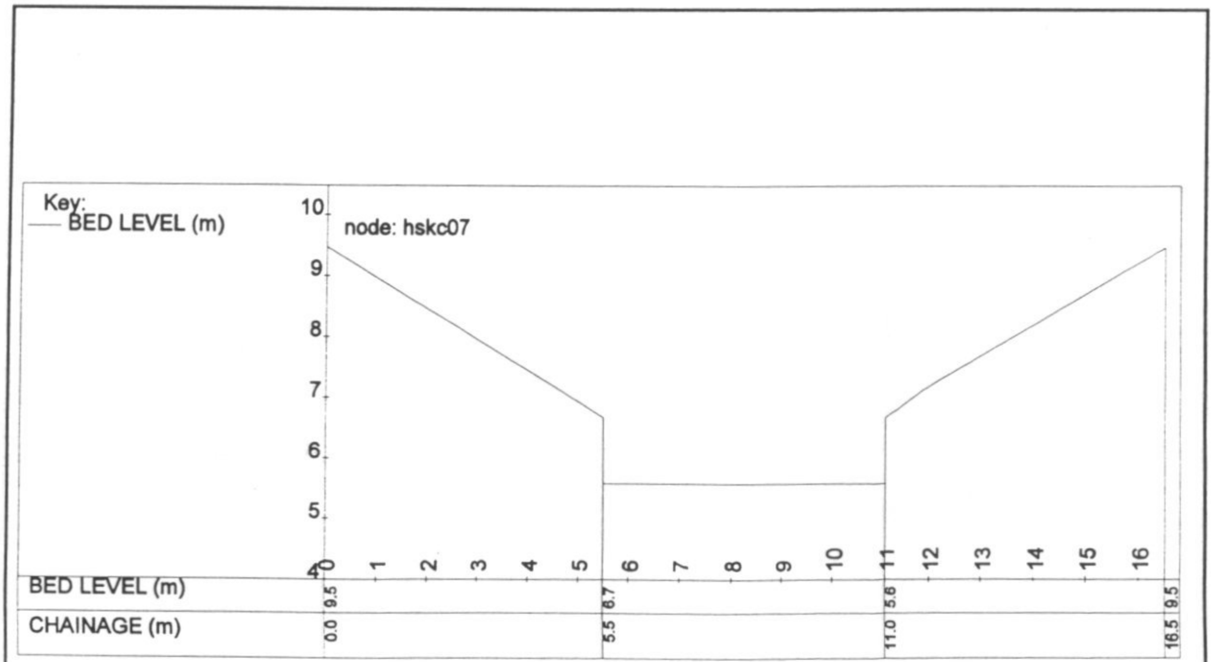


Figure 65 Cross-section at nodehskc07

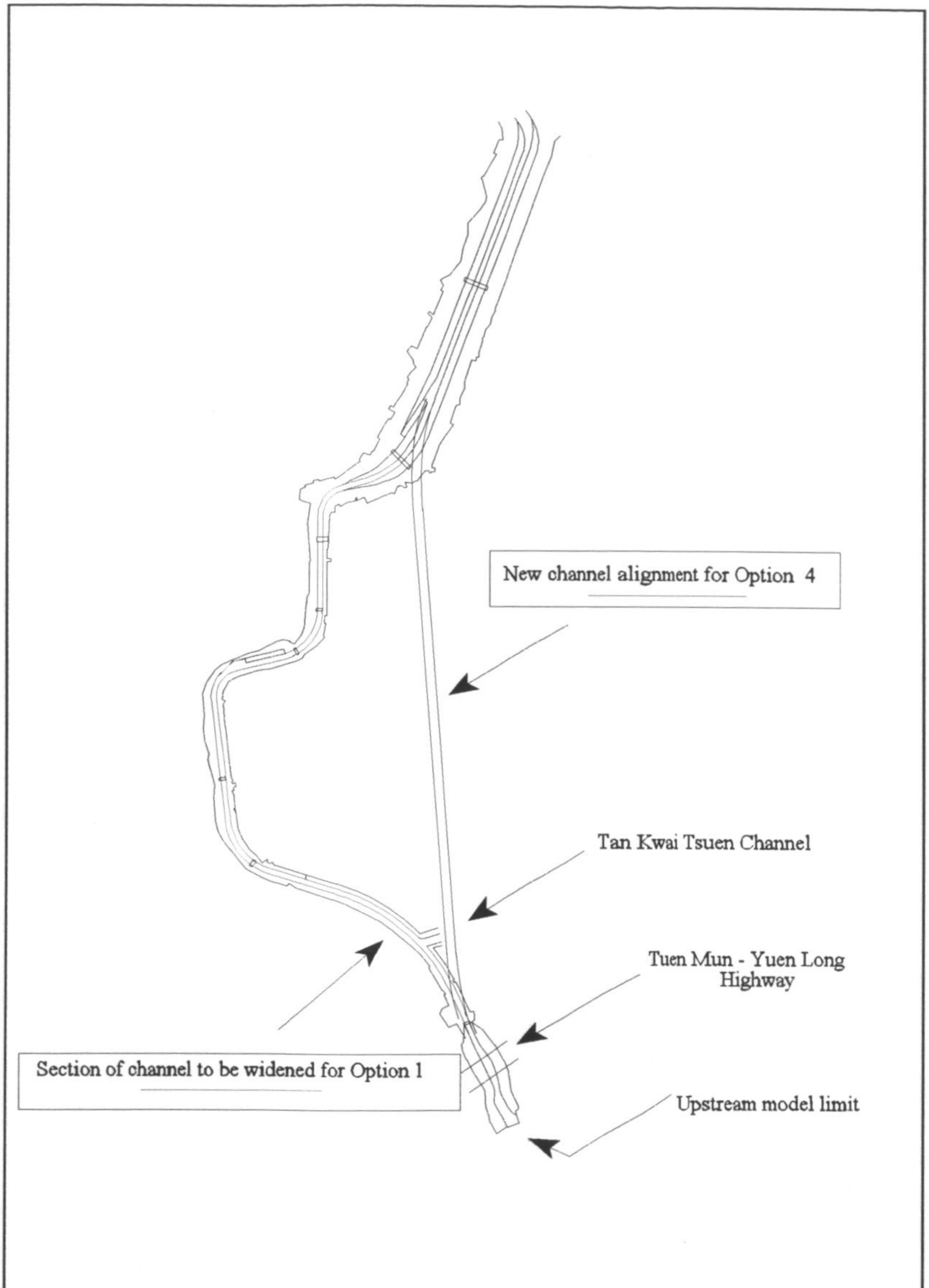


Figure 66 Section of channel to be widened for Option 1 and New channel alignment for Option 4.

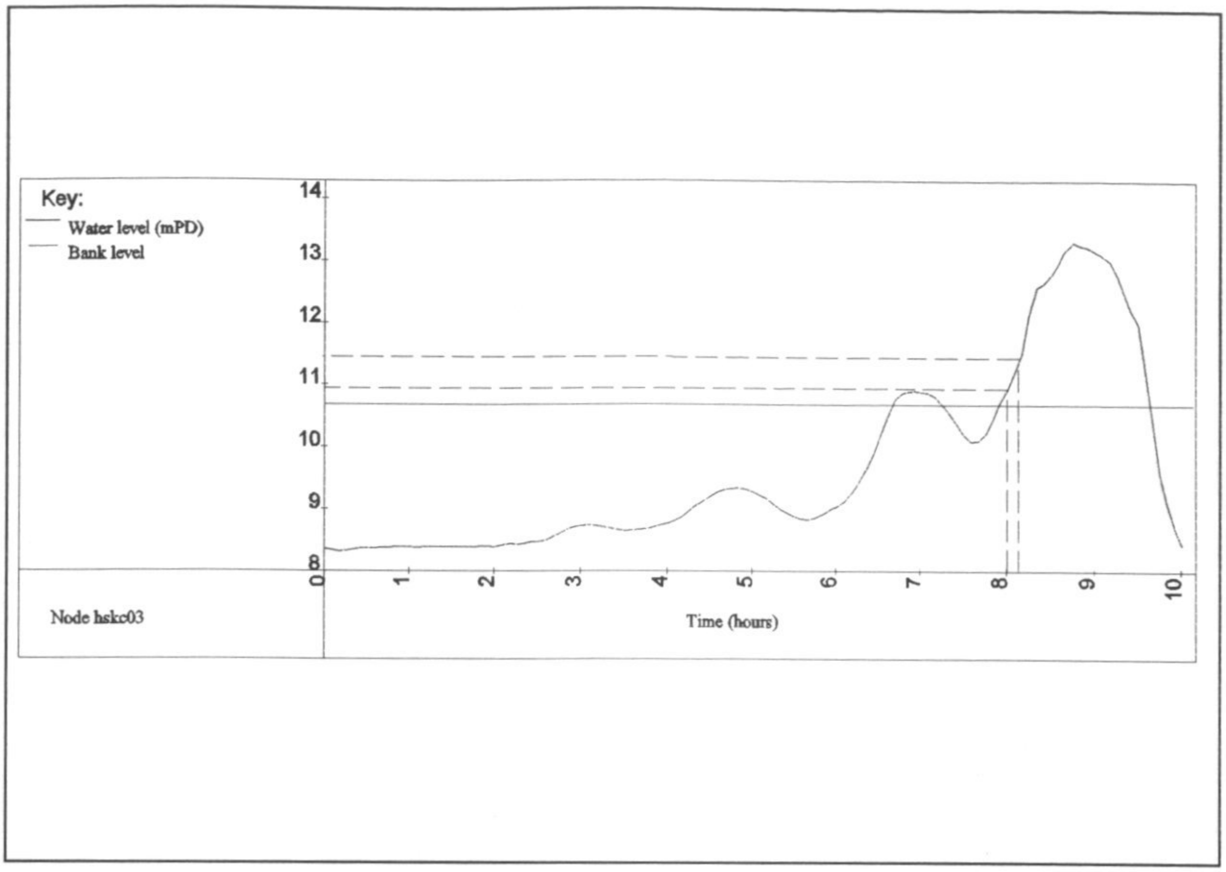


Figure 67 Stage hydrograph for the November, 1993 event at node hskc03

Appendix 1

Summary of data collected

The Hydrological/hydraulic investigation of the Hung Shui Kiu Drainage Channel
Summary of information/data collected

1. DSD/DP (94)

i) Design calculation and reports - 18 items

ii) Drawings:

Item (19): DDN/21CD/0002 -	Catchment plan - main drainage channel for Tin Shun Wai Hinterland.
Item (20): DDN 8364C -	Flood control works of NWNT development.
Item (21): NDD 6380 - 6397 } NDD 6416 - 6418 } NDD 6421 - 6428 }	Hung Shun Kiu drainage channel.
Item (22): NH 7710C } NH7818A } NH 7819A }	YL - TM Eastern Corridor (TM section).

Item (23): 12 aerial photographs

iii) 'Development investigation of North Western New Territories - Base strategy study - Working paper : Preliminary drainage & flooding protection design'.

iv) 'Existing flood and low flow conditions and selection of drainage scheme'.

2. DSD/LD (94)

i) Hydrometric Data of November 93 event, including :
   - a) 15-min rainfall and 60-min moving totals at Yuen Long R.G. Filters (No. 17).
   - b) 5-min rainfall and 60-min moving totals at rain gauge stations Nos N07 and N12.
   - c) Hourly/Daily summary of 13 rain gauges in the New Territories.
   - d) Hourly/Daily summary of 13 river gauges in the New Territories and a tide gage at Tsim Bei Tsui.
   - e) Flood marks.
   - f) Location of automatic reporting rain gauges.

ii) Hydrometric Data of July 94 event
   - a) 15-min rainfall at Yuen Long R.G. Filters (No 17).
   - b) Hourly/Daily summary of 13 rain gauges in the New Territories.
   - c) Hourly/Daily summary of all rain gauges in the Territory.
   - d) Hourly/Daily summary of 13 river gauges in the New Territories and a tide gage at Tsim Bei Tsui.
   - e) Flood marks.
   - f) Location of automatic reporting rain gauges.

iii) 1 copy of survey sheets of Tin Shun Wai drainage basin in 1:1000 and 1:5000.

iv) 3 additional copies of survey sheets of Tin Shun Wai drainage basin in 1:1000 and 1:5000 scale.

v) TEL's BMP planning report no.8 - Tin Shun Wai.

vi) Aerial photographs of the Hung Shun Hang Catchment taken in 11/93.

vii) Initial report on flooding on 22/7/94.

viii) Newspaper cuttings.
ix) Record photos of the event on 22/7/94.
x) Flooding situation report from a resident site office at Tin Shun Wai.
xi) RO's Monthly Weather Summary - 11/93.
xii) RO's Monthly Weather Summary - 7/94.
xiii) Rainfall frequency analysis on RO HQ. data (1884 - 1990) carried out by RO.
xiv) Rainfall frequency analysis on Yuen Long RG filter and Tai Po Tau treatment works (1980 - 1990) carried out by RO.
xv) Design flood for Hong Kong. Public Works Department (1986).
xvi) Flood extent map on 22/7/94.
xvii) Monthly summary of daily rainfall of GEO and RO rain gauges.
xviii) Rain gauges in Hong Kong - location map.
xix) Memo : LD - DS, O&M
'FMRS3 - Initial flood report'.
- ref : LD 1/4/12 - 94 f (25/7/94)
xx) Memo : MN - LD, DP.
'Flooding on 22/7/94 at Tan Kwai Tsuen, Yuen Long'
- ref : MN 5/14/6 - 4 (2/8/94)
xxi) Memo : CE/TSW - LD
'Tin Shun Wai development, DSD, flood monitoring and reporting system'.
- ref : HPH/TSW/S/1128 (25/8/94)
'FMRS2(b) - Post incident flood report (other departments)'.
- ref : HPH/TSW/S/1128
xxii) Memo : HyD - DP
'Contract # HY/89/17 - YL - TM Eastern Corridor (TM section)'.
- ref : HNT 55/384-20/2 (5/9/94)
xxiii) Memo : CRE/TSW - LD
'Comment on November 93 and July 94 event'
- (ref : HPH/TSW/S/1128)
xxiv) 'Stormwater drainage manual - Planning, design and management'.
xxv) Colour site photographs.
xxvi) Newspaper cuttings.
xxvii) Drawings :
a) DLD 3000 (1:500)
b) DLD 3001 (1:500)
c) DLD 3002 (1:500)
d) DLD 3003 (not to scale)
xxviii) Relevant memos on the design of Hung Shun Kiu drainage channel.

3. **DSD/MN (94)**

   i) As - built drawings :
      a) A69R, W57R - W61R, W76R - W61R, W202R - W215R, W217R - W219R
      b) RK1427, RK1427/1 - RK1427/11
   ii) Remedial measures implemented since the July 94 flooding event - ref: MN 5/14/6 - 4A (29/11/94).

4. **GEO (94)**

   i) Geology of Yuen Long. HK Geological survey sheet report No.1
   ii) 8" x 8" aerial photographs - 9 sheets
   iii) HK Geological Survey 1988 - Solid and superficial geology of Yuen Long sheet 6 1:20K.

5. **Watt, W.E., Chow, K.C.A., Hogg, W.D. and Latthem, K.W.**
   'A 1-h urban design storm for Canada', can. J. civ .Eng. 13. 293-300 (1986)

6. **Watt, W.E. and Chow, K.C.A.**
   'A general expression for basin lag time', can. J. civ. Eng. 12. 294-300 (1985)

Note: All the information above except items 2(i), 2(ii) and 3(ii) will be returned to the corresponding departments on completion of this study.

Appendix 2

Comments from DSD and other Government departments and the corresponding responses from Hydraulics and Water Research (Asia) Ltd

Land Drainage Division Comments:

1.	Para 1.2	
	This draft report and the Final Report to be structured in accordance with Section 7.2 and 7.3 of the Brief respectively.	Agreed - text has been modified
2.	Para 2.2.1	
	Not all five rain gauges are autographic.	Agreed - text has been modified
3.	Para 2.2.3	
	Provide a listing of flood marks, the surveyed flood levels, together with a location plan.	Locations have been shown on the revised Figure 34. A table of the flood marks and surveyed levels will be provided
4.	Para 2.2.4	
	How is the tide record used in the hydraulic modelling and tidal effect accounted for in this study?	The follow text has been added: "It was necessary to ascertain whether or not the tide is an important factor in the flooding. For the November, 1993 event, the peak tide level was 2.60 mPD, only 0.10 m higher than the 2.50 mPD channel bed level at Shek Po Tsuen. The observed peak level at the Shek Po Tsuen gauge was 4.83 mPD, significantly higher than the peak tide level and coincident with the peak discharge, negating any possible backwater influence from the tide. The July, 1994 peak tide level was 2.87 mPD against and observed peak water level at Shek Po Tsuen of 5.20 mPD again negating any possible backwater effect of the tide."
5.	Para 2.3	
	a) '400 m' and '40 m' should be amended to read '400 mPD' and '40 mPD' respectively.	Agreed - text has been modified
	b) What does the outfall at 40 mPD refer to?	Due to a typing error the text should read that the outfall is at about 20 mPD, the outfall referring to the irrigation weir just upstream of the TM-YL Highway bridge - text has been modified

6.	Para 3.2	
	Runoff coefficient for Rational Method is different from the proportional loss coefficient in a loss model. For a loss model, calibration of loss rates is based on total volume of rainfall and runoff during an event. Calibration of runoff coefficient is based on peak rainfall and peak flow in the Rational Method and the runoff coefficient includes attenuation effect of catchment response. They refer to different parameters of different hydrological models, and it will be very misleading to compare the two parameters directly as has been done under this draft Final Report.	Agreed - text has been modified
7.	Para 3.3.1	
	a) It is better to suggest the use of Watt's formula based on: i) Physically more realistic to have a Tp/LAG of approx. 1/2 hr. Therefore Bransby-Williams formula is not used. ii) Watt better than Kirpich because Watt based on more data. iii) Ability to provide a good calibration result. The sentence 'Since LAG or Tp is a function ..... should not be applicable in Hong Kong' however seems to suggest Bransby-Williams is also applicable.	Agreed - the text has been modified slightly to reflect these comments
	b) Hourly catchment average rainfall in Figures 13 and 16 should be indicated in bars instead of lines.	We do not concur with this comment and Figures 13 and 16 stand as in the Draft Final Report
	c) Explanations in the reports for the following different approach adopted for the two events are required: i) For 5/11/93 event, hourly catchment average data was desegregated to 15 minutes totals by scaling the 15-minutes totals at raingauge No 174, instead of No 17 which is closer to the centroid of the catchment. As for 22/7/94 event, raingauge N12 was used.	In the derivation of the catchment average rainfall profiles for the calibration events, the reason why two slightly different approaches were adopted was explained previously in our responses to the comments on the second report. However, to summarise, from the information supplied by DSD and RO during the data collection phase of the study, the Consultants understood that no 5-min data were available for the 1994 event. No174 was therefore used as the raingauge on which to base the sub-hourly profile for both events because: (i) it was the only gauge for which sub-hourly data were available for the 1994 event, (ii) it was the only gauge for which sub-hourly data were available for the entire day for both

ii) For 5/11/93 event, the desegregated 15-min rainfall were divided into 3 equal depths for input as 5-min rainfall, which in fact means that 15-min rainfall data is used. On the other hand the hourly catchment average data were desegregated to 5-min totals directly for the 22/7/94 event.	events, and (iii) the Consultants wished to be consistent about which raingauge they used for this purpose and how they derived the catchment average rainfall profiles. Therefore, 5-min profiles were derived for both calibration events based on gauge No174 and disaggregation of 15-min catchment rainfalls into three equal 5-min rainfalls. For the 1993 event the choice of No174 was not critical since all the raingauges had very similar profiles. However, it was acknowledged that for the 1994 event the individual raingauge profiles were quite different with N12 best matching the recorded stage. However, after all the work had been done to derive the 5-min profiles, DSD supplied some previously undiscovered 5-min data for the 1994 event. These data were supplied late, and the Consultants could have ignored them. However, since N12 provided a better fit to the recorded stage for the 1994 event, and data were available for the whole day, the Consultants reworked the catchment rainfall for this event. It was not felt necessary to subsequently rework the catchment rainfall for the 1993 event because: (i) No174 provided an adequate fit to the recorded stage and compared well with other raingauges, and (ii) the disaggregation of the 15-min catchment rainfalls into three equal 5-min rainfalls provided an adequate fit to the recorded stage.
d) In establishing the return period of rainfall depth, a more critical rainfall duration, say 1 hour, should be used. Use of 4-hr duration rainfall tends to indicate a less severe event, especially for 1993.	For the 1993 event, the peak rainfall of 114.35 mm in 1 hr corresponds to a return period of approximately 136 years. For the 1994 event, the peak rainfall of 80 mm in 1 hr corresponds to a return period of 14 years - text has been modified.
e) The inflows from catchments 2, 5, 6 and 7 had been modelled as diffuse lateral inflows in the hydraulic model. Please clarify the followings indifferent observations as shown in Tables 11, 14 and 17-35: i) The abrupt change in flows at sections over Castle Peak Road, ie between sections cpr-us and cpr-ds. ii) Flow decreases gradually from section cpr-ds downstream to hskc15d.	The inflows from catchments 2, 5, 6 & 7 were modelled as discrete inflows rather than diffuse lateral inflows. This reflects the presence of a number of drainage outfalls in the banks of the channel. Inflows from catchment 2 were modelled as a discrete inflow entering the steep trapezoidal channel under the TM-YL highway via the pipe culverts on the right bank. The inflow from catchment 5 was divided equally between the two outfalls on the right bank at approximately chainages 402 m and 1324 m. No data on the location of the outfalls for catchments 6 and 7 were available and no evidence of any outfalls on the left bank was found during the site visits. Thus these inflows were modelled as discrete inflows entering the channel downstream of the Castle Peak Road culverts. This explains the abrupt increase in discharge between nodes cpr-us and cpr-ds. The gradual decrease in flow from nodes cpr-ds to hskc15d is insignificant (less than 2 m^3s^{-1}) and probable a reflection of the slight attenuation caused by take up of storage with in the channel.

8.	<u>Para 3.3.3</u>	
	To tie in with the requirements of the study brief, last paragraph should be reworded to the effect that calibration model is based on 1994 event during when more data is available and that verification is carried out based on 93 events.	Agreed - text has been modified
9.	<u>Para 3.3.4</u>	
	Should 'percentage runoff method' better be named as 'proportional loss method' ?	Agreed - text has been modified
9a.	Should para 4.4 and 4.4.1 in page 21 be amended to read as para 3.4 and 3.4.1 respectively?	Agreed - text has been modified
10.	<u>Para 3.4.1</u> (refer to para 9a above regarding paragraph numbering)	
	Historic rainstorm profiles recorded at WSD autographic rain gauge (ref no. 17) had not been analyzed and considered in the selection of design rainstorm process (para 4.2(d) or the Brief refers). Why?	As explained in section 2.2.1, there appears to be no great merit in analysing significant historical events from the autographic raingauge charts available at N017 Yuen Long RG Filters within the short timescale of the current study since RO have recently published the results of such analysis for the period 1981-90 (Lam & Leung, 1994), which should be more reliable. Furthermore it is unclear how the results of such a study might be used in deriving flood estimates of required return periods. Experience shows that the most appropriate design flood estimates are produced using generalised storm profiles, such as those derived by RO. Such a design storm profile is likely to be somewhat more peaky than most historically observed storms, but this yields conservative, or safe, peak flood estimates - text has been added to section 3.4.1.
11.	<u>Para 3.4.2</u>	
	Hydrograph for 1 hour storm is missing in Figure 28.	Agreed, Figure 28 will be corrected.
12.	<u>Para 4.1.3</u>	
	a) How is the stage discharge rating curve derived and how is it subsequently used in the model? A figure showing the rating curve is also required.	The rating curve was calculated using the conveyance properties of the channel cross-section and value of Manning's n determined during the calibration. A Figure showing the rating curve was sent to DSD

	b) Figures of cross sections, some at odd locations such as access ramp, transitions and section where tributary runs in, are to be provided.	Agreed, figures will be provided
13.	<u>Para 4.1.6</u>	
	There are substantial difference between the modified n estimated at Table 10 and the calibrated result at Table 12 (0.051 c.f. with 0.025). Please explain.	Table 10 gives modified Manning's n values based on the experimental work of Toebes & Sooky. Table 12 gives the values of Manning's n determined during the calibration. The later are believed to be more appropriate because these values, when used in the hydraulic model, reproduce the observed water levels taken during the 1994 event. The work of Toebes & Sooky has been included to support the use of higher than usual Manning's n to represent the additional energy loss at channel bends.
14.	<u>Para 4.2.1 and 4.2.2</u>	
	Explanations on why sudden drop of water level (approx 1.1m and 1.6m for 94 and 93 event respectively) occur near the 4th bend (sections labelled hskc06 to hskc06b2, where major headloss appears) as shown on tables 11 and 14 is required.	The position of node hskc06 is incorrect in Tables 11, 12 and 14 - 16. It should be between nodes hskc06b1 and hskc06b2. However the results are consistent with the node labels. The headloss between nodes hskc06 and hskc06b2 is associated with the surcharging of the vehicular bridge CV 1. the headloss between hskc06b2 and hskc06d is associated with the energy loss caused by bend 4.
15.	<u>Para 4.2.3</u>	
	a) Where is the best location for stage gauge? Is it for short term flow measurement only?	The following extra text will be added.
	b) More detailed recommendation is required as per requirements in para 4.4(c) of the Brief.	"The Consultants recommend that five temporary water level records are installed along the existing rectangular channel. These should be at the following locations: (i) the confluence with the Tan Kwai Tsuen Channel (ii) upstream of bend 2 (iii) between downstream of the access ramp (AR 1) and bend 3 (iv) upstream of bend 4 (v) downstream of bend 4 at the start of the channel transition. Data from the first gauge will provide more information about the energy losses occurring at the confluence. Difference in the water levels measured between

		Comment	Response
			gauges 4 and 5 will provide an improved assessment of the energy loss around bend 4. Similarly difference between levels at gauges 3 and 4 will provide a better assessment of the energy loss at bend 3. This may also be used to relate energy loss to bend radius and thus enable more confident predictions to made for the energy loss associated with bend 1 and 2. Gauges 2 and 3 , along with new estimates for the energy loss for bend 2 will provide further information on the impact of the access ramp (AR 1). These gauges should be installed before the 1995 monsoon season and remain in place for at least one monsoon or until at least three reasonably large events have been recorded. A reasonably large event might be one during which the water level in the channel exceeds 2 m. Ideally the gauges should record the water level at intervals of no more than 5 minutes over a range of channel depths from say 1m to 4.6m. Continuing to record the water level once the flow is out of bank will provide useful calibration data for the flood plain roughness coefficients."
16.	Para 4.3		
	a)	It is useful to point out that, compared to the suggested design flow, the flow simulated in 94 event corresponds to a return period of approx. 10 years and 93 events corresponds to a return period of approx. 50 years. It also indicates that the use of 4 hr duration rainfall for deriving the return period of a flood event is not appropriate (see para 3.3.2).	This comment seems to relate to the perception that the 1993 event produced a more extreme response than that of 1994, which was mainly because the latter event had three peaks and rainfall was more prolonged and less intense at the centre of the storm. The comment appears to be confusing the issues of model calibration and determination of design floods of specified return period, particularly the suggestion that the 4 hour duration is not appropriate. As shown in Section 3.4.1, assuming that the RO design storm profile is adopted, any storm duration exceeding two hours will produce the flood peak of required return period, and the four hour storm was recommended in order to ensure an adequate wetting period for the catchment. As shown in Figure 28 the same flood shape and peak was derived for all storm durations exceeding one hour (although unfortunately the figure did not show the hydrograph for the 1 hour storm correctly). The comment has also in part been responded to under our reply to comment 7(e) above.
	b)	How is the downstream boundary condition specified?	see comment 12
17.	Para 5.1.3		
	a)	For the two events, to use return period on design flow as in my comments on para 4.3 above.	See the response to comment 16(a).

		Comment	Response
	b)	Due to the special catchment configuration, use of short duration rainfall is more appropriate. This will leads to higher runoff derived.	The use of shorter duration storms will not produce higher flood peaks as suggested because of the use of the nested RO profiles. These design storm profiles nest intense, short duration storms within longer storms, and hence once a critical duration has been reached (somewhere between 1 and 2 hours in this case) then increasing the storm duration further will have no impact upon the peak flow, but will merely affect the volume of runoff.
	c)	Runoff coefficients used in the hydrological study is strictly not comparable with those used in the original design (refer to my comments on para 3.2 above).	Agreed, but there is no real merit in comparing the original design parameters with those used in the current study. The parameters adopted in this study, including the runoff coefficient selected, provide an acceptable model calibration.
	d)	Add paragraph stating that energy loss from the bend and access ramps can only be properly and accurately determined using more sophisticated hydrodynamic model that is adopted in this study, and therefore the loss is underestimated in the original design.	Agreed, the following text will be inserted: "Assessment of the impact of energy loss at bends and changes in the channel geometry, such as those at access ramps can not be easily undertaken with out using a sophisticated hydrodynamic model such as the one used in this study. Thus modelling should be adopted as part of DSD design procedures for all but the most simple, straight channels."
	e)	It appears that the villagers refer to the reservoir at upstream of HSK catchment.	If this is the case then the outflow from this reservoir will result from runoff from the upper catchment which was accounted for by hydrological models
18.	Para 5.2		
	a)	In Figures 45-48, the bank level appears not raised. Also, in Figure 46 the max. water level appears wrong.	This observation is incorrect. Figures 45 - 48 show that the banks have been raised as far as the confluence with the Tan Kwai Tsuen Channel, accurately reflecting the existing remedial measures as built at the time of the modelling. Figure 46 is incorrect, it is actually the results of the 1 in 2 year event. This will be amended.
	b)	Table 21 appears to indicate that flooding still occurs for the 1 in 2 year event.	The sentence has been changed to say "... the raised banks ***almost*** contain the flooding..."

No.	Item	Comment	Response
19.	Para 5.3		
	a)	How is the downstream boundary specified in all these modelling work for each proposed remedial measure option?	See comment 12
	b)	Note that the Brief defines flood protection standard for long term measures as 1 in 50 yrs. Options 1-4 should therefore be based on 1 in 50 yrs instead of 1 in 10 yrs.	Tables 27 and 33 clearly demonstrate that the recommended options for long term remedial measures, Option 1A and Option 4 have sufficient capacity to contain runoff from the 1 in 50 year design rainfall event with a freeboard of at least 300 mm along the rectangular channel. These tables also show that there may be some minor flooding from the entry and exit from the rectangular channel. The banks should be raised at these location s to over come this.
20.	Para 5.3.1		
	a)	Are the headloss for ramps also taken out?	Access ramp AR1 was removed.
	b)	According to Fig. 36, increase in width will increase in headloss. Do we need to remove the bends under this scheme? Should provide a minimum radius for the widened scheme.	Figure 36 is based on the work of Leopold (1960) and indicates that none of the current bend would cause any significant increase in energy loss (Para 4.1.6). Widening the channel will change the width to curvature ratio which according to Figure 36 would lead to an increase in the energy loss. However this approach was not adopted for selection of the Manning's n. Should DSD wish to specify a minimum radius for the new channel bends then Figure 36 suggests a radius of 57.5 m or above for a channel width of 11.5 m.
	c)	Long term measure to be based on 1 in 50 yr.	See comment 19 (b)
	d)	Figure 51 and 52 are obviously wrong as they show overtopping flow along the improved sections.	There is no overtopping of the improved section in Figure 51 and only minor of the improved section in Figure 52. There is some minor flooding from the entry and exit from the rectangular channel. The banks should be raised at these location s to over come this.
	e)	Option 1A is to be adopted. This is one of the 2 recommended options found to be feasible. A schematic layout to be provided (see para 4.4(b) of Brief).	Option 1A follows the route of the existing channel therefore no new schematic is needed.

21.	<u>Para 5.3.4</u>	
	This is one of the 2 recommended options found to be feasible. A schematic layout to be provided (see para 4.4(b) of Brief).	This has been sent to DSD
22.	<u>Para 5.3.6</u>	
	a) All those recommended short term measure options should have a schematic layout (see para 4.4(b) of Brief).	DSD have already produced drawing of the proposed modification to the access ramp and channel transition therefore there is no point in doing this.
	b) According to the Brief, the short term measures proposed should achieve a flood protection standard of 2-10 years. The Consultants should specifically point out that this can hardly be achieved and should conclude that the short term measures all together can achieve a certain flood protection.	The short term measures proposed by DSD (1995b) whilst improving the current performance of the channel, will not provide a standard of protect equivalent to the 2 to 10 years specified in the brief. Thus further measures should be considered such as the construction of the siphons explored as Option 5. However as stated in para 5.3.6 these works are still supported by the consultants.
23.	<u>Para 5.4</u>	
	a) Though it is of shorter length, construction cost may be higher for Option 4 than Option 1 as Option 4 involves channel excavation on high ground (approx 7-10m excavation depth c.f. 4m along existing channel) and provision of local drainage along existing channel. Additional revenue made available by the release of land is also limited as local drainage are required along the existing channel once the new channel is completed.	Our recommendation that detailed estimates be prepared for both Options still stands.
	b) Under 4.4(c) of the Brief, the Consultant should recommend a programme of data acquisition e.g. rain gauge and short term flow survey, with suggested locations.	See comment 15
24.	<u>Table</u>	
	Table 14 - should be 1993 flood event, not 1994.	Agreed
25.	<u>Figures</u>	
	a) Figure 34 should be expanded to show major nodes with labels (at least for identification of those labels on Tables no 11 - 35 and Figures 37 - 39). If necessary, scale is to be expanded to fit A3 size paper.	A revised figure has been sent to DSD

b) Figures 45 and 46 - label nos. are missing.	These will be amended
Regarding the preparation and submission of the Final Report, you should take note of the requirements in para 7.3 of the Brief. I suggest that the Final Report to be bound in three separate volumes as follows: Vol. 1 -Contains the main report and comments by DSD and other Government departments on the Draft Final Report, together with Consultant's responses. Vol. 2 -As appendix to Vol. 1, should contain summary of information/data collected and the input and output files for all the modelling works. Vol. 3 -Executive Summary. We will confirm the need of Chinese text for the Executive Summary upon receiving your quotations.	The report will be produced in accordance with clause 7.3 of the brief. One (1) copy of input and results on ascii files will be provided on disk and paper.
Drainage Projects Comments:	
(a) Heavy rainfall has been considered the main cause of flooding during the historic rainstorm events in November 1993 and July 1994. However, it is now noted that the return periods of both events are estimated by the Consultants at 13 years, which is only marginally higher than 10-year design standard of the channel. Perhaps the return periods of the two events should be calculated for various durations. It is recalled that during the November 1993 event, the maximum 60-minute point rainfall at Yuen Long RG Filters was 119.5mm, with a return period exceeding 100 years. It is also likely that the Consultants have underestimated the catchment rainfalls (and thus runoffs) on the Hung Shui Kiu Channel during the two events due to gaps in the data of the raingauge at Yuen Long RG Filters (particularly during the July 1994 event) and the absence of raingauges at higher altitudes which often experience heavier rain. Large stones set into motion were also an evidence of exceptionally high flows.	See the reply to comment 7(d) above. It is possible that the 'true' rainfall over the catchment, particularly during the July 1994 event, may have been underestimated, however, the best use has been made of all available data. Perhaps this comment and others support the consultants suggestion to DSD (personal communication from Mr Farquharson) that Hong Kong urgently requires a thorough review of methods of studying rainfall and flood events throughout the territory.
(b) Besides the limitations of catchment rainfalls as discussed above, other uncertainties in the hydrological calibration include inaccuracies of runoff measurements due to the less-than-ideal configuration of the irrigation weir and the large mismatch between predicted and measured runoffs for the November 1993 event. Therefore, the resulting hydrological parameters (eg 90% runoff) should be taken with some caution.	The suggestion that the hydrological parameters adopted '*should be taken with some caution*' is disputed. The hydrological and hydraulic models have been fitted adequately given the data available.

(c)	We have adopted a Manning's n of 0.015 (which is commonly used in Hong Long) for estimating the hydraulic capacity of the concrete channel. We are concerned to note that the Consultants have advised that n can become as high as 0.025 due to the presence of the bends at high Froude Numbers. Details of the supporting literatures (Leopold et al, 1960; USSCS, 1963) are requested to enable us to fully understand the basis of the consultant's recommendation which has implications on our current design standards. Furthermore, if the runoff has been underestimated as discussed above, the suggested n-value (of 0.025) would have been overestimated in the hydraulic calibration.	Extract of USSCS (1963) have been sent. We will try and provide copies of Leopold (1960). The comment above has been disputed and so is the suggestion that the Manning's n values have been overestimated.
(d)	Our above observations on the limitations of the hydrological/hydraulic calibration processes lead us to believe that the recommended calibration parameters are likely to be 'Worst-Case Scenario' values, which will result in very conservation design of the remedial works. it is also noted that the Consultants do recognise the need for further hydrological data collection to verify the model calibrations.	There is no evidence for this observation. The hydrological/hydraulic parameters have been fitted using all the available data. Whilst the Consultants support further data collection to improve the confidence in the calibration of the models we are not aware of any information that suggests that the parameters are either over or under estimates.
(e)	Our specific comments are: i) <u>Para 3.1.1 page 8</u> Did the hydraulic modelling during the November 1993 event reproduce bankfull condition at 8:00 hrs 1.5m rise between 8:00 hrs and 8:10 hrs ?	The following text has been added: "The villagers observed that the channel was approximately bankfull at 08:00 hrs but that the water level rose by 1.5 m between 08:00 and 08:10. These observations must be treated with caution because they are based on a visual assessment, no actual measurements of water levels were taken, and the exact times of the observation can not be verified. Figure gives the stage hydrograph at node hskc03 for the November, 1993 event and shows that the model predicated the water level to be approximately 0.2 m above bank top at 08:00, rising to about 0.75 m above bank top by 08:00. The water level continues to rise very rapidly reaching 1.5 m above bank top at about 08:20. Although this does not exactly recreate the observations made by the villagers, which are themselves questionable, the model does simulate the extremely rapid increase in water level which occurred at about 08:00 and this is perhaps further evidence to support the choice of hydrological and hydraulic parameters used in this study."

ii) <u>Page 13</u>	
Figure 7 shows the lag times of WSD and FSR unit hydrograph to be similar. This contradicts Table 5 which shows lag based on WSD is much larger than that based on Watt & Chow.	For clarity, the peaks have been aligned in order to demonstrate that the difference in hydrograph shapes is small. Note that the two unit hydrographs have the same runoff volume. The intention was to show that the triangular unit hydrograph used was very similar to the WSD design manual version. The fact that the WSD Tp estimate is unrealistically long is a separate issue.
iii) <u>Para 4.11 page 23</u>	
The unit of q in Eqn(9) should be m^2/s and '=0' is missing from Eqn(10).	Agreed
iv) <u>Para 5.2 page 34</u>	
How are the downstream boundary conditions at Shek Po Tsuen assumed for the design events?	See comment 16 (b)
Drainage Project comments on the reliability of the Hydrological/hydraulic modelling:	
Further to the Consultants' responses to our comments on the draft final report, we still have some reservations on the reliability of the hydrological/hydraulic modelling in light of the quality of the available rainfall-runoff data.	We fully recognise the concerns over the availability of rainfall data, particularly for the higher elevation portion of the Hung Shui Kiu catchment, and there is, of course, no runoff data at all for the catchment. This lack of rainfall data from high elevations was commented on in our report, and in Section 3.3.2 of the report we attempted to relate rainfall to elevation such that better catchment areal rainfall estimates could be produced. This brief study was however unsuccessful. However, we would query the implication in the comment by DSD that there is a problem with data quality, and that in consequence, they have reservations on the reliability of the hydrological/hydraulic modelling. Certainly we do not believe that there is any general problem with the quality of the rainfall data available, apart from possibly its representativeness, and therefore see no reason why this vital input to the modelling work that the quality of the rainfall data is inadequate. There may well be greater concerns over aspects of runoff estimation at the irrigation weir upstream of the TM-YL Highway bridge, and over estimation of hydraulic roughness and head losses at bends for example. However, we believe that in all cases we have made full use of all available data and that the results of

the modelling studies would therefore be as reliable as the data permits. We strongly refute the implication that the modelling results are in some way unsatisfactory, and also question the implied criticism of the quality of the input data on which the modelling has been based. If DSD are aware of any additional data that the consultants have not used, or believe that we have misinterpreted some of the information made available to us, then such knowledge should have been transmitted to us at an earlier stage.

Perhaps the comment is really emphasising the points made by the consultants in a letter to DSD some time ago that, whilst in world terms, Hong Kong has an abundance of good quality rainfall data, and broadly adequate flow records, there has been no comprehensive and thorough analysis of these data. There is a clear requirement that this valuable pool of information be scrutinised and analysed to provide hydrologists and engineers with a range of robust locally derived flood estimation procedures. Currently used methods are too often based upon empirical formulae form elsewhere in the world, or upon outdated, or incomplete analysis of the data. For example, the only rainfall-losses modelling work available was undertaken by WSD as long ago as 1968, and until recently, the only rainfall depth-duration-frequency analysis available was for the record at RO Headquarters. This has recently been extended by RO to three raingauges in the New Territories, but only a partial analysis of 10 years data has been undertaken. Given the sums of money spent annually throughout Hong Kong on drainage matters, there is clearly a need for a comprehensive analysis of all available data leading to a range of revised flood estimation procedures, along the lines of the UK Flood Studies Report.

Mainland North comments:

I refer to your above-referenced memo and the discussion held on 28.2.1995. In accordance with the Consultant's study, the November 1993 event rainfall had a return period of approx 1 in 13 years and the July 1994 event was approx. 1 in 13 years based on peak intensity or 1 in 38 years based on total rainfall. It can be seen that there is significant deference in the return period with different approach. As such, I would consider that it would be prudent to carefully look into the methodology that the Consultant had adopted in deriving these figures as it will

This comment has effectively been responded to in replies to Land Drainage Division's comments number 6, 7(d), 16(a), 17(a), (b) and (c), and Drainage Project's comments (a) and (b).

significantly affect the findings of the causes of the flooding and might also affect the design parameters to be adopted for our subsequent drainage channel design. Apart from the above, I have no other comment on the report.	
Royal Observatory Comments:	
2.It would be more appropriate to change the second paragraph of Section 2.2 to read: '... (pers. comm. K. Wong, 1994). The current RO radar can operate at four different ranges; 512, 256, 128 and 64 km. For both events, data archives were made at 256 km radius .....'	Agreed - text has been modified
Planning, Water Supplies Department Comments:	
No comment.	Noted

Appendix 3

Input and output ASCII data files

(listing of 3.5" floppy disk)

List of Input and result files

Input file name (.dat)	Result file name (.zzr)	Contents
ex2yn	ex2yn	Testing existing channel for 2 year storm
ex10yn	ex10yn	Testing existing channel for 10 year storm
ex50yn	ex50yn	Testing existing channel for 50 year storm
ex200yn	ex200yn	Testing existing channel for 200 year storm
exst10y	exst10y	Detail tests of existing channel for 10 year storm
exstcap	exstcap	
july94	july94	Simulating July 94 event
july94m	july94m	Detail simulation of July 94 event
nov93	nov93	Simulation of Nov 93 event
o1-2yn	o1-2yn	Testing RM option 1A for 2 year storm
o1-10yn	o1-10yn	Testing RM option 1A for 10 year storm
o1-50yn	o1-50yn	Testing RM option 1A for 50 year storm
o1-200yn	o1-200yn	Testing RM option 1A for 200 year storm
o1b50yn	o1b50yn	Testing RM option 1B for 50 year storm
o2-10yn	o2-10yn	Testing RM option 1 for 10 year storm
o4-2yn	o4-2yn	Testing RM option 4 for 2 year storm
o4-10yn	o4-10yn	Testing RM option 4 for 10 year storm
o4-50yn	o4-50yn	Testing RM option 4 for 50 year storm
o4-200yn	o4-200yn	Testing RM option 4 for 200 year storm
rm2yn	rm2yn	Testing existing RM for 2 year storm
rm10yn	rm10yn	Testing existing RM for 10 year storm
rm50yn	rm50yn	Testing existing RM for 50 year storm
rm200yn	rm200yn	Testing existing RM for 200 year storm

RM - remedial measures

Natural
Environment
Research
Council